University of Baghdad
Collage of Engineering
Civil Engineering Department

Traffic Engineering

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Lecture 1

Principle of Traffic Engineering

<u>Traffic Engineering:</u> A branch of transportation engineering is described as "that phase of transportation engineering which deals with planning, geometric design, and traffic operation of roads, streets, and highways, their networks, terminals, and relationships with other modes of transportation.

هندسة المرور : وهي فرع من فروع هندسة النقل تبحث في تخطيط الطرق و التصميم الهندسي لها وكذلك التشغيل المروري للطرق وشبكاتها وتبحث ايضاً في علاقة هندسة المرور مع باقي اشكال النقل .

Traffic Stream Components and their characteristic are based on Roadway network (Grid or ring and radial) and road ways(highways and street) .

تعتمد عناصر المجرى المروري على شبكة الطريق (شبكية او حلقية) ونوعية الطريق (شارع او طريق سريع)

<u>Functional Classification of Highways:</u> التصنيف الوظيفي للطرق

1- Principle Arterial الطرق الرئيسية

- Free way , express way ,other
- High design speed >120 km/hr سرعة تصميمية اكبر او تساوي 120 كم/ ساعة
- Long distance مسافات طويلة
- Full control of access سيطرة تامة على المداخل
- Design level of service B مستوى اداء الخدمة ب

2- Minor Arterial الطرق الرئسية الثانوية

- Moderate design speed ~ 100 km/hr سرعة تصميمة متوسطة تقريبا 100 كم / ساعة
- Design level of service = B-C مستوى اداء الخدمة يتراوح بين المستوى ب و ج

3- Collector طرق تجميعية

- Design speed ~ 80km/hr السرعة التصميمة مقاربة ل 80 كم/ ساعة
- Level of service C –D مستوى اداء الخدمة يتراوح بين المستوى ج و د

4- Local محلي (طرق وشوارع داخل المناطق السكنية)

- Design speed ~ 20-40 km/hr السرعة التصميمة تتراوح بين 20 الى 40 كم/ ساعة
- Local street – urban area (area with population) > 5000 capita شوارع محلية (مناطق حضرية) عدد سكانها اكبر من 5000 نسمة
- Local road - rural area(area with population) < 5000 capita طرق محلية (مناطق ريفية) عدد سكانها لا يتجاوز 5000 نسمة

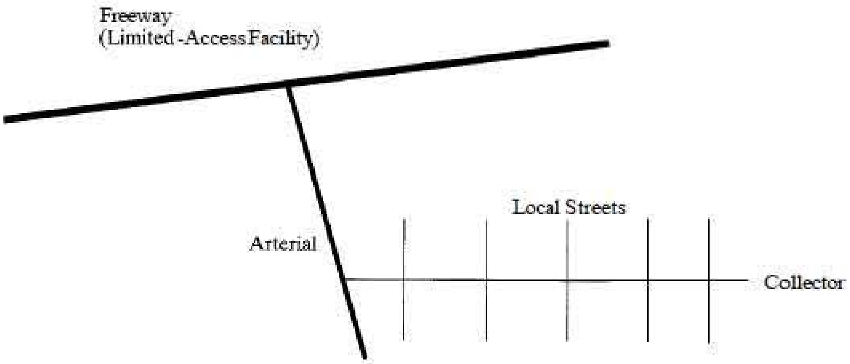
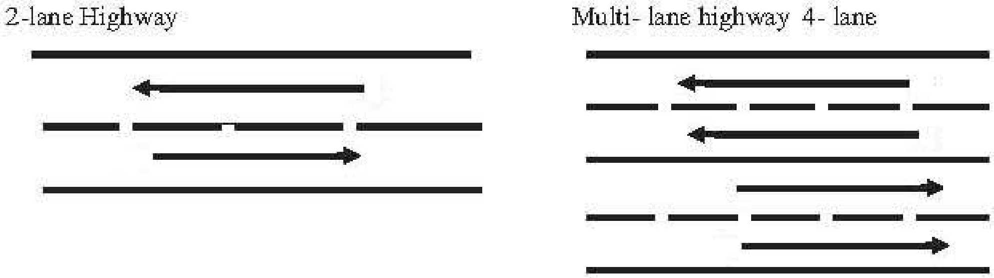
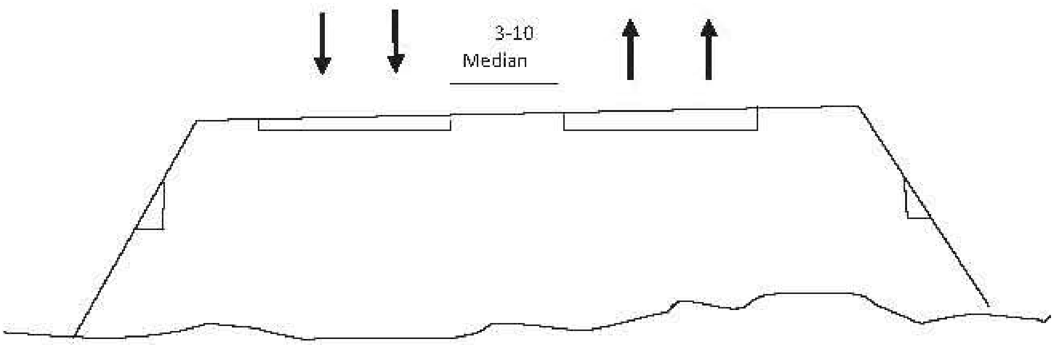


Illustration of highway classification

Highways Classification according to Geometrical Design Feature:





Rural Highway Section

Lecture 2

Road User Characteristics

The four main components of highway mode of transportation are the **driver**, the **pedestrian** , the vehicle , and the road . The bicycle is also becoming an important components in the design of urban highways and street .In order to provide efficient and safe highway transportation, a knowledge of the characteristic and limitation of each of these components is essential. It is also important to be aware of the inter relationship that exists among these component to determine the effect, is any , that they are exists. Their characteristics are also of primary importance when traffic engineer measure such as traffic control are to be used on highway mode .

1- Drivers:

The human response Process

Actions taken by drivers on a road result from their evaluation of and reaction to information they obtain from certain stimuli that they see or hear. However, evaluation and reaction must be carried out within a very short time, as the information being received along the highways is continually changing. It has been suggested that most of the information received by a driver is visual, implying that the ability to see is of fundamental importance in the driving task. It is therefore important that highway and traffic engineers have some fundamental knowledge of visual perception as well as of hearing perception.

Visual reception

The principal characteristics of the eye are visual acuity, peripheral vision, color vision, glare vision and recovery, and depth perception.

Visual Acuity: Visual acuity is the ability to see fine details of an object. It can be represented by the visual angle (ϴ), which is the reciprocal of the smallest pattern detail in minutes of arc that can be resolved and given as:

$$\varphi = 2\arctan\left(\frac{L}{2D}\right)$$

where

L = diameter of the target (letter or symbol)
D = distance from the eye to target in the same units as L

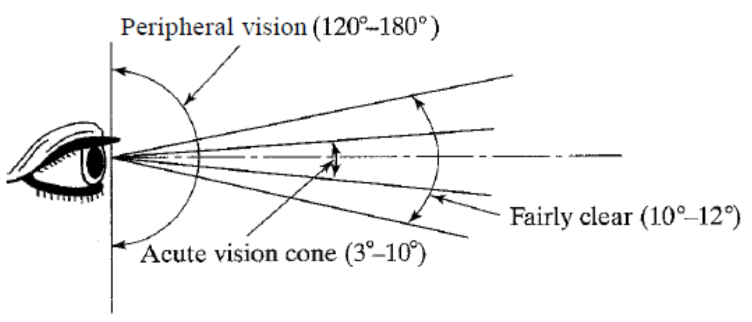


Illustration of Fields of Vision

- Acuity is best within 3^o cone around the line of vision.
- Signs should be place in a 10^o cone.
-Movable object may be detect within ($120\text{-}180^o$) cone.

For example : the ability to resolve a pattern detail with visual acuity of one minute of Arc (1/60) from degree , is considered the normal vision acuity (20/20) (USA System) (6/6) (Iraqi System).

6/6 = 1 can read 8.8 mm letter from 6 m
6/12 = 0.5 can read 17.6 mm letter from 6 m.

Find the letter height ?

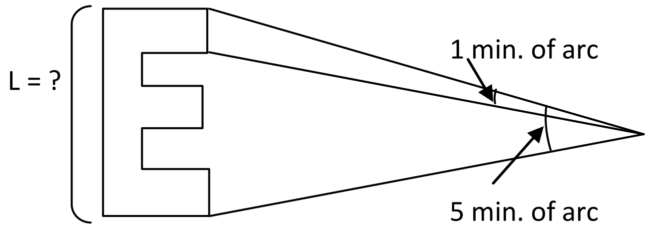
$\Theta = 2\text{Arc tan } (L/2D)$

$(5/60) = 2 \text{ Arc tan } (L \text{ mm} / 2\times6\times1000)$

L= 8.8 mm

$(5/60) = 2 \text{ Arc tan } (L \text{ mm} / 2\times60\times1000)$

L= 87.5mm



There are two type of visual acuity :

1- static visual acuity : both the object and driver are stationary .

2- dynamic visual acuity : the driver ability to detect relatively moving object.

Peripheral Vision:
Peripheral vision is the ability of people to see objects beyond the cone of clearest vision. Although objects can be seen within this zone, details and color are not clear. The cone for peripheral vision could be one subtending up to 160 degrees; this value is affected by the speed of the vehicle.

Color Vision:
Color vision is the ability to differentiate one color from another, but deficiency in this ability, usually referred to as color *blindness*. It can be compensated by the use of different traffic sign shapes. White and black or black eye is most sensitive.

Glare Vision:
The time required by a person to recover from the effects of glare after passing the light source is known as *glare recovery*. This time is about (3 sec.) . moving from dark to light and it can be (6 sec.) or more when moving from light to dark .

Depth perception:
Ability of driver to estimate speed and distance , it is particularly important on two – lane highway during passing movement .

Hearing perception :

Loss of some hearing ability is not a serious problem for driver , since its normally can be corrected by hearing ads.

Perception – reaction time:

Divided into three stages

1- perception: drivers sees a control device , warning signs or object in the road.
2- Identification: identifying the object or control device and thus understand the stimulus.
3- Emotion: the driver decides what action to take in response to the stimuli, for example break, swerve, and pass or changing lane.
4- Reaction or volition: the driver actually executes the action decided.

The time that elapse from the start of perception to the end of reaction is the total time required for perception, identification, emotion and volition referred to as (PIEV or PIER) as perception – reaction time .

AASHTO recommend perception – reaction time :

Perception time = 0.5 sec (Rural)
= 0.5 sec (urban)
Reaction time = 1 sec.
t = perception – reaction time = 1.5 - 2.5 sec.
Urban Rural

2- Pedestrian:

Pedestrian walking speed range between 0.8-1.6 m/sec. average = 1.2 m/s.

Lecture 3

VEHICLE CHARACTERISTICS

Criteria for the geometric design of highways are partly based on the static, kinematic, and dynamic characteristics of vehicles. **Static characteristics** include the weight and size of the vehicle, while **kinematic characteristics** involve the motion of the vehicle without considering the forces that cause the motion. **Dynamic characteristics** involve the forces that cause the motion of the vehicle. Since nearly all highways carry both passenger-automobile and truck traffic, it is essential that design criteria take into account the characteristics of different types of vehicles.

Vehicle classification:

- Passenger car (P.C): vehicles that have four tires touching the pavement (pickup and Minibus).
- Heavy Vehicles (H.V): vehicles that have more than four tires touching the pavement.
- Bus
- Truck
- Tractor – trailer combination.

Static characteristics:

1- Dimension and turning radius (Geometric design of highway).

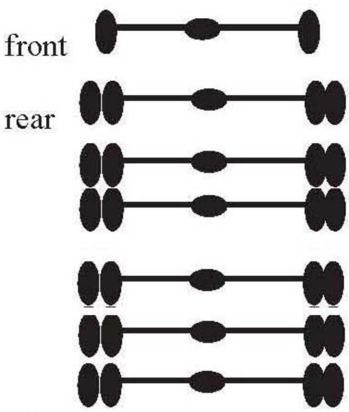
2- Axle loading (structural pavement design).

1- Dimension and turning radius .

	Width(m)	Length (m)	Height (m)	Turning radius (m)
Passenger Car	2.1	5.8	1.3	7.3
Truck	2.6	12	4.1	12.8
Tractor – trailer	2.6	20	4.1	13.7

*These dimensions are maximum.

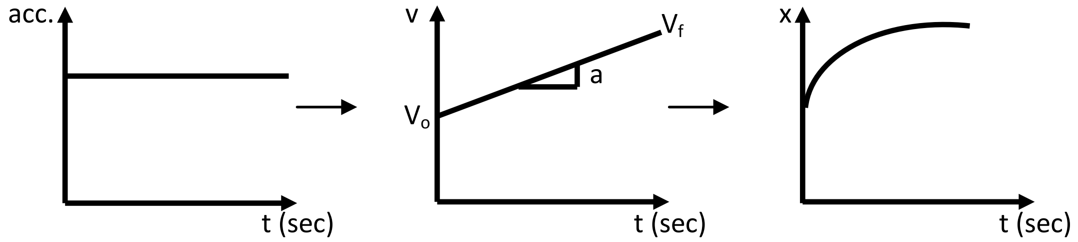
2- Axle loading



Axle type	Max load (ton)
1- single axle	7
	13
2- Dual Axle	20
3- Triple (tridem)	27

Kinematic characteristics:

1- Uniform acceleration : (acceleration , velocity , distance and time equation)



Acceleration assumed constant

$a = \frac{dv}{dt} \longrightarrow \int_0^t a\,dt = \int_{vo}^{vf} dv \longrightarrow at = V_f - V_o \longrightarrow \boxed{V_f = V_o + at}$ ………… 1

Velocity –Time relation

$\frac{dx}{dt} = V_o + at \longrightarrow \int_0^x dx = \int_0^t Vo + at\,.\,dt \longrightarrow \boxed{X = V_o t + \frac{1}{2}at^2}$ ……………… 2

$a = \frac{dv}{dt}\frac{dx}{dx} \longrightarrow = V\frac{dv}{dx} \longrightarrow \int_0^x a\,dx = \int_{vo}^{vf} v\,dv \longrightarrow ax\Big]_0^x = \frac{1}{2}V^2\Big]_{Vo}^{Vf}$

$ax = \frac{1}{2}V_f^2 - V_o^2 \longrightarrow \boxed{X = \frac{V_f^2 - V_o^2}{2a}}$ …………… 3

Example / A motorist is traveling at 80 km/hr when he observed that the traffic light 260m ahead of him turns red , the traffic light stay red for (15 sec) if the motorist wishes to pass the light without stopping just as its turns green again , find :
1- The required uniform deceleration of car 2- the speed of the car as its pass the light?

Sol./ V= 80km/hr
80km/hr ×0.278 = 22.2 m/sec.
1- $X = V_o t + \frac{1}{2}at^2$
$260 = 22.2 \times 15 + \frac{1}{2} \times a \times 15^2 \longrightarrow a = -0.65\ m/s^2$

2- $X = \frac{V_f^2 - V_o^2}{2a} \longrightarrow 260 = \frac{V_f^2 - 22.2^2}{2\times -0.65} \longrightarrow V_f = 12.5\ m/s$
$V_f = 12.5 * 0.278 = 45$ km/hr

Example/ A truck traveling (40km/hr) is approach a stop sign at time to distance (20 m). the track begins to slow down by deceleration of (4m /sec^2) . Will the truck be able to stop in time.

Sol./ $X = \frac{V_f^2 - V_o^2}{2a}$
V_o = 40km/hr * 0.278 = 11.12 m/sec

Sol./ $X = \frac{0 - 11.12^2}{2\times -4} = 15.42 < 20$ m
So, it's able to stop.

2- Non-Uniform acceleration : (acceleration as a function of velocity)

α : is the maximum acceleration rate

β : is the change in acceleration with respect to time

V_t: is the velocity at any time t

X : is the displacement at any time t

max. velocity when acceleration =0 , max. acceleration when V =0

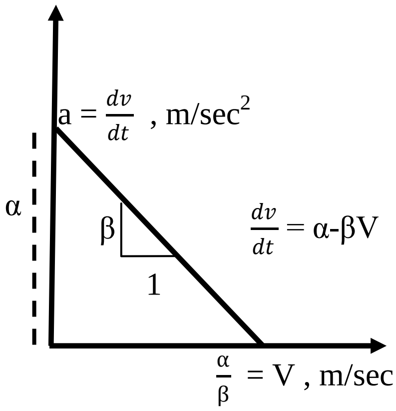
$\frac{dv}{dt} = \alpha - \beta\,(0) \qquad \frac{dv}{dt} = \alpha$

Units : $\alpha = \frac{m}{sec\,2}$, $\beta = \frac{1}{sec}$

$\frac{dv}{dt} = \alpha - \beta v \;\Rightarrow\; \frac{dv}{\alpha - \beta v} = dt$

$-\frac{1}{\beta}\int_{Vo}^{Vf} \frac{dv}{\alpha - \beta v} = \int_0^t dt$

$-\frac{1}{\beta} \ln\left(\alpha - \beta v\right)\Big]_{Vo}^{Vf} = t$



$-\beta t = \ln(\alpha - \beta V_f) - \ln(\alpha - \beta V_o) \;\Rightarrow\; -\beta t = \ln\left(\frac{\alpha - \beta Vf}{\alpha - \beta Vo}\right)$

$e^{-\beta t} = \frac{\alpha - \beta Vf}{\alpha - \beta Vo} \;\Rightarrow\; (\alpha - \beta V_o)\, e^{-\beta t} = \alpha - \beta V_f$

$(\alpha e^{-\beta t} - \beta V_o e^{-\beta t} = \alpha - \beta V_f) / -\beta$

$\frac{-\beta}{-\beta}\, Vf = \frac{\alpha}{\beta} - \frac{\alpha}{\beta}\, e^{-\beta t} - \frac{\beta}{-\beta}\, Vo\, e^{-\beta t} \;\Rightarrow\; Vf = \frac{\alpha}{\beta} - \frac{\alpha}{\beta}\, e^{-\beta t} + Vo\, e^{-\beta t}$

$$\boxed{Vf = \frac{\alpha}{\beta}\left(1 - e^{-\beta t}\right) + Vo\, e^{-\beta t}}$$ ………1 (v-t)

$\frac{dx}{dt} = \frac{-\alpha}{\beta}\, e^{-\beta t} + Vo\, e^{-\beta t} + \frac{\alpha}{\beta} \;\Rightarrow\; \int dx = \int \left(\frac{-\alpha}{\beta}\, e^{-\beta t} + Vo\, e^{-\beta t} + \frac{\alpha}{\beta}\right) dt$

$X = \frac{\alpha}{\beta^2}\, e^{-\beta t}\Big]_0^t - \frac{Vo}{\beta}\, e^{-\beta t}\Big]_0^t + \frac{\alpha}{\beta}\Big]_0^t$

$X = \frac{\alpha}{\beta^2}\, e^{-\beta t} - \frac{\alpha}{\beta^2} - \left(\frac{Vo}{\beta}\, e^{-\beta t} - \frac{Vo}{\beta}\right) + \frac{\alpha}{\beta}\, t$

$$\boxed{X = \frac{\alpha}{\beta} t - \frac{\alpha}{\beta^2}\left(1 - e^{-\beta t}\right) + \frac{Vo}{\beta}\left(1 - e^{-\beta t}\right)}$$ …2 (x-t) distance *x* traveled at any time *t*

$V_f = \frac{\alpha}{-\beta}\, e^{-\beta t} + Vo\, e^{-\beta t} + \frac{\alpha}{\beta} \;\Rightarrow\; \frac{dv}{dt} = \alpha\, e^{-\beta t} - Vo\beta\, e^{-\beta t} + 0$

$$\boxed{\frac{dv}{dt} = e^{-\beta t}(\alpha - Vo\beta)}$$ ………3 (a-t)

Example / A driver traveling at (30 km/hr) behind another car decided to pass it and press the acceleration to the floor . Assume the acceleration behavior of the vehicles may be described $\frac{dv}{dt}$ = 3 -0.1V , find ;

1- the rate at which the vehicles is accelerated after (3 sec) ?

2- the max acceleration and max speed

3- how long it would take the vehicles to reach speed (60 km/hr).

Sol./

$\frac{dv}{dt} = 3 - 0.1V \longrightarrow \frac{dv}{dt} = \alpha - \beta v$

$\alpha = 3$ and $\beta = 0.1$, $V_{o} =$ 30× 0.278 = 8.34 m/sec.

1- $\frac{dv}{dt} = e^{-\beta t}(\alpha - Vo\beta) \longrightarrow \frac{dv}{dt} = e^{-0.1*3}(3 - 8.34 \times 0.1)$ = 1.604 m/sec^2

2- max. acceleration at V= 0 , a = 3 m/sec^2

Max speed $\frac{\alpha}{\beta} = \frac{3}{0.1}$ = 30m/sec. $\longrightarrow$ 107.9 km/hr

$\frac{dv}{dt} = e^{-\beta t}(\alpha - Vo\beta) \longrightarrow 0 = e^{-\beta t}(\alpha - Vo\beta) \longrightarrow V = \frac{\alpha}{\beta}$

3- $Vf = \frac{\alpha}{\beta}(1 - e^{-\beta t}) + Vo\, e^{-\beta t}$

$60*0.278 = \frac{3}{0.1}\left(1 - e^{-0.1t}\right) + 30 * 0.278\, e^{-0.1t}$

t = 4.86 sec.

$X = \frac{\alpha}{\beta} t - \frac{\alpha}{\beta^2}\left(1 - e^{-\beta t}\right) + \frac{Vo}{\beta}\left(1 - e^{-\beta t}\right)$

$X = \frac{3}{0.1} 4.86 - \frac{\alpha}{0.1^2}\left(1 - e^{-0.1*4.86}\right) + \frac{30*0.278}{0.1}\left(1 - e^{-0.1*4.86}\right)$

X= 62.425.

Example / the acceleration of vehicle can be represented by the following equation $\frac{dv}{dt}$ = 3.3 -0.04V where V is the vehicle speed in m/s. if the vehicle travelling at 85 km/hr , determine its velocity after 5 sec of acceleration and the distance during time.

Sol/

Speed 85 km/hr ×0.278 = 23.611m/sec

$\alpha = 3.3$

$\beta = 0.04$

$$Vf = \frac{\alpha}{\beta}(1 - e^{-\beta t}) + Vo\, e^{-\beta t}$$

$$Vf = \frac{3.3}{0.04}\left(1 - e^{-(0.04\times5)}\right) + 23.611\, e^{-(0.04\times5)}$$

Vf = 14.85 +19.33 = 34.13 m/sec = 122 km/hr

To determine distance traveled

$X = \frac{\alpha}{\beta} t - \frac{\alpha}{\beta^2}\left(1 - e^{-\beta t}\right) + \frac{Vo}{\beta}\left(1 - e^{-\beta t}\right)$

$X = \frac{3.3}{0.04} 5 - \frac{3.3}{0.04^2}\left(1 - e^{-0.04\times5}\right) + \frac{23.611}{0.04}\left(1 - e^{-0.04\times5}\right)$

X = 412.5 - 371.25 +90 = 145.631 m

H.w / The impatient car stuck behind slow moving truck traveling at (30 km/hr) decided to overtake the truck . the acceleration characteristics of car is given by $\frac{dv}{dt}$ = 2- 0.07 V find :

1- what is the acceleration after (2 , 3 and 10 second)
2- what is the max speed attains b the car
3- how far will the car travel in 30 sec.
4- what is the max acceleration .

Lecture 4

Dynamic characteristics:

A vehicle motion tend to be retarded by at least five types of resistance

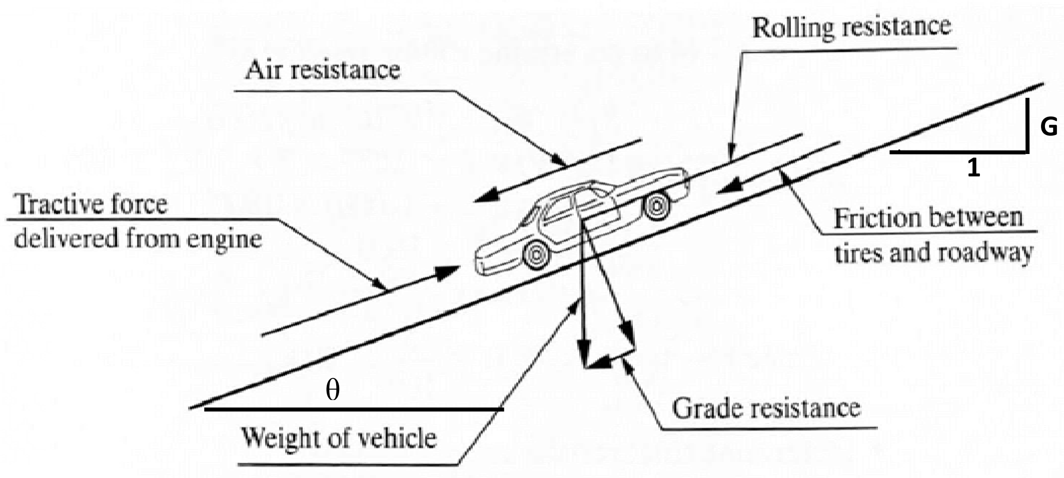
1- Inertia resistance (Fi)

the force required by a vehicle to overcome the tendency to remain at rest or remain in motion in straight line with a constant speed

Fi = m × a , m : vehicles mass (Kg) , a = vehicles acceleration (m/sec^2)

* (Fi) be (+) positive where there is acceleration (+ a)

*(Fi) be (-) negative where there is deceleration (- a)



Forces Acting on a Moving Vehicle

2- Grade resistance (Fg): the force required by a vehicle to overcome the component of gravitational force acting on frictionless inclined surface (WsinΘ)

W= m.g

Fg = Wsin θ

Fg = m .g . sin θ , For most highway applications θ is small, so

sin Θ = tan Θ = G

$\boxed{Fg = M.g. (\frac{G}{100})}$ where g = 9.81 m/sec^2 , G = grade (slope) in percent .

3- Rolling resistance (Fr) : the force required by a vehicle to overcome the friction effects of moving parts of vehicles as well as the friction between tire and pavement surface .

Fr = $(\frac{m.g}{100})\ (1+ \frac{V}{44.73})$, where V = vehicles speed m/sec = V km/hr ×0.278

4- Air resistance (Fa) : the force required by a vehicle to move air from a vehicles path way as well as to overcome the frictional effects of air with vehicle side top and under.

$\boxed{Fa = 0.5 \times Cd \times \rho \times A \times V^2}$,

where Cd = aerodynamic drag coefficient , Cd = P.C typical value = 0.4 , ρ = air density Kg/m^3 = 0.0382 Kg/m^3 , A = Frontal cross - section area m^2 .

Cd =0.363	Cd = 0.381	Cd = 0.5-0.8
P.C with close windows	P.C with open windows	Truck

5- curve resistance (Fc) ; the force required by a vehicle to overcome the tendency to motion in straight line and changing the direction to curved path.

$Fc = \frac{m.V^2}{Rc}$, where Rc = radius of curve (m) , V = speed of vehicle (m/sec)

- **Power requirement** ; the engine – generated power requirement to overcome the various resistance and propel a vehicle can be calculated as following ;

$P = \Sigma F \times V$

$R = \Sigma F = Fi + Fa + Fg + Fc + Fr$ where (in Watts)power or (hp) horse power

hp = 750 watts = 0.75 K watts

Example / determine the horse power product by passing car travelling at speed of 100 km/hr on a straight road of 5% grade with smooth pavement , assume the weight of the car 1800 kg and cross section area of vehicle is 3.6 m^2 .

Sol/

$\Sigma F = Fa + Fg + Fr$

$Fa = 0.5 \times Cd \times \rho \times A \times V^2 = 0.5 \times 0.4 \times 0.0382 \times 3.6 \times (100 \times 0.278)^2 = 21.25N$

$Fr = (M.g / 100)\,(1+\frac{V}{44.73}) = (1800 \times 9.81)/100 \times 1+\frac{100 \times 0.278}{44.73} = 286.3\ N$

$Fg = M.g.\,(\frac{G}{100}) = (1800 \times 9.81 \times \frac{5}{100}) = 882.4\ N$

$\Sigma F = 21.25+286.3+882.4 = 1190.45\ N$

$P = R \times V = 1190.45 \times 100 \times 0.278 = 33094.5$ Watts

33094/750 = 44.13 hp

HW1 / a 1134 Kg passenger vehicle with a frontal cross- sectional area of 2.78 m^2 . straight and level road at sea level . speed :89 km/hr. vehicles enters an 260 m radius horizontal curve, determine:

1- horsepower required in the curve to maintain the original speed.

2- total resistance force on the vehicle when traveling the curve .

HW 2/ estimate the power required to accelerate (1350 Kg) vehicle traveling (48 km/hr) up 5% grade at rate of 1.8 m/sec the vehicle has frontal cross section area of 1.9 m^2 . the roadway has a straight alignment and badly broken and patched asphalt surface ?

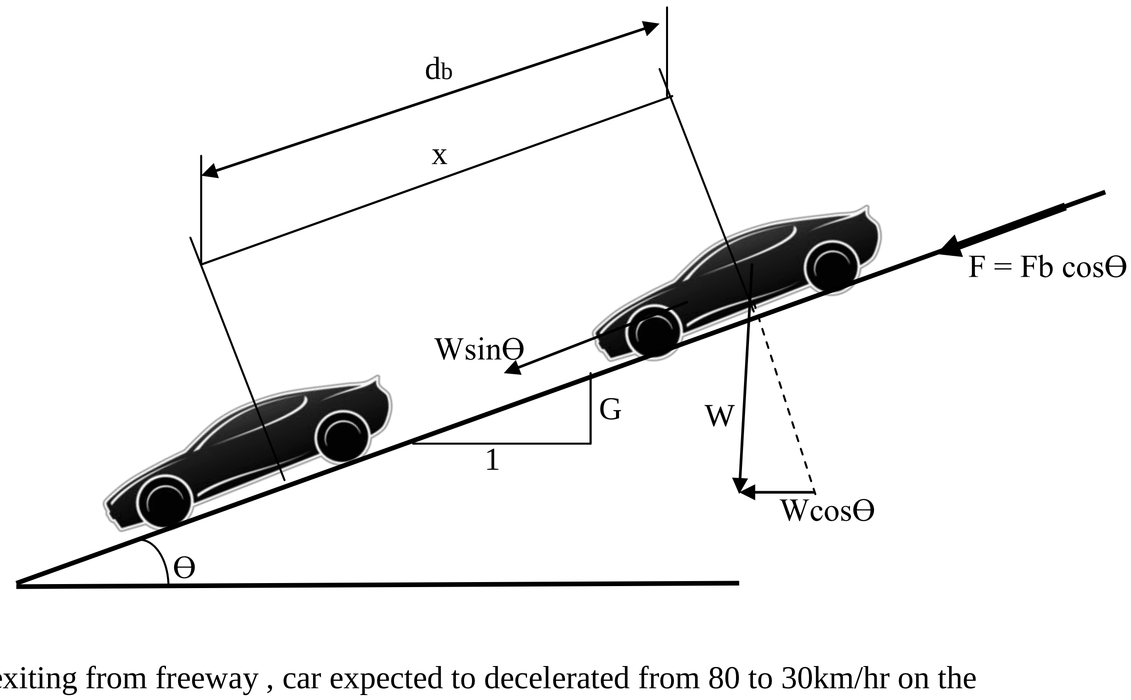
Breaking distance (d_b): distance needed to stop the vehicle after applying brake (skid distance)

$d_b = \frac{V^2}{254\ (Fb \pm G)}$, where V : initial speed (Km/hr) , design speed.

Fb : coefficient of frication due to breaking .

Fb =	0.45-0.62	dry pavement condition
	0.28-0.4	wet pavement condition
	0.1	muddy pavement condition
	0.05	icy pavement

G = grade in percent (%) , G + = upgrade , G - = downgrade



Ex/ in exiting from freeway , car expected to decelerated from 80 to 30km/hr on the exit ramp , assume Fb= 0.2 . compare between the braking distance required in 5% upgrade exit ramp from the depressed freeway with that required of at grade exit ramp .

Sol. /

$$d_b = \frac{Vi^2 - Vf^2}{254\ (Fb \pm G)} = \frac{80^2 - 30^2}{254\ (0.2 + 0.05)} = 84.6 \text{ m}$$

$$d_b = \frac{Vi^2 - Vf^2}{254\ (Fb \pm G)} = \frac{80^2 - 30^2}{254\ (0.2 + 0)} = 105.8 \text{ m}$$

Ex/driver of a car travelling at 70 kph required 40 m to stop after the brakes have been applied . what average coefficient of friction was developed between tires and pavement surface ? let G = 10%.

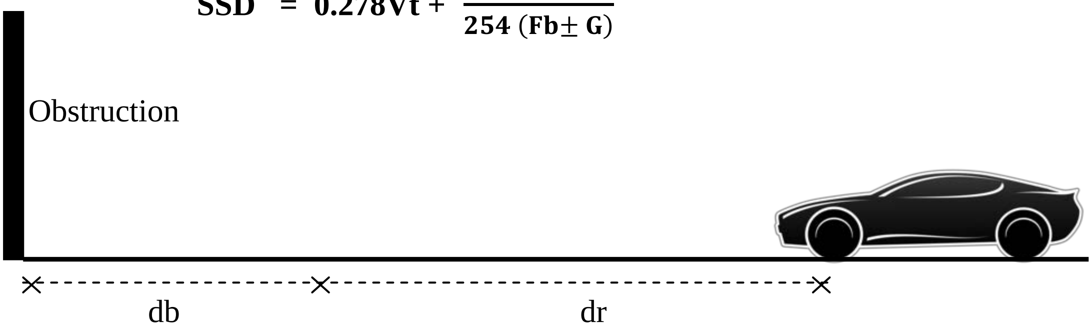
Sol/ $d_b = \frac{Vi^2 - Vf^2}{254\ (Fb \pm G)} = \frac{70^2 - 0^2}{254\ (Fb + 0.1)}$

$$d_b = 40 = \frac{70^2 - 0^2}{254\ (Fb + 0.1)} = Fb = 0.3823$$

Sight Distance : Sight distance is the length of the roadway visible ahead at any particular time. The sight distance available at each point of the highway must be such that, when a driver is traveling at the highways design speed, adequate time is given after an object is observed in the vehicles path to make the necessary evasive maneuvers without colliding with the object. The two types of sight distance are (1) stopping sight distance and (2) passing sight distance.

Stopping sight distance (SSD) : for design purposes, is usually taken as **the minimum sight distance required for a driver to stop a vehicle after observing an object in the vehicles path without ahead hitting that object**. This distance is the sum of the distance traveled during perception-reaction time and the distance traveled during braking. The SSD for a vehicle traveling at **V** km/h is given by

$$\mathbf{SSD} = \overbrace{\mathbf{0.278Vt}}^{\text{Lag distance}} + \overbrace{\frac{\mathbf{Vi^2 - Vf^2}}{\mathbf{254\ (Fb \pm G)}}}^{\text{Breaking distance}}$$



Ex/ if the design speed of multilane highway is 90kph, what is minimum stopping sight distance that should be provided on road if :

1- road has level grade

2- has a max grade -5% , assume t = 2.5 sec.

Sol/

1- SSD = dr+db = $0.278Vt + \frac{Vi^2 - Vf^2}{254\ (Fb \pm G)}$

$= 0.278 \times 90 \times 2.5 + \frac{90^2 - 0^2}{254\ (0.3 \pm 0)} = 170m$

2- SSD = $0.278 \times 90 \times 2.5 + \frac{90^2 - 0^2}{254\ (0.3 - 0.05)} = 190m$

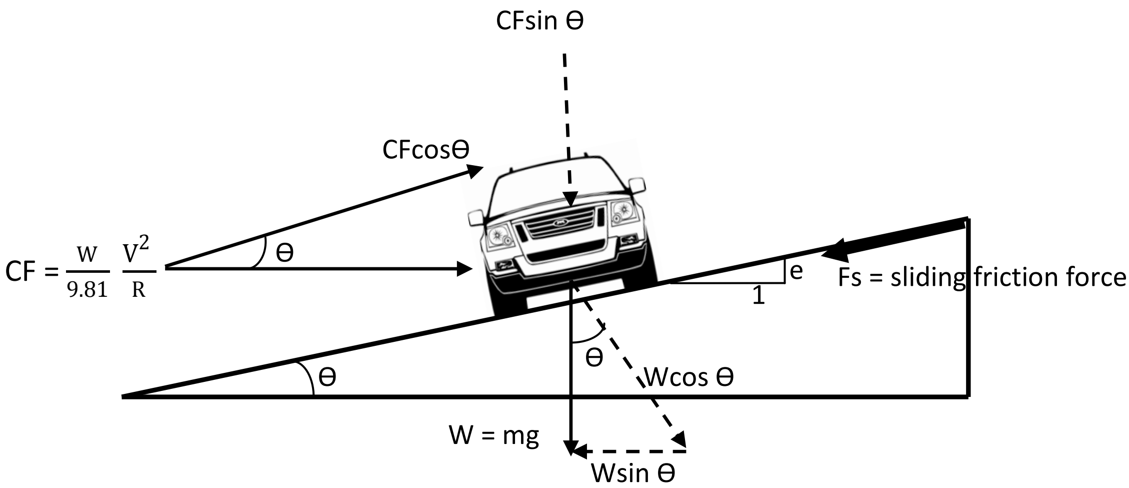
Turning Radius (R) :

When a vehicle is moving around a circular curve, there is outward radial force acting on the vehicle, usually referred to as the centrifugal force. Which may cause the vehicle to slide outward or overturning.

Centrifugal force CF = m×a = $\frac{W.a}{g} = \frac{W}{9.81}\ \frac{V^2}{R}$, where a = acceleration for curve motion

In order to balance the effect of the centripetal acceleration, the road is inclined toward the center of the curve. The inclination of the roadway toward the center of the curve is known as superelevation.

The minimum radius of a circular curve *R* for a vehicle traveling at V k/h can be determined by using with respect to overturning force and stabilizing force in curve , using the following equation :



$$Rc = \frac{V^2}{127\ (e+f)} ,$$

Where ; V vehicle speed (km/hr) , Rc radius of curve (m) , f (coefficient of sliding frication (0.11- 0.17) , e superelevation rate (the inclination of the roadway toward the center of the curve (0.04 – 0.12)

Ex./ Because of the location restrictions, a turning road way is limited to a radius of 150m . if the max rate of superelevation is 0.06 and the coefficient of side frication f is taken 0.14 what would be considered a safe speed ?

Sol/ $R = \frac{V^2}{127\ (e+f)} = 150 = \frac{V^2}{127\ (0.06+0.14)}$ = V = 62 m/sec

Ex./ determine the minimum radius required at a curve section of highway if the design speed 110km/hr and the superelevation is 0.08 ?

Sol./ assume f = 0.14

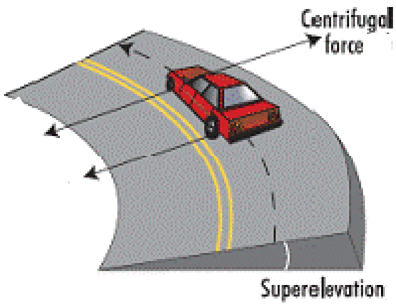
$R = \frac{110^2}{127\ (0.08+0.14)}$ = R = 433 m

Ex./ A horizontal curve is to be designed for a section of highway having a speed 95km/hr ,if the psychological condition restrict the radius to 310 m, what value required for superelevation will be a good for design ?

Sol/ $R = \frac{V^2}{127\ (e+f)}$

$310 = \frac{95^2}{127\ (e+0.14)}$

e = 0.09 = 9%



Lecture 5

Traffic Stream Parameters

Traffic stream parameters classified into two groups:
1- Macroscopic parameter: describe the traffic stream as a whole, includes the following:

- Volume
- Speed
- Density

2- Microscopic parameter: describe the behavior of individual or pair of vehicles, includes the following:

- Headway
- Spacing
- Speed of individual vehicle

Traffic Volume

Volume: The number of vehicles passing a given point or section of a lane or a roadway during a specified period of time. (day or hour).

Units: vpd (vehicle /day)
vph (vehicle /hour)

* Note: Daily volume is given for entire roadway (both direction), otherwise specified, for example , 5000 v/d in both directions .

- Daily Volume is used for :
1- Establishment of traffic volume trend.
2- Computation of accident rates in term of 100 million vehicles – kilometers.
3- Evaluation of the economical feasibility of highway project.
4- Development of maintenance program for highways.
5- Structural design of pavement
6- Planning of highway.

- Daily Volume Types:
1- Average Annual Daily Traffic (AADT) : The total yearly volume divided by the number of days in the year

$$\text{AADT} = \frac{\textit{Total Volume of one Year}}{\text{365 day}} = \textbf{Vpd}$$

2- Annual Average Weekday Traffic (AAWT) : total weekday volume for one year divided by the number of weekdays (usually 260)

$$\text{AAWT} = \frac{\textit{Total weekday volume for one year}}{260} = \textbf{Vpd}$$

3- Average Daily Traffic (ADT) : The total volume during a given time period (greater than one day and less than one year) divided by the number of days in that time period.

$$\text{ADT} = \frac{\textit{Total Volume of a given period}}{\text{time period (days)}}$$

4- Average Weekday Traffic (AWT) : the total weekday volume for a given period divided by the number of weekdays in that time period.

$$\text{AWT} = \frac{\textit{Total weekday volume of a given period}}{\text{no.of weekday on the time period}}$$

Ex/ Find the AADT and AAWT from the following data

1 Month	2 No. of weekdays in month (days)	3 Total Days in Month (days)	4 Total Monthly Volume (veh)	5 Total Weekday Volume (veh)	6 AWT (5/2) (veh/day)	7 ADT (4/3) (veh/day)
Jan	22	31	425000	208,000	9455	13710
Feb	20	28	410000	220,000	11000	14643
Mar	22	31	385000	185000	8409	12119
Apr	22	30	400000	200000	9091	12333
May	21	31	450000	215000	10238	14516
Jun	22	30	500000	230000	10455	16667
Jul	23	31	580000	260000	11304	18710
Aug	21	31	570000	260000	12381	18387
Sep	22	30	490000	205000	9318	16333
Oct	22	31	420000	190000	8636	13548
Nov	21	30	415000	200000	9524	13833
Dec	22	31	400000	210000	9545	12903
Total	**260**	**365**	**5445000**	**2583000**	**--**	**--**

AADT = 5445000/365 = 14918 veh/day
AAWT = 2583000/260 = 9935 veh/day

Traffic Projection Factor (T.P.F) :

Future volume = current volume × T.P.F

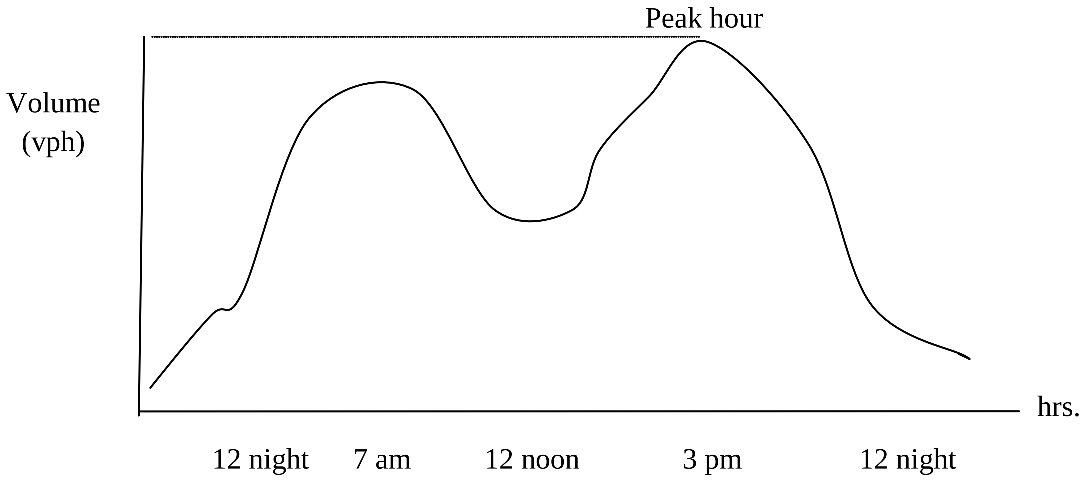
T.P.F = $(1+ r)^{x+n}$

Where ;

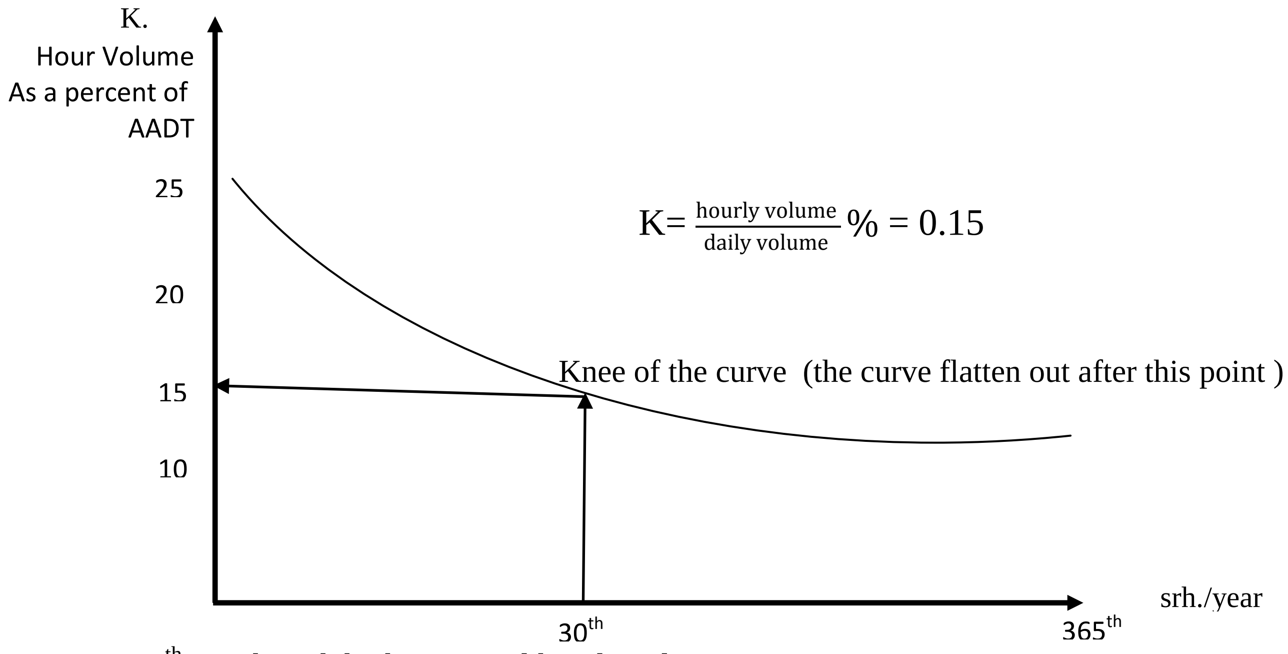
r = annual rate of traffic increase (0-10%)
x = construction period (2-4 years)
n = design life (20- 50years)

for example / T.P.F = $(1+0.05)^{2+20} = 2.9$
current volume = 24000 vpd in 2016
future volume = 24000×2.9 = 69600 vpd in 2038

Hourly Volume : HV (veh/hr.) : the single hour of the day that has the highest hourly volume is referred to as peak hour .the traffic volume within this hour is of great interest to traffic engineer for design and operational analysis for highway section.



DHV : <u>D</u>esign <u>H</u>ourly <u>V</u>olume : the 30th highest hourly volume that is exceeded by 29 hourly volumes during the design year. This volume is usually taken as percentage of expected ADT on the highway .



30th HV thirteth highest annual hourly volu me

Directional design-hour volume (DDHV) : The DDHV is the amount of traffic moving in the peak direction during the design hour. The formula for calculating the directional design hour volume is:

DDHV = K ×D× ADT

K = proportion of daily traffic occurs during the peak hour

$$= \frac{\text{Hourly voulme}}{\text{Daily Volume}} = \frac{\text{DHV}}{\text{AADT}}$$

D = Directional distribution factor reflecting the proportion of peak-hour traffic traveling in the peak direction.

$$= \frac{\text{Volume of one direction}}{\text{Volume of two direction}} \times 100$$ ----------(50-80)%

Type of facility	K factor	D factor
Rural highway	0.15-0.25	0.65-0.8
Suburban highway	0.12-0.15	0.55-0.65
Urban highways	0.07-0.12	0.5-0.6

EX/ rural highway has (20years) forecast of AADT of (30000 vpd) what range of directional design hour volume might be expected for this situation ?

Sol/ DDHV = AADT ×K ×D
DDHV low = 30000×0.15×0.65 = 2925 vph/dir.
DDHV high = 30000×0.25×0.8 = 6000 vph/dir.

Flow Rate and Peak Hourly Factor

Peak Flow Rate : volume observed for a period less than one hour (15 min. or 5 min) expressed as equivalent hourly rate of flow (q)

Example /

Time interval	Road A	Road B
8:00 – 8:15	1000	4000
8:15 – 8:30	1000	0
8:30 – 8:45	1000	0
8:45 – 9:00	100	0
	4000	4000

Peak flow rate = 4 ×Max V_{15} min

PHF = $\frac{\text{Volume}}{4\times\text{Max V15 (min.)}}$

0.25≥ PHF ≥ 1

Normally, between 0.7- 0.98 , lower PHF indicates a greater degree of variation in flow during the peak hour

Peak flow rate = $\frac{\text{Volume (DDHV)}}{\text{PHF}}$

Time interval	Volume (vehicles)
5:00-5:15 pm	950
5:15-5:30 pm	1150
5:30-5:45 pm	1250
5:45-6:00 pm	1000
For 5:00-6:00 pm	4350 veh/h

Peak flow rate = 4*1250 =5000

PHF = $\frac{4350}{4\times1250}$ = 0.87

Passenger Car Equivalency Factor (E_{pc})

To convert different distribution of vehicle, (truck , bus) to one standard type (passenger car p.c)

Terrain	Flat	Rolling	Mountain
E_{pc}	1.5	2.5	4.5

Example/

volume = 3000 vph , tuck = 10% , mountain terrain .

Sol/

Volume (pc ph) = volume (vph) [$P_{pc}\times1 + P_{HV}\times E_{pc}$)

Volume (p.c) = 3000 [0.9×1 +0.1×4.5] = 4050 pc ph.

Summary

ADT

↓ × TPF = traffic projection factor = $(1+r)^{x+n}$

Future ADT

↓ × K (0.15 rural - 0.12 urban)

DHV

↓ ×D (directional distribution factor 0.5-0.8)

DDHV

↓ / (design service flow rate for one lane)

No of lanes

Capacity: it represents the maximum number of vehicles that can pass a given point or section during a period of time under ideal condition, commonly expressed as (pcphpl).

$$\text{No. of lanes/ one dir.} = \frac{\text{Volume of one direction}}{\text{design service flow rate for one lane}}$$

Example / A multilane minor arterial highway is being designed through a rolling rural area. The current daily volume is 7100 vpd with 20% truck . 90% peak hour factor and 60% directional distribution factor . how many lanes are required for this highway if this highway located in an urban area with level terrain.

Sol/

Minor arterial rolling area = design level of service is B

So design service flow for one lane = 1080 pc ph pl

Assume T.P.F = 3.6

Future volume = current volume ×T.P.F

= 7100 × 3.6 = 25560 vpd

DHV(30^{th}) = 0.15× Future ADT = 0.15 × 25560 = 3834 vph

$$\text{Peak flow rate} = \frac{\text{Volume}}{\text{PHF}} = \frac{3834}{0.9} = 4260 \text{ vph}$$

Volume (pc ph) = 4260 [0.2 ×2.5 +0.8 ×1] = 5538 pc ph

Volume of one direction = 5538 × 0.6 = 3323 pc ph /dir

No of lanes in one dir. = 3323/1080 = 4 lanes/ dir .

No of lanes in both dir = 4×2 = 8 lanes / two dir

Lecture 6

Volume Studies

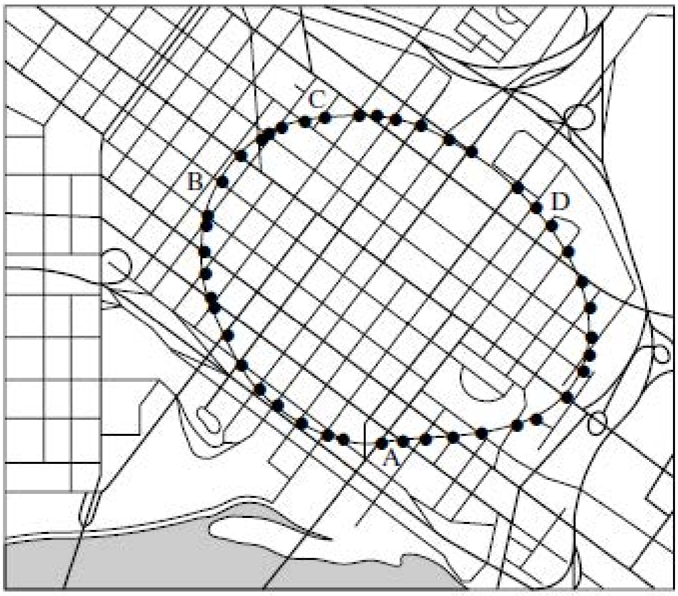
1- Methods of Conducting Volume Counts:

- **Manual Method :** involves one or more persons recording observed vehicles using a stopwatch.

- **Automatic Method :** using surface detectors such as pneumatic road tube or sub-surface detectors such as magnetic or electrical control device .Also, digital camera or lesser scanner can be used for this purpose .

2- Type of Volume Counts :

- **Cordon Count:** this type of count is used when information is required on vehicles accumulation within area such central business district (CBD) .The area is enclosed with imaginary closed loop ABCDA, volume counts for vehicle leaving or entering the cordon area are recorded in the intersection point of each road with the cordon line.



Example of Station Locations for a Cordon Count

- **Screen Line Counts:** this type of count is used when information is required about the change in traffic flow from one area to another. The study area is divided into large section by using imaginary lines knowing as screen line and traffic counts were then taken at each point where a road crosses the screen line .Also, natural and manmade barrier such as river and railway can be used as a screen line .

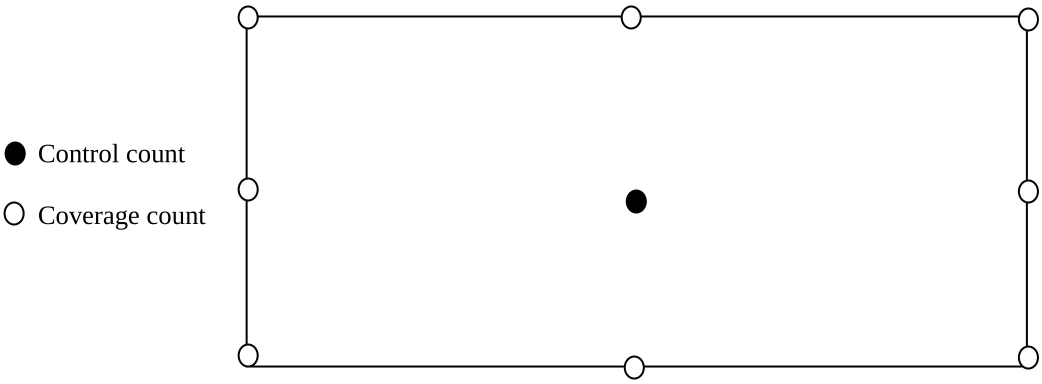
- **Intersection Counts :** This type of count is used to obtain data about the volume of each movement in the intersection approaches, also to obtain data about vehicles classification .

  Note* / All the above counts are conducted for certain period (study period)

- **Periodic Volume Counts:** to obtain a certain traffic volume data such as (AADT) . it is necessary to obtain data continuously. However, it is not feasible to collect continuous data on all road because of the cost involved, therefore periodic volume counts are divided into two types :

  1- Control Count: in this control count station, volumes are recorded continuously (24 hours of 365 days) to determine volume adjustment factor that can be used to determine (AADT).

  2- **Coverage count**: in this coverage count, volumes are counted for period normally ranged (24-48 hrs.) with the aids of volume adjustment factor obtained from control counts, these short time volume can be converted to AADT.



Adjustment of periodic count: using data from continuous counting station (control station) expansion factor can be calculated and used to determine (AADT) for the similar road class from short period count (coverage count station).

- **Hourly Expansion Factor (HEF)** = $\frac{\textbf{Total Volume for 24 hour period}}{\textbf{Volume of particular hour}}$

- **Daily Expansion Factor (DEF)** = $\frac{\textbf{Total Volume for week}}{\textbf{Volume for particular day}}$

- **<u>M</u>onthly <u>E</u>xpansion <u>F</u>actor (MEF)** = $\dfrac{\textbf{AADT}}{\textbf{ADT for particular month}}$

Example/ A traffic engineer urgently needs to determine the AADT on a rural primary road that has the volume distribution characteristics shown in table 1 ,2 and 3 . he collected the data shown below on a Tuesday during the month of May. Determine the AADT of the road ?

7--8 am	400	volume
8--9 am	535	
9--10 am	650	
10--11am	710	
11-12 am	650	

Table 1: Hourly Expansion Factors

Hour	Volume	HEF
6-7 am	294	42
7-8 am	426	29
8-9 am	560	22.05
9-10 am	657	18.8
10-11 am	722	17.1
11-12 am	667	18.52
12-1 pm	660	18.71
1-2 pm	739	16.71
2-3 pm	832	14.84
3-4 pm	836	14.77
4-5 pm	961	12.85
5-6 pm	892	13.85
6-7 pm	743	16.62
7-8 pm	706	17.49
8-9 pm	606	20.38
9-10 pm	489	25.26
10-11 pm	396	31.19
11-12 pm	360	34.31
12-1 am	241	51.24
1-2 am	150	82.33
2-3 am	100	123.52
3-4 am	90	137.2
4-5 am	86	143.6
5-6 am	137	90.1
Total Daily Volume	12350	

Table 2 Daily Expansion Factor for a rural primary road

Day of week	Volume	DEF
Sunday	7895	9.515
Monday	10714	7.012
Tuesday	9722	7.727
Wednesday	11413	6.582
Thursday	10714	7.012
Friday	13125	5.724
Saturday	11539	6.510
Total Weekly Volume	75122	

Table 3 Monthly Expansion Factor for a rural primary road

Month	ADT	MEF
January	1350	1.756
February	1200	1.975
March	1450	1.635
April	1600	1.481
May	1700	1.394
June	2500	0.948
July	4100	0.578
August	4550	0.521
September	3750	0.632
October	2500	0.948
November	2000	1.185
December	1750	1.354
Total yearly volume	28450	
Mean Average Volume	28450/12	= 2370

Solution/

Estimate the 24 hr volume for Tuesday using the factor given in table 1

Total daily volume of month (May and July and Tuesday) =

$$\frac{(\mathbf{400} \times \mathbf{29}) + (\mathbf{535} \times \mathbf{22.05}) + (\mathbf{650} \times \mathbf{18.8}) + (\mathbf{710} \times \mathbf{17.1}) + (\mathbf{650} \times \mathbf{18.52})}{\mathbf{5}}$$

= 11959 vpd

* adjust the 24 hr volume for Tuesday to an average volume for week using the factor given in table 2

$$= \frac{\mathbf{11959} \times \mathbf{7.727}}{\mathbf{7}} = 13201$$

*Since the data were collected in May use factor shown for May in table 3 to obtain AADT

$AADT = ADT_{May} \times MEF_{May}$

$13201 \times 1.394 = 18402$ vpd

Lecture 7

Speed and Travel Time

Speed : is the rate of movement of vehicle in distance per unit time, usually expressed in (km/hr) or (m/sec).

Spot Speed : is the instantaneous speed of a vehicle at any specified point .

there are two types of vehicles average spot speed :

1- Time Mean Speed (TMS) : average speed of all vehicles passing a point over same specified time interval.

$$\text{TMS} = \frac{\sum \text{Vi}}{\text{n}} = \frac{\sum \frac{d}{ti}}{n}$$

where :

V= speed of ith vehicles.

d = distance travelled or length of highway segment

ti = travel time of the (i^{th} vehicles)

2- Space Mean Speed(SMS): Average speed of all vehicle occupying a given section at an instance.

$$\text{SMS} = \frac{d}{\sum \frac{ti}{n}} = \frac{n \times d}{\sum ti}$$

note / TMS is always more than SMS

Example / The travel time shown in the table below are measured for vehicles as they traversed (3.2 km) segment of highway . Compute, TMS,SMS and show why is lower than TMS ?

No. of vehicles	Traveled time (min)	Vi (km/hr)
1	2.6	73.8
2	2.4	80
3	2.4	80
4	2.8	68.6
5	2.2	87.3
6	2.1	91.4
	∑= 14.5	∑= 481.1

$$\text{TMS} = \frac{\sum \text{Vi}}{\text{n}} = \frac{481.1}{6} = 80.2 \text{ km/hr}$$

$$\text{SMS} = \frac{n \times d}{\sum ti} = \frac{6 \times 3.2}{\frac{14.5}{60}} = 79.3 \text{ km/hr}$$

SMS is lower than TMS because it give more weight for slower moving vehicles because they occupy the section for longer time .

Types of Speed :

1- Travel Speed (journey Speed) :

$$\textbf{Travel Speed} = \frac{\textbf{Distance (Km)}}{\textbf{Overall travel time (hr)}}$$

2- Running Speed:

$$\textbf{Running Speed} = \frac{\textbf{Distance (Km)}}{\textbf{Running time (hr)}}$$

***** Running time = Travel time - Stopping time

3- Design Speed : It is a selected speed used to determine the various geometric design features of highways (20-130 km/hr).

for example :

- Principle Arterial = ≥ 120km/hr
- Minor Arterial = 80 km/hr
- Collector = 60 km/hr
- local street = 20-40 km/hr

4- Operating Speed : Max safe speed of vehicle in traffic stream "without exceeding the design speed " usually 85% percentile speed = operating speed.

5- **Percentile Speed :** Different speed like (mean speed , median speed , mode speed ,operating speed , lowest speed)

98% is used for determining the design speed
85% is used for determining the operating speed
50% is used for determining the median speed
15% shows the slower vehicles where speed may be causing interference with the traffic stream .

Ex/ For the given spot speed data which was taken on the ring road at Al-Dora district , conduct the following:

1- Plot the frequency and the cumulative frequency distribution of the speed data.

2- Calculate the:

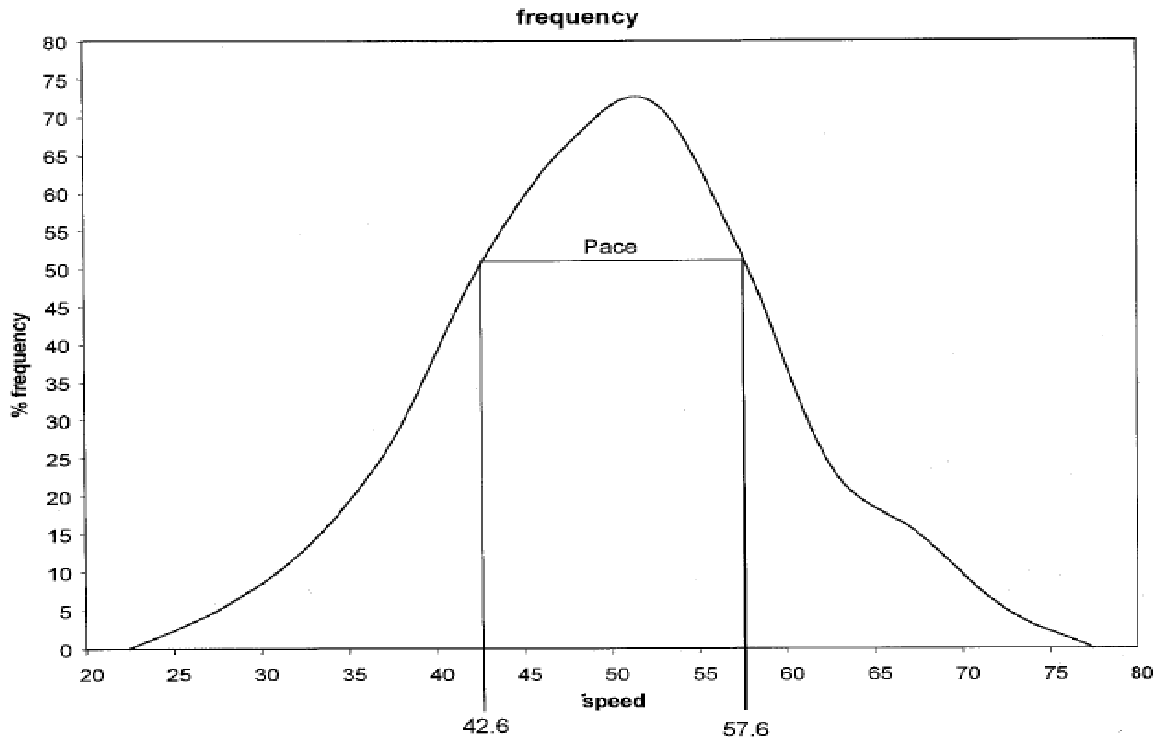
- average speed
- standard deviation
- median and mode speeds

- the 85th and 15th percentile speed
- the pace

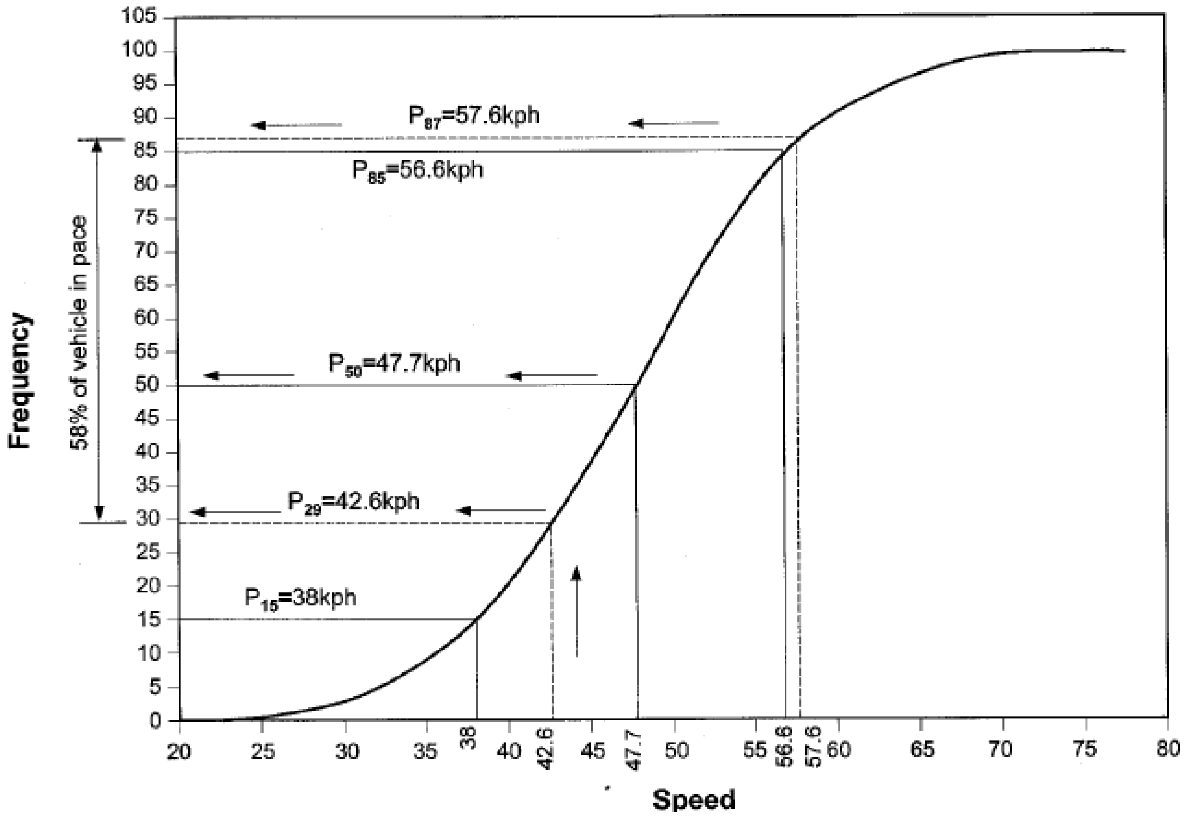
Sol./

The speed data are grouped into 5 kph interval, resulting into 12 speed groups (usually 10-25 groups is adequate) as shown below :

Speed Group	Mean Speed of Group	Number of Vehicle in group (f)	Percent total observation in group	Cumulative percent of total observation	$V\times f$	$(V-\bar{X})^2$	$f(V-\bar{X})^2$
20-24.9	22.45	0	0.0	0.0		760.877	0
25-29.5	27.45	5	1.5	1.5	137	510.037	2550.185
30-34.9	32.45	13	4.0	5.5	421.85	309.197	4019.562
35-39.9	37.45	27	8.2	13.7	1011.15	158.357	4275.641
40-44.9	42.45	50	15.2	28.9	2122.5	57.517	2875.853
45-49.9	47.45	66	20.1	48.9	3131.7	6.677	440.6857
50-54.9	52.45	72	21.9	70.8	3776.4	5.837	420.268
55-59.9	57.45	52	15.8	86.6	2987.4	54.997	2859.847
60-69.9	62.45	24	7.3	93.9	1498.8	154.157	3699.769
65-69.9	67.45	15	4.6	98.5	1011.75	303.317	4549.756
70-74-.9	72.45	5	1.5	100	362.25	502.477	2512.385
75-79.9	77.45	0	0	100	0	751.637	0
	$\sum$	329	100		16461.05	3575.084	28203.95



Frequency distribution curve for spot speed data



Cumulative distribution curve for spot speed data

- Average : $X = \frac{\sum fv}{n} = \frac{16461.05}{n}$ = 50.034

- Standard deviation : S = $\frac{\sqrt{\sum f\,(v-X)2}}{n-1} = \frac{\sqrt{28203.95}}{328}$

  S = 9.273 km/hr

- P50 = 47.7

- Mode ≈ 50

- P85 =56.6 kph , P15 = 38 kph
  pace (15 kph) = 42.6 kph - 57 .6 kph and it contains 58% of observation

Lecture 8

Density

- **Density** (concentration) : no of vehicles occupying 1km (1 mile) length of a road at certain instant.

units : veh/km

vpkm

* **Spacing** (m) = $\frac{1000}{\textbf{Density}}$

ex/ spacing= 10m $\longrightarrow$ **Density** = $\frac{1000}{10}$ = 100 vpkm

- **Level of Service (LOS)** :qualitative measurement describing the operational condition within the traffic stream.

LOS	Description	Operating speed (km/hr)
A	Free flow	96
B	Stable flow	88
C	Stable flow with some restriction	72
D	Approaching unstable flow	56
E	Unstable flow	48
F	Forced flow (stop and go)	<48

- **Design service flow rate :** Maximum hourly flow rate at which vehicle can be expected to pass a point or section of a lane or roadway during one hour without falling below a pre- selected level of service.

LOS	Design service flow rate (pc ph pl)
A	660
B	1080
C	1550
D	1980
E	2200

* **Capacity** = Design service flow rate for LOS E

- **Design level of service**

Highway	Flat	Rolling	Mountain
Principle Arterial	B	B	C
Minor Arterial	B	B	C
Collector	C	C	D
Local	D	D	D

LOS

Although speed is a major concern of drivers as related to service quality, freedom to maneuver within the traffic stream and proximity to other vehicles are equally noticeable concerns. These qualities are related to the density of the traffic stream. Unlike speed, density increases as flow increases up to capacity, resulting in a measure of effectiveness that is sensitive to a broad range of flows.

Operating characteristics for the six LOS are shown in Illustrations 13-5 through 13-10. The LOS are defined to represent reasonable ranges in the three critical flow variables: speed, density, and flow rate.

LOS A describes free-flow operations. Free-flow speeds prevail. Vehicles are almost completely unimpeded in their ability to maneuver within the traffic stream. The effects of incidents or point breakdowns are easily absorbed at this level.

LOS B represents reasonably free flow, and free-flow speeds are maintained. The ability to maneuver within the traffic stream is only slightly restricted, and the general level of physical and psychological comfort provided to drivers is still high. The effects of minor incidents and point breakdowns are still easily absorbed.

ILLUSTRATION 13-5. LOS A.

ILLUSTRATION 13-6. LOS B.

ILLUSTRATION 13-7. LOS C.

ILLUSTRATION 13-8. LOS D.

ILLUSTRATION 13-9. LOS E.

ILLUSTRATION 13-10. LOS F.

LOS C provides for flow with speeds at or near the FFS of the freeway. Freedom to maneuver within the traffic stream is noticeably restricted, and lane changes require more care and vigilance on the part of the driver. Minor incidents may still be absorbed, but the local deterioration in service will be substantial. Queues may be expected to form behind any significant blockage.

LOS D is the level at which speeds begin to decline slightly with increasing flows and density begins to increase somewhat more quickly. Freedom to maneuver within the traffic stream is more noticeably limited, and the driver experiences reduced physical and psychological comfort levels. Even minor incidents can be expected to create queuing, because the traffic stream has little space to absorb disruptions.

At its highest density value, LOS E describes operation at capacity. Operations at this level are volatile, because there are virtually no usable gaps in the traffic stream. Vehicles are closely spaced, leaving little room to maneuver within the traffic stream at speeds that still exceed 49 mi/h. Any disruption of the traffic stream, such as vehicles entering from a ramp or a vehicle changing lanes, can establish a disruption wave that propagates throughout the upstream traffic flow. At capacity, the traffic stream has no ability to dissipate even the most minor disruption, and any incident can be expected to produce a serious breakdown with extensive queuing. Maneuverability within the traffic stream is extremely limited, and the level of physical and psychological comfort afforded the driver is poor.

LOS F describes breakdowns in vehicular flow. Such conditions generally exist within queues forming behind breakdown points. Breakdowns occur for a number of reasons:

- Traffic incidents can cause a temporary reduction in the capacity of a short segment, so that the number of vehicles arriving at the point is greater than the number of vehicles that can move through it.
- Points of recurring congestion, such as merge or weaving segments and lane drops, experience very high demand in which the number of vehicles arriving is greater than the number of vehicles discharged.
- In forecasting situations, the projected peak-hour (or other) flow rate can exceed the estimated capacity of the location.

Note that in all cases, breakdown occurs when the ratio of existing demand to actual capacity or of forecast demand to estimated capacity exceeds 1.00. Operations immediately downstream of such a point, however, are generally at or near capacity, and downstream operations improve (assuming that there are no additional downstream bottlenecks) as discharging vehicles move away from the bottleneck.

LOS F operations within a queue are the result of a breakdown or bottleneck at a downstream point. LOS F is also used to describe conditions at the point of the breakdown or bottleneck and the queue discharge flow that occurs at speeds lower than

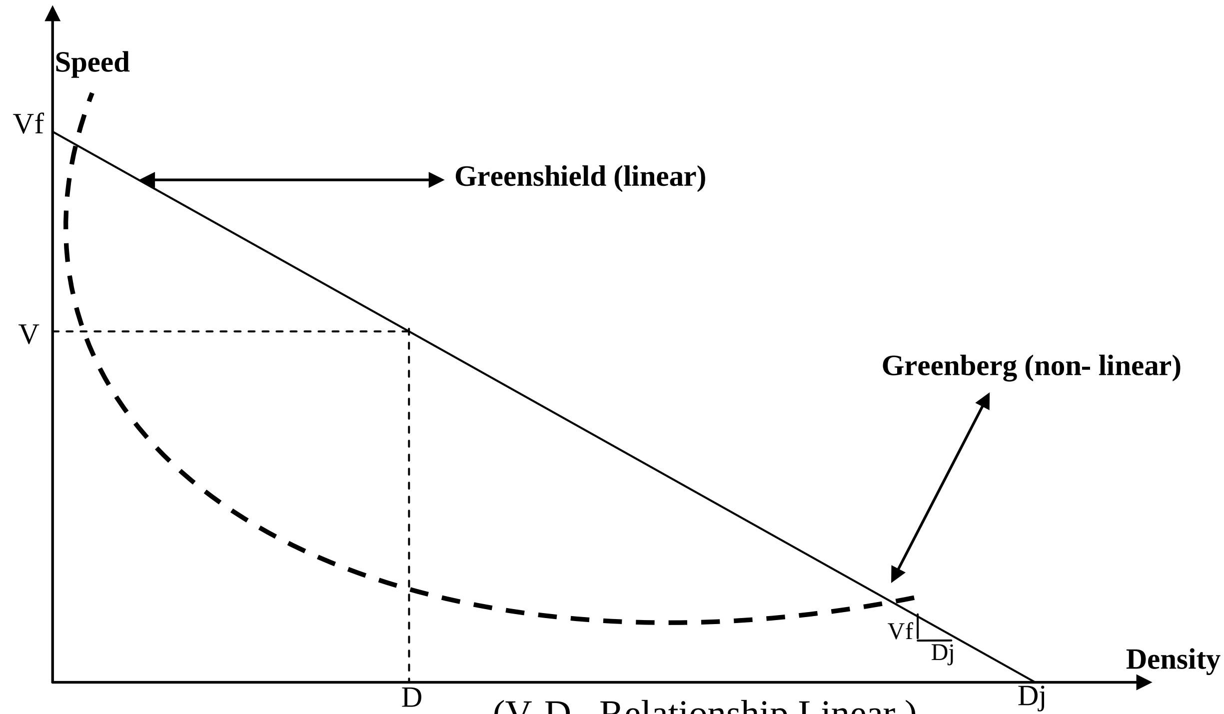
Flow - Density - Speed Relationships

Vf : free flow speed

Dj : jam density

at Vf ⟶ D = 0

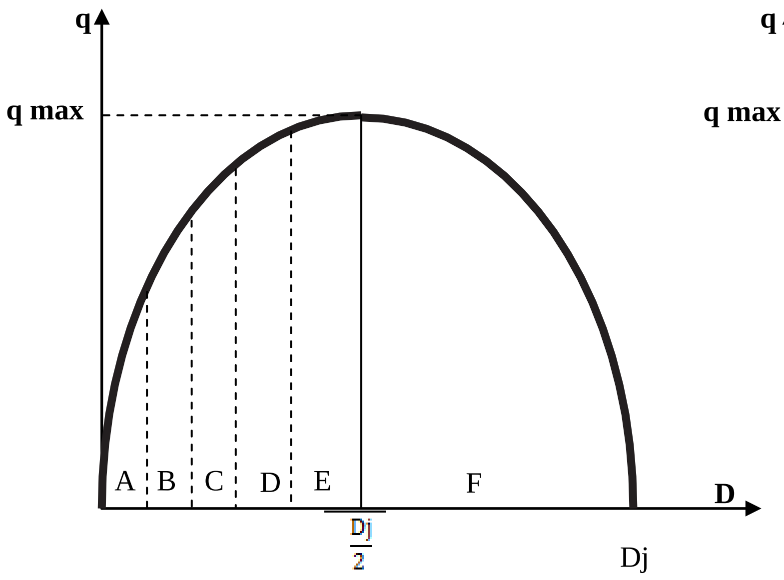
at Dj ⟶ V = 0

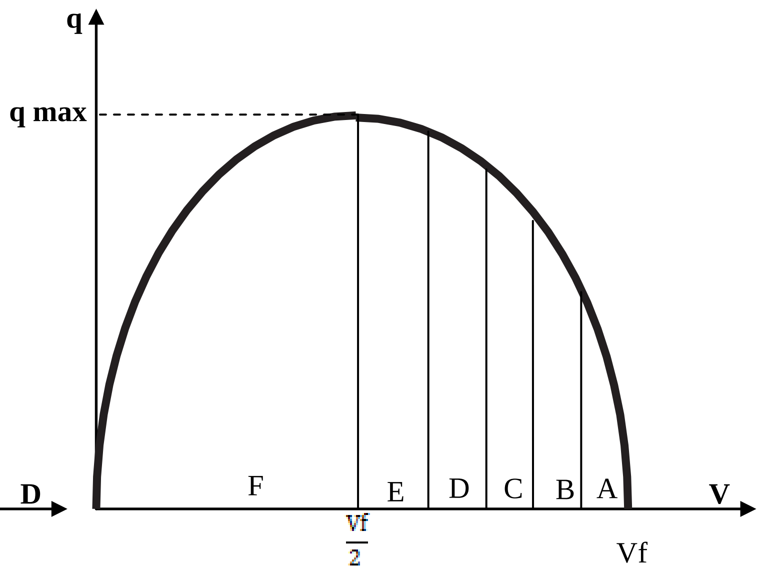


(V-D Relationship Linear)

$$\boxed{V = Vf - D\,\frac{Vf}{Dj} \quad \text{or} \quad D = Dj - V\,\frac{Dj}{Vf}}$$

qmax = capacity





$$\boxed{\mathbf{q = V \times D}}$$

$$\frac{veh}{hr} = \frac{k\cancel{m}}{hr} \times \frac{veh}{k\cancel{m}}$$

$[\ V = Vf - D\ \frac{Vf}{Dj}\ ] \times D$

$q = D\ Vf - D^2\ \frac{Vf}{Dj}$ .... (q-D relationship)

it is parabola function

$[\ D = Dj - V\ \frac{Dj}{Vf}\ ] \times V$

$q = V\ Dj - V^2\ \frac{Dj}{Vf}$ ....(q-V) relationship)

it is parabola function

* At which density (D) , Capacity or q max occur ?

- from the q- D plot , since its symmetrical about the midline , therefore:

$D_{\mathbf{at\ qmax}} = \frac{Dj}{2}$

also

$V_{\mathbf{at\ qmax}} = \frac{Vf}{2}$

- since $q = VD$

therefore :

$q\ max = \frac{Vf}{2} \times \frac{Dj}{2}$

$q\ max = \frac{Vf\ Dj}{4}$

- solution by derivation:

$q = D\ Vf - D^2\ \frac{Vf}{Dj}$ ....(q-D)

$\frac{dq}{dD} = Vf - 2D\ \frac{Vf}{Dj} = 0$

$\cancel{Vf} = 2D\ \frac{\cancel{Vf}}{Dj} \longrightarrow \boxed{D_{\mathbf{at\ qmax}} = \frac{Dj}{2}}$ ....(1)

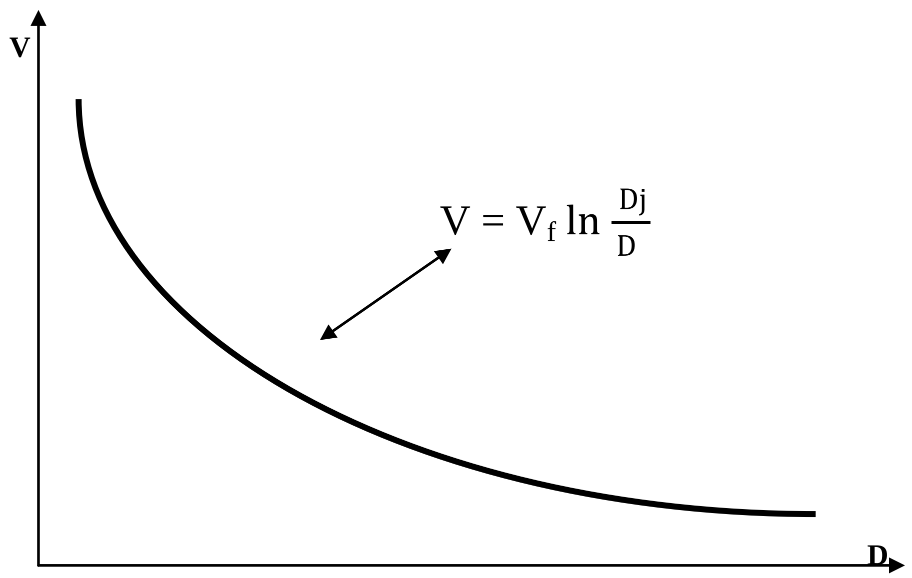
Sub eq. (1) at q-D relation

$$q\ max = \frac{Dj\ Vf}{2} - \left(\frac{Dj}{2}\right)^2 \frac{Vf}{Dj}$$

$$= \frac{Dj\ Vf}{2} - \frac{Dj\ Vf}{4}$$

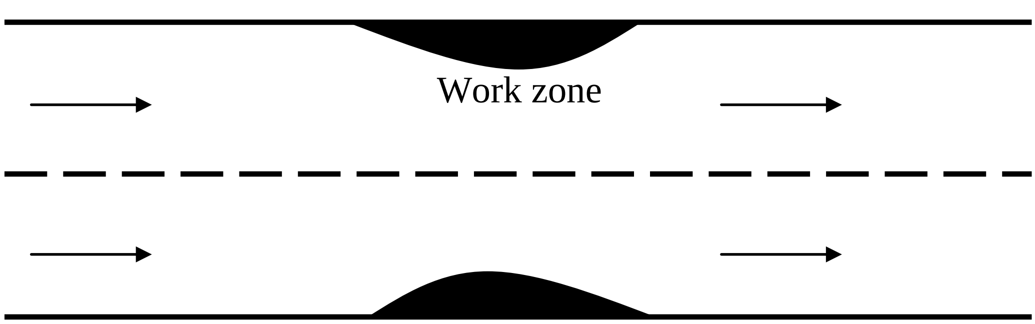
$$qmax = \frac{Vf\ Dj}{4}$$

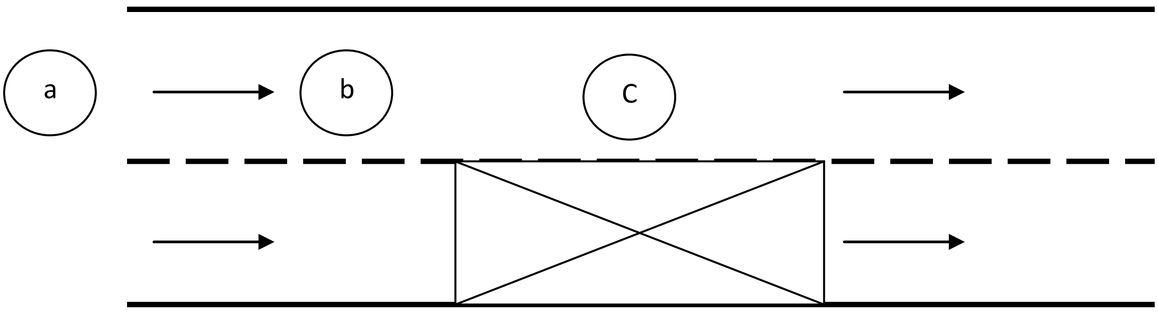
- **Greenberg relationship between V and D**



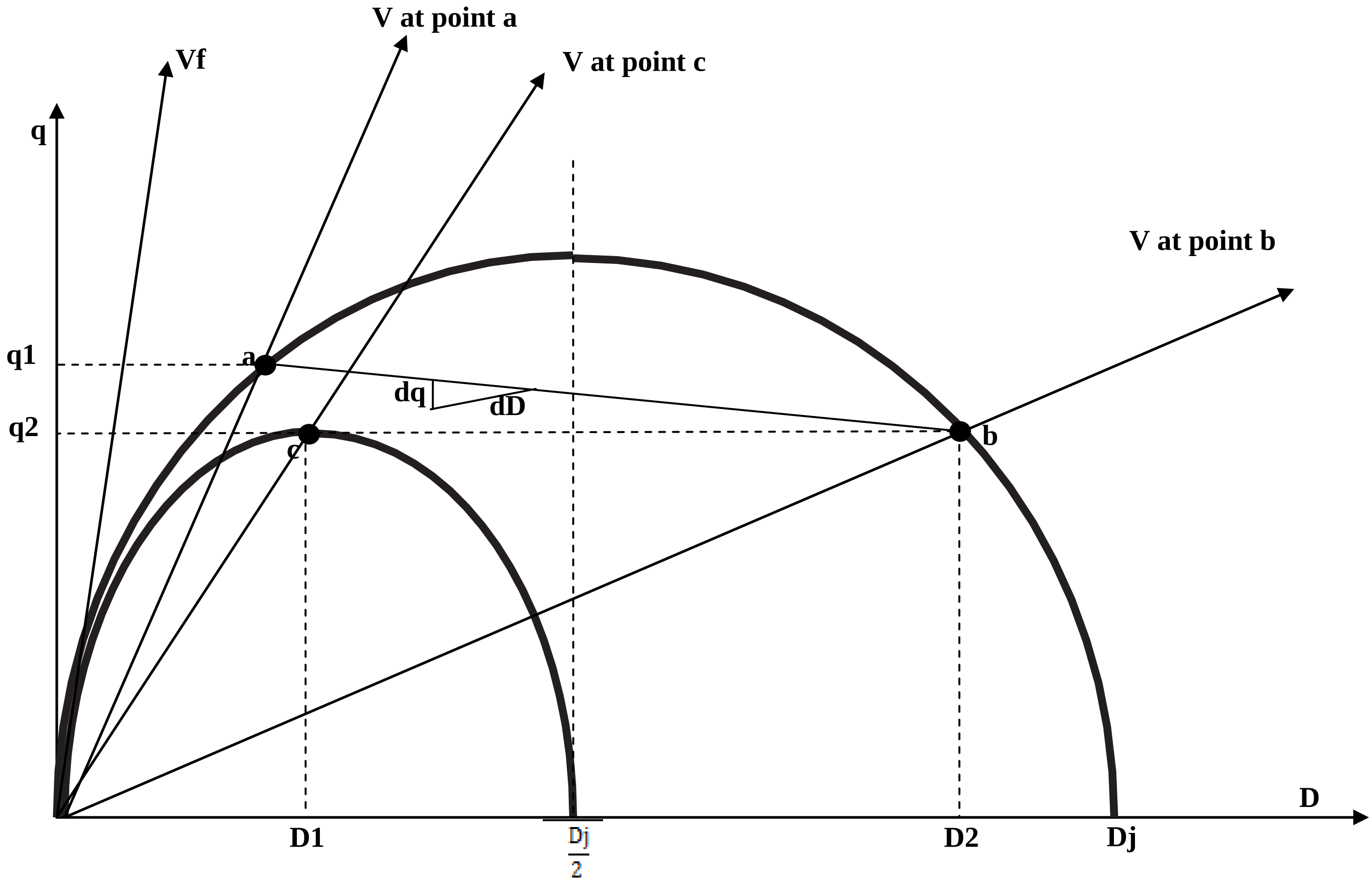
- **Bottleneck**

  Bottleneck or shockwaves occur in traffic stream due to sudden reduction in roadway capacity which may be occur due to the reduction for no. of lanes, work zones, restricted bridge size , crashes or signal turning red .





One Lane blockage



$$Vsw = \frac{q1 - q2}{D2 - D1} = \frac{\Delta q}{\Delta D}$$

Ex1/ Green shield proposed linear relationship between speed and density using the relationship . it was noted that on length of highway the free speed (Vf) was (80 km/hr) and the jam density was (70 vpkm) find the required below :

1- what is the max flow which could be expected on this highway ?

2- at what speed it would be occur ?

3- at what density it would be occur ?

Sol/ Vf = 80km/hr

Dj = 70 vpkm

$q\ max = \frac{80 \times 70}{4} = 21400$ vph

$V_{\text{at qmax}} = \frac{Vf}{2} = 40$ km/hr

$D_{\text{at qmax}} = \frac{Dj}{2} = 35$ km/hr

Ex2/ The speed density relationship of traffic on a section of free way lane was $Vs = 18.2 \ln \frac{220}{D}$

1- what is the max flow , speed and density at this flow ?

2- what is the jam density ?

Ex3/ One lane of 2-lane carriage of a highway is closed for repairs, the free flow speed in the 2-lane portion of highway is 70 km/hr ,the observation showed that the free flow speed in the bottleneck section is also 70 km/hr. the space head way of vehicles in stationary condition is 10 m. if the traffic volume is 2000 veh/hr . find the following?

(assume linear relationship between speed and density).

1- The mean speed of the traffic immediately before bottleneck section.

2- The mean speed of the traffic at a distance clear from the influence of bottleneck.

3- Find the queue length of vehicles after 30 minutes from the beginning of repair operation.

Lecture 9

Headway
- **time headway (sec)**
- **spacing headway (m)**

- time headway (ht) : is the time between the arrival of successive vehicles at a specified point and it is the reciprocal of volume.

$$\mathbf{ht} = \frac{1}{q} \longrightarrow \mathbf{ht_{sec}} = \frac{3600}{q}$$

- space headway (hs) : is the distance between the successive vehicles typically measured from front bumper to front bumper and it's the reciprocal of density.

$$\mathbf{hs} = \frac{1}{D} \longrightarrow \mathbf{hs_{m}} = \frac{1000}{D}$$

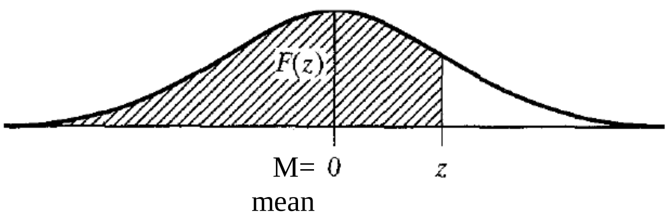
Statistical Application in Traffic Engineering

- Normal Distribution (Continuous distribution)
- Poisson Distribution (Discrete or Counting Distribution)

- Normal Distribution (used in Speed analysis)

$$P(Z)=\int_{-\infty}^{Z} \frac{1}{\sqrt{2\pi}}\, e^{\frac{-Z^2}{2}}\, dz \quad ......(1)$$

P(Z) = area under the curve



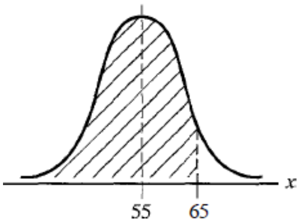
originally, the equation. is

$$P(x) = \frac{1}{\sigma\sqrt{2\pi}}\, e^{-\frac{(x-\mu)^2}{2\sigma^2}} \quad ....(2)$$

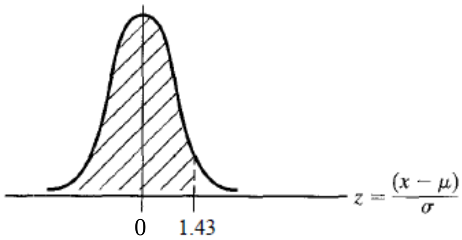
where :
X = normal distribution statistics.
μ = mean of the distribution
σ = standard deviation of distribution

The transformation of eq. 2 into eq. 1 can be done by

$$Z = \frac{\mathbf{X - \mu}}{\boldsymbol{\sigma}}$$



Normal Distribution



Standard Normal Distribution

X = 65
μ = 55 ⟶ $Z = \frac{65-55}{7}$ = 1.43 (Zero Mean)
σ = 7

Sample size determination

$$N \geq \frac{1.96^2\ \sigma^2}{e^2}$$

where :
N= Sample Size
σ = standard deviation
e = standard error of mean
1.96 = z- value corresponding to confidence level of 95% .

Confidence level (area under the curve)	Z value
95%	1.96
99.7 %	3

Example / if the standard deviation of speed sample is (7.2 km/ hr),what is the sample size to estimate the mean speed on highway section plus or minus 1 km/hr based on 95 % and 99.7 % confidence level .

Sol/
for 95 % confidence level
$N = \frac{1.96^2\ 7.2^2}{1^2}$ = 200 veh (sample size)
for 99.7 % confidence level
$N = \frac{3^2\ 7.2^2}{1^2}$ = 467 veh (sample size)
if the required tolerance ± 0.5 km/hr
for 95 %
$N = \frac{1.96^2\ 7.2^2}{0.5^2}$ = 800 veh (sample size 95%)

for 99.7 %

N = $\frac{3^2\ 7.2^2}{0.5^2}$ = 1868 veh (sample size 99.7%)

<u>Analysis of Speed Studies :</u>

In speed studies, a considerable number of speeds are observed . Therefore, it is necessary to use statistics to analysis the data obtained from the observation, the analysis steps can be summarized as follow ;

1- The data are tabulated and converted to frequency and classes, using the following :

k = 1 + 3.3 log N

$$W = \frac{R}{K}$$

where W: class interval width, R: range (max value - min value) , K : no. of classes , N = no. of observation

2- The calculation of cumulative frequency and the percentage cumulative frequency.

3- The calculation of the deviation of each class interval center from the origin.

4- The deviation is multiplied by the corresponding frequency.

5- Multiply each frequency by the square deviation.

6-Assume the data follow the normal distribution.

7- Calculate the mean and the standard deviation from observed data.

8- Calculate the theoretical Probability.

9- Convert the theoretical probability to the theoretical frequency.

10- Apply the chi- square test to check the validity of assumption in step no. (6).

Ex/ Check whether the speed data shown in table below follow the normal distribution or not ?

1	2	3	4	5	6	7	8
Speed class (km/h)	Frequency	Percentage frequency	Cumulative frequency	Percentage cumulative frequency	Deviation	(2) × (6)	$(2) \times (6)^2$
44–47.9	1	0.286	1	0.286	−9	−9	81
48–51.9	2	0.571	3	0.857	−8	−16	128
52–55.9	2	0.571	5	1.429	−7	−14	98
56–59.9	4	1.143	9	2.571	−6	−24	144
60–63.9	11	3.143	20	5.714	−5	−55	275
64–67.9	24	6.875	44	12.571	−4	−96	384
68–71.9	40	11.429	84	24.000	−3	−120	360
72–75.9	48	13.714	132	37.714	−2	−96	192
76–79.9	63	18.000	195	55.714	−1	−63	63
80–83.9	40	11.429	235	67.143	0	0	0
84–87.9	34	9.714	269	76.857	1	34	34
88–91.9	29	8.286	298	85.143	2	58	116
92–95.9	25	7.143	323	92.286	3	75	225
96–99.9	13	3.714	336	96.000	4	52	208
100–103.9	5	1.429	341	97.429	5	25	125
104–107.9	3	0.857	344	98.286	6	18	108
108–111.9	1	0.286	345	98.571	7	7	49
112–115.9	2	0.571	347	99.143	8	16	128
116–119.9	2	0.571	349	99.714	9	18	162
120–123.9	1	0.286	350	100.000	10	10	100
	Σ 350					Σ −180	Σ 2980

The mean speed is then given by :

mid- class mark of selected class + $\frac{\text{Class interval } \sum(\text{column } 7)}{\sum(\text{column } 2)}$

$$82 - \frac{4 \times 180}{350} = 79.9 \text{ km/h}$$

The standard deviation is given by :

$$\text{class interval} \sqrt{\left[\frac{\Sigma\,(\text{frequency (deviation)}^2)}{\Sigma\,(\text{column 2})} - \left(\frac{\Sigma\,(\text{frequency} \times \text{deviation})}{\Sigma\,(\text{column 2})}\right)^2\right]}$$

The value of Σ (frequency × deviation) has already been calculated in column 6 and it is now necessary to calculate the frequency (deviation)² for each speed class. These values are given in column 8.

$$4\sqrt{\left[\frac{2980}{350} - \left(\frac{-180}{350}\right)^2\right]} = 11.6 \text{ km/h}$$

$$\text{class interval} \times \sqrt{\frac{(\text{column } 8)}{(\text{column } 2)} - \left(\frac{(\text{column } 7)}{(\text{column } 2)}\right)^2}$$

1	2	3	4	5	6	7	8
Upper speed class limit (km/h)	Column 1 minus mean speed	Column 2 divided by standard deviation	Normal area	Probability	Theoretical frequency	Observed frequency	$\frac{((6)-(7))^2}{(6)}$
44	−35.9	−3.10	−0.499				
48	−31.9	−2.75	−0.497	0.002	0.7	1	
52	−27.9	−2.40	−0.492	0.005	1.8	2	2.27
56	−23.9	−2.06	−0.480	0.012	4.2	2	
60	−19.9	−1.72	−0.457	0.023	8.1	4	
64	−15.9	−1.37	−0.415	0.042	14.7	11	0.93
68	−11.9	−1.025	−0.349	0.066	23.1	24	0.04
72	−7.9	−0.680	−0.252	0.097	33.9	40	1.10
76	−3.9	−0.336	−0.132	0.119	41.9	48	0.89
80	+0.1	0.009	0.004	0.137	48.0	63	4.69
84	+4.1	0.354	0.138	0.134	46.9	40	1.02
88	+8.1	0.70	0.258	0.120	42.0	34	1.52
92	+12.1	1.04	0.351	0.093	32.6	29	0.40
96	+16.1	1.39	0.418	0.067	23.4	25	0.11
100	+20.1	1.74	0.459	0.041	14.3	13	0.12
104	+24.1	2.08	0.481	0.022	7.7	5	
108	+28.1	2.42	0.492	0.011	3.8	3	
112	+32.1	2.76	0.497	0.005	1.8	1	
116	+36.1	3.11	0.499	0.002	0.7	2	0.01
120	+40.1	3.46	0.500	0.001	0.4	2	
124	+44.1	3.81	0.500	0.000	0	1	
						Chi^2 cal	Σ 13.10

<u>Chi^2 from table</u>

- df (degree of fredoom)

based on df = (no.of rows -3) = 12-3 = 9

- significant level (x = 0.05)

Chi^2 = 16.9

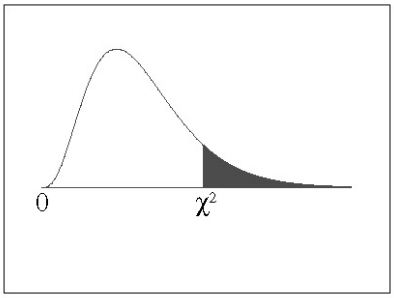
Chi^2 cal < Chi^2 table

so the data follow normal distribution

Table 1. area (z) under normal curve

Z	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	0.0000	0.0040	0.0080	0.0120	0.0160	0.0199	0.0239	0.0279	0.0319	0.0359
0.1	0.0398	0.0438	0.0478	0.0517	0.0557	0.0596	0.0636	0.0675	0.0714	0.0753
0.2	0.0793	0.0832	0.0871	0.0910	0.0948	0.0987	0.1026	0.1064	0.1103	0.1141
0.3	0.1179	0.1217	0.1255	0.1293	0.1331	0.1368	0.1406	0.1443	0.1480	0.1517
0.4	0.1554	0.1591	0.1628	0.1664	0.1700	0.1736	0.1772	0.1808	0.1844	0.1879
0.5	0.1915	0.1950	0.1985	0.2019	0.2054	0.2088	0.2123	0.2157	0.2190	0.2224
0.6	0.2257	0.2291	0.2324	0.2357	0.2389	0.2422	0.2454	0.2486	0.2517	0.2549
0.7	0.2580	0.2611	0.2642	0.2673	0.2704	0.2734	0.2764	0.2794	0.2823	0.2852
0.8	0.2881	0.2910	0.2939	0.2967	0.2995	0.3023	0.3051	0.3078	0.3106	0.3133
0.9	0.3159	0.3186	0.3212	0.3238	0.3264	0.3289	0.3315	0.3340	0.3365	0.3389
1.0	0.3413	0.3438	0.3461	0.3485	0.3508	0.3531	0.3554	0.3577	0.3599	0.3621
1.1	0.3643	0.3665	0.3686	0.3708	0.3729	0.3749	0.3770	0.3790	0.3810	0.3830
1.2	0.3849	0.3869	0.3888	0.3907	0.3925	0.3944	0.3962	0.3980	0.3997	0.4015
1.3	0.4032	0.4049	0.4066	0.4082	0.4099	0.4115	0.4131	0.4147	0.4162	0.4177
1.4	0.4192	0.4207	0.4222	0.4236	0.4251	0.4265	0.4279	0.4292	0.4306	0.4319
1.5	0.4332	0.4345	0.4357	0.4370	0.4382	0.4394	0.4406	0.4418	0.4429	0.4441
1.6	0.4452	0.4463	0.4474	0.4484	0.4495	0.4505	0.4515	0.4525	0.4535	0.4545
1.7	0.4554	0.4564	0.4573	0.4582	0.4591	0.4599	0.4608	0.4616	0.4625	0.4633
1.8	0.4641	0.4649	0.4656	0.4664	0.4671	0.4678	0.4686	0.4693	0.4699	0.4706
1.9	0.4713	0.4719	0.4726	0.4732	0.4738	0.4744	0.4750	0.4756	0.4761	0.4767
2.0	0.4772	0.4778	0.4783	0.4788	0.4793	0.4798	0.4803	0.4808	0.4812	0.4817
2.1	0.4821	0.4826	0.4830	0.4834	0.4838	0.4842	0.4846	0.4850	0.4854	0.4857
2.2	0.4861	0.4864	0.4868	0.4871	0.4875	0.4878	0.4881	0.4884	0.4887	0.4890
2.3	0.4893	0.4896	0.4898	0.4901	0.4904	0.4906	0.4909	0.4911	0.4913	0.4916
2.4	0.4918	0.4920	0.4922	0.4925	0.4927	0.4929	0.4931	0.4932	0.4934	0.4936
2.5	0.4938	0.4940	0.4941	0.4943	0.4945	0.4946	0.4948	0.4949	0.4951	0.4952
2.6	0.4953	0.4955	0.4956	0.4957	0.4959	0.4960	0.4961	0.4962	0.4963	0.4964
2.7	0.4965	0.4966	0.4967	0.4968	0.4969	0.4970	0.4971	0.4972	0.4973	0.4974
2.8	0.4974	0.4975	0.4976	0.4977	0.4977	0.4978	0.4979	0.4979	0.4980	0.4981
2.9	0.4981	0.4982	0.4982	0.4983	0.4984	0.4984	0.4985	0.4985	0.4986	0.4986
3.0	0.4987	0.4987	0.4987	0.4988	0.4988	0.4989	0.4989	0.4989	0.4990	0.4990
3.1	0.4990	0.4991	0.4991	0.4991	0.4992	0.4992	0.4992	0.4992	0.4993	0.4993
3.2	0.4993	0.4993	0.4994	0.4994	0.4994	0.4994	0.4994	0.4995	0.4995	0.4995
3.3	0.4995	0.4995	0.4995	0.4996	0.4996	0.4996	0.4996	0.4996	0.4996	0.4997
3.4	0.4997	0.4997	0.4997	0.4997	0.4997	0.4997	0.4997	0.4997	0.4997	0.4998
3.5	0.4998	0.4998	0.4998	0.4998	0.4998	0.4998	0.4998	0.4998	0.4998	0.4998
3.6	0.4998	0.4998	0.4999	0.4999	0.4999	0.4999	0.4999	0.4999	0.4999	0.4999
3.7	0.4999	0.4999	0.4999	0.4999	0.4999	0.4999	0.4999	0.4999	0.4999	0.4999
3.8	0.4999	0.4999	0.4999	0.4999	0.4999	0.4999	0.4999	0.4999	0.4999	0.4999
3.9	0.5000	0.5000	0.5000	0.5000	0.5000	0.5000	0.5000	0.5000	0.5000	0.5000

Chi-Square Distribution Table



The shaded area is equal to α for $\chi^2 = \chi^2_\alpha$.

df	$\chi^2_{.995}$	$\chi^2_{.990}$	$\chi^2_{.975}$	$\chi^2_{.950}$	$\chi^2_{.900}$	$\chi^2_{.100}$	$\chi^2_{.050}$	$\chi^2_{.025}$	$\chi^2_{.010}$	$\chi^2_{.005}$
1	0.000	0.000	0.001	0.004	0.016	2.706	3.841	5.024	6.635	7.879
2	0.010	0.020	0.051	0.103	0.211	4.605	5.991	7.378	9.210	10.597
3	0.072	0.115	0.216	0.352	0.584	6.251	7.815	9.348	11.345	12.838
4	0.207	0.297	0.484	0.711	1.064	7.779	9.488	11.143	13.277	14.860
5	0.412	0.554	0.831	1.145	1.610	9.236	11.070	12.833	15.086	16.750
6	0.676	0.872	1.237	1.635	2.204	10.645	12.592	14.449	16.812	18.548
7	0.989	1.239	1.690	2.167	2.833	12.017	14.067	16.013	18.475	20.278
8	1.344	1.646	2.180	2.733	3.490	13.362	15.507	17.535	20.090	21.955
9	1.735	2.088	2.700	3.325	4.168	14.684	16.919	19.023	21.666	23.589
10	2.156	2.558	3.247	3.940	4.865	15.987	18.307	20.483	23.209	25.188
11	2.603	3.053	3.816	4.575	5.578	17.275	19.675	21.920	24.725	26.757
12	3.074	3.571	4.404	5.226	6.304	18.549	21.026	23.337	26.217	28.300
13	3.565	4.107	5.009	5.892	7.042	19.812	22.362	24.736	27.688	29.819
14	4.075	4.660	5.629	6.571	7.790	21.064	23.685	26.119	29.141	31.319
15	4.601	5.229	6.262	7.261	8.547	22.307	24.996	27.488	30.578	32.801
16	5.142	5.812	6.908	7.962	9.312	23.542	26.296	28.845	32.000	34.267
17	5.697	6.408	7.564	8.672	10.085	24.769	27.587	30.191	33.409	35.718
18	6.265	7.015	8.231	9.390	10.865	25.989	28.869	31.526	34.805	37.156
19	6.844	7.633	8.907	10.117	11.651	27.204	30.144	32.852	36.191	38.582
20	7.434	8.260	9.591	10.851	12.443	28.412	31.410	34.170	37.566	39.997
21	8.034	8.897	10.283	11.591	13.240	29.615	32.671	35.479	38.932	41.401
22	8.643	9.542	10.982	12.338	14.041	30.813	33.924	36.781	40.289	42.796
23	9.260	10.196	11.689	13.091	14.848	32.007	35.172	38.076	41.638	44.181
24	9.886	10.856	12.401	13.848	15.659	33.196	36.415	39.364	42.980	45.559
25	10.520	11.524	13.120	14.611	16.473	34.382	37.652	40.646	44.314	46.928
26	11.160	12.198	13.844	15.379	17.292	35.563	38.885	41.923	45.642	48.290
27	11.808	12.879	14.573	16.151	18.114	36.741	40.113	43.195	46.963	49.645
28	12.461	13.565	15.308	16.928	18.939	37.916	41.337	44.461	48.278	50.993
29	13.121	14.256	16.047	17.708	19.768	39.087	42.557	45.722	49.588	52.336
30	13.787	14.953	16.791	18.493	20.599	40.256	43.773	46.979	50.892	53.672
40	20.707	22.164	24.433	26.509	29.051	51.805	55.758	59.342	63.691	66.766
50	27.991	29.707	32.357	34.764	37.689	63.167	67.505	71.420	76.154	79.490
60	35.534	37.485	40.482	43.188	46.459	74.397	79.082	83.298	88.379	91.952
70	43.275	45.442	48.758	51.739	55.329	85.527	90.531	95.023	100.425	104.215
80	51.172	53.540	57.153	60.391	64.278	96.578	101.879	106.629	112.329	116.321
90	59.196	61.754	65.647	69.126	73.291	107.565	113.145	118.136	124.116	128.299
100	67.328	70.065	74.222	77.929	82.358	118.498	124.342	129.561	135.807	140.169

Poisson Distribution (Discrete or Counting Distribution)

suitable for parking studies, accident or vehicles arrivals .

$P(n) = \frac{\mathbf{m^n.e^{-m}}}{\mathbf{n!}}$

p(n) : Probability that n events will occur during specified time interval .

m : mean number of events during the specified time interval.

$m = \frac{\text{Total No.of events}}{\text{Total No.of time interval}}$

$P(n) = \frac{\mathbf{m^n.e^{-m}}}{\mathbf{n!}}$

$P(n\text{-}1) = \frac{\mathbf{m^{n-1}.e^{-m}}}{\mathbf{(n-1)!}}$

$\frac{P(n)}{P(n-1)} = \frac{\mathbf{m^n.e^{-m}\,(n-1)!}}{\mathbf{m^{n-1}.e^{-m}\;n!}} = \frac{\mathbf{m}}{\mathbf{n}}$

Ex/on a motor way, the number of vehicles arriving from one direction in successive (10 seconds) was counted as shown in table below, is the vehicle arrival random.

1 vehicles arrival 10 seconds	**2** observed frequency	**3= 1×2** Total No. of vehicles	**Theo. Probability**	**Theo. freq. = ∑F×Theo. Probability**	**Chi^2** **(theo.-observed)^2/ (theo)**
0	11	0	0.135	13.5	0.462
1	28	28	0.2736	27.06	0.032
2	30	60	0.2706	27.06	0.319
3	18	54	0.1804	18.04	8.8×10^{-5}
4	8	32	0.0902	9.02	0.113
5	4	20	0.036	3.6	0.0076
6	1	6	0.012	1.2	
7 or more	0	0	0.0098	0.4	
	∑= 100	∑= 200 veh			∑Chi^2 = 0.882 cal.

$m = \frac{\text{Total No.of events}}{\text{Total No.of time interval}} = \frac{200}{100} = 2$ arrival /time interval

does the arrival pattern is random ?

theo. probability

$P(0) = \frac{\mathbf{2^0.e^{-2}}}{\mathbf{0!}} = \mathbf{0.13}$

$P(1) = \frac{\mathbf{2^1.e^{-2}}}{\mathbf{1!}} = \mathbf{0.2706}$

$P(2) = \frac{\mathbf{2^2.e^{-2}}}{\mathbf{2!}} = \mathbf{0.2706}$

.

P(6) = 0.01

(P≥7) = 1-P(< 7)

= 1-P(0 ,1 ,2, 3, 4,5,6)

= 0.0098

df (6-2) = 4, X = 0.05%
chi square from table = 9.488
0.882 <9.488
random

H.W1/ The number of traffic accident that occur on particular stretch of road during a month follows a Poisson distribution with a mean of 7. Find the probability of observing exactly three accidents on this stretch of road next month.

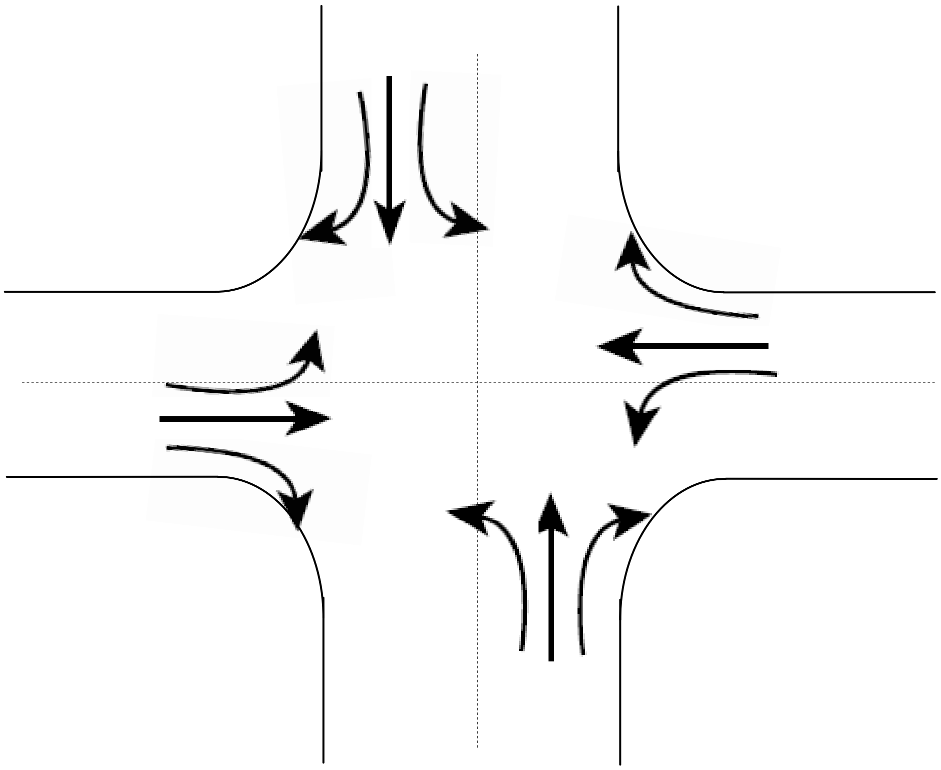
H.W2/ In the entrance of car parking, the vehicle arrival in each counting period of 100 sec. is shown in table below, check whether the arrival distribution of vehicle can be assumed random or not .

Vehicle per 100 sec.	0	1	2	3	≥4
Frequency	60	28	16	8	0

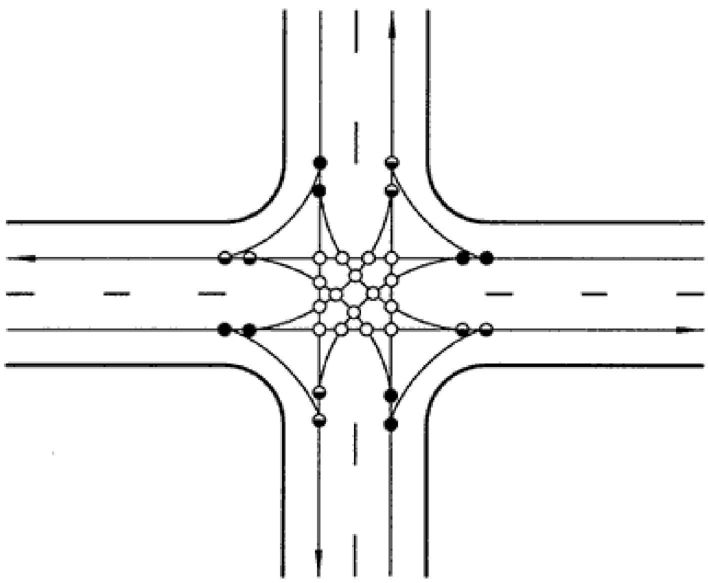
Lecture 10

Intersections

defined as the general area where two or more highway join or cross.



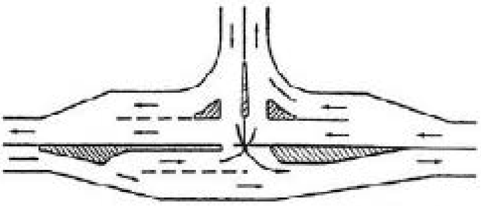
two way intersection = 24 conflict point



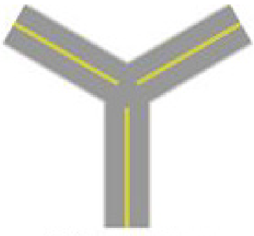
Intersection Types :

A- At- grade Intersection

1 - 3-leg

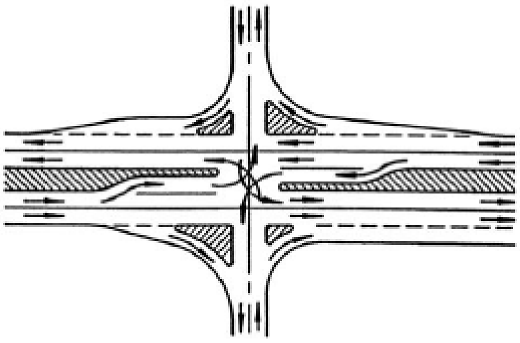


T- intersection

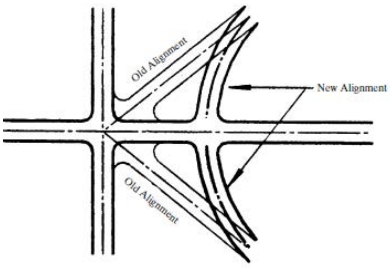


Y- intersection

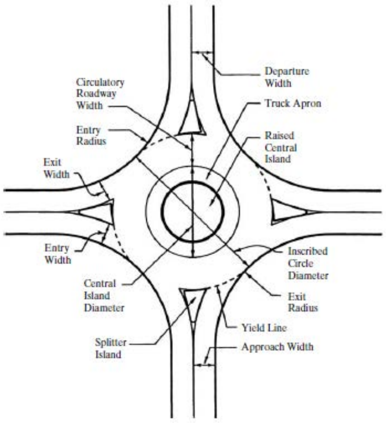
2- 4 Leg- intersection



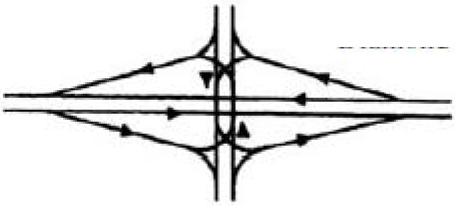
3- Multi leg- intersection



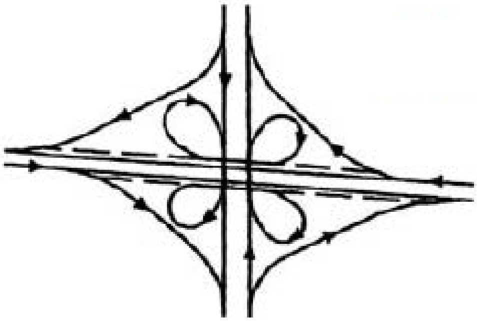
4- Roundabout



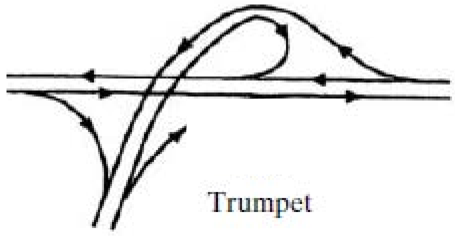
Interchange



Diamond



Full cloverleaf

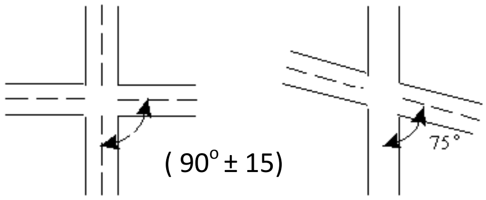


Trumpet

At- Grade Intersection

Requirement

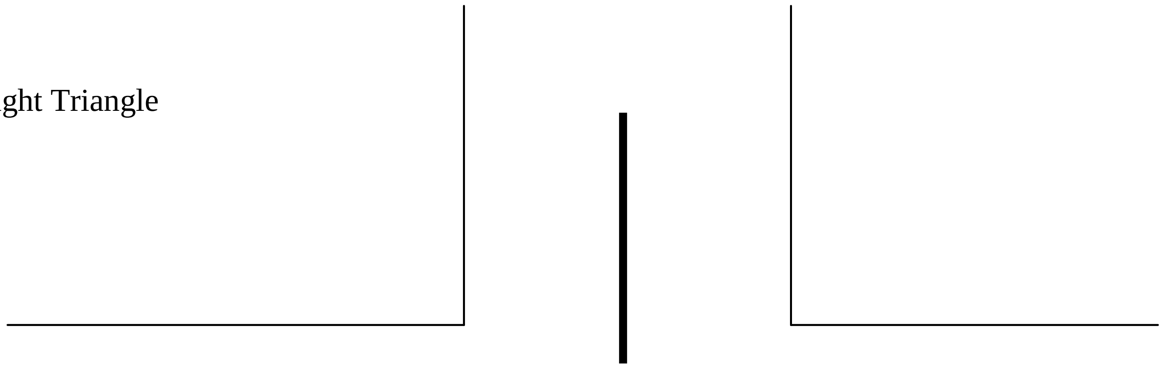
1- Intersection Angel $\approx 90 \pm 15^{o}$ (75^{o} -105^{o})

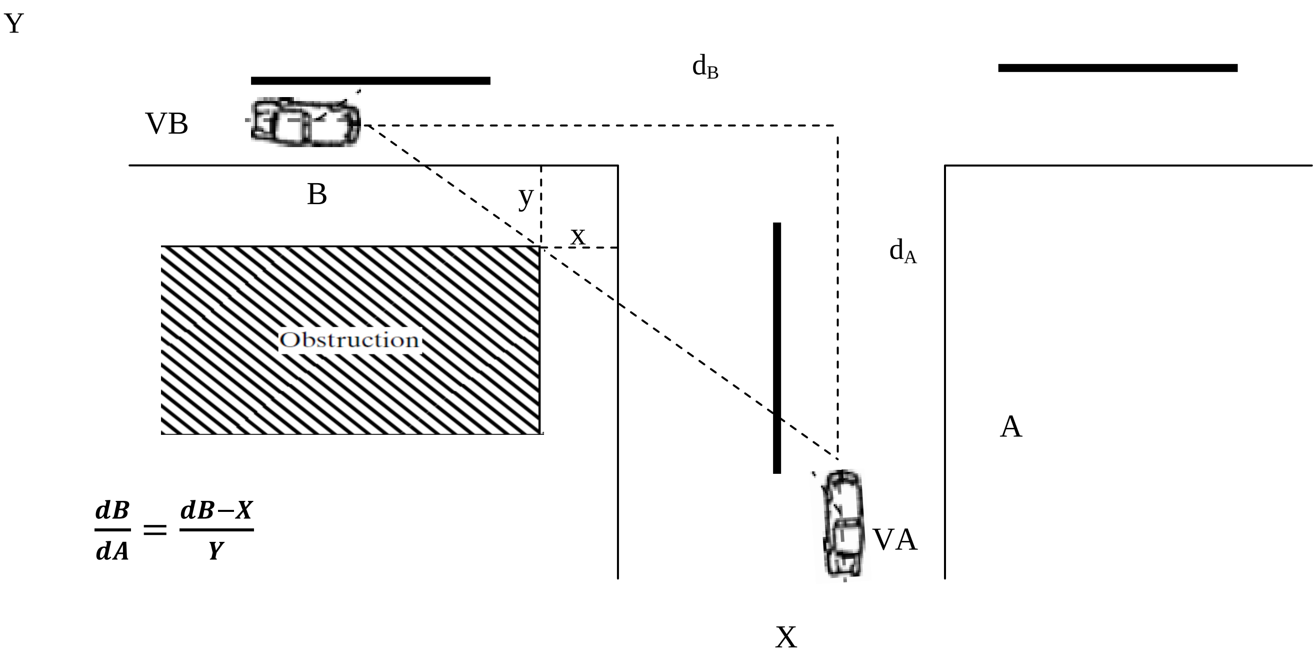


2- Slopes (grade)

max grade 3% (preferred ≤ 1%)

3- Sight Triangle





$$\frac{dB}{dA} = \frac{dB - X}{Y}$$

$d_B = 0.278V_B t + \frac{V_B{}^2}{254\ (fb \pm g)}$

where :

V_B = speed of vehicle B in km/hr

t = perception - reaction time (urban = 1.5 sec , rural = 2.5 sec)

fb = coefficient of breaking friction = 0.45 if dry pavement

= 0.35 if wet pavement

= 0.1 if icy

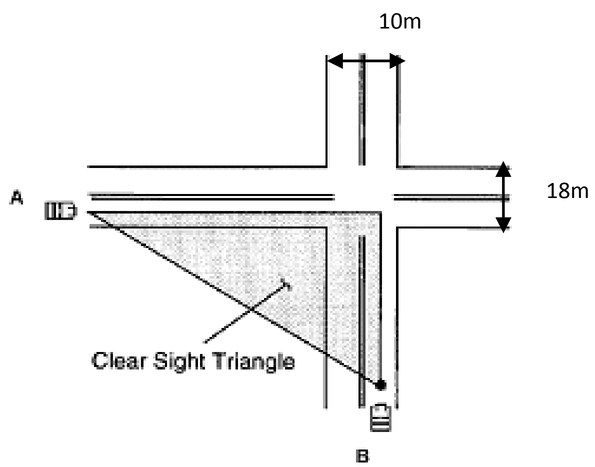
= 0.05 muddy

g = grade high speed ≤ 0.05 (5%)

low speed ≥ 0.05

$V_A = 0.278V_A t + \frac{V_A{}^2}{254\ (fb \pm g)}$

Example/ At a 4-leg intersection, the approach speed of vehicle A is 70 km/hr .as it approaches from west .The values of (x, y)equals to 8m and 10m respectively, calculate the distances (X, Y) and the safe speed of vehicle B which is approaching from south using AASHO method .Take f =0.4, t =1.5 sec, Note: median width is 2 m for both sides (see plot in figure 1)



Intersections

Design of Signalized intersection:

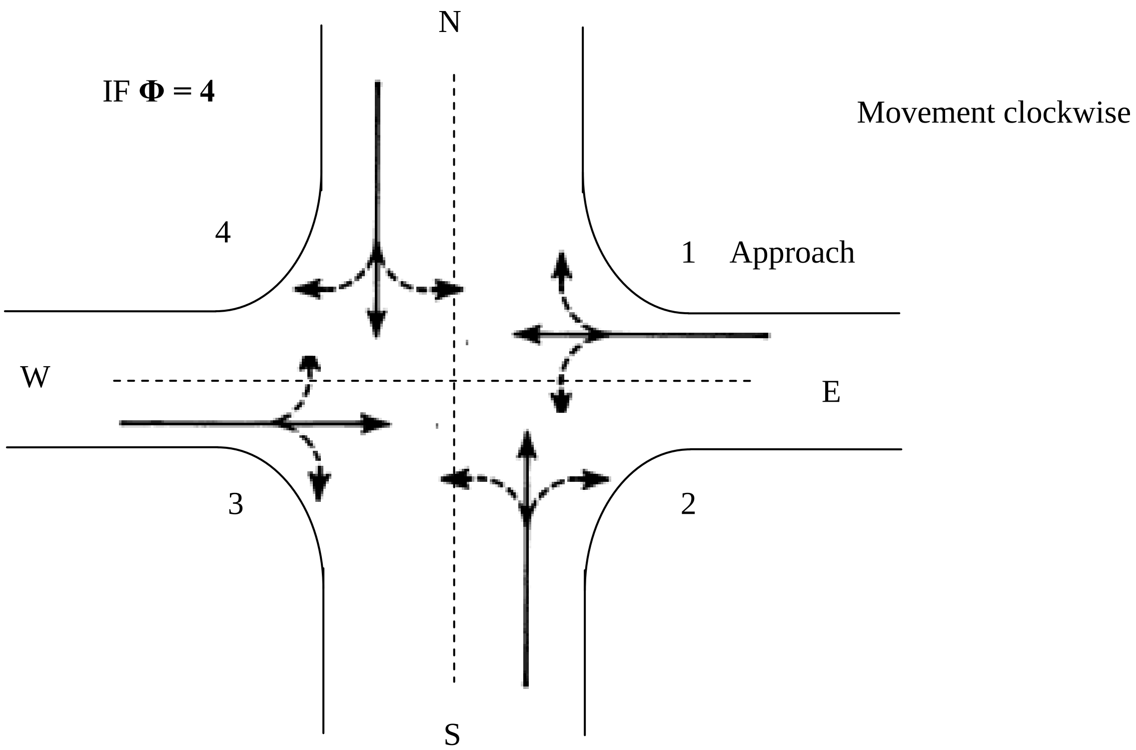
- pre timed (fixed) : cycle length of signal

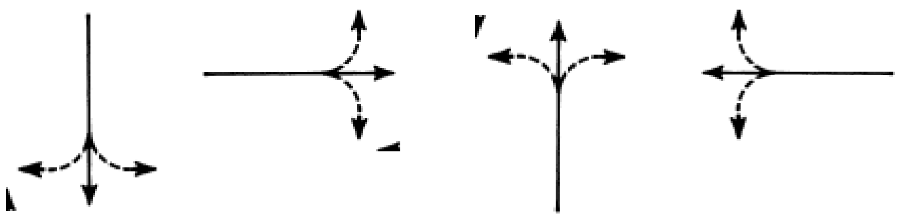
- Traffic actuated (detector + control) = moveable cycle length

- Traffic adjusted (master computer) = moveable cycle length

Phase Φ : one or more direction receiving the same signal at the same time to eliminate the conflicts.

n : no. of phases : 2,3,4,........,8

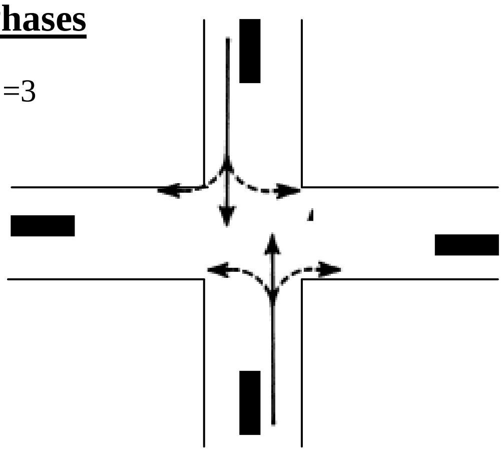




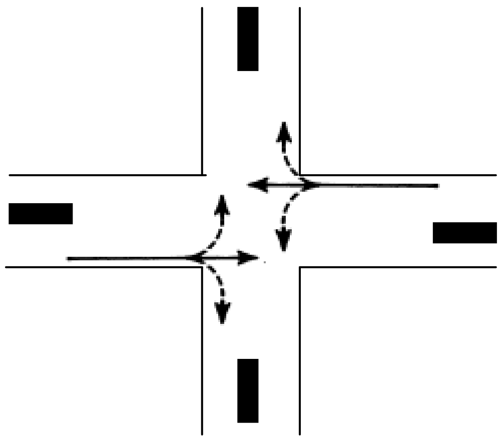
Φ4 **Φ**3 **Φ**2 **Φ**1

Phases

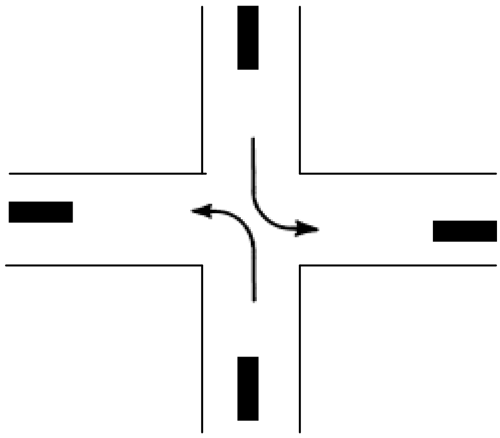
n =3



Φ2

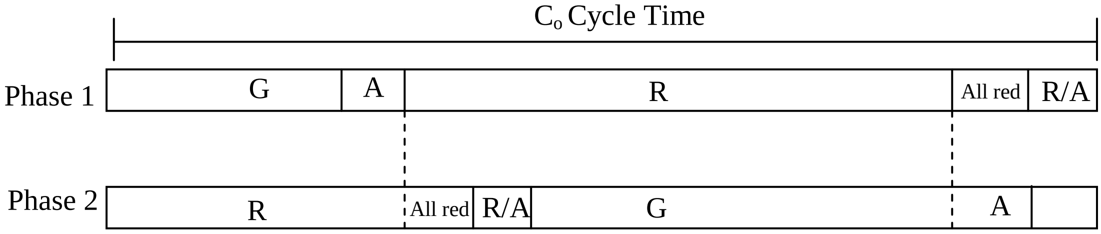


Φ1



Φ3

Signal Indication



- Amber(Yalow) : Warning , red is coming

- Green : go

- All red : clearance for pedestrian and vehicle (1-2 sec)

- Red/ Amber : to reduce starting delay (to use green time effectively, 2-3 sec)

- Red : stop

- **The yellow time can be calculated as follow (ITE) :**

$$A = \frac{0.278\ V_{85}}{2d + (2 \times 9.81 \times \frac{G}{100})}$$

where :

d : vehicle declaration rate (m/sec^2) ≈ (2.5-3.5 m/sec^2)

G : grade percent

V85 : operating speed

so, if v85 = 60 km/hr , G = 0% , d= 3 m/sec^2

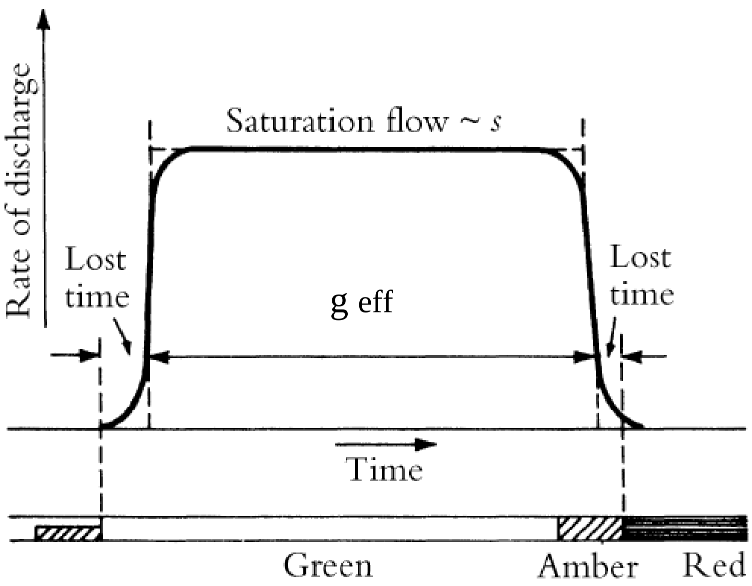
A = 2.78 sec use 3 sec

- **All red can be calculated**

$$AR = \frac{W + L}{0.278\ V_{15}}$$

where :
W : width of intersection (m)
L : length of standard vehicle 6-7 m
V15 : percentiles speed of approach traffic
so, if W= 20 m , L= 6 m , V15 = 50km/hr
then AR = 1.87 sec use 2 sec

- **Saturation Flow (Q_s) :**max rate of discharge which can be maintained at certain approach .



* all red (1-2) sec
red/amber (2-3) sec
Amber (4) sec

G :controller Green time
A: controller Amber time
g : effective green time
ts: lost time during starting and falling discharge for each phase
tr: lost time (all red and red/ amber) for each phase
TL = n (ts + tr) , where n = No. of phases/ cycle and TL = total lost time / cycle

Q_s = 525W ....webester Model

where :

W : Approach width (m)

Q_s : saturation flow (veh/hr)

- Condition for using Webester Model

1- W = 5.5m -18 m

2- No. left turn

3- No side parked vehicle

- If W < 5.5 m then :

W	Q_s
4.0	1960
5.0	2575

- If left turn vehicle exist (1 left turn vehicle = 1.75on going vehicles)

Q_s corrected = $Q_{s\ calculated}$ - 0.75 × No. of left turn vehicles

= $Q_{s\ calculated}$ - 0.75 ×Pl × $Q_{s\ calculated}$

* Pl = proportion of left turn vehicle

- If there is side parking

loss in width (ΔW) = $1.68 - \frac{d-7.62}{G}$

* d = distance in m from parking edge to approach stop line

So, we use in webester model (W- ΔW)

- **Actual Flow Q_a** : DDHV (30 HV) or
  0.15 × ADT (rural)
  0.1 × ADT (urban) or
  K × ADT

- **Phase Efficiency :** $yi = \frac{Qa}{Qs}$

for phase with more than one approach, use y max

$y_{available} = \sum_{i=1}^{n} yi$

$= y1+y2+y3+................yn$

$y_{max} = [\ 1 - \frac{TL}{C}\ ] \times 0.9$

for t =120 sec

$y_{max} = 0.9-0.0075\ TL$

$y_{available} \leq y_{max}$

Cycle time (Co) = $\frac{1.5\ TL+5}{1-y_{avaliable}}$

Optimum cycle time Co = 60- 140 sec

gt = Co - TL

$g1 = \frac{y_1}{y_{avaliable}} \times gt = G1 = g1+ ts -A$

$g2 = \frac{y_2}{y_{avaliable}} \times gt = G2 = g2+ ts -A$

$g3 = \frac{y_3}{y_{avaliable}} \times gt = G3 = g3+ ts -A$

$gi = \frac{y_i}{y_{avaliable}} \times gt = Gi = gi+ ts -A$

No. of vehicle in queue of approach I during one cycle = $Q_{ai} \times$ (Co -gi).....effective red

Ex/At a certain at- grade intersection , its to use a 4 - phase traffic signal for the four approaches show below :

Approach	East	West	South	North
Width (m)	12	10.5	8	7
Actual flow (DHV) pcph	1104	715	427	331
Left turn pcph Percent from saturation flow	31%	19%	15%	13%

Assuming the following for each phase :

ts (starting delay and falling discharge) = 3 sec , amber time = 5 sec , all red = 1 sec , red amber = 1 sec . find the optimum cycle time and prepare a plot for the seqience of controller timing in one complete cycle .

Sol/

	E	W	S	N
Q_a	1104	715	427	331
Q_s	6300	5513	4200	3675
$Q_{a\ correct}$	4835	4727	3727	3317

Q_s E = Q_s E- 0.75 ×percent × Q_s E

= 6300 - 0.75 ×0.31×6300 = 4835 vph

Q_s W corrected = 5513- 0.75× 0.19×5513 = 4727 vph

Q_s N corrected = 4200- 0.75× 0.15×4200 = 3727 vph

Q_s S corrected = 3673- 0.75× 0.13×3673 = 3317 vph

$yi = \frac{\mathbf{Qa}}{Qs}$

E	W	S	N
0.228	0.151	0.115	0.1

$y_{available} = \sum_{i=1}^{n} yi$

= 0.594

TL = n(ts+tr) = 4(3+2) = TL = 20 sec

$y_{max} =$ 0.9 - 0.0075 TL = 0.9-0.0075 ×20

=0.75> $y_{available}$ ....ok

$$\mathbf{Co} = \frac{\mathbf{1.5\ TL+5}}{\mathbf{1-y_{avaliable}}} = \frac{1.5 \times 20+5}{1-0.594} = 86 \text{ sec}$$

gt = C- TL = 86-20 = 66 sec

$g_1 = \frac{\mathbf{y_1}}{\mathbf{y_{avaliable}}} \times gt$ = G1 = g1+ ts -A

$= \frac{\mathbf{0.228}}{\mathbf{0.549}} \times 66 = 25$

G = g1+ts-A

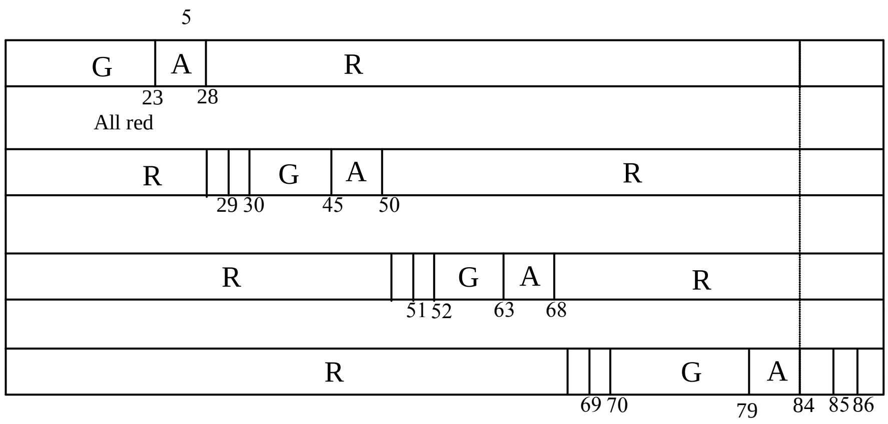
= 25+3-5 = 23

g_2 = 17 G = 15

g_3 = 13 G = 11

g_4 =11 G = 9

∑g = 66................ok


5
G
A
R
23
28
All red
R
G
A
R
29 30
45
50
R
G
A
R
51 52
63
68
R
G
A
69 70
79
84
85 86

Lecture 11

CHAPTER 20

TWO-LANE HIGHWAYS

CONTENTS

EXHIBITS

I. INTRODUCTION

This chapter presents a comprehensive study of two-lane highway operation (*1*). The development of the methodology used microscopic simulation, field data, and theoretical concepts. Analytical procedures are provided for two applications, operational and planning. Chapter 12, "Highway Concepts," presents definitions of basic parameters and important concepts related to the methodology. Appendix A also covers design treatments not addressed by the methodology.

For background and concepts, see Chapter 12, "Highway Concepts"

SCOPE OF THE METHODOLOGY

This chapter presents operational analysis for two-way and directional segments of two-lane highways. Two-way segments may include longer sections of two-lane highway with homogeneous cross sections and relatively constant demand volumes and vehicle mixes over the length of the segment. Two-way segments may be located in level or rolling terrain. Two-lane highways in mountainous terrain or with grades of 3 percent or more for lengths of 1.0 km or more cannot be analyzed as two-lane segments. Instead, they are analyzed as specific upgrades or downgrades. Performance measures for the two-way segment methodology apply to both directions of travel combined.

The analysis can consider two directions combined or only one direction

Directional segments carry one direction of travel on a two-lane highway with homogeneous cross sections and relatively constant demand volume and vehicle mix. Any roadway segment can be evaluated with the directional segment procedure, but separate analysis by direction of travel is particularly appropriate for steep grades and for segments containing passing lanes.

The types of directional segments addressed by the operational applications include directional segments in level or rolling terrain, specific upgrades, and specific downgrades. When only one direction of travel on a two-way segment is analyzed, the procedure for directional segments in level and rolling terrain is used. All directional segments in mountainous terrain and all grades of 3 percent or more with a length of 1.0 km or more must be analyzed as specific upgrades or downgrades.

For analysis of specific upgrades or downgrades, the length of grade is its tangent length plus a portion of the vertical curves at its beginning and end. About one-fourth of the length of the vertical curves at the beginning and end of a grade are included. If two grades (in the same direction) are joined by a vertical curve, one-half the length of the curve is included in each grade segment. The performance measures determined by the directional segment methodology apply only to the direction of travel being analyzed. However, the traffic performance measures for the analysis direction are influenced by the flow rate and traffic characteristics in the opposing direction.

The objective of operational analysis is to determine the level of service (LOS) for an existing or proposed facility operating under current or projected traffic demand. Operational analysis also may be used to determine the capacity of a two-lane highway segment, or the service flow rate that can be accommodated at any given LOS.

LIMITATIONS OF THE METHODOLOGY

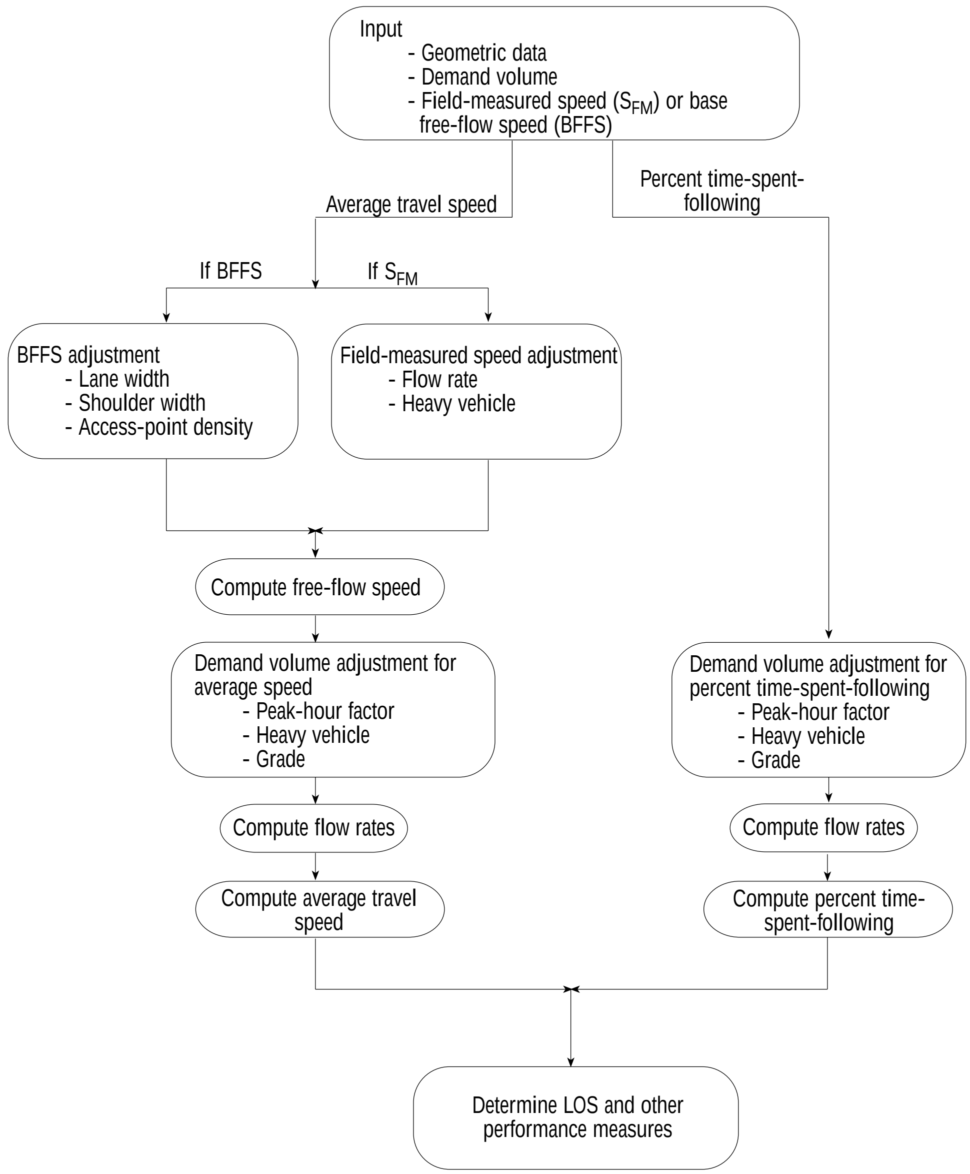
Some two-lane highways—particularly those that involve interactions among several passing or climbing lanes—are too complex to be addressed with the procedures of this chapter. For analytical problems beyond the scope of this chapter, see Part V of this manual, which describes the application of simulation modeling to two-lane highway analyses. Several design treatments discussed in Appendix A are not accounted for by the methodology.

The operational analysis methodologies in this chapter do not address two-lane highways with signalized intersections. Isolated signalized intersections on two-lane highways can be evaluated with the methodology in Chapter 16, "Signalized Intersections." Two-lane highways in urban and suburban areas with multiple signalized intersections at spacings of 3.2 km or less can be evaluated with the methodology of Chapter 15, "Urban Streets."

II. METHODOLOGY

The following discussion presents estimates of two-lane highway capacity, defines the LOS for two-lane highways, and documents the methodology for operational and for planning applications. Exhibit 20-1 summarizes the basic methodology for two-lane highways.

EXHIBIT 20-1. TWO-LANE HIGHWAY METHODOLOGY



CAPACITY

The capacity of a two-lane highway is 1,700 pc/h for each direction of travel. The capacity is nearly independent of the directional distribution of traffic on the facility, except that for extended lengths of two-lane highway, the capacity will not exceed 3,200 pc/h for both directions of travel combined. For short lengths of two-lane highway—such as tunnels or bridges—a capacity of 3,200 to 3,400 pc/h for both directions of travel combined may be attained but cannot be expected for an extended length.

Capacity = 1,700 pc/h for each direction, and 3,200 for both directions combined

LEVELS OF SERVICE

The service measures for a two-lane highway are defined in Chapter 12, "Highway Concepts." On Class I highways, efficient mobility is paramount, and LOS is defined in terms of both percent time-spent-following and average travel speed. On Class II highways, mobility is less critical, and LOS is defined only in terms of percent time-spent-following, without consideration of average travel speed. Drivers will tolerate higher levels of percent time-spent-following on a Class II facility than on a Class I facility, because Class II facilities usually serve shorter trips and different trip purposes.

For definitions of the service measures for two-lane highways, percent time-spent-following, and average travel speed, see Chapter 12, "Highway Concepts"

LOS criteria for two-lane highways in Classes I and II are presented in Exhibits 20-2, 20-3, and 20-4. Exhibit 20-2 reflects the maximum values of percent time-spent-following and average travel speed for each LOS for Class I highways. A segment of a Class I highway must meet the criteria for both the percent time-spent-following and the average travel speed shown in Exhibit 20-2 to be classified in any particular LOS. Exhibit 20-3 illustrates the LOS criteria for Class I highways. For example, a Class I two-lane highway with percent time-spent-following equal to 45 percent and an average travel speed of 65 km/h would be classified as LOS D based on Exhibit 20-2. However, a Class II highway with the same conditions would be classified as LOS B based on Exhibit 20-4. The difference between these LOS assessments represents the difference in motorist expectations for Class I and II facilities.

For definitions of Class I and II highways, also see Chapter 12

The LOS criteria in Exhibits 20-2 through 20-4 apply to all types of two-lane highways, including extended two-way segments, extended directional segments, specific upgrades, and specific downgrades.

TWO-WAY SEGMENTS

The two-way segment methodology estimates measures of traffic operation along a section of highway, based on terrain, geometric design, and traffic conditions. Terrain is classified as level or rolling, as described below. Mountainous terrain is addressed in the operational analysis of specific upgrades and downgrades, presented below. This methodology typically is applied to highway sections of at least 3.0 km.

Traffic data needed to apply the two-way segment methodology include the two-way hourly volume, a peak-hour factor (PHF), and the directional distribution of traffic flow. The PHF may be computed from field data, or appropriate default values may be selected from the tabulated values presented in Chapter 12. Traffic data also include the proportion of trucks and recreational vehicles (RVs) in the traffic stream. The operational analysis of extended two-way segments for a two-lane highway involves several steps, described in the following sections.

EXHIBIT 20-2. LOS CRITERIA FOR TWO-LANE HIGHWAYS IN CLASS I

LOS	Percent Time-Spent-Following	Average Travel Speed (km/h)
A	≤ 35	> 90
B	> 35–50	> 80–90
C	> 50–65	> 70–80
D	> 65–80	> 60–70
E	> 80	≤ 60

Note:
LOS F applies whenever the flow rate exceeds the segment capacity.

EXHIBIT 20-3. LOS CRITERIA (GRAPHICAL) FOR TWO-LANE HIGHWAYS IN CLASS I

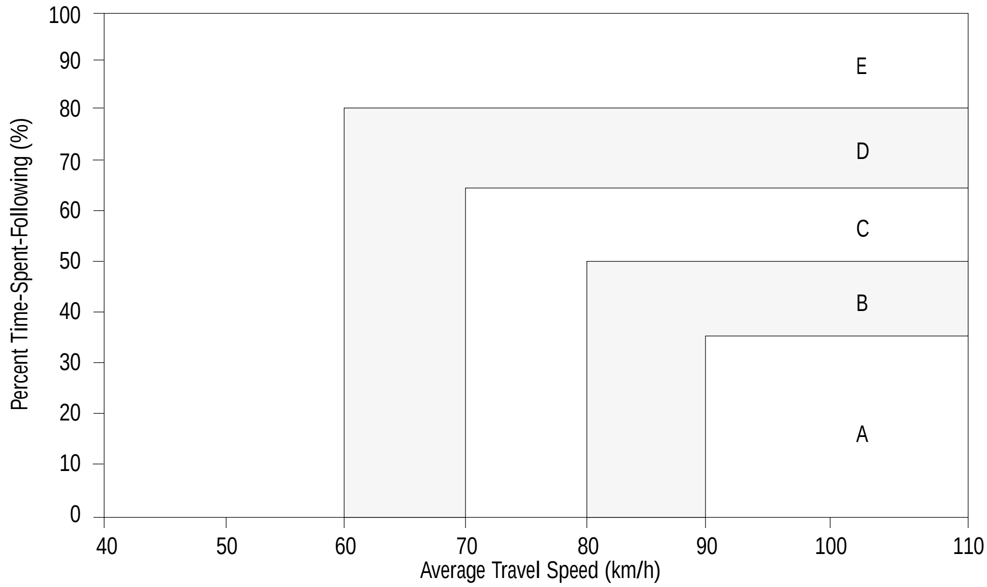


EXHIBIT 20-4. LOS CRITERIA FOR TWO-LANE HIGHWAYS IN CLASS II

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

Note:
LOS F applies whenever the flow rate exceeds the segment capacity.

Determining Free-Flow Speed

Free-flow speed occurs at two-way flows of 200 pc/h or less

A key step in the assessment of the LOS of a two-lane highway is to determine the free-flow speed (FFS). The FFS is measured using the mean speed of traffic under low flow conditions (up to two-way flows of 200 pc/h). If field measurements must be made with two-way flow rates of more than 200 pc/h, a volume adjustment must be made in determining FFS. This volume adjustment is discussed below.

Two general methods can be used to determine the FFS for a two-lane highway: field measurement and estimation with the guidelines provided in this chapter. The field-measurement procedure assists in gathering these data directly or incorporating the measurements into a speed monitoring program. However, field measurements are not necessary for an operational analysis—the FFS can be estimated from field data and user knowledge of conditions on the highway.

Field Measurement

The FFS of a highway can be determined directly from a speed study conducted in the field. No adjustments are made to the field-measured data. The speed study should be conducted at a representative location within the highway segment being evaluated; for example, a site on a short upgrade should not be selected within a segment that is generally level. Any speed measurement technique acceptable for other types of traffic engineering speed studies may be used. The field study should be conducted in periods of low traffic flow (up to a two-way flow of 200 pc/h) and should measure the speeds of all vehicles or of a systematic sampling (e.g., of every 10th vehicle). A representative sample of the speeds of at least 100 vehicles, impeded or unimpeded, should be obtained. Further guidance on speed studies is found in standard traffic engineering texts such as the *Manual of Transportation Engineering Studies* (*2*).

Speed measurements taken at flows exceeding 200 pc/h can be adjusted to FFS

If the speed study must be conducted at a two-way flow rate of more than 200 pc/h, the FFS can be found by using the speed-flow relationships shown in Chapter 12, assuming that data on traffic volumes are recorded at the same time. The FFS can be computed based on field data as shown in Equation 20-1.

$$FFS = S_{FM} + 0.0125 \frac{V_f}{f_{HV}} \tag{20-1}$$

where

FFS = estimated free-flow speed (km/h),
S_{FM} = mean speed of traffic measured in the field (km/h),
V_f = observed flow rate for the period when field data were obtained (veh/h), and
f_{HV} = heavy-vehicle adjustment factor, determined as shown in Equation 20-4.

If field measurement of the highway is not feasible, data taken at a similar facility may be used. The surrogate roadway should be similar with respect to the variables affecting FFS, which are identified in this chapter.

Highway agencies with ongoing speed-monitoring programs or with speed data on file may prefer to use those rather than conducting a new speed study or using indirect speed estimates. However, these data should be used directly only if collected in accordance with the previously described procedures.

Estimating FFS

The FFS can be estimated indirectly if field data are not available. This is a greater challenge on two-lane highways than on other types of uninterrupted-flow facilities because the FFS of a two-lane highway can range from 70 to 110 km/h. To estimate FFS, the analyst must characterize the operating conditions of the facility in terms of a base free-flow speed (BFFS) that reflects the character of traffic and the alignment of the facility. Because of the broad range of speed conditions on two-lane highways and the importance of local and regional factors that influence driver-desired speeds, no guidance on estimating the BFFS is provided. Estimates of BFFS can be developed based on speed data and local knowledge of operating conditions on similar facilities. The design speed and posted speed limit of the facility may be considered in determining the BFFS; however, the design speeds and speed limits for many facilities are not based on current operating conditions. Once BFFS is estimated, adjustments can be made for the influence of lane width, shoulder width, and access-point density. The FFS is estimated using Equation 20-2.

$$FFS = BFFS - f_{LS} - f_A \tag{20-2}$$

where

FFS = estimated FFS (km/h);
$BFFS$ = base FFS (km/h);
f_{LS} = adjustment for lane width and shoulder width, from Exhibit 20-5; and
f_A = adjustment for access points, from Exhibit 20-6.

The first adjustment to the estimated FFS relates to the effects of lane and shoulder widths. Base conditions for a two-lane highway require 3.6-m lane widths and 1.8-m shoulder widths. Exhibit 20-5 lists the adjustments to the estimated FFS for narrower lanes and shoulders. The data in Exhibit 20-5 indicate, for example, that a two-lane highway with 3.3-m lanes and full shoulder widths has an FFS that is 0.7 km/h less than a highway with base lane and shoulder widths. Similarly, a two-lane highway with 3.6-m lanes and 0.6-m shoulders has an FFS 4.2 km/h less than a highway with base lane and shoulder widths.

EXHIBIT 20-5. ADJUSTMENT (f_{LS}) FOR LANE WIDTH AND SHOULDER WIDTH

	Reduction in FFS (km/h)			
	Shoulder Width (m)			
Lane Width (m)	≥ 0.0 < 0.6	≥ 0.6 < 1.2	≥ 1.2 < 1.8	≥ 1.8
2.7 < 3.0	10.3	7.7	5.6	3.5
≥ 3.0 < 3.3	8.5	5.9	3.8	1.7
≥ 3.3 < 3.6	7.5	4.9	2.8	0.7
≥ 3.6	6.8	4.2	2.1	0.0

Exhibit 20-6 lists the adjustments for access-point density. The data indicate that each access point per kilometer decreases the estimated FFS by about 0.4 km/h. The access-point density is found by dividing the total number of intersections and driveways on both sides of the roadway segment by the length of the segment in kilometers. An intersection or driveway should only be included if it influences traffic flow; access points unnoticed by the driver or with little activity should not be included.

EXHIBIT 20-6. ADJUSTMENT (f_A) FOR ACCESS-POINT DENSITY

Access Points per km	Reduction in FFS (km/h)
0	0.0
6	4.0
12	8.0
18	12.0
≥ 24	16.0

When data on the number of access points on a two-lane highway segment are unavailable (e.g., when the highway has not yet been constructed), the guidelines in Chapter 12 may be used.

If a highway segment contains sharp horizontal curves with design speeds substantially below the rest of the segment, it may be desirable to determine the FFS separately for curves and tangents and compute a weighted-average FFS for the segment as a whole.

The data for the FFS relationships in this chapter include both commuter and noncommuter traffic. There were no significant differences between the two. However, it is expected that commuters or other regular drivers will use a facility more efficiently than recreational users and other occasional drivers. If the effect of a driver population is a concern, the FFS should be measured in the field. If field measurements cannot be made, an FFS should be selected to reflect the anticipated effect of the driver population. Care should be taken not to underestimate the BFFS of a highway by overstating the effect of a given driver population.

Determining Demand Flow Rate

Three adjustments must be made to hourly demand volumes, whether based on traffic counts or estimates, to arrive at the equivalent passenger-car flow rate used in LOS analysis. These adjustments are the PHF, the grade adjustment factor, and the heavy-vehicle adjustment factor. These adjustments are applied according to Equation 20-3.

$$v_p = \frac{V}{PHF * f_G * f_{HV}} \quad (20\text{-}3)$$

where

v_p = passenger-car equivalent flow rate for peak 15-min period (pc/h),
V = demand volume for the full peak hour (veh/h),
PHF = peak-hour factor,
f_G = grade adjustment factor, and
f_{HV} = heavy-vehicle adjustment factor.

PHF

PHF represents the variation in traffic flow within an hour. Two-lane highway analysis is based on demand volumes for a peak 15-min period within the hour of interest—usually the peak hour. For operational analysis, the full-hour demand volumes must be converted to flow rates for the peak 15 min, as shown in Equation 20-3.

Grade Adjustment Factor

The grade adjustment factor, f_G, accounts for the effect of the terrain on travel speeds and percent time-spent-following, even if no heavy vehicles are present. The values of the grade adjustment factor are listed in Exhibit 20-7 for estimating average travel speeds and in Exhibit 20-8 for estimating percent time-spent-following.

EXHIBIT 20-7. GRADE ADJUSTMENT FACTOR (f_G) TO DETERMINE SPEEDS ON TWO-WAY AND DIRECTIONAL SEGMENTS

Range of Two-Way Flow Rates (pc/h)	Range of Directional Flow Rates (pc/h)	Type of Terrain	
		Level	Rolling
0–600	0–300	1.00	0.71
> 600–1200	> 300–600	1.00	0.93
> 1200	> 600	1.00	0.99

EXHIBIT 20-8. GRADE ADJUSTMENT FACTOR (f_G) TO DETERMINE PERCENT TIME-SPENT-FOLLOWING ON TWO-WAY AND DIRECTIONAL SEGMENTS

Range of Two-Way Flow Rates (pc/h)	Range of Directional Flow Rates (pc/h)	Type of Terrain	
		Level	Rolling
0–600	0–300	1.00	0.77
> 600–1200	> 300–600	1.00	0.94
> 1200	> 600	1.00	1.00

Adjustment for Heavy Vehicles

The presence of heavy vehicles in the traffic stream decreases the FFS, because at base conditions the traffic stream is assumed to consist only of passenger cars—a rare occurrence. Therefore, traffic volumes must be adjusted to an equivalent flow rate expressed in passenger cars per hour. This adjustment is accomplished by using the factor f_{HV}.

Adjustment for the presence of heavy vehicles in the traffic stream applies to two types of vehicles: trucks and RVs. Buses should not be treated as a separate type of heavy vehicle but should be included with trucks. The heavy-vehicle adjustment factor requires two steps. First, the passenger-car equivalency factors for trucks (E_T) and RVs (E_R) for the prevailing operating conditions must be found. Then, using these values, an adjustment factor must be computed to correct for all heavy vehicles in the traffic stream.

Heavy-vehicle adjustment considers trucks and RVs. Buses are included with trucks.

Passenger-car equivalents for extended two-way segments are determined from Exhibit 20-9 for estimating speeds and from Exhibit 20-10 for estimating percent time-spent-following. The terrain of extended two-way segments should be categorized as level or rolling.

EXHIBIT 20-9. PASSENGER-CAR EQUIVALENTS FOR TRUCKS AND RVS TO DETERMINE SPEEDS ON TWO-WAY AND DIRECTIONAL SEGMENTS

Vehicle Type	Range of Two-Way Flow Rates (pc/h)	Range of Directional Flow Rates (pc/h)	Type of Terrain	
			Level	Rolling
Trucks, E_T	0–600	0–300	1.7	2.5
	> 600–1,200	> 300–600	1.2	1.9
	> 1,200	> 600	1.1	1.5
RVs, E_R	0–600	0–300	1.0	1.1
	> 600–1,200	> 300–600	1.0	1.1
	> 1,200	> 600	1.0	1.1

EXHIBIT 20-10. PASSENGER-CAR EQUIVALENTS FOR TRUCKS AND RVS TO DETERMINE PERCENT TIME-SPENT-FOLLOWING ON TWO-WAY AND DIRECTIONAL SEGMENTS

Vehicle Type	Range of Two-Way Flow Rates (pc/h)	Range of Directional Flow Rates (pc/h)	Type of Terrain	
			Level	Rolling
Trucks, E_T	0–600	0–300	1.1	1.8
	> 600–1,200	> 300–600	1.1	1.5
	> 1,200	> 600	1.0	1.0
RVs, E_R	0–600	0–300	1.0	1.0
	> 600–1,200	> 300–600	1.0	1.0
	> 1,200	> 600	1.0	1.0

Level Terrain

Level terrain is any combination of horizontal and vertical alignment permitting heavy vehicles to maintain approximately the same speed as passenger cars; this generally includes short grades of no more than 1 or 2 percent.

Rolling Terrain

Rolling terrain is any combination of horizontal and vertical alignment causing heavy vehicles to reduce their speeds substantially below those of passenger cars, but not to operate at crawl speeds for any significant length of time or at frequent intervals; generally, this includes short- and medium-length grades of no more than 4 percent. Segments with substantial lengths of more than a 4 percent grade should be analyzed with the specific grade procedure for directional segments.

Heavy-Vehicle Adjustment Factor

Once values for E_T and E_R have been determined, the adjustment factor for heavy vehicles is computed using Equation 20-4.

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)} \tag{20-4}$$

where

P_T = proportion of trucks in the traffic stream, expressed as a decimal;
P_R = proportion of RVs in the traffic stream, expressed as a decimal;

E_T = passenger-car equivalent for trucks, obtained from Exhibit 20-9 or Exhibit 20-10; and
E_R = passenger-car equivalent for RVs, obtained from Exhibit 20-9 or Exhibit 20-10.

Iterative Computations

Exhibits 20-7 through 20-10—the grade adjustment factor f_G and the passenger-car equivalents for trucks (E_T) and RVs (E_R)—are stratified by flow rates expressed in passenger cars per hour. However, until Equation 20-3 is applied, the flow rate in passenger cars per hour is not known. Therefore, an iterative approach must be applied to determine the passenger-car equivalent flow rate v_p, and from that, either average travel speed or percent time-spent-following.

First, determine the flow rate, in vehicles per hour, as V/PHF. Second, select values of f_G, E_T, and E_R appropriate for that flow rate from the tables. Then, determine the v_p from those values using Equations 20-3 and 20-4. If the computed value of v_p is less than the upper limit of the selected flow-rate range for which f_G, E_T, and E_R were determined, then the computed value of v_p should be used. If the v_p is higher than the upper limit of the selected flow-rate range, repeat the process for successively higher ranges until an acceptable value of v_p is found. Because the highest range includes all flow rates greater than 1,200 pc/h in both directions of travel combined, it can be used if a computed value exceeds the upper limit of both lower flow-rate ranges.

Determining Average Travel Speed

The average travel speed is estimated from the FFS, the demand flow rate, and an adjustment factor for the percentage of no-passing zones. The demand flow rate for estimating average travel speed is determined with Equation 20-3 using the value of f_{HV} computed with the passenger-car equivalents in Exhibit 20-9. Average travel speed is then estimated using Equation 20-5.

$$ATS = FFS - 0.0125v_p - f_{np} \tag{20-5}$$

where

ATS = average travel speed for both directions of travel combined (km/h),
f_{np} = adjustment for percentage of no-passing zones (see Exhibit 20-11), and
v_p = passenger-car equivalent flow rate for peak 15-min period (pc/h).

The FFS used in Equation 20-5 is the value estimated with Equation 20-1 or Equation 20-2. The adjustment for the effect of the percentage of no-passing zones on average travel speed (f_{np}) is listed in Exhibit 20-11. The exhibit shows that the effect of no-passing zones on average travel speed increases to a maximum at a two-way flow rate of 400 pc/h and then decreases at higher volumes. The maximum value of f_{np} is 7.3 km/h.

Determining Percent Time-Spent-Following

The percent time-spent-following is estimated from the demand flow rate, the directional distribution of traffic, and the percentage of no-passing zones. The demand flow rate (v_p) for estimating percent time-spent-following is determined with Equation 20-3 using the value of f_{HV} computed with passenger-car equivalents from Exhibit 20-10. Percent time-spent-following is then estimated using Equation 20-6. Appropriate values of base percent-time-spent following can be determined from Equation 20-7.

$$PTSF = BPTSF + f_{d/np} \tag{20-6}$$

where

$PTSF$ = percent-time-spent following,

BPTSF = base percent time-spent-following for both directions of travel combined (use Equation 20-7), and

$f_{d/np}$ = adjustment for the combined effect of the directional distribution of traffic and of the percentage of no-passing zones on percent time-spent-following.

$$BPTSF = 100\left(1 - e^{-0.000879 v_p}\right) \qquad (20\text{-}7)$$

An adjustment representing the combined effect of directional distribution of traffic and percentage of no-passing zones ($f_{d/np}$) is presented in Exhibit 20-12.

EXHIBIT 20-11. ADJUSTMENT (f_{np}) FOR EFFECT OF NO-PASSING ZONES ON AVERAGE TRAVEL SPEED ON TWO-WAY SEGMENTS

Two-Way Demand Flow Rate, v_p (pc/h)	Reduction in Average Travel Speed (km/h)					
	No-Passing Zones (%)					
	0	20	40	60	80	100
0	0.0	0.0	0.0	0.0	0.0	0.0
200	0.0	1.0	2.3	3.8	4.2	5.6
400	0.0	2.7	4.3	5.7	6.3	7.3
600	0.0	2.5	3.8	4.9	5.5	6.2
800	0.0	2.2	3.1	3.9	4.3	4.9
1000	0.0	1.8	2.5	3.2	3.6	4.2
1200	0.0	1.3	2.0	2.6	3.0	3.4
1400	0.0	0.9	1.4	1.9	2.3	2.7
1600	0.0	0.9	1.3	1.7	2.1	2.4
1800	0.0	0.8	1.1	1.6	1.8	2.1
2000	0.0	0.8	1.0	1.4	1.6	1.8
2200	0.0	0.8	1.0	1.4	1.5	1.7
2400	0.0	0.8	1.0	1.3	1.5	1.7
2600	0.0	0.8	1.0	1.3	1.4	1.6
2800	0.0	0.8	1.0	1.2	1.3	1.4
3000	0.0	0.8	0.9	1.1	1.1	1.3
3200	0.0	0.8	0.9	1.0	1.0	1.1

Determining LOS

The first step in determining LOS is to compare the passenger-car equivalent flow rate (v_p) to the two-way capacity of 3,200 pc/h. If v_p is greater than the capacity, then the roadway is oversaturated and the LOS is F. Similarly, if the demand flow rate in either direction of travel—as determined from the two-way flow rate and the directional split—is greater than 1,700 pc/h, then the roadway is oversaturated and the LOS is F. In LOS F, percent time-spent-following is nearly 100 percent and speeds are highly variable and difficult to estimate.

When a segment of a Class I facility has a demand less than its capacity, the LOS is determined by locating a point on Exhibit 20-3 that corresponds to the estimated percent time-spent-following and average travel speed. If a segment of a Class II facility has a demand less than its capacity, the LOS is determined by comparing the percent time-spent-following with the criteria in Exhibit 20-4. The analysis should include the LOS and the estimated values of percent time-spent-following and average travel speed. Although average travel speed is not considered in the LOS determination for a Class II highway, the estimate may be useful in evaluating the quality of service of two-lane highway facilities, highway networks, or systems including the segment.

EXHIBIT 20-12. ADJUSTMENT ($f_{d/np}$) FOR COMBINED EFFECT OF DIRECTIONAL DISTRIBUTION OF TRAFFIC AND PERCENTAGE OF NO-PASSING ZONES ON PERCENT TIME-SPENT-FOLLOWING ON TWO-WAY SEGMENTS

	Increase in Percent Time-Spent-Following (%)					
	No-Passing Zones (%)					
Two-Way Flow Rate, v_p (pc/h)	0	20	40	60	80	100
Directional Split = 50/50						
≤ 200	0.0	10.1	17.2	20.2	21.0	21.8
400	0.0	12.4	19.0	22.7	23.8	24.8
600	0.0	11.2	16.0	18.7	19.7	20.5
800	0.0	9.0	12.3	14.1	14.5	15.4
1400	0.0	3.6	5.5	6.7	7.3	7.9
2000	0.0	1.8	2.9	3.7	4.1	4.4
2600	0.0	1.1	1.6	2.0	2.3	2.4
3200	0.0	0.7	0.9	1.1	1.2	1.4
Directional Split = 60/40						
≤ 200	1.6	11.8	17.2	22.5	23.1	23.7
400	0.5	11.7	16.2	20.7	21.5	22.2
600	0.0	11.5	15.2	18.9	19.8	20.7
800	0.0	7.6	10.3	13.0	13.7	14.4
1400	0.0	3.7	5.4	7.1	7.6	8.1
2000	0.0	2.3	3.4	3.6	4.0	4.3
≥ 2600	0.0	0.9	1.4	1.9	2.1	2.2
Directional Split = 70/30						
≤ 200	2.8	13.4	19.1	24.8	25.2	25.5
400	1.1	12.5	17.3	22.0	22.6	23.2
600	0.0	11.6	15.4	19.1	20.0	20.9
800	0.0	7.7	10.5	13.3	14.0	14.6
1400	0.0	3.8	5.6	7.4	7.9	8.3
≥ 2000	0.0	1.4	4.9	3.5	3.9	4.2
Directional Split = 80/20						
≤ 200	5.1	17.5	24.3	31.0	31.3	31.6
400	2.5	15.8	21.5	27.1	27.6	28.0
600	0.0	14.0	18.6	23.2	23.9	24.5
800	0.0	9.3	12.7	16.0	16.5	17.0
1400	0.0	4.6	6.7	8.7	9.1	9.5
≥ 2000	0.0	2.4	3.4	4.5	4.7	4.9
Directional Split = 90/10						
≤ 200	5.6	21.6	29.4	37.2	37.4	37.6
400	2.4	19.0	25.6	32.2	32.5	32.8
600	0.0	16.3	21.8	27.2	27.6	28.0
800	0.0	10.9	14.8	18.6	19.0	19.4
≥1400	0.0	5.5	7.8	10.0	10.4	10.7

Other Traffic Performance Measures

The v/c ratio for an extended two-way segment can be computed using Equation 20-8.

$$v/c = \frac{v_p}{c} \qquad (20\text{-}8)$$

where

v/c = volume to capacity ratio;
c = two-way segment capacity—normally 3,200 pc/h for two-way segment and 1,700 for a directional segment; and
v_p = passenger-car equivalent flow rate for peak 15-min period (pc/h).

The total travel on the extended two-way segment during the peak 15-min period is computed using Equation 20-9.

$$VkmT_{15} = 0.25\left(\frac{V}{PHF}\right)L_t \quad (20\text{-}9)$$

where

$VkmT_{15}$ = total travel on the analysis segment during the peak 15-min period (veh-km), and
L_t = total length of the analysis segment (km).

The total travel on the two-way segment during the peak hour is computed using Equation 20-10.

$$VkmT_{60} = V * L_t \quad (20\text{-}10)$$

where

$VkmT_{60}$ = total travel on the analysis segment during the peak hour (veh-km).

Equation 20-11 can be used to compute the total travel time during the peak 15-min period using Equations 20-5 and 20-9.

$$TT_{15} = \frac{VkmT_{15}}{ATS} \quad (20\text{-}11)$$

where

TT_{15} = total travel time for all vehicles on the analyzed segment during the peak 15-min period (veh-h).

DIRECTIONAL SEGMENTS

The methodology addresses three types of directional segments: extended directional segments, specific upgrades, and specific downgrades. The methodology for directional segments is analogous to the two-way segment methodology, except that it estimates traffic performance measures and LOS for one direction of travel at a time. However, the operational assessment of one direction of travel on a two-lane highway necessarily considers the opposing traffic volume. There is a strong interaction between the directions of travel on a two-lane highway because passing opportunities are reduced and eventually eliminated as the opposing traffic increases.

Directional analyses usually are applied to segments ≥3 km, or to grades ≥3 percent and ≥1 km in length

The directional segment methodology applies on level or rolling terrain, usually to highway sections of at least 3.0 km. Any grade of 3 percent or more and at least 1.0 km long must be addressed with the procedures for specific upgrades and downgrades. Mountainous terrain is addressed through an analysis of individual upgrades and downgrades. The specific upgrade and downgrade procedures differ from the extended segment procedure primarily in the handling of heavy-vehicle effects.

The basic directional segment methodology applies to segments on highways with one lane in each direction. However, there is a supplementary procedure to estimate the operational effect of an added passing lane within a directional segment. The operational analysis of a directional segment on a two-lane highway involves several steps, described below.

Determining FFS

The first step in the analysis of a directional segment is to determine FFS, using either of the methods for extended two-way segments. These methods should be applied on a directional basis rather than to both directions combined. If the FFS for a particular direction of travel is determined in the field, it should be under conditions of low traffic flow in both directions.

Determining Demand Flow Rate

The demand flow rate for the peak 15-min period in the direction analyzed is determined with Equation 20-12, which is analogous to Equation 20-3.

$$v_d = \frac{V}{PHF * f_G * f_{HV}} \qquad (20\text{-}12)$$

where

v_d = passenger-car equivalent flow rate for the peak 15-min period in the direction analyzed (pc/h),
V = demand volume for the full peak hour in the direction analyzed (veh/h),
f_G = grade adjustment factor, and
f_{HV} = heavy-vehicle adjustment factor.

This demand flow rate should be based on the PHF, the traffic composition, and the terrain or actual grade in the specific direction of travel. As in the two-way segment procedure, different values of v_d are used for estimating average travel speed and percent time-spent-following, because the value of f_{HV} will differ for these applications.

Directional analysis also requires consideration of the demand flow rate in the opposing direction. The opposing demand flow rate is computed using Equation 20-13, which is analogous to Equation 20-12.

$$v_o = \frac{V_o}{PHF * f_G * f_{HV}} \qquad (20\text{-}13)$$

where

v_o = passenger-car equivalent flow rate for the peak 15-min period in the opposing direction of travel, and
V_o = demand volume for the full peak hour in the opposing direction of travel.

The values of PHF and f_{HV} used in Equation 20-13 also should apply to the opposing direction of travel.

PHF

The PHF used in the directional segment procedure should be the same as that applied to a single direction of travel. If possible, the PHF should be determined from local field data, but if field data are not available, the default values given in Chapter 12 can be used.

Adjustments for Grade and Heavy Vehicles

The adjustment for the presence of heavy vehicles in directional segments is analogous to that for two-way segments in that the passenger-car equivalents for trucks (E_T) and RVs (E_R) are determined and used together with the proportions of trucks and RVs in Equation 20-4. However, the procedures for determining the values of E_T and E_R differ for extended directional segments, specific upgrades, and specific downgrades.

The values of E_T and E_R for an extended directional segment in level or rolling terrain are determined from Exhibits 20-9 and 20-10, based on the methodology for two-

Analysis of upgrades is only for segments with grades ≥ 3 percent and ≥ 0.4 km in length

way segments. For directional segments, the value of the grade adjustment factor, f_G, is given in Exhibits 20-7 and 20-8.

Any upgrade of 3 percent or more and a length of 0.4 km or more may be analyzed as a specific upgrade; however, any upgrade of 3 percent or more and a length of 1.0 km or more must be analyzed as a specific upgrade. This includes all upgrades on directional segments in mountainous terrain. If the grade varies, it should be analyzed as a single composite, using an average computed by dividing the total change in elevation by the total length of grade and expressing the result as a percentage.

The values of the grade adjustment factor f_G, used in estimating average travel speed for specific upgrades, are presented in Exhibit 20-13. The f_G for estimating percent time-spent-following on specific upgrades is presented in Exhibit 20-14. The grade adjustment factor accounts for the effect of the grade on average travel speeds and percent time-spent-following in a traffic stream composed entirely of passenger cars.

Passenger-car equivalents (E_T) for trucks used in estimating average travel speed and percent time-spent-following are presented in Exhibits 20-15 and 20-16, respectively. These factors account for the effect of trucks on average travel speed and percent time-spent-following on the specific upgrade, over and above the effect of the grade on passenger cars.

Exhibit 20-17 presents passenger-car equivalents (E_R) for RVs for estimating average travel speed on a specific upgrade. For estimating percent time-spent-following on specific upgrades, E_R is always 1.0, as shown in Exhibit 20-16.

Analysis of downgrades

Any downgrade of 3 percent or more and a length of 1.0 km or more must be analyzed as a specific downgrade. This includes all downgrades on directional segments in mountainous terrain. If the grade of a downgrade varies, it should be analyzed as a single composite using an average computed by dividing the total change in elevation by the total length of grade and expressing the result as a percentage. Because the definitions of specific upgrades and downgrades are similar, the opposing direction of any specific upgrade should be analyzed as a specific downgrade.

For most specific downgrades, the grade adjustment factor f_G is 1.0, and the heavy-vehicle adjustment factor f_{HV} is determined with passenger-car equivalencies from Exhibits 20-9 and 20-10. Some specific downgrades are long and steep enough that some heavy vehicles must travel at crawl speeds to avoid loss of control. This, of course, impedes other vehicles, increases percent time-spent-following, and decreases average travel speed. When this occurs, the heavy-vehicle adjustment factor f_{HV}, used to determine average travel speed, should be based on Equation 20-14 rather than on Equation 20-4.

$$f_{HV} = \frac{1}{1 + P_{TC} * P_T (E_{TC} - 1) + (1 - P_{TC}) P_T (E_T - 1) + P_R (E_R - 1)} \qquad (20\text{-}14)$$

where

P_{TC} = proportion (expressed as a decimal) of all trucks in the traffic stream using crawl speeds on the specific downgrade, and

E_{TC} = passenger-car equivalent for trucks using crawl speeds, obtained from Exhibit 20-18.

In applying Equation 20-14, the passenger-car equivalent for trucks that use crawl speeds (E_{TC}) should be determined from Exhibit 20-18, based on the directional flow rate and the difference between the FFS and the truck crawl speed. The passenger-car equivalents for other trucks (E_T) and RVs (E_R) should be the values for level terrain in Exhibit 20-9. If more specific data are not available, the proportion of all trucks that use crawl speeds can be estimated as equal to the proportion of all trucks that are tractor-trailer combinations.

EXHIBIT 20-13. GRADE ADJUSTMENT FACTOR (f_G) FOR ESTIMATING AVERAGE TRAVEL SPEED ON SPECIFIC UPGRADES

		Grade Adjustment Factor, f_G		
		Range of Directional Flow Rates v_d (pc/h)		
Grade (%)	Length of Grade (km)	0–300	> 300–600	> 600
≥ 3.0 < 3.5	0.4	0.81	1.00	1.00
	0.8	0.79	1.00	1.00
	1.2	0.77	1.00	1.00
	1.6	0.76	1.00	1.00
	2.4	0.75	0.99	1.00
	3.2	0.75	0.97	1.00
	4.8	0.75	0.95	0.97
	≥ 6.4	0.75	0.94	0.95
≥ 3.5 < 4.5	0.4	0.79	1.00	1.00
	0.8	0.76	1.00	1.00
	1.2	0.72	1.00	1.00
	1.6	0.69	0.93	1.00
	2.4	0.68	0.92	1.00
	3.2	0.66	0.91	1.00
	4.8	0.65	0.91	0.96
	≥ 6.4	0.65	0.90	0.96
≥ 4.5 < 5.5	0.4	0.75	1.00	1.00
	0.8	0.65	0.93	1.00
	1.2	0.60	0.89	1.00
	1.6	0.59	0.89	1.00
	2.4	0.57	0.86	0.99
	3.2	0.56	0.85	0.98
	4.8	0.56	0.84	0.97
	≥ 6.4	0.55	0.82	0.93
≥ 5.5 < 6.5	0.4	0.63	0.91	1.00
	0.8	0.57	0.85	0.99
	1.2	0.52	0.83	0.97
	1.6	0.51	0.79	0.97
	2.4	0.49	0.78	0.95
	3.2	0.48	0.78	0.94
	4.8	0.46	0.76	0.93
	≥ 6.4	0.45	0.76	0.93
≥ 6.5	0.4	0.59	0.86	0.98
	0.8	0.48	0.76	0.94
	1.2	0.44	0.74	0.91
	1.6	0.41	0.70	0.91
	2.4	0.40	0.67	0.91
	3.2	0.39	0.67	0.89
	4.8	0.39	0.66	0.88
	≥ 6.4	0.38	0.66	0.87

EXHIBIT 20-14. GRADE ADJUSTMENT FACTOR (f_G) FOR ESTIMATING PERCENT TIME-SPENT-FOLLOWING ON SPECIFIC UPGRADES

		Grade Adjustment Factor, f_G		
		Range of Directional Flow Rates, v_d (pc/h)		
Grade (%)	Length of Grade (km)	0–300	> 300–600	> 600
≥ 3.0 < 3.5	0.4	1.00	0.92	0.92
	0.8	1.00	0.93	0.93
	1.2	1.00	0.93	0.93
	1.6	1.00	0.93	0.93
	2.4	1.00	0.94	0.94
	3.2	1.00	0.95	0.95
	4.8	1.00	0.97	0.96
	≥ 6.4	1.00	1.00	0.97
≥ 3.5 < 4.5	0.4	1.00	0.94	0.92
	0.8	1.00	0.97	0.96
	1.2	1.00	0.97	0.96
	1.6	1.00	0.97	0.97
	2.4	1.00	0.97	0.97
	3.2	1.00	0.98	0.98
	4.8	1.00	1.00	1.00
	≥ 6.4	1.00	1.00	1.00
≥ 4.5 < 5.5	0.4	1.00	1.00	0.97
	0.8	1.00	1.00	1.00
	1.2	1.00	1.00	1.00
	1.6	1.00	1.00	1.00
	2.4	1.00	1.00	1.00
	3.2	1.00	1.00	1.00
	4.8	1.00	1.00	1.00
	≥ 6.4	1.00	1.00	1.00
≥ 5.5 < 6.5	0.4	1.00	1.00	1.00
	0.8	1.00	1.00	1.00
	1.2	1.00	1.00	1.00
	1.6	1.00	1.00	1.00
	2.4	1.00	1.00	1.00
	3.2	1.00	1.00	1.00
	4.8	1.00	1.00	1.00
	≥ 6.4	1.00	1.00	1.00
≥ 6.5	0.4	1.00	1.00	1.00
	0.8	1.00	1.00	1.00
	1.2	1.00	1.00	1.00
	1.6	1.00	1.00	1.00
	2.4	1.00	1.00	1.00
	3.2	1.00	1.00	1.00
	4.8	1.00	1.00	1.00
	≥ 6.4	1.00	1.00	1.00

EXHIBIT 20-15. PASSENGER-CAR EQUIVALENTS FOR TRUCKS FOR ESTIMATING AVERAGE SPEED ON SPECIFIC UPGRADES

		Passenger-Car Equivalent for Trucks, E_T		
		Range of Directional Flow Rates, v_d (pc/h)		
Grade (%)	Length of Grade (km)	0–300	> 300–600	> 600
≥ 3.0 < 3.5	0.4	2.5	1.9	1.5
	0.8	3.5	2.8	2.3
	1.2	4.5	3.9	2.9
	1.6	5.1	4.6	3.5
	2.4	6.1	5.5	4.1
	3.2	7.1	5.9	4.7
	4.8	8.2	6.7	5.3
	≥ 6.4	9.1	7.5	5.7
≥ 3.5 < 4.5	0.4	3.6	2.4	1.9
	0.8	5.4	4.6	3.4
	1.2	6.4	6.6	4.6
	1.6	7.7	6.9	5.9
	2.4	9.4	8.3	7.1
	3.2	10.2	9.6	8.1
	4.8	11.3	11.0	8.9
	≥ 6.4	12.3	11.9	9.7
≥ 4.5 < 5.5	0.4	4.2	3.7	2.6
	0.8	6.0	6.0	5.1
	1.2	7.5	7.5	7.5
	1.6	9.2	9.0	8.9
	2.4	10.6	10.5	10.3
	3.2	11.8	11.7	11.3
	4.8	13.7	13.5	12.4
	≥ 6.4	15.3	15.0	12.5
≥ 5.5 < 6.5	0.4	4.7	4.1	3.5
	0.8	7.2	7.2	7.2
	1.2	9.1	9.1	9.1
	1.6	10.3	10.3	10.2
	2.4	11.9	11.8	11.7
	3.2	12.8	12.7	12.6
	4.8	14.4	14.3	14.2
	≥ 6.4	15.4	15.2	15.0
≥ 6.5	0.4	5.1	4.8	4.6
	0.8	7.8	7.8	7.8
	1.2	9.8	9.8	9.8
	1.6	10.4	10.4	10.3
	2.4	12.0	11.9	11.8
	3.2	12.9	12.8	12.7
	4.8	14.5	14.4	14.3
	≥ 6.4	15.4	15.3	15.2

EXHIBIT 20-16. PASSENGER-CAR EQUIVALENTS FOR TRUCKS AND RVS FOR ESTIMATING PERCENT TIME-SPENT-FOLLOWING ON SPECIFIC UPGRADES

		Passenger-Car Equivalent for Trucks, E_T			
		Range of Directional Flow Rates, v_d (pc/h)			
Grade (%)	Length of Grade (km)	0–300	> 300–600	> 600	RVs, E_R
≥ 3.0 < 3.5	0.4	1.0	1.0	1.0	1.0
	0.8	1.0	1.0	1.0	1.0
	1.2	1.0	1.0	1.0	1.0
	1.6	1.0	1.0	1.0	1.0
	2.4	1.0	1.0	1.0	1.0
	3.2	1.0	1.0	1.0	1.0
	4.8	1.4	1.0	1.0	1.0
	≥ 6.4	1.5	1.0	1.0	1.0
≥ 3.5 < 4.5	0.4	1.0	1.0	1.0	1.0
	0.8	1.0	1.0	1.0	1.0
	1.2	1.0	1.0	1.0	1.0
	1.6	1.0	1.0	1.0	1.0
	2.4	1.1	1.0	1.0	1.0
	3.2	1.4	1.0	1.0	1.0
	4.8	1.7	1.1	1.2	1.0
	≥ 6.4	2.0	1.5	1.4	1.0
≥ 4.5 < 5.5	0.4	1.0	1.0	1.0	1.0
	0.8	1.0	1.0	1.0	1.0
	1.2	1.0	1.0	1.0	1.0
	1.6	1.0	1.0	1.0	1.0
	2.4	1.1	1.2	1.2	1.0
	3.2	1.6	1.3	1.5	1.0
	4.8	2.3	1.9	1.7	1.0
	≥ 6.4	3.3	2.1	1.8	1.0
≥ 5.5 < 6.5	0.4	1.0	1.0	1.0	1.0
	0.8	1.0	1.0	1.0	1.0
	1.2	1.0	1.0	1.0	1.0
	1.6	1.0	1.2	1.2	1.0
	2.4	1.5	1.6	1.6	1.0
	3.2	1.9	1.9	1.8	1.0
	4.8	3.3	2.5	2.0	1.0
	≥ 6.4	4.3	3.1	2.0	1.0
≥ 6.5	0.4	1.0	1.0	1.0	1.0
	0.8	1.0	1.0	1.0	1.0
	1.2	1.0	1.0	1.3	1.0
	1.6	1.3	1.4	1.6	1.0
	2.4	2.1	2.0	2.0	1.0
	3.2	2.8	2.5	2.1	1.0
	4.8	4.0	3.1	2.2	1.0
	≥ 6.4	4.8	3.5	2.3	1.0

EXHIBIT 20-17. PASSENGER-CAR EQUIVALENTS FOR RVS FOR ESTIMATING AVERAGE TRAVEL SPEED ON SPECIFIC UPGRADES

		Passenger-Car Equivalent for RVs, E_R		
		Range of Directional Flow Rates, v_d(pc/h)		
Grade (%)	Length of Grade (km)	0–300	> 300–600	> 600
≥ 3.0 < 3.5	0.4	1.1	1.0	1.0
	0.8	1.2	1.0	1.0
	1.2	1.2	1.0	1.0
	1.6	1.3	1.0	1.0
	2.4	1.4	1.0	1.0
	3.2	1.4	1.0	1.0
	4.8	1.5	1.0	1.0
	≥ 6.4	1.5	1.0	1.0
≥ 3.5 < 4.5	0.4	1.3	1.0	1.0
	0.8	1.3	1.0	1.0
	1.2	1.3	1.0	1.0
	1.6	1.4	1.0	1.0
	2.4	1.4	1.0	1.0
	3.2	1.4	1.0	1.0
	4.8	1.4	1.0	1.0
	≥ 6.4	1.5	1.0	1.0
≥ 4.5 < 5.5	0.4	1.5	1.0	1.0
	0.8	1.5	1.0	1.0
	1.2	1.5	1.0	1.0
	1.6	1.5	1.0	1.0
	2.4	1.5	1.0	1.0
	3.2	1.5	1.0	1.0
	4.8	1.6	1.0	1.0
	≥ 6.4	1.6	1.0	1.0
≥ 5.5 < 6.5	0.4	1.5	1.0	1.0
	0.8	1.5	1.0	1.0
	1.2	1.5	1.0	1.0
	1.6	1.6	1.0	1.0
	2.4	1.6	1.0	1.0
	3.2	1.6	1.0	1.0
	4.8	1.6	1.2	1.0
	≥ 6.4	1.6	1.5	1.2
≥ 6.5	0.4	1.6	1.0	1.0
	0.8	1.6	1.0	1.0
	1.2	1.6	1.0	1.0
	1.6	1.6	1.0	1.0
	2.4	1.6	1.0	1.0
	3.2	1.6	1.0	1.0
	4.8	1.6	1.3	1.3
	≥ 6.4	1.6	1.5	1.4

EXHIBIT 20-18. PASSENGER-CAR EQUIVALENTS FOR ESTIMATING THE EFFECT ON AVERAGE TRAVEL SPEED OF TRUCKS THAT OPERATE AT CRAWL SPEEDS ON LONG STEEP DOWNGRADES

Difference Between FFS and Truck Crawl Speed (km/h)	Passenger-Car Equivalent for Trucks at Crawl Speeds, E_{TC}		
	Range of Directional Flow Rates, v_d (pc/h)		
	0–300	> 300–600	> 600
≤ 20	4.4	2.8	1.4
40	14.3	9.6	5.7
≥ 60	34.1	23.1	13.0

Iterative Computations

As with the two-way segment procedure, Equations 20-12 and 20-13 must be applied iteratively in some situations to determine appropriate values of v_d and v_o. This iterative process for directional segments is analogous to that for two-way segments, but with the following differences:

- For extended segments in level and rolling terrain and for specific downgrades, the directional flow rates from Exhibits 20-7 through 20-10 are used instead of the two-way rates;
- For specific upgrades, Exhibits 20-13 through 20-17 are used instead of Exhibits 20-7 through 20-10; and
- For specific downgrades on which some trucks travel at crawl speeds, Equation 20-14 is used instead of Equation 20-4.

Determining Average Travel Speed

The average travel speed is estimated from the FFS, the demand flow rate, the opposing flow rate, and an adjustment factor for the percentage of no-passing zones in the analysis direction. Average travel speed is then estimated using Equation 20-15.

$$ATS_d = FFS_d - 0.0125(v_d + v_o) - f_{np} \qquad (20\text{-}15)$$

where

ATS_d = average travel speed in the analysis direction (km/h),
FFS_d = free-flow speed in the analysis direction (km/h),
v_d = passenger-car equivalent flow rate for the peak 15-min period in the analysis direction (pc/h),
v_o = passenger-car equivalent flow rate for the peak 15-min period in the opposing direction (pc/h), determined from Equation 20-13; and
f_{np} = adjustment for percentage of no-passing zones in the analysis direction (see Exhibit 20-19).

The term containing v_d and v_o in Equation 20-15 represents the relationship between average travel speed and the directional and opposing flow rates presented in Chapter 12. The adjustment f_{np} accounts for the effect of the percentage of no-passing zones in the analysis direction. As shown in Exhibit 20-19, this effect is greatest when opposing flow rates are low; as the opposing flow rates increase, the effect decreases to zero, since passing and no-passing zones become irrelevant if the opposing flow allows no opportunities to pass.

EXHIBIT 20-19. ADJUSTMENT (f_{np}) TO AVERAGE TRAVEL SPEED FOR PERCENTAGE OF NO-PASSING ZONES IN DIRECTIONAL SEGMENTS

Opposing Demand Flow Rate, v_o (pc/h)	No-Passing Zones (%)				
	≤ 20	40	60	80	100
FFS = 110 km/h					
≤ 100	1.7	3.5	4.5	4.8	5.0
200	3.5	5.3	6.2	6.5	6.8
400	2.6	3.7	4.4	4.5	4.7
600	2.2	2.4	2.8	3.1	3.3
800	1.1	1.6	2.0	2.2	2.4
1000	1.0	1.3	1.7	1.8	1.9
1200	0.9	1.3	1.5	1.6	1.7
1400	0.9	1.2	1.4	1.4	1.5
≥ 1600	0.9	1.1	1.2	1.2	1.3
FFS = 100 km/h					
≤ 100	1.2	2.7	4.0	4.5	4.7
200	3.0	4.6	5.9	6.4	6.7
400	2.3	3.3	4.1	4.4	4.6
600	1.8	2.1	2.6	3.0	3.2
800	0.9	1.4	1.8	2.1	2.3
1000	0.9	1.1	1.5	1.7	1.9
1200	0.8	1.1	1.4	1.5	1.7
1400	0.8	1.0	1.3	1.3	1.4
≥ 1600	0.8	1.0	1.1	1.1	1.2
FFS = 90 km/h					
≤ 100	0.8	1.9	3.6	4.2	4.4
200	2.4	3.9	5.6	6.3	6.6
400	2.1	3.0	3.8	4.3	4.5
600	1.4	1.8	2.5	2.9	3.1
800	0.8	1.1	1.7	2.0	2.2
1000	0.8	0.9	1.3	1.5	1.8
1200	0.8	0.9	1.2	1.4	1.6
1400	0.8	0.9	1.1	1.2	1.4
≥ 1600	0.8	0.8	0.9	0.9	1.1
FFS = 80 km/h					
≤ 100	0.3	1.1	3.1	3.9	4.1
200	1.9	3.2	5.3	6.2	6.5
400	1.8	2.6	3.5	4.2	4.4
600	1.0	1.5	2.3	2.8	3.0
800	0.6	0.9	1.5	1.9	2.1
1000	0.6	0.7	1.1	1.4	1.8
1200	0.6	0.7	1.1	1.3	1.6
1400	0.6	0.7	1.0	1.1	1.3
≥ 1600	0.6	0.7	0.8	0.8	1.0
FFS = 70 km/h					
≤ 100	0.1	0.6	2.7	3.6	3.8
200	1.5	2.6	5.0	6.1	6.4
400	1.5	0.8	3.2	4.1	4.3
600	0.7	0.5	2.1	2.7	2.9
800	0.5	0.5	1.3	1.8	2.0
1000	0.5	0.5	1.0	1.3	1.8
1200	0.5	0.5	1.0	1.2	1.6
1400	0.5	0.5	1.0	1.0	1.2
≥ 1600	0.5	0.5	0.7	0.7	0.9

Determining Percent Time-Spent-Following

The percent time-spent-following is estimated from the demand flow rate, the opposing flow rate, and an adjustment factor for the percentage of no-passing zones in the analysis direction. Percent time-spent-following is estimated using Equation 20-16.

$$PTSF_d = BPTSF_d + f_{np} \qquad (20\text{-}16)$$

where

$PTSF_d$ = percent time-spent-following in the direction analyzed,
$BPTSF_d$ = base percent time-spent-following in the direction analyzed, and
f_{np} = adjustment for percentage of no-passing zones in the analysis direction (see Exhibit 20-20).

The percent time-spent-following for base conditions under actual traffic volumes in the direction analyzed in Equation 20-16 is estimated by Equation 20-17.

$$BPTSF_d = 100\left(1 - e^{a v_d^{\,b}}\right) \qquad (20\text{-}17)$$

The values of the coefficients a and b in Equation 20-17 are determined from the flow rate in the opposing direction of travel, as shown in Exhibit 20-21.

The adjustment f_{np} in Equation 20-16 accounts for the effect of the percentage of no-passing zones in the analysis direction. This effect, shown in Exhibit 20-20, is greatest at low opposing flow rates and decreases as the opposing flow rate increases, since passing and no-passing zones become irrelevant if the opposing flow rate is so high that there are no opportunities to pass.

Determining LOS

The first step in determining level of service is to compare the passenger-car equivalent flow rate (v_d) to the roadway capacity of 1,700 pc/h. If v_d is greater than the capacity, then the roadway is oversaturated and the LOS is F. In LOS F, percent time-spent-following is nearly 100 percent and speeds are highly variable and difficult to estimate.

For a segment on a Class I facility with demand less than capacity, the LOS is determined by locating the point corresponding to the estimated percent time-spent-following and average travel speed in Exhibit 20-3. For a segment on a Class II facility with demand less than capacity, the LOS is determined by comparing the directional percent time-spent-following to the criteria in Exhibit 20-4. The reported results of the analysis should include the LOS and the estimated values of percent time-spent-following and average travel speed. Although average travel speed is not considered in the LOS determination for a Class II roadway, the estimate of average travel speed may be useful in evaluating the quality of service of two-lane highway facilities, highway networks, or systems of which the roadway segment is a part.

Other Traffic Performance Measures

Other traffic performance measures, including v/c ratio, total travel, and total travel time, can be determined from Equations 20-8 through 20-11, but using directional volumes, flow rates, and speeds, rather than their two-way equivalents.

DIRECTIONAL SEGMENTS WITH PASSING LANES

The procedure should not be applied to mountainous terrain or specific upgrades

Providing a passing lane on a two-lane highway in level or rolling terrain has an effect on its LOS; an operational analysis procedure allows this effect to be estimated. This procedure, however, does not address added lanes in mountainous terrain or on specific upgrades, which are known as climbing lanes. A separate operational analysis procedure for climbing lanes is presented later in this chapter.

EXHIBIT 20-20. ADJUSTMENT (f_{np}) TO PERCENT TIME-SPENT-FOLLOWING FOR PERCENTAGE OF NO-PASSING ZONES IN DIRECTIONAL SEGMENTS

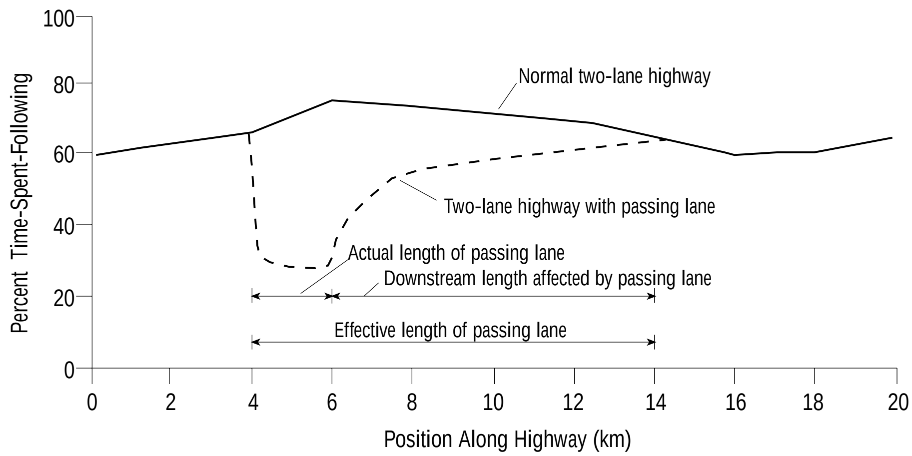
Opposing Demand Flow Rate, v_o (pc/h)	No-Passing Zones (%) ≤ 20	40	60	80	100
FFS = 110 km/h					
≤ 100	10.1	17.2	20.2	21.0	21.8
200	12.4	19.0	22.7	23.8	24.8
400	9.0	12.3	14.1	14.4	15.4
600	5.3	7.7	9.2	9.7	10.4
800	3.0	4.6	5.7	6.2	6.7
1000	1.8	2.9	3.7	4.1	4.4
1200	1.3	2.0	2.6	2.9	3.1
1400	0.9	1.4	1.7	1.9	2.1
≥ 1600	0.7	0.9	1.1	1.2	1.4
FFS = 100 km/h					
≤ 100	8.4	14.9	20.9	22.8	26.6
200	11.5	18.2	24.1	26.2	29.7
400	8.6	12.1	14.8	15.9	18.1
600	5.1	7.5	9.6	10.6	12.1
800	2.8	4.5	5.9	6.7	7.7
1000	1.6	2.8	3.7	4.3	4.9
1200	1.2	1.9	2.6	3.0	3.4
1400	0.8	1.3	1.7	2.0	2.3
≥ 1600	0.6	0.9	1.1	1.2	1.5
FFS = 90 km/h					
≤ 100	6.7	12.7	21.7	24.5	31.3
200	10.5	17.5	25.4	28.6	34.7
400	8.3	11.8	15.5	17.5	20.7
600	4.9	7.3	10.0	11.5	13.9
800	2.7	4.3	6.1	7.2	8.8
1000	1.5	2.7	3.8	4.5	5.4
1200	1.0	1.8	2.6	3.1	3.8
1400	0.7	1.2	1.7	2.0	2.4
≥ 1600	0.6	0.9	1.2	1.3	1.5
FFS = 80 km/h					
≤ 100	5.0	10.4	22.4	26.3	36.1
200	9.6	16.7	26.8	31.0	39.6
400	7.9	11.6	16.2	19.0	23.4
600	4.7	7.1	10.4	12.4	15.6
800	2.5	4.2	6.3	7.7	9.8
1000	1.3	2.6	3.8	4.7	5.9
1200	0.9	1.7	2.6	3.2	4.1
1400	0.6	1.1	1.7	2.1	2.6
≥ 1600	0.5	0.9	1.2	1.3	1.6
FFS = 70 km/h					
≤ 100	3.7	8.5	23.2	28.2	41.6
200	8.7	16.0	28.2	33.6	45.2
400	7.5	11.4	16.9	20.7	26.4
600	4.5	6.9	10.8	13.4	17.6
800	2.3	4.1	6.5	8.2	11.0
1000	1.2	2.5	3.8	4.9	6.4
1200	0.8	1.6	2.6	3.3	4.5
1400	0.5	1.0	1.7	2.2	2.8
≥ 1600	0.4	0.9	1.2	1.3	1.7

EXHIBIT 20-21. VALUES OF COEFFICIENTS USED IN ESTIMATING PERCENT TIME-SPENT-FOLLOWING FOR DIRECTIONAL SEGMENTS

Opposing Demand Flow Rate, v_o (pc/h)	a	b
≤ 200	–0.013	0.668
400	–0.057	0.479
600	–0.100	0.413
800	–0.173	0.349
1000	–0.320	0.276
1200	–0.430	0.242
1400	–0.522	0.225
≥ 1600	–0.665	0.199

Exhibit 20-22 illustrates the operational effect of a passing lane on percent time-spent-following. The figure shows that installation of a passing lane provides operational benefits for some distance downstream before percent time-spent-following returns to its former level. Thus, the effective length of a passing lane is greater than its actual length. Exhibit 20-23 shows how the traffic flow rate on a downstream length of a two-lane highway benefits from a passing lane in terms of both percent time-spent-following and average travel speed.

EXHIBIT 20-22. OPERATIONAL EFFECT OF A PASSING LANE ON PERCENT TIME-SPENT-FOLLOWING



Source: Harwood and Hoban (*3*).

EXHIBIT 20-23. DOWNSTREAM LENGTH OF ROADWAY AFFECTED BY PASSING LANES ON DIRECTIONAL SEGMENTS IN LEVEL AND ROLLING TERRAIN

	Downstream Length of Roadway Affected, L_{de} (km)	
Directional Flow Rate (pc/h)	Percent Time-Spent-Following	Average Travel Speed
≤ 200	20.9	2.8
400	13.0	2.8
700	9.1	2.8
≥ 1000	5.8	2.8

For complex systems of passing lanes, consider use of a computer simulation model

The operational analysis procedures presented here for passing lanes in level or rolling terrain are applicable to directional segments of two-lane highways that include the entire passing lane. Sections of two-lane highway upstream and downstream of the passing lane also may be included. Whenever possible, the directional segment should include not only the passing lane but also its full effective downstream length, as indicated in Exhibit 20-23. There are special procedures for directional segments that include only part of the effective downstream length of the passing lane (e.g., when an analysis segment must end because of the proximity of a small town or due to a change in the demand volume). The effects of providing another passing lane in the same direction of travel within the effective length of the first passing lane are too complex to evaluate. In such situations, an evaluation with a traffic simulation model is recommended. The operational analysis procedures for passing lanes in level or rolling terrain are described below.

Analysis of a Directional Segment with a Passing Lane

The first step in the operational analysis of a passing lane is to apply the procedure for directional segments in level or rolling terrain to the normal cross section without the passing lane. The data required are the demand volume in the analysis direction, demand volume in the opposing direction, vehicle mix, lane width, shoulder width, and percentage of no-passing zones. The result is the percent time-spent-following and the average travel speed for the normal two-lane cross section.

Dividing the Segment into Regions

The next step is to divide the analysis segment into four regions. These regions are

1. Upstream of the passing lane,
2. The passing lane,
3. Downstream of the passing lane but within its effective length, and
4. Downstream of the passing lane but beyond its effective length.

These four lengths must add up to the total length of the analysis segment. The analysis regions and their lengths will differ for estimations of percent time-spent-following and average travel speed, because the downstream lengths for these measures differ, as shown in Exhibit 20-23.

For constraints on applicable lengths of lane additions see Chapter 12, "Highway Concepts"

The length of the passing lane, L_{pl}, used in the analysis, is either the length of the passing lane as constructed or its planned length. The passing lane length should include the lengths of the lane addition and lane drop tapers. The analysis procedure is calibrated for passing lanes within the optimal ranges of length shown in Chapter 12. Passing lane lengths substantially shorter or longer than the optimum may provide less operational benefit than predicted by this procedure.

The length of the conventional two-lane highway segment upstream of the passing lane, L_u, is determined by the actual or planned placement of the passing lane within the analysis section. The length of the downstream highway segment within the effective length of the passing lane, L_{de}, is determined from Exhibit 20-23. Any remaining length within the analysis segment downstream of the passing lane is included in L_d as shown in Equation 20-18.

$$L_d = L_t - \left(L_u + L_{pl} + L_{de}\right) \qquad (20\text{-}18)$$

where

- L_d = length of two-lane highway downstream of the passing lane and beyond its effective length (km),
- L_t = total length of analysis segment (km),
- L_u = length of two-lane highway upstream of the passing lane (km),
- L_{pl} = length of the passing lane including tapers (km), and
- L_{de} = downstream length of two-lane highway within the effective length of the passing lane (km) (from Exhibit 20-23).

Determining Percent Time-Spent-Following

Percent time-spent-following within lengths L_u and L_d is assumed to be equal to $PTSF_d$, as predicted by the directional segment procedure. Within the passing lane, percent time-spent-following is generally equal to 58 to 62 percent of its upstream value. This effect varies as a function of the directional flow rate, as shown in Exhibit 20-24. Within the downstream length, L_{de}, percent time-spent-following is assumed to increase linearly with distance from the passing-lane value to its normal upstream value. Thus, the average percent time-spent-following with the passing lane in place can be computed using Equation 20-19.

$$PTSF_{pl} = \frac{PTSF_d \left[L_u + L_d + f_{pl} L_{pl} + \left(\frac{1 + f_{pl}}{2} \right) L_{de} \right]}{L_t} \qquad (20\text{-}19)$$

where

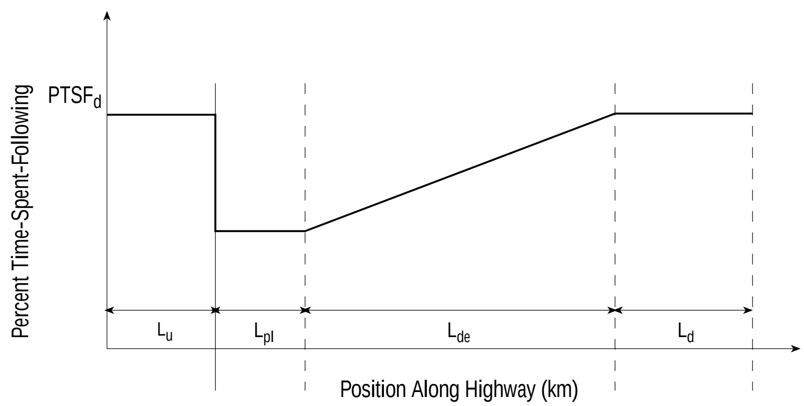
- $PTSF_{pl}$ = percent time-spent-following for the entire segment including the passing lane,
- $PTSF_d$ = percent time-spent-following for the entire segment without the passing lane from Equation 20-16, and
- f_{pl} = factor for the effect of a passing lane on percent time-spent-following (see Exhibit 20-24).

The variations in percent time-spent-following are shown in Exhibit 20-25.

EXHIBIT 20-24. FACTORS (f_{pl}) FOR ESTIMATION OF AVERAGE TRAVEL SPEED AND PERCENT TIME-SPENT-FOLLOWING WITHIN A PASSING LANE

Directional Flow Rate (pc/h)	Average Travel Speed	Percent Time-Spent-Following
0–300	1.08	0.58
> 300–600	1.10	0.61
> 600	1.11	0.62

EXHIBIT 20-25. EFFECT OF A PASSING LANE ON PERCENT TIME-SPENT-FOLLOWING AS REPRESENTED IN THE OPERATIONAL ANALYSIS METHODOLOGY



Special case: downstream effective length is truncated

If the analysis section is truncated by a town or a major intersection before the full downstream effective length of the passing lane has been reached, then distance L_d is not used and the actual downstream length within the analysis segment, L'_{de}, is less than the value of L_{de} tabulated in Exhibit 20-23. In this case, Equation 20-19 should be replaced by Equation 20-20. Equation 20-20 applies whenever distance, L_d, computed with Equation 20-8, is negative.

$$PTSF_{pl} = \frac{PTSF_d \left[L_u + f_{pl} L_{pl} + f_{pl} L'_{de} + \left(\frac{1 - f_{pl}}{2} \right) \left(\frac{(L'_{de})^2}{L_{de}} \right) \right]}{L_t} \qquad (20\text{-}20)$$

where

L'_{de} = actual distance from end of passing lane to end of analysis segment (km). L'_{de} must be less than or equal to the value of L_{de} from Exhibit 20-23.

Determining Average Travel Speed

Average travel speed within lengths L_u and L_d is assumed to equal ATS_d, as predicted by the directional segment procedure. Within the passing lane, average travel speed is generally 8 to 11 percent higher than its upstream value. This effect varies as a function of directional flow rate, as shown in Exhibit 20-24. Within the downstream length, L_{de}, average travel speed is assumed to decrease linearly with distance from the within-passing-lane value to its normal upstream value. Thus, the average travel speed with the passing lane in place can be computed using Equation 20-21.

$$ATS_{pl} = \frac{ATS_d * L_t}{L_u + L_d + \frac{L_{pl}}{f_{pl}} + \frac{2 L_{de}}{1 + f_{pl}}} \qquad (20\text{-}21)$$

where

ATS_{pl} = average travel speed for the entire segment including the passing lane (km/h),

ATS_d = average travel speed for the entire segment without the passing lane from Equation 20-15 (km/h), and

f_{pl} = factor for the effect of a passing lane on average travel speed (see Exhibit 20-23).

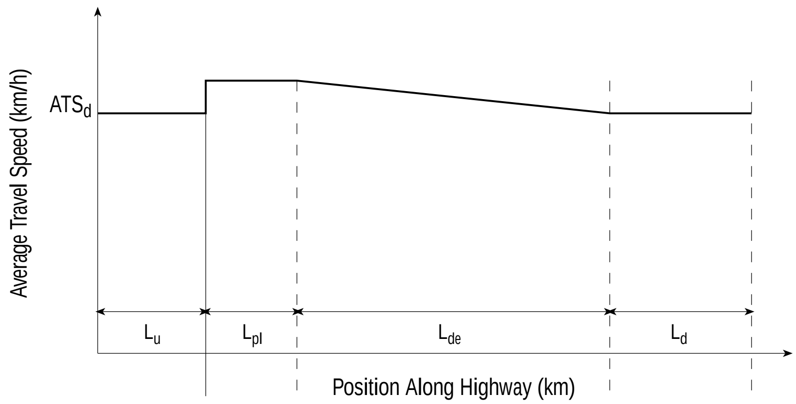
The variations in average travel speed are shown in Exhibit 20-26. If the analysis section is truncated by the presence of a town or a major intersection before the full downstream effective length of the passing lane has been reached, then distance L_d is not used and the actual downstream length within the analysis segment, L'_{de}, is less than the value of L_{de} tabulated in Exhibit 20-23. In this case, Equation 20-21 should be replaced by Equation 20-22. Equation 20-22 applies whenever distance, L_d, computed with Equation 20-18, is negative.

$$ATS_{pl} = \frac{ATS_d * L_t}{L_u + \frac{L_{pl}}{f_{pl}} + \frac{2 L'_{de}}{\left[1 + f_{pl} + (f_{pl} - 1) \frac{L_{de} - L'_{de}}{L_{de}} \right]}} \qquad (20\text{-}22)$$

Determining LOS

Determining LOS for a directional segment with a passing lane is similar to determining LOS for a directional segment without a passing lane; however, for the passing lane, the values of $PTSF_{pl}$ and ATS_{pl} are used instead of $PTSF_d$ and ATS_d. The LOS for a Class I highway segment with a passing lane is determined by locating the point corresponding to $PTSF_{pl}$ and AST_{pl} in Exhibit 20-3. The LOS for a Class II highway segment with a passing lane is determined by comparing $PTSF_{pl}$ to the LOS thresholds in Exhibit 20-4. If the directional demand flow rate, v_d, exceeds 1,700 pc/h, the roadway is oversaturated, and the LOS is F. Although a passing lane section with two lanes in the same direction can serve more than 1,700 pc/h, the sections with a single directional lane between passing lanes will be oversaturated and will become bottlenecks.

EXHIBIT 20-26. EFFECT OF A PASSING LANE ON AVERAGE TRAVEL SPEED AS REPRESENTED IN THE OPERATIONAL ANALYSIS METHODOLOGY



Effects of Passing Lanes on Opposing Traffic

If the installation of a passing lane on a directional segment changes the percentage of no-passing zones for the opposing direction of travel, the directional analysis for the opposing direction must be revised. This can occur, for example, if a highway agency routinely prohibits passing in the opposing direction of travel to a passing lane. However, if passing is permitted in the opposing direction of travel, the passing lane may have little effect on the LOS for the opposing direction. It is possible that a passing lane, by breaking up platoons in one direction of travel, may reduce passing opportunities for the other direction. However, this effect has not been quantified and is not reflected in the operational analysis procedure.

When passing lanes are provided in both directions of travel, the operational analyses for the two directions can proceed independently, unless the addition of the passing lane in one direction substantially changes the percentage of no-passing zones outside the passing lane in the other direction.

DIRECTIONAL SEGMENTS WITH CLIMBING LANES ON UPGRADES

The operational analysis procedure does not address traffic operations on the roadway downstream of a climbing lane beyond the top of the grade. Consider use of a computer simulation model of two-lane highway operations to analyze such segments.

A climbing lane is a passing lane added on an upgrade to allow traffic to pass heavy vehicles whose speeds are reduced. According to the American Association of State Highway and Transportation Officials (AASHTO) *Policy on Geometric Design of Highways and Streets* (*4*), climbing lanes on two-lane highway upgrades are warranted when

- The directional flow rate on the upgrade exceeds 200 veh/h,
- The directional flow rate for trucks on the upgrade exceeds 20 veh/h, and
- Any of the following conditions apply: a speed reduction of 15 km/h for a typical heavy truck, LOS E or F on the grade, or a reduction of two or more levels of service from the approach segment to the grade.

The AASHTO policy on climbing lanes directly refers to the LOS determined with the HCM operational analysis procedures. Operational analysis of climbing lanes on specific upgrades can be performed with the same procedures for passing lanes in level and rolling terrain, with two major differences. First, in applying the directional segment procedure to the roadway without the added lane, the grade adjustment factor, f_G, and the heavy-vehicle adjustment factor, f_{HV}, should be the values for specific upgrades. If the grade on which the lane is added is not sufficiently long or steep to be analyzed as a specific upgrade, then it should be analyzed as a passing lane rather than a climbing lane. Second, the values of the adjustment factors for average travel speed and percent time-spent-following in Exhibit 20-27 should be used instead of those in Exhibit 20-24.

EXHIBIT 20-27. FACTORS (f_{pl}) FOR ESTIMATION OF AVERAGE TRAVEL SPEED AND PERCENT TIME-SPENT-FOLLOWING WITHIN A CLIMBING LANE

Directional Flow Rate (pc/h)	Average Travel Speed	Percent Time-Spent-Following
0–300	1.02	0.20
> 300–600	1.07	0.21
> 600	1.14	0.23

Because climbing lanes are analyzed as part of a specific upgrade, the lengths L_u and L_d used in Equations 20-19 through 20-22 generally equal zero. The downstream effective length, L_{de}, also generally equals zero unless the climbing lane ends before the top of the grade. In this case, a value of L_{de} shorter than the values shown in Exhibit 20-23 should be considered.

LOS ASSESSMENT FOR DIRECTIONAL TWO-LANE FACILITIES

A directional two-lane highway facility is a series of contiguous directional two-lane highway segments. If an operational analysis has been conducted for each segment in the series, the results can be combined to obtain an operational assessment of the facility as a whole. The same approach can be used to combine operational results from directional segments in the opposing directions of travel on a two-lane highway. In either case, the applicable LOS criteria are shown in Exhibits 20-3 and 20-4 for Class I and II highways, respectively.

The combined percent time-spent-following for several directional segments can be determined by Equation 20-23.

$$PTSF_c = \frac{TT_1 * PTSF_1 + TT_2 * PTSF_2 + \ldots + TT_n * PTSF_n}{TT_1 + TT_2 + \ldots + TT_n} \qquad (20\text{-}23)$$

where

$PTSF_c$ = percent time-spent-following for all segments combined,
TT_x = total travel time (veh-h) for Segment x (determined from Equation 20-11), and
$PTSF_x$ = percent time-spent-following for Segment x.

The combined average travel speed for several directional segments can be determined using Equation 20-24.

$$ATS_c = \frac{VkmT_1 + VkmT_2 + \ldots + VkmT_n}{TT_1 + TT_2 + \ldots + TT_n} \qquad (20\text{-}24)$$

where

ATS_c = average travel speed for all segments combined (km/h) and
$VkmT_x$ = total travel for Segment x, determined from Equation 20-9 (veh-km).

LOS ASSESSMENT FOR UNINTERRUPTED-FLOW FACILITIES AND CORRIDORS WITH TWO-LANE HIGHWAYS

Results from individual segments may be combined

A directional analysis procedure has been provided in this chapter so that operational analysis results for directional segments on two-lane facilities can be combined readily with results for interrupted-flow facilities, including multilane highways (see Chapter 21) and basic freeway segments (see Chapter 23). Operational analysis across different types of uninterrupted-flow facilities should be based solely on average travel speed, because percent time-spent-following generally is a consideration only for two-lane highways. Equations 20-20, 20-21, and 20-22 can be used to combine estimates of average travel speed from segments on different types of facilities.

III. APPLICATIONS

Guidelines on required inputs and estimated values are in Chapter 12, "Highway Concepts"

The methodology of this chapter can be used to analyze the capacity and LOS of two-lane highways. The analyst must address two fundamental questions. First, what is the primary output? Primary outputs typically solved for in a variety of applications include LOS and achievable flow rate (v_p). Performance measures related to average travel speed (ATS) and percent time-spent-following (PTSF) are also achievable but are secondary.

Second, what are the default values or estimated values for use in the analysis? Basically, there are three sources of input data:

1. Default values found in this manual,
2. Estimates and locally derived default values developed by the user, and
3. Values derived from field measurements and observation.

For each of the input variables, a value must be supplied to calculate the primary and secondary outputs.

A common application of the method is to compute the LOS of a current or a changed facility in the near term or in the future. This type of application is often termed operational, and its primary output is LOS, with secondary outputs for ATS and PTSF. The achievable flow rate, v_p, can be solved for as the primary output. This analysis requires a LOS goal and geometric data as inputs for estimating when a flow rate will be exceeded, causing the highway to operate at an unacceptable LOS. Using the methodology to determine the number of lanes required (known as a design application in this manual) is of course not necessary for two-lane highways. Modifications to grade, alignment, and cross section, however, can improve the operational efficiency of a two-lane facility. Computational examples are provided for two design-related applications—the addition of a passing or climbing lane to a two-lane highway and the addition of through lanes to convert a two-lane highway to a four-lane highway. The latter example involves a comparison of results obtained from this chapter with results obtained from Chapter 21, "Multilane Highways."

Another general type of analysis is the planning analysis, which uses estimates, HCM default values, and local default values as inputs. As outputs, LOS or flow rate can be determined along with the secondary outputs of average travel speed and percent time-spent-following. The difference between this type of analysis and operational analysis is that most or all of the input values come from estimates or default values, while the operational analyses use field-measured values or known values.

COMPUTATIONAL STEPS

The worksheet for two-way, two-lane highway segment computations is shown in Exhibit 20-28. The worksheets for directional two-lane highway segments with or without a passing lane are included in Appendix B. For all applications, the analyst provides general information and site information.

Operational (LOS) analysis

For operational (LOS) analysis, the analyst inputs all the required data. For estimating average travel speed, equivalent flow is computed with the aid of exhibits for passenger-car equivalencies. FFS is estimated by applying adjustments to the base FFS. Then average travel speed is estimated. Similarly, equivalent flow is estimated by using passenger-car equivalency exhibits to estimate percent time-spent-following. Percent time-spent-following is estimated by adjusting the base percent time-spent-following value for the percentage of no-passing zones. Finally, LOS is determined by average travel speed, percent time-spent-following, or both, depending on the highway classification.

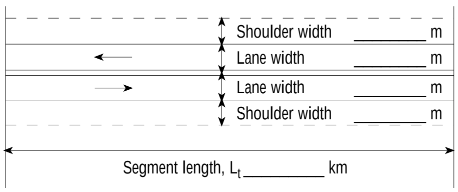
EXHIBIT 20-28. TWO-WAY TWO-LANE HIGHWAY SEGMENT WORKSHEET

TWO-WAY TWO-LANE HIGHWAY SEGMENT WORKSHEET

General Information		**Site Information**	
Analyst	____________	Highway	____________
Agency or Company	____________	From/To	____________
Date Performed	____________	Jurisdiction	____________
Analysis Time Period	____________	Analysis Year	____________

❑ Operational (LOS) ❑ Design (v_p) ❑ Planning (LOS) ❑ Planning (v_p)

Input Data



❑ Class I highway ❑ Class II highway
Terrain ❑ Level ❑ Rolling
Two-way hourly volume ________ veh/h
Directional split ______ /_____
Peak-hour factor, PHF ________
% Trucks and buses, P_T ________ %
% Recreational vehicles, P_R ________ %
% No-passing zone ________ %
Access points/km ________/km

Average Travel Speed

Grade adjustment factor, f_G (Exhibit 20-7)	
Passenger-car equivalents for trucks, E_T (Exhibit 20-9)	
Passenger-car equivalents for RVs, E_R (Exhibit 20-9)	
Heavy-vehicle adjustment factor, f_{HV} $f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$	
Two-way flow rate,[1] v_p (pc/h) $v_p = \frac{V}{PHF * f_G * f_{HV}}$	
v_p * highest directional split proportion[2] (pc/h)	

Free-Flow Speed from Field Measurement	Estimated Free-Flow Speed
Field measured speed, S_{FM} ____________km/h	Base free-flow speed, BFFS ____________km/h
Observed volume, V_f ____________veh/h	Adj. for lane width and shoulder width, f_{LS} (Exhibit 20-5) _____km/h
Free-flow speed, FFS ____________km/h	Adj. for access points, f_A (Exhibit 20-6) ____________km/h
$FFS = S_{FM} + 0.0125\left(\frac{V_f}{f_{HV}}\right)$	Free-flow speed, FFS ____________km/h
	$FFS = BFFS - f_{LS} - f_A$

Adj. for no-passing zones, f_{np} (km/h) (Exhibit 20-11)	
Average travel speed, ATS (km/h) $ATS = FFS - 0.0125v_p - f_{np}$	

Percent Time-Spent-Following

Grade adjustment factor, f_G (Exhibit 20-8)	
Passenger-car equivalents for trucks, E_T (Exhibit 20-10)	
Passenger-car equivalents for RVs, E_R (Exhibit 20-10)	
Heavy-vehicle adjustment factor, f_{HV} $f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$	
Two-way flow rate,[1] v_p (pc/h) $v_p = \frac{V}{PHF * f_G * f_{HV}}$	
v_p * highest directional split proportion[2] (pc/h)	
Base percent time-spent-following, BPTSF (%) $BPTSF = 100(1 - e^{-0.000879v_p})$	
Adj. for directional distribution and no-passing zone, $f_{d/np}$ (%) (Exhibit 20-12)	
Percent time-spent-following, PTSF (%) $PTSF = BPTSF + f_{d/np}$	

Level of Service and Other Performance Measures

Level of service, LOS (Exhibit 20-3 for Class I or 20-4 for Class II)	
Volume to capacity ratio, v/c $v/c = \frac{v_p}{3{,}200}$	
Peak 15-min vehicle-kilometers of travel, $VkmT_{15}$ (veh-km) $VkmT_{15} = 0.25L_t\left(\frac{V}{PHF}\right)$	
Peak-hour vehicle-kilometers of travel, $VkmT_{60}$ (veh-km) $VkmT_{60} = V * L_t$	
Peak 15-min total travel time, TT_{15} (veh-h) $TT_{15} = \frac{VkmT_{15}}{ATS}$	

Notes

1. If $v_p \geq 3{,}200$ pc/h, terminate analysis—the LOS is F.
2. If highest directional split $v_p \geq 1{,}700$ pc/h, terminate analysis—the LOS is F.

Design (v_p) analysis

The objective of design (v_p) analysis is to estimate the flow rate in passenger cars per hour given a set of traffic, roadway, and FFS conditions. A desired LOS is stated and entered in the worksheet. Then a flow rate is assumed and the procedure for operational (LOS) analysis is performed. This computed LOS is then compared with the desired LOS. If the desired LOS is not met, another flow rate is assumed. This iteration continues until the maximum flow rate for the desired LOS is achieved.

PLANNING APPLICATIONS

Planning (LOS) and planning (v_p) analyses

The two planning applications, planning for LOS and for v_p, correspond directly to the procedures for operational and design analyses. The criterion that categorizes these as planning applications is the use of estimates, HCM default values, and local default values as inputs. Another characteristic of a planning application is the use of annual average daily traffic (AADT) to estimate directional design-hour volume (DDHV). For guidelines in computing DDHV, see Chapter 8. Chapter 12 also lists the required data and estimated values to perform a planning application.

ANALYSIS TOOLS

Worksheets for two-way, directional, and directional with passing lane segments are provided in Appendix B. These worksheets can be used to perform operational (LOS), design (v_p), planning (LOS), and planning (v_p) analyses.

IV. EXAMPLE PROBLEMS

Problem No.	Description	Application
1	Find the two-way LOS of a Class I two-lane highway	Operational (LOS)
2	Find the two-way LOS of a Class II two-lane highway	Operational (LOS)
3	Find the directional LOS of a Class I two-lane highway	Operational (LOS)
4	Find the directional LOS of a Class I two-lane highway including a passing lane	Operational (LOS)

EXAMPLE PROBLEM 1

The Highway A Class I two-lane highway segment.

The Question What is the two-way segment LOS for the peak hour?

The Facts

- √ 1,600 veh/h (two-way volume),
- √ 14 percent trucks and buses,
- √ 0.95 PHF,
- √ Rolling terrain,
- √ 1.2-m shoulder width,
- √ 50 percent no-passing zones,
- √ 50/50 directional split,
- √ 4 percent RVs,
- √ 100-km/h base FFS,
- √ 3.4-m lane width,
- √ 10-km length, and
- √ 12 access points/km.

Outline of Solution Two-way average travel speed and percent time-spent-following will be determined, and from these parameters, the LOS.

Steps

1. Determine grade adjustment factor for average travel speed (use Exhibit 20-7).	$f_G = 0.99$
2. Compute f_{HV} for average travel speed (use Exhibit 20-9 and Equation 20-4).	$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$ $f_{HV} = \frac{1}{1 + 0.14(1.5 - 1) + 0.04(1.1 - 1)} = 0.931$
3. Compute v_p (use Equation 20-3).	$v_p = \frac{V}{PHF * f_G * f_{HV}}$ $v_p = \frac{1,600}{(0.95)(0.99)(0.931)} = 1,827$ pc/h
4. Calculate highest directional flow rate.	v_p * 0.50 = 1,827 * 0.50 = 914 pc/h
5. Check the highest directional flow rate and two-way flow rate against capacity values of 1,700 pc/h and 3,200 pc/h, respectively.	914 pc/h < 1,700 pc/h 1,827 pc/h < 3,200 pc/h
6. Compute the FFS (use Exhibits 20-5 and 20-6 and Equation 20-2).	FFS = BFFS – f_{LS} – f_A FFS = 100 – 2.8 – 8.0 = 89.2 km/h
7. Compute the average travel speed (use Exhibit 20-11 and Equation 20-5).	ATS = FFS – 0.0125v_p – f_{np} ATS = 89.2 – 0.0125(1827) – 1.3 = 65.1 km/h
8. Determine grade adjustment factor for percent time-spent-following (use Exhibit 20-8).	$f_G = 1.00$
9. Compute f_{HV} for time-spent-following (use Exhibit 20-10 and Equation 20-4).	$f_{HV} = \frac{1}{1 + 0.14(1.0 - 1) + 0.04(1.0 - 1)} = 1.000$
10. Compute v_p (use Equation 20-3).	$v_p = \frac{1,600}{(0.95)(1.000)(1.00)} = 1,684$ pc/h
11. Calculate the highest directional flow rate.	$v_p * 0.50 = 1,684 * 0.50 = 842$ pc/h
12. Check the highest directional flow rate and two-way flow rate against the capacity values of 1,700 pc/h and 3,200 pc/h, respectively.	842 pc/h < 1,700 pc/h 1,684 pc/h < 3,200 pc/h

13. Compute base percent time-spent-following (use Equation 20-7).	$BPTSF = 100\left(1 - e^{-0.000879v_p}\right)$ $BPTSF = 100\left[1 - e^{-0.000879(1{,}684)}\right] = 77.2\%$
14. Compute percent time-spent-following (use Exhibit 20-12 and Equation 20-6).	PTSF = BPTSF + $f_{d/np}$ PTSF = 77.2 + 4.8 = 82.0%
15. Determine LOS (use Exhibit 20-3).	ATS = 65.1 km/h and PTSF = 82.0% LOS E

Results The two-lane highway operates at LOS E.

Other Performance Measures

$$v/c = \frac{v_p}{3{,}200} = \frac{1{,}827}{3{,}200} = 0.57$$

$$VkmT_{15} = 0.25L_t\left(\frac{V}{PHF}\right) = 0.25(10)\left(\frac{1{,}600}{0.95}\right) = 4{,}211 \text{ veh-km}$$

$$VkmT_{60} = V * L_t = (1{,}600)(10) = 16{,}000 \text{ veh-km}$$

$$TT_{15} = \frac{VkmT_{15}}{ATS} = \frac{4{,}211}{65.1} = 64.7 \text{ veh-h}$$

Example Problem 1

TWO-WAY TWO-LANE HIGHWAY SEGMENT WORKSHEET

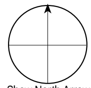
General Information		Site Information	
Analyst	*M.E.*	Highway	*US 391*
Agency or Company	*CEI*	From/To	*SR-33/Adams Rd.*
Date Performed	*5/20/99*	Jurisdiction	
Analysis Time Period		Analysis Year	*1999*

☒ Operational (LOS) ☐ Design (v_p) ☐ Planning (LOS) ☐ Planning (v_p)

Input Data

Shoulder width *1.2* m
Lane width *3.4* m
Lane width *3.4* m
Shoulder width *1.2* m

Segment length, L_t *10* km


Show North Arrow

☒ Class I highway ☐ Class II highway
Terrain ☐ Level ☒ Rolling
Two-way hourly volume *1,600* veh/h
Directional split *50* / *50*
Peak-hour factor, PHF *0.95*
% Trucks and buses, P_T *14* %
% Recreational vehicles, P_R *4* %
% No-passing zone *50* %
Access points/km *12* /km

Average Travel Speed	
Grade adjustment factor, f_G (Exhibit 20-7)	*0.99*
Passenger-car equivalents for trucks, E_T (Exhibit 20-9)	*1.5*
Passenger-car equivalents for RVs, E_R (Exhibit 20-9)	*1.1*
Heavy-vehicle adjustment factor, f_{HV} $f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$	*0.931*
Two-way flow rate,[1] v_p (pc/h) $v_p = \frac{V}{PHF * f_G * f_{HV}}$	*1,827*
v_p * highest directional split proportion[2] (pc/h)	*914*

Free-Flow Speed from Field Measurement		Estimated Free-Flow Speed	
Field measured speed, S_{FM}	km/h	Base free-flow speed, BFFS	*100* km/h
Observed volume, V_f	veh/h	Adj. for lane width and shoulder width, f_{LS} (Exhibit 20-5)	*2.8* km/h
Free-flow speed, FFS	km/h	Adj. for access points, f_A (Exhibit 20-6)	*8.0* km/h
$FFS = S_{FM} + 0.0125\left(\frac{V_f}{f_{HV}}\right)$		Free-flow speed, FFS	*89.2* km/h
		$FFS = BFFS - f_{LS} - f_A$	

Adj. for no-passing zones, f_{np} (km/h) (Exhibit 20-11)	*1.3*
Average travel speed, ATS (km/h) $ATS = FFS - 0.0125v_p - f_{np}$	*65.1*

Percent Time-Spent-Following	
Grade adjustment factor, f_G (Exhibit 20-8)	*1.00*
Passenger-car equivalents for trucks, E_T (Exhibit 20-10)	*1.0*
Passenger-car equivalents for RVs, E_R (Exhibit 20-10)	*1.0*
Heavy-vehicle adjustment factor, f_{HV} $f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$	*1.000*
Two-way flow rate,[1] v_p (pc/h) $v_p = \frac{V}{PHF * f_G * f_{HV}}$	*1,684*
v_p * highest directional split proportion[2] (pc/h)	*842*
Base percent time-spent-following, BPTSF (%) $BPTSF = 100(1 - e^{-0.000879v_p})$	*77.2*
Adj. for directional distribution and no-passing zone, $f_{d/np}$ (%) (Exhibit 20-12)	*4.8*
Percent time-spent-following, PTSF (%) $PTSF = BPTSF + f_{d/np}$	*82.0*

Level of Service and Other Performance Measures	
Level of service, LOS (Exhibit 20-3 for Class I or 20-4 for Class II)	*E*
Volume to capacity ratio, v/c $v/c = \frac{v_p}{3,200}$	*0.57*
Peak 15-min vehicle-kilometers of travel, $VkmT_{15}$ (veh-km) $VkmT_{15} = 0.25L_t\left(\frac{V}{PHF}\right)$	*4,211*
Peak-hour vehicle-kilometers of travel, $VkmT_{60}$ (veh-km) $VkmT_{60} = V * L_t$	*16,000*
Peak 15-min total travel time, TT_{15} (veh-h) $TT_{15} = \frac{VkmT_{15}}{ATS}$	*64.7*

Notes

1. If $v_p \geq 3,200$ pc/h, terminate analysis—the LOS is F.
2. If highest directional split $v_p \geq 1,700$ pc/h, terminate analysis—the LOS is F.

EXAMPLE PROBLEM 2

The Highway A Class II two-lane highway segment on a scenic and recreational route.

The Question What is the two-way segment LOS?

The Facts

- √ 1,050 veh/h (two-way volume),
- √ 5 percent trucks and buses,
- √ 0.85 PHF,
- √ Rolling terrain,
- √ 0.6-m shoulder width,
- √ 60 percent no-passing zones,
- √ 70/30 directional split,
- √ 7 percent RVs,
- √ 90-km/h base FFS,
- √ 3.0-m lane width,
- √ 10-km roadway length, and
- √ 6 access points/km.

Outline of Solution Two-way average travel speed and percent time-spent-following will be determined, and with these parameters, the LOS. Since V/PHF = 1,050/0.85 = 1,235, select truck equivalencies and grade adjustment factors for flow rates greater than 1,200 pc/h.

Steps

1. Determine grade adjustment factor for average travel speed (use Exhibit 20-7).	$f_G = 0.99$
2. Compute f_{HV} for average travel speed (use Exhibit 20-9 and Equation 20-4).	$f_{HV} = \frac{1}{1+P_T(E_T-1)+P_R(E_R-1)}$ $f_{HV} = \frac{1}{1+0.05(1.5-1)+0.07(1.1-1)} = 0.969$
3. Compute v_p (use Equation 20-3).	$v_p = \frac{V}{PHF * f_G * f_{HV}}$ $v_p = \frac{1,050}{(0.85)(0.99)(0.969)} = 1,288$ pc/h
4. Calculate the highest directional flow rate.	v_p * 0.70 = 1,288 * 0.70 = 902 pc/h
5. Check the highest directional flow rate and two-way flow rate against capacity values of 1,700 pc/h and 3,200 pc/h, respectively.	902 pc/h < 1,700 pc/h 1,288 pc/h < 3,200 pc/h
6. Compute FFS (use Exhibit 20-5 and 20-6 and Equation 20-2).	FFS = BFFS – f_{LS} – f_A FFS = 90 – 5.9 – 4.0 = 80.1 km/h
7. Compute ATS (use Exhibit 20-11 and Equation 20-5).	ATS = FFS – 0.0125v_p – f_{np} ATS = 80.1 – 0.0125(1288) – 2.3 = 61.7 km/h
8. Determine grade adjustment factor of percent time-spent-following (use Exhibit 20-8).	$f_G = 1.00$
9. Compute f_{HV} for percent time-spent-following (use Exhibit 20-10 and Equation 20-4).	$f_{HV} = \frac{1}{1+0.05(1.0-1)+0.07(1.0-1)} = 1.000$
10. Compute v_p (use Equation 20-3).	$v_p = \frac{1,050}{(0.85)(1.000)(1.00)} = 1,235$ pc/h
11. Calculate the highest directional flow rate.	v_p * 0.70 = 1,235 * 0.70 = 865 pc/h

12. Check the highest directional flow rate and two-way flow rate against the capacity values of 1,700 pc/h and 3,200 pc/h, respectively.	865 pc/h < 1,700 pc/h 1,235 pc/h < 3,200 pc/h
13. Compute base percent time-spent-following (use Equation 20-7).	$BPTSF = 100\left(1 - e^{-0.000879v_p}\right)$ $BPTSF = 100\left[1 - e^{-0.000879(1{,}235)}\right] = 66.2\%$
14. Compute percent time-spent-following (use Exhibit 20-12 and Equation 20-6).	$PTSF = BPTSF + f_{d/np}$ PTSF = 66.2 + 9.0 = 75.2%
15. Determine LOS (use Exhibit 20-4).	PTSF = 75.2% LOS D

Results The two-lane highway operates at LOS D.

Other Performance Measures

$$v/c = \frac{v_p}{3{,}200} = \frac{1{,}288}{3{,}200} = 0.40$$

$$VkmT_{15} = 0.25L_t\left(\frac{V}{PHF}\right) = 0.25(10)\left(\frac{1{,}050}{0.85}\right) = 3{,}088 \text{ veh-km}$$

$$VkmT_{60} = V * L_t = (1{,}050)(10) = 10{,}500 \text{ veh-km}$$

$$TT_{15} = \frac{VkmT_{15}}{ATS} = \frac{3{,}088}{61.7} = 50.0 \text{ veh-h}$$

Example Problem 2

TWO-WAY TWO-LANE HIGHWAY SEGMENT WORKSHEET

General Information		Site Information	
Analyst	*M.E.*	Highway	*State Highway 34*
Agency or Company	*CEI*	From/To	*US 24/Creek Rd.*
Date Performed	*5/20/99*	Jurisdiction	
Analysis Time Period		Analysis Year	*1999*

☒ Operational (LOS) ❑ Design (v_p) ❑ Planning (LOS) ❑ Planning (v_p)

Input Data

Shoulder width *0.6* m
Lane width *3.0* m
Lane width *3.0* m
Shoulder width *0.6* m
Segment length, L_t *10* km
Show North Arrow

❑ Class I highway ☒ Class II highway
Terrain ❑ Level ☒ Rolling
Two-way hourly volume *1,050* veh/h
Directional split *70* / *30*
Peak-hour factor, PHF *0.85*
% Trucks and buses, P_T *5* %
% Recreational vehicles, P_R *7* %
% No-passing zone *60* %
Access points/km *6* /km

Average Travel Speed

Grade adjustment factor, f_G (Exhibit 20-7)	*0.99*
Passenger-car equivalents for trucks, E_T (Exhibit 20-9)	*1.5*
Passenger-car equivalents for RVs, E_R (Exhibit 20-9)	*1.1*
Heavy-vehicle adjustment factor, f_{HV} $f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$	*0.969*
Two-way flow rate,[1] v_p (pc/h) $v_p = \frac{V}{PHF * f_G * f_{HV}}$	*1,288*
v_p * highest directional split proportion[2] (pc/h)	*902*

Free-Flow Speed from Field Measurement	Estimated Free-Flow Speed
Field measured speed, S_{FM} ____ km/h	Base free-flow speed, BFFS *90* km/h
Observed volume, V_f ____ veh/h	Adj. for lane width and shoulder width, f_{LS} (Exhibit 20-5) *5.9* km/h
Free-flow speed, FFS ____ km/h	Adj. for access points, f_A (Exhibit 20-6) *4.0* km/h
$FFS = S_{FM} + 0.0125\left(\frac{V_f}{f_{HV}}\right)$	Free-flow speed, FFS *80.1* km/h
	$FFS = BFFS - f_{LS} - f_A$

Adj. for no-passing zones, f_{np} (km/h) (Exhibit 20-11)	*2.3*
Average travel speed, ATS (km/h) $ATS = FFS - 0.0125v_p - f_{np}$	*61.7*

Percent Time-Spent-Following

Grade adjustment factor, f_G (Exhibit 20-8)	*1.00*
Passenger-car equivalents for trucks, E_T (Exhibit 20-10)	*1.0*
Passenger-car equivalents for RVs, E_R (Exhibit 20-10)	*1.0*
Heavy-vehicle adjustment factor, f_{HV} $f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$	*1.000*
Two-way flow rate,[1] v_p (pc/h) $v_p = \frac{V}{PHF * f_G * f_{HV}}$	*1,235*
v_p * highest directional split proportion[2] (pc/h)	*865*
Base percent time-spent-following, BPTSF (%) $BPTSF = 100(1 - e^{-0.000879v_p})$	*66.2*
Adj. for directional distribution and no-passing zone, $f_{d/np}$ (%) (Exhibit 20-12)	*9.0*
Percent time-spent-following, PTSF (%) $PTSF = BPTSF + f_{d/np}$	*75.2*

Level of Service and Other Performance Measures

Level of service, LOS (Exhibit 20-3 for Class I or 20-4 for Class II)	*D*
Volume to capacity ratio, v/c $v/c = \frac{v_p}{3,200}$	*0.40*
Peak 15-min vehicle-kilometers of travel, $VkmT_{15}$ (veh-km) $VkmT_{15} = 0.25L_t\left(\frac{V}{PHF}\right)$	*3,088*
Peak-hour vehicle-kilometers of travel, $VkmT_{60}$ (veh-km) $VkmT_{60} = V * L_t$	*10,500*
Peak 15-min total travel time, TT_{15} (veh-h) $TT_{15} = \frac{VkmT_{15}}{ATS}$	*50.0*

Notes

1. If $v_p \geq 3,200$ pc/h, terminate analysis—the LOS is F.
2. If highest directional split $v_p \geq 1,700$ pc/h, terminate analysis—the LOS is F.

Example Problem 3

The Highway A Class I two-lane highway segment.

The Question What is the LOS of the peak direction?

The Facts

- √ 1,200 veh/h (analysis direction volume),
- √ 14 percent trucks and buses,
- √ 4 percent RVs,
- √ 100-km/h base FFS,
- √ 3.3-m lane width,
- √ 10-km roadway length,
- √ 12 access points/km,
- √ 400 veh/h (opposing direction volume),
- √ 0.95 PHF,
- √ Rolling terrain,
- √ 1.2-m shoulder width, and
- √ 50 percent no-passing zones.

Outline of Solution Analysis direction average travel speed and percent time-spent-following will be determined, and with these parameters, the LOS.

Steps

1. Determine the grade adjustment factor, f_G, for average travel speed for the analysis direction (use Exhibit 20-7).	$f_G = 0.99$
2. Compute f_{HV} and v_d for average travel speed in the analysis direction (use Exhibit 20-9 and Equations 20-4 and 20-12).	$f_{HV} = \frac{1}{1 + 0.14(1.5 - 1) + 0.04(1.1 - 1)} = 0.931$ $v_d = \frac{1,200}{(0.95)(0.99)(0.931)} = 1,370 \text{ pc/h}$
3. Determine the grade adjustment factor, f_G, for average travel speed for the opposing direction (use Exhibit 20-7).	$f_G = 0.93$
4. Compute f_{HV} and v_o for average travel speed in the opposing direction (use Exhibit 20-9 and Equations 20-4 and 20-13).	$f_{HV} = \frac{1}{1 + 0.14(1.9 - 1) + 0.04(1.1 - 1)} = 0.885$ $v_o = \frac{400}{(0.95)(0.93)(0.885)} = 512 \text{ pc/h}$
5. Check v_d and v_o with the capacity value of 1,700 pc/h.	1,370 pc/h < 1,700 pc/h 512 pc/h < 1,700 pc/h
6. Compute FFS (use Exhibits 20-5 and 20-6, and Equation 20-2).	$FFS = BFFS - f_{LS} - f_A$ FFS = 100 – 2.8 – 8.0 = 89.2 km/h
7. Compute average travel speed (use Exhibit 20-19 and Equation 20-15).	$ATS_d = FFS_d - 0.0125(v_d + v_o) - f_{np}$ $ATS_d = 89.2 - 0.0125(1370 + 512) - 2.7 =$ 63.0 km/h
8. Determine the grade adjustment factor, f_G, for percent time-spent-following for the analysis direction (use Exhibit 20-8).	$f_G = 1.00$
9. Compute f_{HV} and v_d for percent time-spent-following in the analysis direction (use Exhibit 20-10 and Equations 20-4 and 20-12).	$f_{HV} = \frac{1}{1 + 0.14(1.0 - 1) + 0.04(1.0 - 1)} = 1.000$ $v_d = \frac{1,200}{(0.95)(1.00)(1.000)} = 1,263 \text{ pc/h}$
10. Determine the grade adjustment factor, f_G, for percent time-spent-following for the opposing direction (use Exhibit 20-8).	$f_G = 0.94$

11. Compute f_{HV} and v_o for percent time-spent-following in the opposing direction (use Exhibit 20-10 and Equations 20-4 and 20-14).	$f_{HV} = \frac{1}{1 + 0.14(1.5 - 1) + 0.04(1.0 - 1)} = 0.935$ $v_o \ \frac{400}{(0.95)(0.94)(0.935)} = 479$ pc/h
12. Check v_d and v_o against the capacity value of 1,700 pc/h.	1,263 pc/h < 1,700 pc/h 479 pc/h < 1,700 pc/h
13. Compute base percent time-spent-following the analysis direction (use Exhibit 20-21 and Equation 20-17).	$BPTSF_d = 100\left(1 - e^{av_d^b}\right)$ $BPTSF_d = 100\left[1 - e^{(-0.074)(1,263)^{0.453}}\right] = 84.7\%$
14. Compute percent time-spent-following for the analysis direction (use Exhibit 20-20 and Equation 20-16).	$PTSF_d = BPTSF_d + f_{np}$ $PTSF_d$ = 84.7 + 11.7 = 96.4%
15. Determine LOS (use Exhibit 20-3).	ATS_d = 63.0 km/h and $PTSF_d$ = 96.4% LOS E

Results The two-lane highway operates at LOS E in the analysis direction.

Other Performance Measures

$$v/c = \frac{v_d}{1,700} = \frac{1,370}{1,700} = 0.81$$

$$VkmT_{15} = 0.25L_t\left(\frac{V_d}{PHF}\right) = 0.25(10)\left(\frac{1,200}{0.95}\right) = 3,158 \text{ veh-km}$$

$$VkmT_{60} = V_d * L_t = (1,200)(10) = 12,000 \text{ veh-km}$$

$$TT_{15} = \frac{VkmT_{15}}{ATS_d} = \frac{3,158}{63.0} = 50.1 \text{ veh-h}$$

Example Problem 3

DIRECTIONAL TWO-LANE HIGHWAY SEGMENT WORKSHEET

General Information		Site Information	
Analyst	M.E.	Highway/Direction of Travel	State Highway 45
Agency or Company	CEI	From/To	RD 20/RD 35
Date Performed	5/20/99	Jurisdiction	
Analysis Time Period		Analysis Year	1999

☒ Operational (LOS) ☐ Design (v_p) ☐ Planning (LOS) ☐ Planning (v_p)

Input Data

Shoulder width 1.2 m
Opposing direction — Lane width 3.3 m
Analysis direction — Lane width 3.3 m
Shoulder width 1.2 m
Segment length, L_t 10 km

Show North Arrow

☒ Class I highway ☐ Class II highway
Terrain ☐ Level ☒ Rolling
Grade Length ____ km Up/down ____ %
Peak-hour factor, PHF 0.95
% Trucks and buses, P_T 14 %
% Recreational vehicles, P_R 4 %
% No-passing zone 50 %
Access points/km 12 /km

Analysis direction volume, V_d 1,200 veh/h Opposing direction volume, V_o 400 veh/h

Average Travel Speed

	Analysis Direction (d)	Opposing Direction (o)
Passenger-car equivalent for trucks, E_T (Exhibit 20-9 or 20-15)	1.5	1.9
Passenger-car equivalent for RVs, E_R (Exhibit 20-9 or 20-17)	1.1	1.1
Heavy-vehicle adjustment factor,[5] f_{HV} $f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$	0.931	0.885
Grade adjustment factor,[1] f_G (Exhibit 20-6 or 20-12)	0.99	0.93
Directional flow rate,[2] v_i (pc/h) $v_i = \frac{V_i}{PHF * f_{HV} * f_G}$	1,370	512

Free-Flow Speed from Field Measurement		Estimated Free-Flow Speed	
Field measured speed,[3] S_{FM}	____ km/h	Base free-flow speed,[3] BFFS	100 km/h
Observed volume,[3] V_f	____ veh/h	Adj. for lane width and shoulder width,[3] f_{LS} (Exh. 20-5)	2.8 km/h
Free-flow speed, FFS_d	____ km/h	Adj. for access points,[3] f_A (Exhibit 20-6)	8.0 km/h
$FFS_d = S_{FM} + 0.0125 \left(\frac{V_f}{f_{HV}}\right)$		Free-flow speed, FFS_d	89.2 km/h
		$FFS_d = BFFS - f_{LS} - f_A$	

Adjustment for no-passing zones, f_{np} (km/h) (Exhibit 20-19)	2.7
Average travel speed, ATS_d (km/h) $ATS_d = FFS_d - 0.0125(v_d + v_o) - f_{np}$	63.0

Percent Time-Spent-Following

	Analysis Direction (d)	Opposing Direction (o)
Passenger-car equivalent for trucks, E_T (Exhibit 20-10 or 20-16)	1.0	1.5
Passenger-car equivalent for RVs, E_R (Exhibit 20-10 or 20-16)	1.0	1.0
Heavy-vehicle adjustment factor, f_{HV} $f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$	1.000	0.935
Grade adjustment factor,[1] f_G (Exhibit 20-8 or 20-14)	1.0	0.94
Directional flow rate,[2] v_i (pc/h) $v_i = \frac{V_i}{PHF * f_{HV} * f_G}$	1,263	479

Base percent time-spent-following,[4] $BPTSF_d$ (%) $BPTSF_d = 100(1 - e^{av_d^b})$	84.7
Adjustment for no-passing zone, f_{np} (Exhibit 20-20)	11.7
Percent time-spent-following, $PTSF_d$ (%) $PTSF_d = BPTSF_d + f_{np}$	96.4

Level of Service and Other Performance Measures

Level of service, LOS (Exhibit 20-3 or 20-4)	E
Volume to capacity ratio, v/c $v/c = \frac{V_d}{1700}$	0.81
Peak 15-min vehicle-kilometers of travel, $VkmT_{15}$ (veh-km) $VkmT_{15} = 0.25L_t\left(\frac{V_d}{PHF}\right)$	3,158
Peak-hour vehicle-kilometers of travel, $VkmT_{60}$ (veh-km) $VkmT_{60} = V_d * L_t$	12,000
Peak 15-min total travel time, TT_{15} (veh-h) $TT_{15} = \frac{VkmT_{15}}{ATS_d}$	50.1

Notes

1. If the highway is extended segment (level) or rolling terrain, $f_G = 1.0$
2. If v_i (v_d or v_o) ≥ 1,700 pc/h, terminate analysis—the LOS is F.
3. For the analysis direction only.
4. Exhibit 20-21 provides factors a and b.
5. Use alternative Equation 20-14 if some trucks operate at crawl speeds on a specific downgrade.

EXAMPLE PROBLEM 4

The Highway A Class I two-lane highway segment described in Example Problem 3. In this analysis, a 2-km passing lane is to be added beginning at a location 2 km downstream from the beginning of the 10-km two-lane highway in the analysis direction.

The Question What is the LOS in the peak direction including the passing lane?

The Facts

- √ All input parameters listed in Example Problem 3,
- √ 2-km length of two-lane highway upstream of the passing lane, and
- √ 2-km length of passing lane including tapers.

Outline of Solution The length of roadway expected to be affected downstream of the passing lane will be determined. These lengths will be applied to the average travel speed and percent time-spent-following without a passing lane to compute the average travel speed and percent time-spent-following with the passing lane. Using these parameters, the LOS will be determined.

Steps

1. Compute L_d for average travel speed (use Exhibit 20-23 and Equation 20-18).	$L_d = L_t – (L_u + L_{pl} + L_{de})$ $L_d = 10 – (2 + 2 + 2.8) = 3.2$ km
2. Compute average travel speed of the analysis direction including passing lane (use Exhibit 20-24 and Equation 20-21).	$ATS_{pl} = \dfrac{ATS_d * L_t}{L_u + L_d + \dfrac{L_{pl}}{f_{pl}} + \dfrac{2L_{de}}{1+f_{pl}}}$ $ATS_{pl} = \dfrac{63.0(10)}{2 + 3.2 + \left(\dfrac{2}{1.11}\right) + \dfrac{2(2.8)}{1+1.11}} = 65.2$ km/h
3. Compute L_d for percent time-spent-following (use Exhibit 20-23 and Equation 20-18).	$L_d = L_t – (L_u + L_{pl} + L_{de})$ $L_d = 10 – (2 + 2 + 5.8) = 0.2$ km
4. Compute percent time-spent-following of the analysis direction including passing lane (use Exhibit 20-24 and Equation 20-19).	$PTSF_{pl} = \dfrac{PTSF_d\left[L_u + L_d + f_{pl}L_{pl} + \left(\dfrac{1+f_{pl}}{2}\right)L_{de}\right]}{L_t}$ $PTSF_{pl} = \dfrac{96.4\left[2 + 0.2 + 0.62(2) + \left(\dfrac{1+0.62}{2}\right)5.8\right]}{10} =$ 78.5%
5. Determine LOS (use Exhibit 20-3).	$ATS_{pl} = 65.2$ km/h and $PTSF_{pl} = 78.5\%$ LOS D

Results The two-lane highway operates at LOS D in the analysis direction with the passing lane as compared to LOS E without the passing lane (from Example Problem 3).

Other Performance Measures

$$v/c = \frac{v_d}{1,700} = \frac{1,370}{1,700} = 0.81$$

$$VkmT_{15} = 0.25L_t\left(\frac{V_d}{PHF}\right) = 0.25(10)\left(\frac{1,200}{0.95}\right) = 3,158 \text{ veh-km}$$

$$VkmT_{60} = V_d * L_t = (1{,}200)(10) = 12{,}000 \text{ veh-km}$$

$$TT_{15} = \frac{VkmT_{15}}{ATS_{pl}} = \frac{3{,}158}{65.2} = 48.4 \text{ veh-h}$$

Example Problem 4

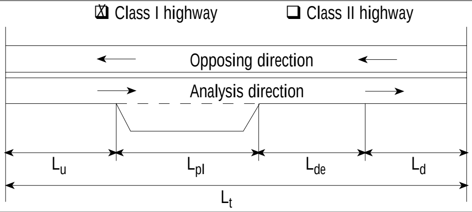
DIRECTIONAL TWO-LANE HIGHWAY SEGMENT WITH PASSING LANE WORKSHEET

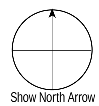
General Information		Site Information	
Analyst	*M.E.*	Highway/Direction of Travel	*State Highway 45*
Agency or Company	*CEI*	From/To	*RD 20/RD 35*
Date Performed	*5/20/99*	Jurisdiction	
Analysis Time Period		Analysis Year	*1999*

☒ Operational (LOS) ☐ Design (v_p) ☐ Planning (LOS) ☐ Planning (v_p)

Input Data

☒ Class I highway ☐ Class II highway





Total length of analysis segment, L_t (km)	*10*
Length of two-lane highway upstream of the passing lane, L_u (km)	*2*
Length of passing lane including tapers, L_{pl} (km)	*2*
Average travel speed, ATS_d (from Directional Two-Lane Highway Segment Worksheet)	*63.0*
Percent time-spent-following, $PTSF_d$ (from Directional Two-Lane Highway Segment Worksheet)	*96.4*
Level of service,[1] LOS_d (from Directional Two-Lane Highway Segment Worksheet)	*E*
Average Travel Speed	
Downstream length of two-lane highway within effective length of passing lane for average travel speed, L_{de} (km) (Exhibit 20-23)	*2.8*
Length of two-lane highway downstream of effective length of the passing lane for average travel speed, L_d (km) $L_d = L_t - (L_u + L_{pl} + L_{de})$	*3.2*
Adj. factor for the effect of passing lane on average speed, f_{pl} (Exhibit 20-24)	*1.11*
Average travel speed including passing lane,[2] ATS_{pl} $ATS_{pl} = \frac{ATS_d * L_t}{L_u + L_d + \frac{L_{pl}}{f_{pl}} + \frac{2L_{de}}{1 + f_{pl}}}$	*65.2*
Percent Time-Spent-Following	
Downstream length of two-lane highway within effective length of passing lane for percent time-spent-following, L_{de} (km) (Exhibit 20-23)	*5.8*
Length of two-lane highway downstream of effective length of the passing lane for percent time-spent-following, L_d (km) $L_d = L_t - (L_u + L_{pl} + L_{de})$	*0.2*
Adj. factor for the effect of passing lane on percent time-spent-following, f_{pl} (Exhibit 20-24)	*0.62*
Percent time-spent-following including passing lane,[3] $PTSF_{pl}$ (%) $PTSF_{pl} = \frac{PTSF_d\left[L_u + L_d + f_{pl}L_{pl} + \left(\frac{1 + f_{pl}}{2}\right)L_{de}\right]}{L_t}$	*78.5*
Level of Service and Other Performance Measures [4]	
Level of service including passing lane, LOS_{pl} (Exhibits 20-3 or 20-4)	*D*
Peak 15-min total travel time, TT_{15} (veh-h) $TT_{15} = \frac{VkmT_{15}}{ATS_{pl}}$	*48.4*

Notes

1. If LOS_d = F, passing lane analysis cannot be performed.
2. If $L_d < 0$, use alternative Equation 20-22.
3. If $L_d < 0$, use alternative Equation 20-20.
4. v/c, $VkmT_{15}$, and $VkmT_{60}$ are calculated on Directional Two-Lane Highway Segment Worksheet.

V. REFERENCES

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4. American Association of State Highway and Transportation Officials. *A Policy on Geometric Design of Highways and Streets*. Washington, D.C., 1994.

APPENDIX A. DESIGN AND OPERATIONAL TREATMENTS

Two-lane highways comprise approximately 80 percent of all paved rural highways in the United States but carry only about 30 percent of all traffic. For the most part, two-lane highways carry light volumes and experience few operational problems. Some two-lane highways, however, periodically experience significant operational and safety problems due to a variety of traffic, geometric, and environment causes. Such highways may require design or operation improvements to alleviate congestion.

When traffic operational problems occur on a two-lane highway, many highway agencies consider widening the highway to four lanes. Another effective method for alleviating operational problems on two-lane highways is to provide passing lanes at intervals in each direction of travel or to provide climbing lanes on steep upgrades. Passing and climbing lanes cannot increase the capacity of a two-lane highway but can improve its level of service. Short sections of four-lane highway can function as a pair of passing lanes in opposite directions of travel. Operational analysis procedures for passing and climbing lanes on two-lane highways are included in this chapter.

There are a number of other design and operational treatments that are effective in alleviating operational congestion on two-lane highways. These include

- Turnouts,
- Shoulder use,
- Wide cross sections,
- Intersection turn lanes, and
- Two-way left-turn lanes.

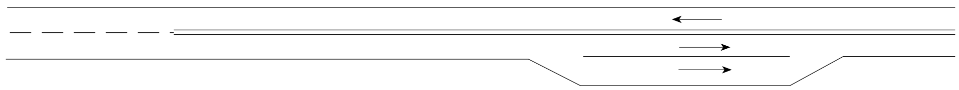
No calculation methodologies are provided in this chapter for these treatments; however, the treatments are discussed below to indicate their potential for improving traffic operations on two-lane highways.

TURNOUTS

A turnout is a widened, unobstructed shoulder area on a two-lane highway that allows slow-moving vehicles to pull out of the through lane so that vehicles following may pass. Turnouts are relatively short, generally less than 190 m. At a turnout, the driver of a slow-moving vehicle that is delaying one or more following vehicles is expected to pull out of the through lane, allowing the vehicles to pass. The driver of the slow-moving vehicle is expected to remain in the turnout only long enough for the following vehicles to pass and then should return to the through lane. When there are only one or two following vehicles, this maneuver usually can be completed smoothly and there is no need for the vehicle to stop in the turnout. When there are three or more following vehicles, however, the vehicle in the turnout will usually need to stop so that all of the following vehicles may pass. Signs inform motorists of the turnout's location and reinforce the legal requirements concerning turnout use.

Turnouts have been used in several countries to provide additional passing opportunities on two-lane highways. In the United States, turnouts have been used most extensively in the western states. Exhibit A20-1 illustrates a typical turnout.

EXHIBIT A20-1. TYPICAL TURNOUT TO INCREASE PASSING OPPORTUNITIES ON A TWO-LANE HIGHWAY



Turnouts may be used on nearly any type of two-lane highway that offers limited passing opportunities. Most often they appear on lower-volume highways in level or rolling terrain, on which long platoons are rare, and on difficult terrain with steep grades or with isolated slow-moving vehicles because the construction of a passing or climbing lane may not be cost-effective. To avoid confusing drivers, turnouts and passing lanes should not be intermixed on the same highway.

A single well-designed and well-located turnout can be expected to provide 20 to 50 percent of the number of passes that would occur in a 1.6-km passing lane in level terrain (*1, 2*). Turnouts have been found to operate safely—according to safety researchers turnout accidents occur at a rate of only 1 per 80,000 to 400,000 users (*2–4*).

SHOULDER USE

The primary purpose of the shoulder on a two-lane highway is to provide a stopping and recovery area for disabled or errant vehicles. However, paved shoulders also may be used to increase passing opportunities on a two-lane highway.

In some parts of the United States and Canada, if the paved shoulders are adequate, there is a longstanding custom for slower vehicles to move to the shoulder when another vehicle approaches from the rear and then return to the travel lane once the passing vehicle has cleared. This custom is regarded as a courtesy and requires little or no sacrifice in speed by either motorist. In this way, paved shoulders can function as continuous turnouts. A few highway agencies encourage drivers of slow-moving vehicles to use the shoulder in this way because it improves the LOS of two-lane highways without the expense of adding passing lanes or widening the highway. On the other hand, many highway agencies discourage this practice because their shoulders are not designed for frequent use by heavy vehicles.

One highway agency in the western United States generally does not permit shoulder use by slow-moving vehicles but designates specific sections on which the shoulder may be used by slow-moving vehicles. These shoulder-use sections range in length from 0.3 to 5.0 km and are identified by traffic signs.

Research has shown that a shoulder-use section is about 20 percent as effective in reducing platoons as a passing lane of comparable length (*1, 2*).

WIDE CROSS SECTIONS

Two-lane highways with lanes about 50 percent wider than normal have been used in several European countries as a less expensive alternative to passing lanes. Sweden, for example, has built approximately 800 km of roadways with two 5.5-m travel lanes and relatively narrow 1.0-m shoulders. The wider lane permits faster vehicles to pass slower vehicles while encroaching only slightly on the opposing lane of traffic. Opposing vehicles must move toward the shoulder to permit such maneuvers. Roadway sections with wider lanes can be provided at intervals, like passing lanes, to increase passing opportunities on two-lane highways.

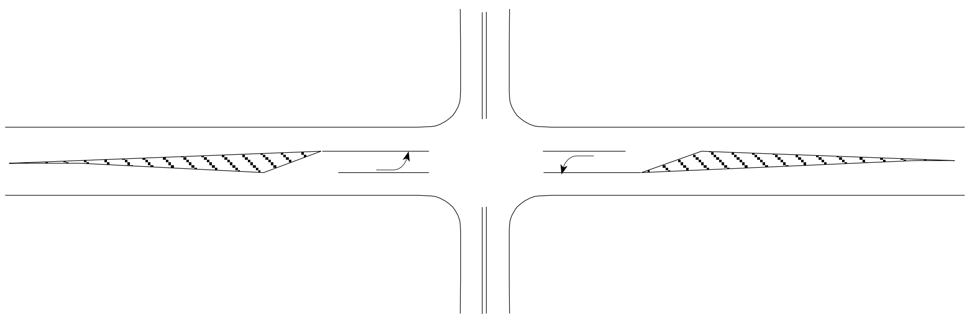
Research has found that speeds at low traffic volumes tend to increase on wider lanes, but the effect on speeds at higher volumes varies (*5*). More than 70 percent of drivers indicated that they appreciate the increased passing opportunities on the wider lanes. No safety problems have been associated with the wider lanes (*5*).

Formal procedures have not yet been established for evaluating the traffic operational effectiveness of wider lanes in increasing the passing opportunities on a two-lane highway. It is reasonable to estimate the traffic operational performance of a directional two-lane highway segment containing a section with widened lanes as midway between the same segment with and without a passing lane of comparable length.

INTERSECTION TURN LANES

Intersection turn lanes are desirable at selected locations on two-lane highways to reduce delays to through vehicles caused by turning vehicles and to reduce accidents related to turning. Separate right-turn and left-turn lanes may be provided, as appropriate, to remove turning vehicles from the through-travel lanes. Left-turn lanes, in particular, provide a protected location for turning vehicles to wait for a gap in opposing traffic. This reduces the potential for collisions from the rear and also may encourage drivers of left-turning vehicles to wait for an adequate gap in opposing traffic before turning. Exhibit A20-2 shows a typical two-lane highway intersection with left-turn lanes.

EXHIBIT A20-2. TYPICAL TWO-LANE HIGHWAY INTERSECTION WITH LEFT-TURN LANES

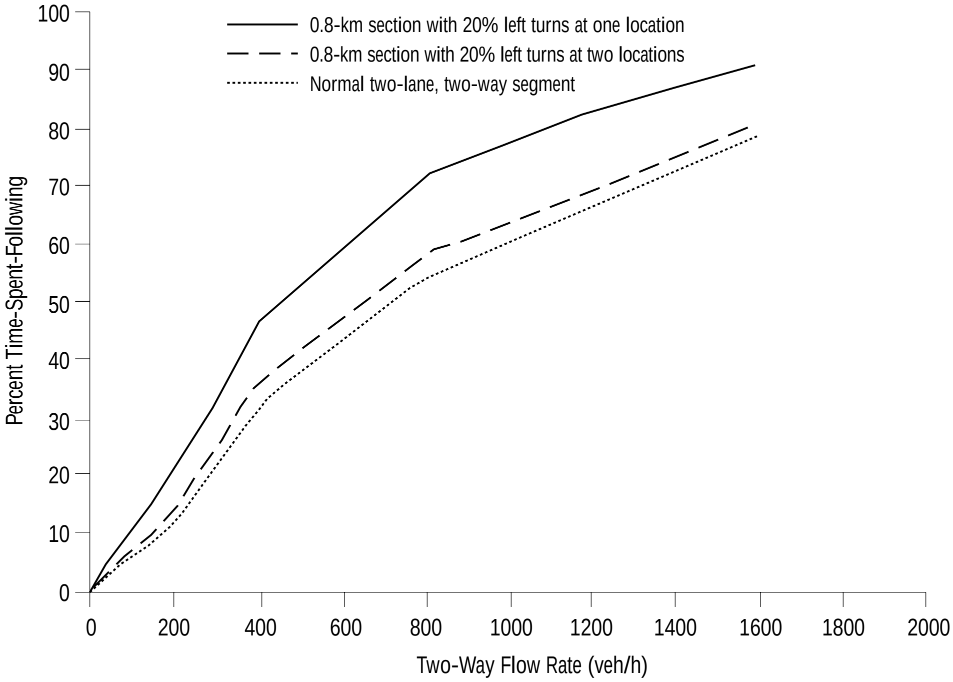


Research recommends specific traffic operational warrants for left-turn lanes at intersections on two-lane highways based on the directional volumes and the percentage of left turns (*6*). Intersection analysis with the methodologies of Chapter 16 for signalized intersections and Chapter 17 for unsignalized intersections can be used to quantify the effects of intersection turn lanes on delay at the intersection. There is no general methodology for estimating the effect of intersection turn lanes in increasing speed or reducing delay on the two-lane highway downstream. However, modeling of intersection delays shows the relative magnitude of likely effects of turning delays on percent time-spent-following (*7*); the results are shown in Exhibit A20-3. The top line in the exhibit shows that turning vehicles can increase percent time-spent-following substantially over a short road section. However, when these effects are averaged over a longer road section, the increase in percent time-spent-following is greatly reduced, as indicated by the dashed line in the exhibit. Provision of intersection turn lanes has the potential to minimize these delays.

Several highway agencies in the United States provide shoulder bypass lanes at three-leg intersections as a low-cost alternative to a left-turn lane. As shown in Exhibit A20-4, a portion of the paved shoulder opposite the minor-road leg may be marked as a lane for through traffic to bypass vehicles that are slowing or stopped to make a left turn. Shoulder bypass lanes may be appropriate for intersections that do not have volumes high enough to warrant a left-turn lane.

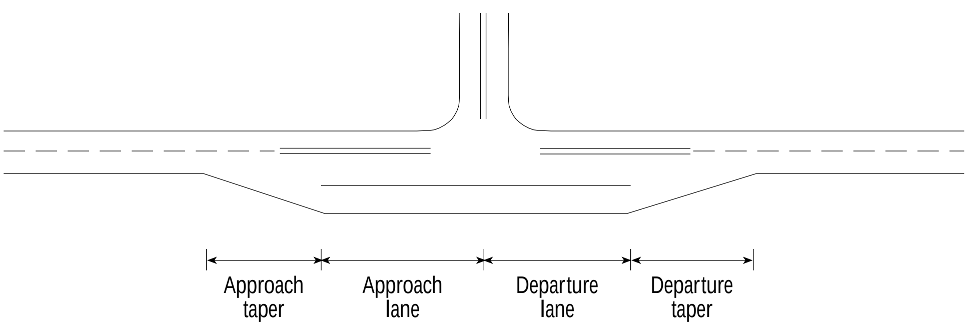
The delay reduction benefits of shoulder bypass lanes have not been quantified, but field studies have indicated that 97 percent of drivers who need to avoid delay will make use of an available shoulder bypass lane. One state has reported a marked decrease in rear-end collisions after shoulder bypass lanes were provided (*8*).

EXHIBIT A20-3. EFFECT OF TURNING DELAYS AT INTERSECTIONS ON PERCENT TIME-SPENT-FOLLOWING IN A TWO-LANE HIGHWAY



Source: Hoban (*7*).

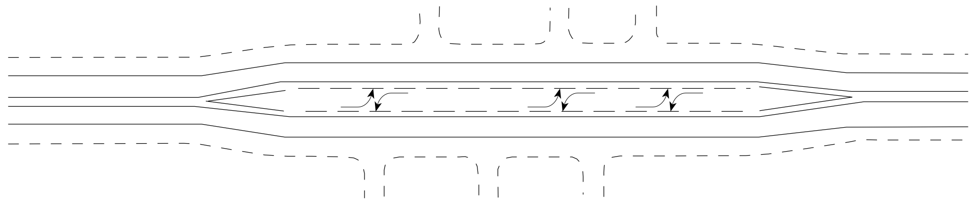
EXHIBIT A20-4. TYPICAL SHOULDER BYPASS LANE AT A THREE-LEG INTERSECTION ON A TWO-LANE HIGHWAY



TWO-WAY LEFT-TURN LANES

A two-way left-turn lane (TWLTL) is a paved area in the highway median that extends continuously along a roadway section and is marked to provide a deceleration and storage area, for vehicles traveling in either direction and making left turns at intersections and driveways. TWLTLs have been used for many years on urban and suburban streets with high driveway densities and turning demands to improve safety and reduce delays to through vehicles. TWLTLs also can be used on two-lane highways in rural and urban fringe areas to obtain these same types of operational and safety benefits. Exhibit A20-5 illustrates a typical TWLTL on a two-lane highway.

EXHIBIT A20-5. TYPICAL TWO-WAY LEFT-TURN LANE ON A TWO-LANE HIGHWAY

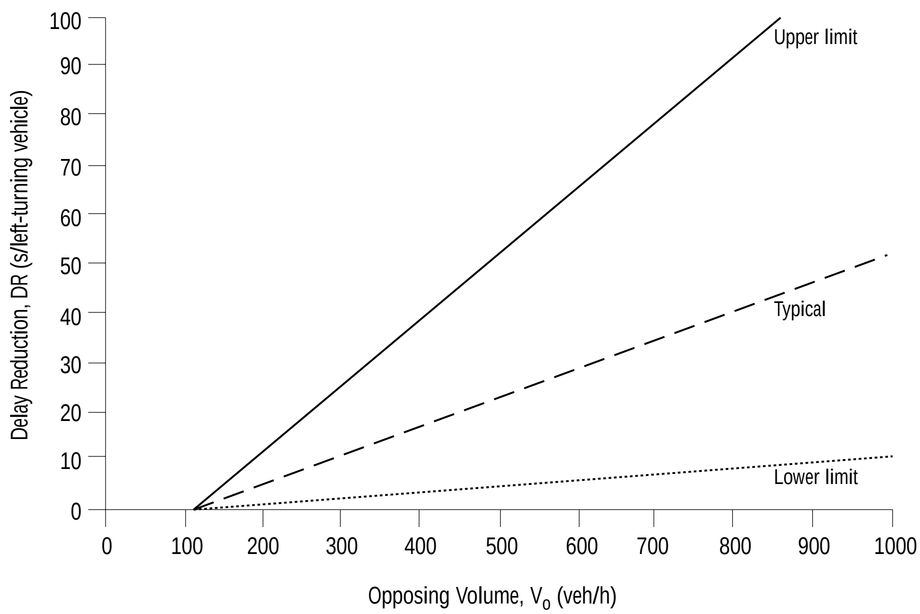


There is no formal methodology for evaluating the traffic operational effectiveness of TWLTLs on two-lane highways. Research has found that the delay reduction provided by a TWLTL depends on both the left-turn demand and the opposing traffic volume (*2*). Without a TWLTL or other left-turn treatment, vehicles that are slowing or stopped to make a left turn may create delays for following through vehicles. A TWLTL minimizes these delays and makes the roadway section operate more like two-way and directional segments with 100 percent no-passing zones. These research results apply to sites that do not have paved shoulders available for following vehicles to bypass turning vehicles. Paved shoulders may alleviate as much of the delay as a TWLTL.

Research has found little delay reduction at rural TWLTL sites with traffic volumes below 300 veh/h in one direction of travel (*2*). At several low-volume sites, no reduction was observed. The highest delay reduction observed was 3.4 s per left-turning vehicle. At low-volume rural sites, therefore, TWLTLs generally should be considered for reducing accidents but should not be expected to increase the operational performance of the highway.

At higher-volume urban fringe sites, greater delay reduction was found with TWLTLs on a two-lane highway. Exhibit A20-6 shows the expected delay reduction per left-turning vehicle as a function of opposing volume. As the delay reduction increases, a TWLTL can be justified for improving both traffic operation and safety.

EXHIBIT A20-6. ESTIMATED DELAY REDUCTION WITH A TWO-WAY LEFT-TURN LANE ON A TWO-LANE HIGHWAY WITHOUT PAVED SHOULDERS



Note:
$DR = -6.87 + 0.058V_o$
Source: Harwood and St. John (*2*).

REFERENCES

1. Harwood, D. W., and C. J. Hoban. *Low Cost Methods for Improving Traffic Operations on Two-Lane Roads—Informational Guide*. Report FHWA-IP-87/2, Federal Highway Administration, U.S. Department of Transportation, January 1987.
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8. Sebastian, O. L., and R. S. Pusey. *Paved-Shoulder Left-Turn By-Pass Lanes: A Report on the Delaware Experience*. Delaware Department of Transportation, October 1982.

APPENDIX B. WORKSHEETS

TWO-WAY TWO-LANE HIGHWAY SEGMENT WORKSHEET

DIRECTIONAL TWO-LANE HIGHWAY SEGMENT WORKSHEET

DIRECTIONAL TWO-LANE HIGHWAY SEGMENT WITH PASSING LANE WORKSHEET

TWO-WAY TWO-LANE HIGHWAY SEGMENT WORKSHEET

General Information		Site Information	
Analyst	____________	Highway	____________
Agency or Company	____________	From/To	____________
Date Performed	____________	Jurisdiction	____________
Analysis Time Period	____________	Analysis Year	____________

❑ Operational (LOS) ❑ Design (v_p) ❑ Planning (LOS) ❑ Planning (v_p)

Input Data

Shoulder width ________ m
Lane width ________ m
Lane width ________ m
Shoulder width ________ m

Segment length, L_t ________ km

Show North Arrow

❑ Class I highway ❑ Class II highway
Terrain ❑ Level ❑ Rolling
Two-way hourly volume ________ veh/h
Directional split ______ /_____
Peak-hour factor, PHF ________
% Trucks and buses, P_T ________ %
% Recreational vehicles, P_R ________ %
% No-passing zone ________ %
Access points/km ________/km

Average Travel Speed

Grade adjustment factor, f_G (Exhibit 20-7)	
Passenger-car equivalents for trucks, E_T (Exhibit 20-9)	
Passenger-car equivalents for RVs, E_R (Exhibit 20-9)	
Heavy-vehicle adjustment factor, f_{HV} $f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$	
Two-way flow rate,[1] v_p (pc/h) $v_p = \frac{V}{PHF * f_G * f_{HV}}$	
v_p * highest directional split proportion[2] (pc/h)	

Free-Flow Speed from Field Measurement	Estimated Free-Flow Speed
Field measured speed, S_{FM} ____________km/h	Base free-flow speed, BFFS ____________km/h
Observed volume, V_f ____________veh/h	Adj. for lane width and shoulder width, f_{LS} (Exhibit 20-5) _____km/h
Free-flow speed, FFS ____________km/h	Adj. for access points, f_A (Exhibit 20-6) ____________km/h
$FFS = S_{FM} + 0.0125\left(\frac{V_f}{f_{HV}}\right)$	Free-flow speed, FFS ____________km/h
	$FFS = BFFS - f_{LS} - f_A$

Adj. for no-passing zones, f_{np} (km/h) (Exhibit 20-11)	
Average travel speed, ATS (km/h) $ATS = FFS - 0.0125v_p - f_{np}$	

Percent Time-Spent-Following

Grade adjustment factor, f_G (Exhibit 20-8)	
Passenger-car equivalents for trucks, E_T (Exhibit 20-10)	
Passenger-car equivalents for RVs, E_R (Exhibit 20-10)	
Heavy-vehicle adjustment factor, f_{HV} $f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$	
Two-way flow rate,[1] v_p (pc/h) $v_p = \frac{V}{PHF * f_G * f_{HV}}$	
v_p * highest directional split proportion[2] (pc/h)	
Base percent time-spent-following, BPTSF (%) $BPTSF = 100(1 - e^{-0.000879 v_p})$	
Adj. for directional distribution and no-passing zone, $f_{d/np}$ (%) (Exhibit 20-12)	
Percent time-spent-following, PTSF (%) $PTSF = BPTSF + f_{d/np}$	

Level of Service and Other Performance Measures

Level of service, LOS (Exhibit 20-3 for Class I or 20-4 for Class II)	
Volume to capacity ratio, v/c $v/c = \frac{v_p}{3{,}200}$	
Peak 15-min vehicle-kilometers of travel, $VkmT_{15}$ (veh-km) $VkmT_{15} = 0.25L_t\left(\frac{V}{PHF}\right)$	
Peak-hour vehicle-kilometers of travel, $VkmT_{60}$ (veh-km) $VkmT_{60} = V * L_t$	
Peak 15-min total travel time, TT_{15} (veh-h) $TT_{15} = \frac{VkmT_{15}}{ATS}$	

Notes

1. If $v_p \geq 3{,}200$ pc/h, terminate analysis—the LOS is F.
2. If highest directional split $v_p \geq 1{,}700$ pc/h, terminate analysis—the LOS is F.

DIRECTIONAL TWO-LANE HIGHWAY SEGMENT WORKSHEET

General Information		**Site Information**	
Analyst	______	Highway/Direction of Travel	______
Agency or Company	______	From/To	______
Date Performed	______	Jurisdiction	______
Analysis Time Period	______	Analysis Year	______

❑ Operational (LOS) ❑ Design (v_p) ❑ Planning (LOS) ❑ Planning (v_p)

Input Data

Shoulder width ______ m

Opposing direction — Lane width ______ m

Analysis direction — Lane width ______ m

Shoulder width ______ m

Segment length, L_t ______ km

Show North Arrow

❑ Class I highway ❑ Class II highway

Terrain ❑ Level ❑ Rolling

Grade Length ____km Up/down ____%

Peak-hour factor, PHF ______

% Trucks and buses, P_T ______ %

% Recreational vehicles, P_R ______ %

% No-passing zone ______ %

Access points/km ______/km

Analysis direction volume, V_d ______veh/h Opposing direction volume, V_o ______veh/h

Average Travel Speed

	Analysis Direction (d)	Opposing Direction (o)
Passenger-car equivalent for trucks, E_T (Exhibit 20-9 or 20-15)		
Passenger-car equivalent for RVs, E_R (Exhibit 20-9 or 20-17)		
Heavy-vehicle adjustment factor,[5] f_{HV} $f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$		
Grade adjustment factor,[1] f_G (Exhibit 20-6 or 20-12)		
Directional flow rate,[2] v_i (pc/h) $v_i = \frac{V_i}{PHF * f_{HV} * f_G}$		

Free-Flow Speed from Field Measurement	Estimated Free-Flow Speed
Field measured speed,[3] S_{FM} ______km/h	Base free-flow speed,[3] BFFS ______km/h
Observed volume,[3] V_f ______veh/h	Adj. for lane width and shoulder width,[3] f_{LS} (Exh. 20-5) ______km/h
Free-flow speed, FFS_d ______km/h	Adj. for access points,[3] f_A (Exhibit 20-6) ______km/h
$FFS_d = S_{FM} + 0.0125 \left(\frac{V_f}{f_{HV}}\right)$	Free-flow speed, FFS_d ______km/h
	$FFS_d = BFFS - f_{LS} - f_A$
Adjustment for no-passing zones, f_{np} (km/h) (Exhibit 20-19)	
Average travel speed, ATS_d (km/h) $ATS_d = FFS_d - 0.0125(v_d + v_o) - f_{np}$	

Percent Time-Spent-Following

	Analysis Direction (d)	Opposing Direction (o)
Passenger-car equivalent for trucks, E_T (Exhibit 20-10 or 20-16)		
Passenger-car equivalent for RVs, E_R (Exhibit 20-10 or 20-16)		
Heavy-vehicle adjustment factor, f_{HV} $f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$		
Grade adjustment factor,[1] f_G (Exhibit 20-8 or 20-14)		
Directional flow rate,[2] v_i (pc/h) $v_i = \frac{V_i}{PHF * f_{HV} * f_G}$		
Base percent time-spent-following,[4] $BPTSF_d$ (%) $BPTSF_d = 100(1 - e^{av_d^b})$		
Adjustment for no-passing zone, f_{np} (Exhibit 20-20)		
Percent time-spent-following, $PTSF_d$ (%) $PTSF_d = BPTSF_d + f_{np}$		

Level of Service and Other Performance Measures

Level of service, LOS (Exhibit 20-3 or 20-4)	
Volume to capacity ratio, v/c $v/c = \frac{V_d}{1700}$	
Peak 15-min vehicle-kilometers of travel, $VkmT_{15}$ (veh-km) $VkmT_{15} = 0.25L_t\left(\frac{V_d}{PHF}\right)$	
Peak-hour vehicle-kilometers of travel, $VkmT_{60}$ (veh-km) $VkmT_{60} = V_d * L_t$	
Peak 15-min total travel time, TT_{15} (veh-h) $TT_{15} = \frac{VkmT_{15}}{ATS_d}$	

Notes

1. If the highway is extended segment (level) or rolling terrain, $f_G = 1.0$
2. If v_i (v_d or v_o) ≥ 1,700 pc/h, terminate analysis—the LOS is F.
3. For the analysis direction only.
4. Exhibit 20-21 provides factors a and b.
5. Use alternative Equation 20-14 if some trucks operate at crawl speeds on a specific downgrade.

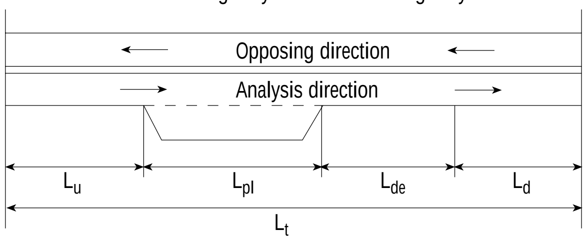
DIRECTIONAL TWO-LANE HIGHWAY SEGMENT WITH PASSING LANE WORKSHEET

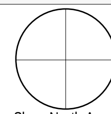
General Information		Site Information	
Analyst	____________	Highway/Direction of Travel	____________
Agency or Company	____________	From/To	____________
Date Performed	____________	Jurisdiction	____________
Analysis Time Period	____________	Analysis Year	____________

❑ Operational (LOS) ❑ Design (v_p) ❑ Planning (LOS) ❑ Planning (v_p)

Input Data

❑ Class I highway ❑ Class II highway





Show North Arrow

Total length of analysis segment, L_t (km)	
Length of two-lane highway upstream of the passing lane, L_u (km)	
Length of passing lane including tapers, L_{pl} (km)	
Average travel speed, ATS_d (from Directional Two-Lane Highway Segment Worksheet)	
Percent time-spent-following, $PTSF_d$ (from Directional Two-Lane Highway Segment Worksheet)	
Level of service,[1] LOS_d (from Directional Two-Lane Highway Segment Worksheet)	
Average Travel Speed	
Downstream length of two-lane highway within effective length of passing lane for average travel speed, L_{de} (km) (Exhibit 20-23)	
Length of two-lane highway downstream of effective length of the passing lane for average travel speed, L_d (km) $L_d = L_t - (L_u + L_{pl} + L_{de})$	
Adj. factor for the effect of passing lane on average speed, f_{pl} (Exhibit 20-24)	
Average travel speed including passing lane,[2] ATS_{pl} $ATS_{pl} = \dfrac{ATS_d * L_t}{L_u + L_d + \dfrac{L_{pl}}{f_{pl}} + \dfrac{2L_{de}}{1 + f_{pl}}}$	
Percent Time-Spent-Following	
Downstream length of two-lane highway within effective length of passing lane for percent time-spent-following, L_{de} (km) (Exhibit 20-23)	
Length of two-lane highway downstream of effective length of the passing lane for percent time-spent-following, L_d (km) $L_d = L_t - (L_u + L_{pl} + L_{de})$	
Adj. factor for the effect of passing lane on percent time-spent-following, f_{pl} (Exhibit 20-24)	
Percent time-spent-following including passing lane,[3] $PTSF_{pl}$ (%) $PTSF_{pl} = \dfrac{PTSF_d\left[L_u + L_d + f_{pl}L_{pl} + \left(\dfrac{1 + f_{pl}}{2}\right)L_{de}\right]}{L_t}$	
Level of Service and Other Performance Measures [4]	
Level of service including passing lane, LOS_{pl} (Exhibits 20-3 or 20-4)	
Peak 15-min total travel time, TT_{15} (veh-h) $TT_{15} = \dfrac{VkmT_{15}}{ATS_{pl}}$	

Notes

1. If LOS_d = F, passing lane analysis cannot be performed.
2. If $L_d < 0$, use alternative Equation 20-22.
3. If $L_d < 0$, use alternative Equation 20-20.
4. v/c, $VkmT_{15}$, and $VkmT_{60}$ are calculated on Directional Two-Lane Highway Segment Worksheet.

Lecture 12

CHAPTER 21

MULTILANE HIGHWAYS

CONTENTS

EXHIBITS

I. INTRODUCTION

For background and concepts, see Chapter 12, "Highway Concepts"

The procedures in this chapter are used to analyze the capacity, level of service (LOS), lane requirements, and impacts of traffic and design features of rural and suburban multilane highways.

The methodology in this chapter is based on the results of a National Cooperative Highway Research study (*1*). The study used additional references in developing the original methodology (*2–6*), which subsequently has been updated (*7*).

BASE CONDITIONS FOR MULTILANE HIGHWAYS

The procedures in this chapter determine the reduction in travel speed that occurs for less-than-base conditions. Under base conditions, the full speed and capacity of a multilane highway are achieved. These conditions include good weather, good visibility, and no incidents or accidents.

Studies of the flow characteristics of multilane highways have defined base conditions for developing flow relationships and adjustments to speed. The base conditions for multilane highways are as follows:

- 3.6-m minimum lane widths;
- 3.6-m minimum total lateral clearance in the direction of travel—this represents the total lateral clearances from the edge of the traveled lanes to obstructions along the edge of the road and in the median (in computations, lateral clearances greater than 1.8 m are considered in computations to be equal to 1.8 m);
- Only passenger cars in the traffic stream;
- No direct access points along the roadway;
- A divided highway; and
- Free-flow speed (FFS) higher than 100 km/h.

These base conditions represent the highest operating level of multilane rural and suburban highways.

LIMITATIONS OF THE METHODOLOGY

The methodology in this chapter does not take into account the following conditions:

- Transitory blockages caused by construction, accidents, or railroad crossings;
- Interference caused by parking on the shoulders (such as in the vicinity of a country store, flea market, or tourist attraction);
- Three-lane cross sections;
- The effect of lane drops and additions at beginning or end of segments;
- Possible queuing delays when transitions from a multilane segment into a two-lane segment are neglected;
- Differences between median barriers and two-way left-turn lanes; and
- FFS below 70 km/h or above 100 km/h.

II. METHODOLOGY

Methodology applies to signal spacing greater than 3.0 km

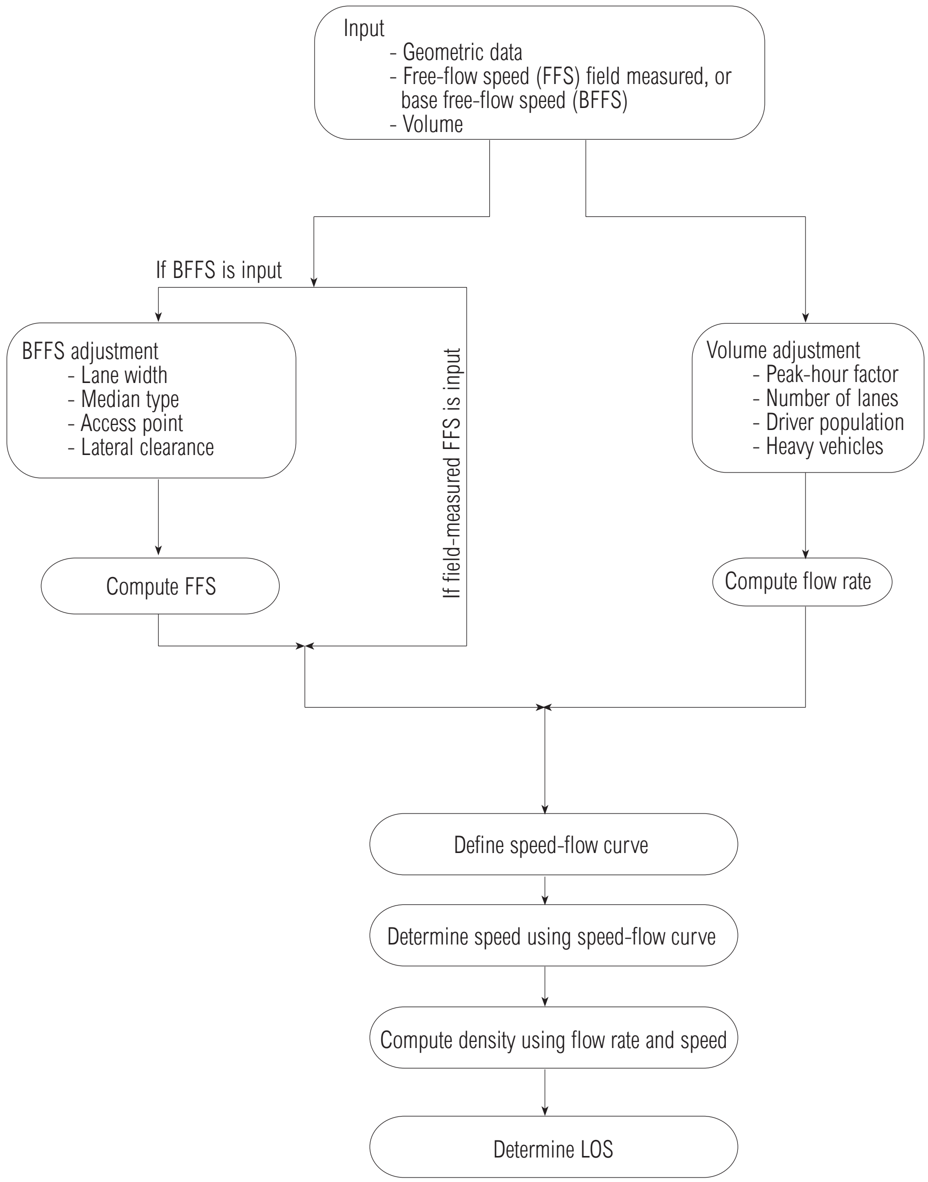
The methodology described in this chapter is intended for analysis of uninterrupted-flow highway segments. Chapter 15 presents the methodology for analyzing urban streets that have one or more of the following characteristics:

- Flow significantly influenced by other signals (i.e., a signal spacing less than or equal to 3.0 km),
- Significant presence of on-street parking,
- Presence of bus stops that have significant use, or
- Significant pedestrian activity.

Exhibit 21-1 illustrates the inputs and the basic computational order for the method described in this chapter. The primary output is LOS.

Uninterrupted-flow facilities that allow access solely through a system of on-ramps and off-ramps from grade separations or service roads are considered freeways and should be evaluated using the methodology presented in Chapter 23.

EXHIBIT 21-1. MULTILANE HIGHWAY METHODOLOGY



LOS

Although speed is a major concern of drivers, freedom to maneuver within the traffic stream and the proximity to other vehicles are also important. LOS criteria are listed in Exhibit 21-2. The criteria are based on the typical speed-flow and density-flow relationships shown in Exhibits 12-1 and 12-2. Exhibit 21-3 shows LOS boundaries as sloped lines, each corresponding to a constant value of density.

EXHIBIT 21-2. LOS CRITERIA FOR MULTILANE HIGHWAYS

		LOS				
Free-Flow Speed	Criteria	A	B	C	D	E
100 km/h	Maximum density (pc/km/ln)	7	11	16	22	25
	Average speed (km/h)	100.0	100.0	98.4	91.5	88.0
	Maximum volume to capacity ratio (v/c)	0.32	0.50	0.72	0.92	1.00
	Maximum service flow rate (pc/h/ln)	700	1100	1575	2015	2200
90 km/h	Maximum density (pc/km/ln)	7	11	16	22	26
	Average speed (km/h)	90.0	90.0	89.8	84.7	80.8
	Maximum v/c	0.30	0.47	0.68	0.89	1.00
	Maximum service flow rate (pc/h/ln)	630	990	1435	1860	2100
80 km/h	Maximum density (pc/km/ln)	7	11	16	22	27
	Average speed (km/h)	80.0	80.0	80.0	77.6	74.1
	Maximum v/c	0.28	0.44	0.64	0.85	1.00
	Maximum service flow rate (pc/h/ln)	560	880	1280	1705	2000
70 km/h	Maximum density (pc/km/ln)	7	11	16	22	28
	Average speed (km/h)	70.0	70.0	70.0	69.6	67.9
	Maximum v/c	0.26	0.41	0.59	0.81	1.00
	Maximum service flow rate (pc/h/ln)	490	770	1120	1530	1900

Note:
The exact mathematical relationship between density and volume to capacity ratio (v/c) has not always been maintained at LOS boundaries because of the use of rounded values. Density is the primary determinant of LOS. LOS F is characterized by highly unstable and variable traffic flow. Prediction of accurate flow rate, density, and speed at LOS F is difficult.

The LOS criteria reflect the shape of the speed-flow and density-flow curves, particularly as speed remains relatively constant across LOS A to D but is reduced as capacity is approached. For FFS of 100, 90, 80, and 70 km/h, Exhibit 21-2 gives the average speed, the maximum value of v/c, the maximum density, and the corresponding maximum service flow rate for each LOS.

As with other LOS criteria, the maximum service flow rates in Exhibit 21-2 are stated in terms of flow rate based on the peak 15-min volume. Demand or forecast hourly volumes generally are divided by the peak-hour factor (PHF) to reflect a maximum hourly flow rate before comparison with the criteria of Exhibit 21-2. Using the basic speed-flow curves (see Exhibit 21-3), the relationships between LOS, flow, and speed can be analyzed.

DETERMINING FFS

FFS is measured using the mean speed of passenger cars operating in low-to-moderate flow conditions (up to 1,400 pc/h/ln). Mean speed is virtually constant across this range of flow rates. Field measurement and estimation with guidelines provided in this chapter are methods that can be used to determine FFS.

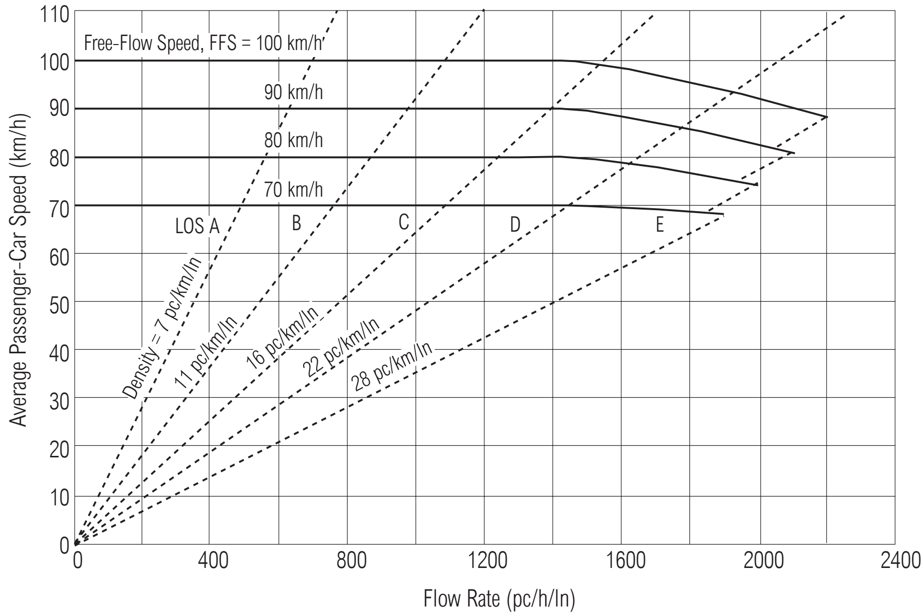
The field measurement procedure is for those who prefer to gather data directly or to incorporate the measurements into a speed-monitoring program. However, field measurements are not necessary to apply the method.

The FFS of a highway can be determined directly from a speed study conducted in the field. If field-measured data are used, no adjustments need to be made to FFS. The speed study should be conducted along a reasonable length of highway within the segment under evaluation; for example, an upgrade should not be selected within a site that is generally level. Any speed measurement technique acceptable for other types of traffic engineering speed studies can be used.

FFS occurs at flow rates ≤ 1,400 pc/h/ln

The field study should be conducted in the more stable regime of low-to-moderate flow conditions (up to 1,400 pc/h/ln). If the speed study must be conducted at a flow rate of more than 1,400 pc/h/ln, the FFS can be found by using the model speed-flow curve, assuming that data on traffic volumes are recorded at the same time.

EXHIBIT 21-3. SPEED-FLOW CURVES WITH LOS CRITERIA



Note:
Maximum densities for LOS E occur at a v/c ratio of 1.0. They are 25, 26, 27, and 28 pc/km/ln at FFS of 100, 90, 80, and 70 km/h, respectively. Capacity varies by FFS. Capacity is 2,200, 2,100, 2,000, and 1,900 pc/h/ln at FFS of 100, 90, 80, and 70 km/h, respectively.

For flow rate (v_p), v_p > 1400 and
$90 < FFS \le 100$ then

$$S = FFS - \left[\left(\frac{9.3}{25} FFS - \frac{630}{25} \right) \left(\frac{v_p - 1,400}{15.7FFS - 770} \right)^{1.31} \right]$$

For $v_p > 1,400$ and
$80 < FFS \le 90$ then

$$S = FFS - \left[\left(\frac{10.4}{26} FFS - \frac{696}{26} \right) \left(\frac{v_p - 1,400}{15.6FFS - 704} \right)^{1.31} \right]$$

For $v_p > 1,400$ and
$70 < FFS \le 80$ then

$$S = FFS - \left[\left(\frac{11.1}{27} FFS - \frac{728}{27} \right) \left(\frac{v_p - 1,400}{15.9FFS - 672} \right)^{1.31} \right]$$

For $v_p > 1,400$ and
FFS = 70 then

$$S = FFS - \left[\left(\frac{3}{28} FFS - \frac{75}{14} \right) \left(\frac{v_p - 1,400}{25FFS - 1,250} \right)^{1.31} \right]$$

For $v_p \le 1,400$, then
S = FFS

The speed study should measure the speeds of all passenger cars or of a systematic sampling of passenger cars (e.g., of every 10th passenger car). The speed study not only should measure speeds for unimpeded vehicles but also should include representative numbers of impeded vehicles. A sample should obtain at least 100 passenger-car speeds. Further guidance on the conduct of speed studies available in standard traffic engineering publications, such as the *Manual of Traffic Engineering Studies*, published by the Institute of Transportation Engineers (*6*).

The average passenger-car speed under low-volume conditions can be used as the free-flow speed if the field measurements were made at flow rates at or below 1,400 pc/h/ln. This FFS reflects the net effects of all conditions at the site that influence speed, including those identified in this procedure (lane width, lateral clearance, type of median, and access points), as well as others, such as speed limit and vertical and horizontal alignment.

Highway agencies with ongoing speed-monitoring programs or with speed data on file might prefer to use those data rather than conduct a new speed study or use an indirect method to estimate speed. The data can be used directly if collected in accordance with the procedures presented above. Data including both passenger-car and heavy-vehicle speeds probably can be used for level terrain or moderate downgrades, but they should not be used for rolling or mountainous terrain.

ESTIMATING FFS

The FFS can be estimated indirectly when field data are not available.

$$FFS = BFFS - f_{LW} - f_{LC} - f_M - f_A \tag{21-1}$$

where

- $BFFS$ = base FFS (km/h);
- FFS = estimated FFS (km/h);
- f_{LW} = adjustment for lane width, from Exhibit 21-4 (km/h);
- f_{LC} = adjustment for lateral clearance, from Exhibit 21-5 (km/h);
- f_M = adjustment for median type, from Exhibit 21-6 (km/h); and
- f_A = adjustment for access points, from Exhibit 21-7 (km/h).

Base FFS

When it is not possible to use data from a similar roadway, an estimate might be necessary, based on available data, experience, and consideration of the variety of factors that have an identified effect on FFS. The speed limit is one factor that affects FFS. Recent research suggests that FFS on multilane highways under base conditions is approximately 11 km/h higher than the speed limit for 65 and 70 km/h speed limits, and it is 8 km/h higher for 80 and 90 km/h speed limits. Chapter 12 provides default values for base FFS.

Adjustment for Lane Width

Base conditions for multilane highways require 3.6-m lane widths. Exhibit 21-4 presents the adjustment to modify the estimated FFS to account for narrower lanes. Exhibit 21-4 shows that 3.0 m and 3.3 m lanes reduce free-flow speeds by 10.6 km/h and 3.1 km/h, respectively. For Exhibit 21-4, lane widths greater than 3.6 m are considered 3.6 m. There are no research data for lane widths less than 3.0 m.

EXHIBIT 21-4. ADJUSTMENT FOR LANE WIDTH

Lane Width (m)	Reduction in FFS (km/h)
3.6	0.0
3.5	1.0
3.4	2.1
3.3	3.1
3.2	5.6
3.1	8.1
3.0	10.6

For undivided highways and highways with two-way left-turn lanes (TWLTL), the left edge lateral clearance equals 1.8 m

Adjustment for Lateral Clearance

Exhibit 21-5 lists the speed reductions caused by the lateral clearance for fixed obstructions on the roadside or in the median. Fixed obstructions with lateral clearance effects include light standards, signs, trees, abutments, bridge rails, traffic barriers, and retaining walls. Standard raised curbs are not considered obstructions. Exhibit 21-5 shows the appropriate reduction in FFS based on the total lateral clearance, which is defined as

$$TLC = LC_R + LC_L \tag{21-2}$$

where

TLC = total lateral clearance (m),
LC_R = lateral clearance (m), from the right edge of the travel lanes to roadside obstructions (if greater than 1.8 m, use 1.8 m), and
LC_L = lateral clearance (m), from the left edge of the travel lanes to obstructions in the roadway median (if the lateral clearance is greater than 1.8 m, use 1.8 m). For undivided roadways, there is no adjustment for left-side lateral clearance. The undivided design is taken into account by the median adjustment. To use Exhibit 21-5 for undivided highways, the lateral clearance on the left edge is always 1.8 m. Lateral clearance in the median of roadways with two-way left-turn lanes (TWLTLs) is considered to be 1.8 m.

EXHIBIT 21-5. ADJUSTMENT FOR LATERAL CLEARANCE

Four-Lane Highways		Six-Lane Highways	
Total Lateral Clearance[a] (m)	Reduction in FFS (km/h)	Total Lateral Clearance[a] (m)	Reduction in FFS (km/h)
3.6	0.0	3.6	0.0
3.0	0.6	3.0	0.6
2.4	1.5	2.4	1.5
1.8	2.1	1.8	2.1
1.2	3.0	1.2	2.7
0.6	5.8	0.6	4.5
0.0	8.7	0.0	6.3

Note:
a. Total lateral clearance is the sum of the lateral clearances of the median (if greater than 1.8 m, use 1.8 m) and shoulder (if greater than 1.8 m, use 1.8 m). Therefore, for purposes of analysis, total lateral clearance cannot exceed 3.6 m.

Thus, a total lateral clearance of 3.6 m is used for a completely unobstructed roadside and median; however, the actual value is used when obstructions are located closer to the roadway. The adjustment for lateral clearance on six-lane highways is slightly less than for four-lane highways because lateral obstructions have a minimal effect on traffic operations in the center lane of a three-lane roadway.

Median Type

The values in Exhibit 21-6 indicate that the average FFS should be decreased by 2.6 km/h for undivided highways to account for the friction caused by opposing traffic in an adjacent lane.

EXHIBIT 21-6. ADJUSTMENT FOR MEDIAN TYPE

Median Type	Reduction in FFS (km/h)
Undivided highways	2.6
Divided highways (including TWLTLs)	0.0

Adjustment for Access-Point Density

Exhibit 21-7 presents the adjustment to FFS for various levels of access-point density. The data indicate that for each access point per kilometer the estimated FFS decreases by approximately 0.4 km/h, regardless of the type of median. The access-point density on a divided roadway is determined by dividing the total number of access points (i.e., intersections and driveways) on the right side of the roadway in the direction of travel by the segment's total length in kilometers. An intersection or driveway should only be included if it influences traffic flow. Access points unnoticed by the driver or with little activity should not be included in determining access-point density.

EXHIBIT 21-7. ACCESS-POINT DENSITY ADJUSTMENT

Access Points/Kilometer	Reduction in FFS (km/h)
0	0.0
6	4.0
12	8.0
18	12.0
≥ 24	16.0

Guidelines for one-way highways

Although the access-point adjustments do not include data for one-way multilane highways, it might be appropriate to include intersections and driveways on both sides of a one-way roadway to determine the total number of access points per kilometer.

DETERMINING FLOW RATE

Two adjustments must be made to hourly volume counts or estimates to arrive at the equivalent passenger-car flow rate used in LOS analyses. These adjustments are the PHF and the heavy vehicle adjustment factor. The number of lanes also is used so that the flow rate can be expressed on a per-lane basis. These adjustments are applied in the following manner using Equation 21-3.

$$v_p = \frac{V}{PHF * N * f_{HV} * f_p} \tag{21-3}$$

where

v_p = 15-min passenger-car equivalent flow rate (pc/h/ln),
V = hourly volume (veh/h),
PHF = peak-hour factor,
N = number of lanes,
f_{HV} = heavy-vehicle adjustment factor, and
f_p = driver population factor.

PHF

PHF represents the variation in traffic flow within an hour. Observations of traffic flow consistently indicate that the flow rates found in the peak 15-min period within an hour are not sustained throughout the entire hour. The application of PHF in Equation 21-3 accounts for this phenomenon.

Heavy-Vehicle Adjustments

The presence of heavy vehicles in the traffic stream decreases the FFS because base conditions allow a traffic stream of passenger cars only. Therefore, traffic volumes must be adjusted to reflect an equivalent flow rate expressed in passenger cars per hour per lane (pc/h/ln). This is accomplished by applying the heavy-vehicle factor (f_{HV}). Once values for E_T and E_R have been determined, the adjustment factor for heavy vehicles may be computed as shown in Equation 21-4.

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)} \tag{21-4}$$

where

E_T and E_R = passenger-car equivalents for trucks and buses and for recreational vehicles (RVs), respectively;

P_T and P_R = proportion of trucks and buses, and RVs, respectively, in the traffic stream (expressed as a decimal fraction); and

f_{HV} = adjustment factor for heavy vehicles.

Adjustment for heavy vehicles in the traffic stream applies to three types of vehicles: trucks, RVs, and buses. No evidence indicates any distinct differences in the performance characteristics of trucks and buses on multilane highways; therefore, buses are considered trucks in this method. Finding the heavy-vehicle adjustment factor requires two steps. First, find an equivalent truck factor (E_T) and RV factor (E_R) for prevailing operating conditions. Second, using E_T and E_R, compute an adjustment factor for all heavy vehicles in the traffic stream.

Extended General Highway Segments

Passenger-car equivalents can be selected for two conditions: extended general highway segments and specific grades. Values of passenger-car equivalents are selected from Exhibits 21-8 through 21-11. For long segments of highway in which no single grade has a significant impact on operations, Exhibit 21-8 is used to select passenger-car equivalents for trucks and buses (E_T) and for RVs (E_R).

EXHIBIT 21-8. PASSENGER-CAR EQUIVALENTS ON EXTENDED GENERAL HIGHWAY SEGMENTS

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

A long multilane highway segment can be classified as an extended general highway segment if no grade exceeding 3 percent is longer than 0.8 km, and if grades of 3 percent or less do not exceed 1.6 km.

Specific Grade

Any grade of 3 percent or less that is longer than 1.6 km or a grade greater than 3 percent that is longer than 0.8 km should be treated as an isolated, specific grade. In addition, the upgrade and downgrade must be treated separately, because the impact of heavy vehicles differs substantially in each.

Equivalents for Extended General Highway Segments

For an extended general segment analysis, the terrain of the highway must be classified as level, rolling, or mountainous. These three classifications are discussed below.

Level Terrain

Level terrain is any combination of horizontal and vertical alignment that permits heavy vehicles to maintain approximately the same speed as passenger cars. This type of terrain generally includes short grades of no more than 1 to 2 percent.

Rolling Terrain

Rolling terrain is any combination of horizontal and vertical alignment that causes heavy vehicles to reduce their speeds substantially below those of passenger cars. However, the terrain does not cause heavy vehicles to operate at crawl speeds for any significant length of time or at frequent intervals.

Mountainous Terrain

Mountainous terrain is any combination of horizontal and vertical alignment that causes heavy vehicles to operate at crawl speeds for significant distances or at frequent intervals. For these general highway segments, values of E_T and E_R are selected from Exhibit 21-8.

Equivalents for Specific Grades

Any highway grade of more than 1.6 km for grades less than 3 percent or of 0.8 km for grades of 3 percent or more should be considered a separate segment. Analysis of such segments must consider the upgrade and downgrade conditions and whether the grade is single and isolated, with a constant percentage of change, or part of a series forming a composite grade.

Equivalents for Specific Upgrades

Exhibits 21-9 and 21-10 give passenger-car equivalents for trucks and buses (E_T) and for RVs (E_R), respectively, on uniform upgrades on four- and six-lane highways. Exhibit 21-9 is based on an average weight-to-power ratio of 100 kg/kW, which is typical of trucks on multilane highways in the United States.

Weight-to-power ratio for trucks

Equivalents for Specific Downgrades

Downgrade conditions for trucks and buses on four- or six-lane highways are analyzed using equivalents from Exhibit 21-11. For all downgrades less than 4 percent and for steeper downgrades less than or equal to 3.2 km long, use the passenger-car equivalents for trucks and buses in level terrain, given in Exhibit 21-8. For grades of at least 4 percent and longer than 3.2 km, use the specific values shown in Exhibit 21-11. For all cases of RVs on downgrades, use the passenger-car equivalents for level terrain, given in Exhibit 21-8.

Equivalents for Composite Grades

When several consecutive grades of different steepness form a composite grade, an average, uniform grade is computed and used in analysis. The average grade is commonly computed as the total rise from the beginning of the grade divided by the total horizontal distance over which the rise occurs.

The composite grade technique is reasonably accurate for segment lengths of 1200 m or less, or for grades of 4 percent or less. For steeper grades and longer segment lengths, a more exact technique is described in Appendix A of Chapter 23. If a large change in grade occurs for a significant length, the analyst should consider segmenting the roadway to apply the composite grade technique.

Generally, an average grade can be used to represent consecutive grades, but for a more detailed method, see Appendix A of Chapter 23

Sometimes a single, steep grade creates a critical effect that might not be identified in a length of highway to be analyzed; in this case, the composite grade technique can be supplemented by a specific grade analysis.

EXHIBIT 21-9. PASSENGER-CAR EQUIVALENTS FOR TRUCKS AND BUSES ON UNIFORM UPGRADES

Upgrade (%)	Length (km)	E_T Percentage of Trucks and Buses 2	4	5	6	8	10	15	20	25
<2	All	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
≥ 2–3	0.0–0.4	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	> 0.4–0.8	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	> 0.8–1.2	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	> 1.2–1.6	2.0	2.0	2.0	2.0	1.5	1.5	1.5	1.5	1.5
	> 1.6–2.4	2.5	2.5	2.5	2.5	2.0	2.0	2.0	2.0	2.0
	> 2.4	3.0	3.0	2.5	2.5	2.0	2.0	2.0	2.0	2.0
> 3–4	0.0–0.4	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	> 0.4–0.8	2.0	2.0	2.0	2.0	2.0	2.0	1.5	1.5	1.5
	> 0.8–1.2	2.5	2.5	2.0	2.0	2.0	2.0	2.0	2.0	2.0
	> 1.2–1.6	3.0	3.0	2.5	2.5	2.5	2.5	2.0	2.0	2.0
	> 1.6–2.4	3.5	3.5	3.0	3.0	3.0	3.0	2.5	2.5	2.5
	> 2.4	4.0	3.5	3.0	3.0	3.0	3.0	2.5	2.5	2.5
> 4–5	0.0–0.4	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	> 0.4–0.8	3.0	2.5	2.5	2.5	2.0	2.0	2.0	2.0	2.0
	> 0.8–1.2	3.5	3.0	3.0	3.0	2.5	2.5	2.5	2.5	2.5
	> 1.2–1.6	4.0	3.5	3.5	3.5	3.0	3.0	3.0	3.0	3.0
	> 1.6	5.0	4.0	4.0	4.0	3.5	3.5	3.0	3.0	3.0
> 5–6	0.0–0.4	2.0	2.0	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	> 0.4–0.5	4.0	3.0	2.5	2.5	2.0	2.0	2.0	2.0	2.0
	> 0.5–0.8	4.5	4.0	3.5	3.0	2.5	2.5	2.5	2.5	2.5
	> 0.8–1.2	5.0	4.5	4.0	3.5	3.0	3.0	3.0	3.0	3.0
	> 1.2–1.6	5.5	5.0	4.5	4.0	3.0	3.0	3.0	3.0	3.0
	> 1.6	6.0	5.0	5.0	4.5	3.5	3.5	3.5	3.5	3.5
> 6	0.0–0.4	4.0	3.0	2.5	2.5	2.5	2.5	2.0	2.0	2.0
	> 0.4–0.5	4.5	4.0	3.5	3.5	3.5	3.0	2.5	2.5	2.5
	> 0.5–0.8	5.0	4.5	4.0	4.0	3.5	3.0	2.5	2.5	2.5
	> 0.8–1.2	5.5	5.0	4.5	4.5	4.0	3.5	3.0	3.0	3.0
	> 1.2–1.6	6.0	5.5	5.0	5.0	4.5	4.0	3.5	3.5	3.5
	>1.6	7.0	6.0	5.5	5.5	5.0	4.5	4.0	4.0	4.0

EXHIBIT 21-10. PASSENGER-CAR EQUIVALENTS FOR RVS ON UNIFORM UPGRADES

Grade (%)	Length (km)	E_R Percentage of RVs 2	4	5	6	8	10	15	20	25
≤2	All	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
> 2–3	0.0–0.8	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
	> 0.8	3.0	1.5	1.5	1.5	1.5	1.5	1.2	1.2	1.2
> 3–4	0.0–0.4	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
	> 0.4–0.8	2.5	2.5	2.0	2.0	2.0	2.0	1.5	1.5	1.5
	> 0.8	3.0	2.5	2.5	2.5	2.0	2.0	2.0	1.5	1.5
> 4–5	0.0–0.4	2.5	2.0	2.0	2.0	1.5	1.5	1.5	1.5	1.5
	> 0.4–0.8	4.0	3.0	3.0	3.0	2.5	2.5	2.0	2.0	2.0
	> 0.8	4.5	3.5	3.0	3.0	3.0	2.5	2.5	2.0	2.0
> 5	0.0–0.4	4.0	3.0	2.5	2.5	2.5	2.0	2.0	2.0	1.5
	> 0.4–0.8	6.0	4.0	4.0	3.5	3.0	3.0	2.5	2.5	2.0
	> 0.8	6.0	4.5	4.0	4.5	3.5	3.0	3.0	2.5	2.0

EXHIBIT 21-11. PASSENGER-CAR EQUIVALENTS FOR TRUCKS ON DOWNGRADES

Downgrade (%)	Length (km)	E_T			
		Percentage of Trucks			
		5	10	15	20
< 4	All	1.5	1.5	1.5	1.5
4–5	≤ 6.4	1.5	1.5	1.5	1.5
4–5	> 6.4	2.0	2.0	2.0	1.5
> 5–6	≤ 6.4	1.5	1.5	1.5	1.5
> 5–6	> 6.4	5.5	4.0	4.0	3.0
> 6	≤ 6.4	1.5	1.5	1.5	1.5
> 6	> 6.4	7.5	6.0	5.5	4.5

Driver Population Factor

The adjustment factor f_p reflects the effect weekend recreational and perhaps even midday drivers have on the facility. The values for f_p range from 0.85 to 1.00. Typically, the analyst should select 1.00, which reflects weekday commuter traffic (i.e., users familiar with the highway), unless there is sufficient evidence that a lesser value, reflecting more recreational or weekend traffic characteristics, should be applied. When greater accuracy is needed, comparative field studies of weekday and weekend traffic flow and speeds are recommended.

DETERMINING LOS

The LOS on a multilane highway can be determined directly from Exhibit 21-3 on the basis of the FFS and the service flow rate (v_p) in pc/h/ln. The procedure is as follows:

- Step 1. Define and segment the highway as appropriate.
- Step 2. On the basis of the measured or estimated FFS, construct an appropriate speed-flow curve of the same shape as the typical curves shown in Exhibit 21-3. The curve should intercept the y-axis at the FFS.
- Step 3. Based on the flow rate v_p, read up to the FFS curve identified in Step 2 and determine the average passenger-car speed and LOS corresponding to that point.
- Step 4. Determine the density of flow according to Equation 21-5.

$$D = \frac{v_p}{S} \tag{21-5}$$

where

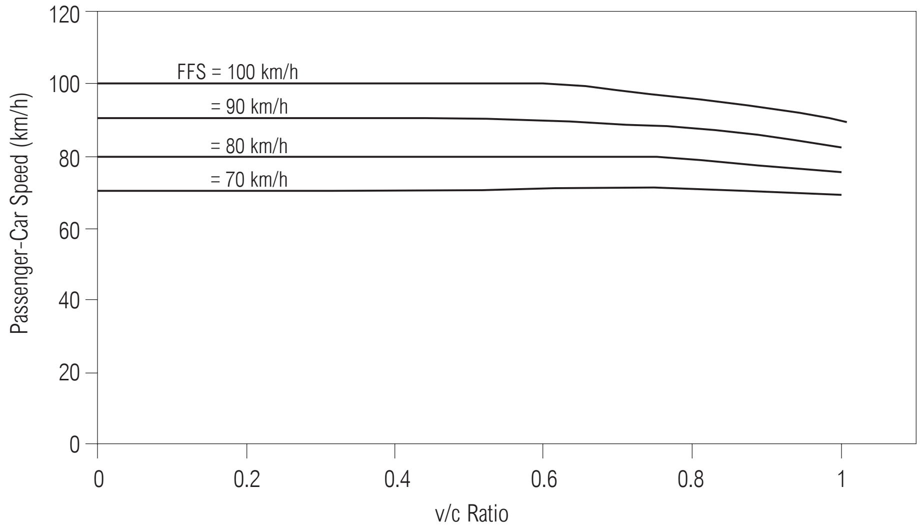
D = density (pc/km/ln),
v_p = flow rate (pc/h/ln), and
S = average passenger-car travel speed (km/h).

The LOS also can be determined by comparing the computed density with the density ranges provided in Exhibit 21-2.

SENSITIVITY OF RESULTS TO INPUT VARIABLES

Exhibit 21-12 shows the impact of v/c ratios on passenger-car speed for multilane highways. Note that speed is insensitive to demand until demand is at least 70 percent of capacity; also note that the mean speed on lower-speed segments is not sensitive to demand until the demand reaches at least 90 percent of capacity.

EXHIBIT 21-12. EFFECT OF v/c RATIO ON MEAN SPEED



For guidelines on required inputs and estimated values, see Chapter 12, “Highway Concepts”

III. APPLICATIONS

The methodology of this chapter can be used to analyze the capacity and LOS of multilane highways. The analyst must address two fundamental questions. First, the primary output must be identified. Primary outputs typically solved for in a variety of applications include LOS, number of lanes required (N), and flow rate achievable (v_p). Performance measures related to density (D) and speed (S) are also achievable but are considered secondary outputs.

Second, the analyst must identify the default values or estimated values for use in the analysis. Basically, the analyst has three sources of input data:

1. Default values found in this manual;
2. Estimates and locally derived default values developed by the user; and
3. Values derived from field measurements and observation.

For each of the input variables, a value must be supplied to calculate the outputs, both primary and secondary.

A common application of the method is to compute the LOS of an existing segment or of a changed segment in the near term or distant future. This type of application is often termed operational, and its primary output is LOS, with secondary outputs for density and speed. Another application is to check the adequacy or to recommend the required number of lanes for a multilane highway given the volume or flow rate and LOS goal. This is termed a design application since its primary output is the number of lanes required to serve the assumed conditions. Other outputs from this application include speed and density. Finally, the achievable flow rate v_p can be calculated as a primary output. This analysis requires stating a LOS goal and a number of lanes as inputs. This analysis typically estimates the point at which a flow rate will cause the highway to operate at an unacceptable LOS.

Another general type of analysis can be termed planning. These analyses use estimates, *Highway Capacity Manual* (HCM) default values, and local default values as inputs in the calculation. As outputs, LOS, number of lanes, or flow rate can be determined, along with the secondary outputs of density and speed. The difference between planning analysis and operational or design analysis is that most or all of the input values in planning come from estimates or default values, but the operational and design analyses tend to use field measurements or known values for most or all of the input variables. Note that for each of the analyses, FFS, either measured or estimated, is required as an input for the computation.

SEGMENTING THE HIGHWAY

The procedures described in this chapter are best applied to homogeneous segments of roadway, for which the variables affecting travel speeds are constant. Therefore, it is often necessary for the analyst to divide a section of highway into separate segments for analysis. The following conditions generally necessitate segmenting the highway:

- A change in the basic number of travel lanes along the highway,
- A change in the median treatment along the highway,
- A change of grade of 2 percent or more or a constant upgrade over 1220 m,
- The presence of a traffic signal or a stop sign along the multilane highway,
- A significant change in the density of access points,
- A change in speed limits, and
- The presence of a bottleneck condition.

Study segments should be at least 760 m long and 0.4 km from a signal

In general, when segmenting a highway for analysis, the minimum length of a study segment should be 760 m. Also, the limits of study segments should be no closer than 0.4 km to a signalized intersection. The procedures in this chapter are based on average conditions observed over an extended highway segment with generally consistent physical characteristics.

COMPUTATIONAL STEPS

The multilane highways worksheet for computations is shown in Exhibit 21-13. For all applications, the analyst provides general information and site information.

Operational (LOS) analysis

For operational (LOS) analysis, all speed and flow data are entered as inputs. Equivalent flow is then computed with the aid of the exhibits for passenger-car equivalencies. FFS is estimated by adjusting a base FFS. Finally, LOS is determined by entering (with v_p) the speed-flow graph at the top of the worksheet and intersecting the specific curve that has been selected or constructed for the highway segment.

This point of intersection identifies the LOS and, on the vertical axis of the graph, the estimated speed S. If the analyst requires a value for density D, it is calculated as v_p/S.

Design (N) analysis

The key to design analysis for number of lanes N is establishing an hourly volume. All information, with the exception of number of lanes, can be entered in the flow input and speed input portion of the worksheet (see Exhibit 21-13). An FFS, either computed or measured directly, is entered on the worksheet. The appropriate curve representing the FFS is established on the graph. The required or desired LOS is also entered. Then, the analyst assumes N and computes flow v_p with the aid of the exhibits for passenger-car equivalencies. LOS is determined by entering the speed-flow graph with v_p at the top of the worksheet. The derived LOS is compared with the desired LOS. This process is then repeated, adding one lane to the previously assumed number of lanes, until the determined LOS matches or is better than the desired LOS. Density is calculated using v_p and S.

Design (v_p) analysis

The objective of design analysis for flow rate v_p is to estimate the flow rate in pc/h/ln given a set of traffic, roadway, and FFS conditions. A desired LOS is entered on the worksheet. Then, the FFS of the segment is established using either the base FFS and the four adjustment factors or an FFS measured in the field. Once this segment speed-flow curve is established, the analyst can determine what flow rate is achievable with the given LOS. This would be considered the maximum flow rate achievable or allowable for the given level. Also directly available from the graph is the average passenger-car speed. Finally, if required, a value for density can be calculated, using v_p and S.

PLANNING APPLICATIONS

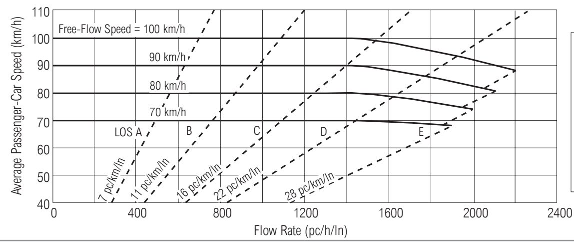
Planning (LOS), Planning (v_p), and Planning (N) applications

The three planning applications—planning for LOS, flow rate v_p, and number of lanes N—correspond directly to the procedures described for operations and design. The primary criterion categorizing these as planning applications is the use of estimates, HCM default values, and local default values for inputs into the calculations. The use of annual average daily traffic (AADT) to estimate directional design-hour volume (DDHV) also characterizes a planning application. (For guidelines on computing DDHV, refer to Chapter 8.)

To perform planning applications, the analyst typically has few, if any, of the required input values. Chapter 12 contains more information on the use of default values.

EXHIBIT 21-13. MULTILANE HIGHWAYS WORKSHEET

MULTILANE HIGHWAYS WORKSHEET



Application	Input	Output
Operational (LOS)	FFS, N, v_p	LOS, S, D
Design (N)	FFS, LOS, v_p	N, S, D
Design (v_p)	FFS, LOS, N	v_p, S, D
Planning (LOS)	FFS, N, AADT	LOS, S, D
Planning (N)	FFS, LOS, AADT	N, S, D
Planning (v_p)	FFS, LOS, N	v_p, S, D

General Information

Analyst ________
Agency or Company ________
Date Performed ________
Analysis Time Period ________

Site Information

Highway/Direction of Travel ________
From/To ________
Jurisdiction ________
Analysis Year ________

❑ Operational (LOS) ❑ Design (N) ❑ Design (v_p) ❑ Planning (LOS) ❑ Planning (N) ❑ Planning (v_p)

Flow Inputs

Volume, V ________veh/h
Annual avg. daily traffic, AADT ________veh/day
Peak-hour proportion of AADT, K ________
Peak-hour direction proportion, D ________
DDHV = AADT * K * D ________veh/h
Driver type
❑ Commuter/Weekday ❑ Recreational/Weekend

Peak-hour factor, PHF ________
% Trucks and buses, P_T ________
% RVs, P_R ________
General terrain
❑ Level ❑ Rolling ❑ Mountainous
Grade: Length ________km Up/Down ________%
Number of lanes ________

Calculate Flow Adjustments

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

Speed Inputs

Lane width, LW ________m
Total lateral clearance, TLC ________m
Access points, A ________A/km
Median type, M ❑ Undivided ❑ Divided
FFS (measured) ________km/h
Base free-flow Speed, BFFS ________km/h

Calculate Speed Adjustments and FFS

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

Operational, Planning (LOS); Design, Planning (v_p)

Operational (LOS) or Planning (LOS)

$v_p = \dfrac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$ ________pc/h/ln
S ________km/h
$D = v_p / S$ ________pc/km/ln
LOS ________

Design (v_p) or Planning (v_p)

LOS ________
v_p ________pc/h/ln
$V = v_p * PHF * N * f_{HV} * f_p$ ________veh/h
S ________km/h
$D = v_p/S$ ________pc/km/ln

Design, Planning (N)

Design (N) or Planning (N) 1st Iteration

N ________assumed
$v_p = \dfrac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$ ________pc/h/ln
LOS ________

Design (N) or Planning (N) 2nd Iteration

N ________assumed
$v_p = \dfrac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$ ________pc/h/ln
LOS ________
S ________km/h
$D = v_p / S$ ________pc/km/ln

Glossary

N - Number of lanes
V - Hourly volume
v_p - Flow rate
LOS - Level of service
DDHV - Directional design-hour volume
S - Speed
D - Density
FFS - Free-flow speed
BFFS - Base free-flow speed

Factor Location

E_T - Exhibit 21-8, 21-9, 21-11
E_R - Exhibit 21-8, 21-10
f_p - Page 21-11
LOS, S, FFS, v_p - Exhibit 21-2, 21-3
f_{LW} - Exhibit 21-4
f_{LC} - Exhibit 21-5
f_M - Exhibit 21-6
f_A - Exhibit 21-7

ANALYSIS TOOLS

The multilane highways worksheet shown in Exhibit 21-13 and provided in Appendix A can be used to perform all applications, including operational for LOS; design for flow rate v_p and number of lanes N; and planning for LOS, v_p, and N.

IV. EXAMPLE PROBLEMS

Problem No.	Description	Application
1	Find LOS on an undivided four-lane highway	Operational (LOS)
2	Find LOS on a five-lane highway with TWLTL	Operational (LOS)
3	Find the cross section required within a right-of-way to achieve desired LOS	Planning (N)
4	Find how much additional traffic can be accommodated by grade separation of a signalized intersection on a highway segment	Planning (v_p)
5	Find opening-day volume and number of lanes on a new suburban highway facility	Planning (N)

EXAMPLE PROBLEM 1 (PART I)

The Highway A 5.23-km undivided four-lane highway on level terrain. A 975-m segment with 2.5 percent grade also is included in the study.

The Question What are the peak-hour LOS, speed, and density for the level terrain portion of the highway?

The Facts

- √ Level terrain,
- √ 74.0-km/h field-measured FFS,
- √ 3.4-m lane width,
- √ 1,900-veh/h peak-hour volume,
- √ 13 percent trucks and buses,
- √ 2 percent RVs, and
- √ 0.90 PHF.

Outline of Solution All input parameters are known. Demand will be computed in terms of pc/h/ln, and the LOS determined from the speed-flow diagram. An estimate of passenger-car speed is determined from the graph, and a value of density is calculated using speed and flow rate.

Steps

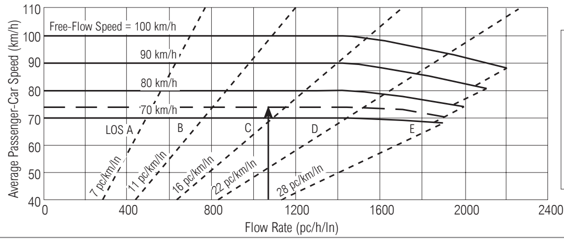
1. Find f_{HV} (use Exhibit 21-8 and Equation 21-4)	$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$ $f_{HV} = \frac{1}{1 + 0.13(1.5 - 1) + 0.02(1.2 - 1)}$ $f_{HV} = 0.935$
2. Find v_p (use Equation 21-3)	$v_p = \frac{V}{PHF * N * f_{HV} * f_p}$ $v_p = \frac{1,900}{0.90 * 2 * 0.935 * 1.00}$ $v_p = 1,129$ pc/h/ln
3. Determine LOS (use Exhibit 21-3)	LOS C

The Results

- LOS C,
- Speed = 74.0 km/h, and
- Density = 15.3 pc/km/ln.

Example Problem 1 (Part I)

MULTILANE HIGHWAYS WORKSHEET



Application	Input	Output
Operational (LOS)	FFS, N, v_p	LOS, S, D
Design (N)	FFS, LOS, v_p	N, S, D
Design (v_p)	FFS, LOS, N	v_p, S, D
Planning (LOS)	FFS, N, AADT	LOS, S, D
Planning (N)	FFS, LOS, AADT	N, S, D
Planning (v_p)	FFS, LOS, N	v_p, S, D

General Information

Analyst	JMYE
Agency or Company	EHI
Date Performed	5/16/99
Analysis Time Period	AM

Site Information

Highway/Direction of Travel	US 80 (East)
From/To	MP 17 - MP 20
Jurisdiction	M. County
Analysis Year	1999

☒ Operational (LOS) ☐ Design (N) ☐ Design (v_p) ☐ Planning (LOS) ☐ Planning (N) ☐ Planning (v_p)

Flow Inputs

Volume, V	1,900 veh/h	Peak-hour factor, PHF	0.90
Annual avg. daily traffic, AADT	______ veh/day	% Trucks and buses, P_T	13
Peak-hour proportion of AADT, K	______	% RVs, P_R	2
Peak-hour direction proportion, D	______	General terrain	
DDHV = AADT * K * D	______ veh/h	☒ Level ☐ Rolling ☐ Mountainous	
Driver type		Grade: Length ______ km	Up/Down ______ %
☒ Commuter/Weekday ☐ Recreational/Weekend		Number of lanes	2

Calculate Flow Adjustments

f_p	1.00	E_R	1.2
E_T	1.5	$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$	0.935

Speed Inputs

Lane width, LW	3.4 m
Total lateral clearance, TLC	______ m
Access points, A	______ A/km
Median type, M	☒ Undivided ☐ Divided
FFS (measured)	74.0 km/h
Base free-flow Speed, BFFS	______ km/h

Calculate Speed Adjustments and FFS

f_{LW}	______ km/h
f_{LC}	______ km/h
f_A	______ km/h
f_M	______ km/h
$FFS = BFFS - f_{LW} - f_{LC} - f_A - f_M$	______ km/h

Operational, Planning (LOS); Design, Planning (v_p)

Operational (LOS) or Planning (LOS)

$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$	1129 pc/h/ln
S	74.0 km/h
$D = v_p/S$	15.3 pc/km/ln
LOS	C

Design (v_p) or Planning (v_p)

LOS	______
v_p	______ pc/h/ln
$V = v_p * PHF * N * f_{HV} * f_p$	______ veh/h
S	______ km/h
$D = v_p/S$	______ pc/km/ln

Design, Planning (N)

Design (N) or Planning (N) 1st Iteration

N	______ assumed
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$	______ pc/h/ln
LOS	______

Design (N) or Planning (N) 2nd Iteration

N	______ assumed
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$	______ pc/h/ln
LOS	______
S	______ km/h
$D = v_p/S$	______ pc/km/ln

Glossary

- N - Number of lanes
- V - Hourly volume
- v_p - Flow rate
- LOS - Level of service
- DDHV - Directional design-hour volume
- S - Speed
- D - Density
- FFS - Free-flow speed
- BFFS - Base free-flow speed

Factor Location

- E_T - Exhibit 21-8, 21-9, 21-11
- E_R - Exhibit 21-8, 21-10
- f_p - Page 21-11
- LOS, S, FFS, v_p - Exhibit 21-2, 21-3
- f_{LW} - Exhibit 21-4
- f_{LC} - Exhibit 21-5
- f_M - Exhibit 21-6
- f_A - Exhibit 21-7

EXAMPLE PROBLEM 1 (PART II)

The Highway A 5.23-km undivided four-lane highway on level terrain. A 975-m segment with 2.5 percent grade also is included in the study.

The Question What are peak-hour LOS, speed, and density of traffic on the 2.5 percent grade? Does this operation still meet the minimum required LOS D?

The Facts
- √ 2.5 percent grade (upgrade and downgrade),
- √ 74.0-km/h field-measured FFS,
- √ 3.4-m lane width,
- √ 1,900-veh/h peak-hour volume,
- √ 13 percent trucks and buses,
- √ 2 percent RVs, and
- √ 0.90 PHF.

Comments
- √ For the 2.5 percent downgrade, trucks, buses, and RVs all operate as though on level terrain. Therefore, results obtained in Part I are applicable for downgrade results of the 2.5 percent grade segment.
- √ Assume FFS of 74.0 km/h applies to both upgrade and downgrade segments.

Outline of Solution All input parameters are known. Demand will be computed in terms of pc/h/ln, and the LOS determined from the speed-flow diagram. An estimate of passenger-car speed is determined from the graph, and a value of density is calculated using speed and flow rate.

Steps

1. Find f_{HV} (use Exhibits 21-9 and 21-10).	$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$ $f_{HV} = \frac{1}{1 + 0.13(1.5 - 1) + 0.02(3.0 - 1)}$ $f_{HV} = 0.905$
2. Find v_p.	$v_p = \frac{V}{PHF * N * f_{HV} * f_p}$ $v_p = \frac{1,900}{0.90 * 2 * 0.905 * 1.00}$ $v_p = 1,166$ pc/h/ln
3. Determine LOS (use Exhibit 21-3).	LOS C (upgrade) LOS C (downgrade)

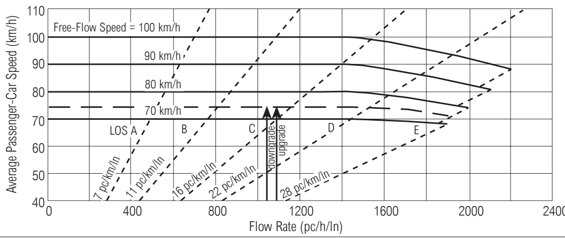
The Results

Downgrade:
- LOS C,
- Speed = 74.0 km/h, and
- Density = 15.3 pc/km/ln.

Upgrade:
- LOS C,
- Speed = 74.0 km/h, and
- Density = 15.8 pc/km/ln.

Example Problem 1 (Part II)

MULTILANE HIGHWAYS WORKSHEET



Application	Input	Output
Operational (LOS)	FFS, N, v_p	LOS, S, D
Design (N)	FFS, LOS, v_p	N, S, D
Design (v_p)	FFS, LOS, N	v_p, S, D
Planning (LOS)	FFS, N, AADT	LOS, S, D
Planning (N)	FFS, LOS, AADT	N, S, D
Planning (v_p)	FFS, LOS, N	v_p, S, D

General Information

Analyst	JMYE
Agency or Company	EHI
Date Performed	5/16/99
Analysis Time Period	AM

Site Information

Highway/Direction of Travel	US 80 (East)
From/To	MP 17 - MP 20
Jurisdiction	M. County
Analysis Year	1999

☒ Operational (LOS) ☐ Design (N) ☐ Design (v_p) ☐ Planning (LOS) ☐ Planning (N) ☐ Planning (v_p)

Flow Inputs

Volume, V	1,900 veh/h	Peak-hour factor, PHF	0.90
Annual avg. daily traffic, AADT	veh/day	% Trucks and buses, P_T	13
Peak-hour proportion of AADT, K		% RVs, P_R	2
Peak-hour direction proportion, D		General terrain	
DDHV = AADT * K * D	veh/h	☐ Level ☐ Rolling ☐ Mountainous	
Driver type		Grade: Length 0.975 km	Up/Down 2.5 (up) %
☒ Commuter/Weekday ☐ Recreational/Weekend		Number of lanes	2

Calculate Flow Adjustments

f_p	1.00	E_R	3.0
E_T	1.5	$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$	0.905

Speed Inputs

Lane width, LW	3.4 m
Total lateral clearance, TLC	m
Access points, A	A/km
Median type, M	☒ Undivided ☐ Divided
FFS (measured)	74.0 km/h
Base free-flow Speed, BFFS	km/h

Calculate Speed Adjustments and FFS

f_{LW}	km/h
f_{LC}	km/h
f_A	km/h
f_M	km/h
FFS = BFFS – f_{LW} – f_{LC} – f_A – f_M	km/h

Operational, Planning (LOS); Design, Planning (v_p)

Operational (LOS) or Planning (LOS)

$v_p = \frac{V \text{ or DDHV}}{PHF * N * f_{HV} * f_p}$	1166 pc/h/ln
S	74.0 km/h
D = v_p /S	15.8 pc/km/ln
LOS	C

Design (v_p) or Planning (v_p)

LOS	
v_p	pc/h/ln
V = v_p * PHF * N * f_{HV} * f_p	veh/h
S	km/h
D = v_p/S	pc/km/ln

Design, Planning (N)

Design (N) or Planning (N) 1st Iteration

N	assumed
$v_p = \frac{V \text{ or DDHV}}{PHF * N * f_{HV} * f_p}$	pc/h/ln
LOS	

Design (N) or Planning (N) 2nd Iteration

N	assumed
$v_p = \frac{V \text{ or DDHV}}{PHF * N * f_{HV} * f_p}$	pc/h/ln
LOS	
S	km/h
D = v_p /S	pc/km/ln

Glossary

- N - Number of lanes
- V - Hourly volume
- v_p - Flow rate
- LOS - Level of service
- DDHV - Directional design-hour volume
- S - Speed
- D - Density
- FFS - Free-flow speed
- BFFS - Base free-flow speed

Factor Location

- E_T - Exhibit 21-8, 21-9, 21-11
- E_R - Exhibit 21-8, 21-10
- f_p - Page 21-11
- LOS, S, FFS, v_p - Exhibit 21-2, 21-3
- f_{LW} - Exhibit 21-4
- f_{LC} - Exhibit 21-5
- f_M - Exhibit 21-6
- f_A - Exhibit 21-7

EXAMPLE PROBLEM 2 (PART I)

The Highway A 3.4-km segment of an east-west five-lane highway with two travel lanes in each direction separated by a two-way left-turn lane (TWLTL). The highway includes a 4 percent grade, 1830-m in length, followed by level terrain of 1570 m.

The Question What is the LOS of the highway on level terrain during the peak hour?

The Facts
- √ Level terrain,
- √ 3.6-m lane width,
- √ 6 percent trucks and buses,
- √ 6 access points/km (eastbound),
- √ 3.6-m and greater lateral clearance for westbound and eastbound,
- √ 83.0-km/h 85th-percentile speed,
- √ 1,500-veh/h peak-hour volume,
- √ 8 access points/km (westbound), and
- √ 0.90 PHF.

Comments
- √ Assume base FFS to be 3.0 km/h less than 85th-percentile speed.
  BFFS = 83.0 – 3.0 = 80.0 km/h
- √ Assume no RVs, since none is indicated.

Outline of Solution All input parameters are known. Demand will be computed in terms of pc/h/ln, an FFS estimate, and the LOS determined from the speed-flow diagram. An estimate of passenger-car speed is determined from the graph, and a value of density is calculated using speed and flow rate.

Steps

1. Find f_{HV} (EB and WB) (use Exhibit 21-8).	$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$ $f_{HV} = \frac{1}{1 + 0.06(1.5 - 1) + 0}$ $f_{HV} = 0.971$
2. Find v_p (EB and WB) (use Equation 21-3).	$v_p = \frac{V}{PHF * N * f_{HV} * f_p}$ $v_p = \frac{1{,}500}{0.90 * 2 * 0.971 * 1.00}$ v_p = 858 pc/h/ln
3. Compute EB and WB free-flow speeds (use Exhibits 21-4, 21-5, 21-6, 21-7, and Equation 21-1).	$FFS = BFFS - f_{LW} - f_{LC} - f_A - f_M$ FFS = 80 – 0.0 – 0.0 – 4.0 – 0.0 FFS = 76.0 km/h (EB) FFS = 80 – 0.0 – 0.0 – 5.3 – 0.0 FFS = 74.7 km/h (WB)
4. Determine LOS (use Exhibit 21-3).	LOS C (EB and WB)

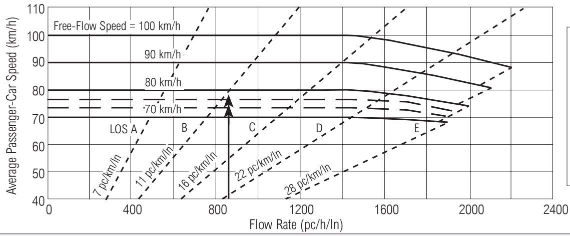
The Results

Eastbound:
- LOS C,
- Speed = 76.0 km/h, and
- Density = 11.3 pc/km/ln.

Westbound:
- LOS C,
- Speed = 74.7 km/h, and
- Density = 11.5 pc/km/ln.

Example Problem 2 (Part I)

MULTILANE HIGHWAYS WORKSHEET



Application	Input	Output
Operational (LOS)	FFS, N, v_p	LOS, S, D
Design (N)	FFS, LOS, v_p	N, S, D
Design (v_p)	FFS, LOS, N	v_p, S, D
Planning (LOS)	FFS, N, AADT	LOS, S, D
Planning (N)	FFS, LOS, AADT	N, S, D
Planning (v_p)	FFS, LOS, N	v_p, S, D

General Information

Analyst	JMYE
Agency or Company	EHI
Date Performed	5/16/99
Analysis Time Period	PM

Site Information

Highway/Direction of Travel	Buckeye Rd. (EB/WB)
From/To	50th - 58th St.
Jurisdiction	N. County
Analysis Year	1999

☒ Operational (LOS) ☐ Design (N) ☐ Design (v_p) ☐ Planning (LOS) ☐ Planning (N) ☐ Planning (v_p)

Flow Inputs

Volume, V	1,500 veh/h	Peak-hour factor, PHF	0.90
Annual avg. daily traffic, AADT	veh/day	% Trucks and buses, P_T	6
Peak-hour proportion of AADT, K		% RVs, P_R	0
Peak-hour direction proportion, D		General terrain	☒ Level ☐ Rolling ☐ Mountainous
DDHV = AADT * K * D	veh/h	Grade: Length ____ km	Up/Down ____ %
Driver type: ☒ Commuter/Weekday ☐ Recreational/Weekend		Number of lanes	2

Calculate Flow Adjustments

f_p	1.00	E_R	
E_T	1.5	$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$	0.971

Speed Inputs

Lane width, LW	3.6 m
Total lateral clearance, TLC	> 3.6 m
Access points, A	6/8 A/km
Median type, M	☐ Undivided ☒ Divided
FFS (measured)	km/h
Base free-flow Speed, BFFS	80 km/h

Calculate Speed Adjustments and FFS

f_{LW}	km/h
f_{LC}	km/h
f_A	4.0/5.3 km/h
f_M	km/h
FFS = BFFS – f_{LW} – f_{LC} – f_A – f_M	76.0/74.7 km/h

Operational, Planning (LOS); Design, Planning (v_p)

Operational (LOS) or Planning (LOS)

$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$	858 pc/h/ln
S	76.0/74.7 km/h
D = v_p /S	11.3/11.5 pc/km/ln
LOS	C/C

Design (v_p) or Planning (v_p)

LOS	
v_p	pc/h/ln
V = v_p * PHF * N * f_{HV} * f_p	veh/h
S	km/h
D = v_p/S	pc/km/ln

Design, Planning (N)

Design (N) or Planning (N) 1st Iteration

N	assumed
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$	pc/h/ln
LOS	

Design (N) or Planning (N) 2nd Iteration

N	assumed
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$	pc/h/ln
LOS	
S	km/h
D = v_p /S	pc/km/ln

Glossary

- N - Number of lanes
- V - Hourly volume
- v_p - Flow rate
- LOS - Level of service
- DDHV - Directional design-hour volume
- S - Speed
- D - Density
- FFS - Free-flow speed
- BFFS - Base free-flow speed

Factor Location

- E_T - Exhibit 21-8, 21-9, 21-11
- E_R - Exhibit 21-8, 21-10
- f_p - Page 21-11
- LOS, S, FFS, v_p - Exhibit 21-2, 21-3
- f_{LW} - Exhibit 21-4
- f_{LC} - Exhibit 21-5
- f_M - Exhibit 21-6
- f_A - Exhibit 21-7

EXAMPLE PROBLEM 2 (PART II)

The Highway A 3.4-km segment of an east-west five-lane highway with two travel lanes in each direction separated by a TWLTL. The highway characteristics include a 4 percent grade, 1830 m in length, followed by level terrain of 1570 m.

The Question What is the LOS of the 4 percent grade segment during the peak hour?

Additional Facts

- √ 4.0 percent grade (EB downgrade, WB upgrade),
- √ 87.0-km/h eastbound 85th-percentile speed,
- √ 77.0-km/h westbound 85th-percentile speed,
- √ 3.6-m lane width,
- √ 6 access points/km (EB), and
- √ 0 access points (WB).

Comments

- √ Assume base FFS to be 3.0 km/h less than 85th-percentile speed.
  BFFS (EB) = 87.0 – 3.0 = 84.0 km/h
- √ BFFS (WB) = 77.0 – 3.0 = 74.0 km/h
- √ Assume no RVs, since none indicated.

Outline of Solution All input parameters are known. Demand will be computed in terms of pc/h/ln, an FFS estimate, and the LOS determined from the speed-flow diagram. An estimate of passenger-car speed is determined from the graph, and a value of density is calculated using speed and flow rate.

Steps

1. Find f_{HV} (EB and WB) (use Exhibits 21-9 and 21-11).	$f_{HV} = \dfrac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$ $f_{HV} = \dfrac{1}{1 + 0.06(1.5 - 1) + 0} = 0.971$ (EB) $f_{HV} = \dfrac{1}{1 + 0.06(3.0 - 1) + 0} = 0.893$ (WB)
2. Find v_p (EB and WB).	$v_p = \dfrac{V}{PHF * N * f_{HV} * f_p}$ $v_p = \dfrac{1{,}500}{0.90 * 2 * 0.971 * 1.00} = 858$ pc/h/ln (EB) $v_p = \dfrac{1{,}500}{0.90 * 2 * 0.893 * 1.00} = 933$ pc/h/ln (WB)
3. Compute EB and WB FFS (use Exhibits 21-4, 21-5, 21-6, and 21-7).	$FFS = BFFS - f_{LW} - f_{LC} - f_A - f_M$ FFS = 84.0 – 0.0 – 0.0 – 4.0 – 0.0 = 80.0 km/h (EB) FFS = 74.0 – 0.0 – 0.0 – 0.0 – 0.0 = 74.0 km/h (WB)
4. Determine LOS (use Exhibit 21-3).	LOS B (EB) LOS C (WB)

The Results

Eastbound:

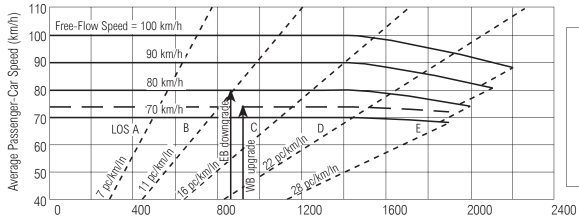
- LOS B,
- Speed = 80.0 km/h, and
- Density = 10.7 pc/km/ln.

Westbound:

- LOS C,
- Speed = 74.0 km/h, and
- Density = 12.6 pc/km/ln.

Example Problem 2 (Part II)

MULTILANE HIGHWAYS WORKSHEET



Application	Input	Output
Operational (LOS)	FFS, N, v_p	LOS, S, D
Design (N)	FFS, LOS, v_p	N, S, D
Design (v_p)	FFS, LOS, N	v_p, S, D
Planning (LOS)	FFS, N, AADT	LOS, S, D
Planning (N)	FFS, LOS, AADT	N, S, D
Planning (v_p)	FFS, LOS, N	v_p, S, D

General Information

Analyst	JMYE
Agency or Company	EHI
Date Performed	5/16/99
Analysis Time Period	PM

Site Information

Highway/Direction of Travel	Buckeye Rd. (EB/WB)
From/To	50th - 58th St.
Jurisdiction	N. County
Analysis Year	1999

☒ Operational (LOS) ☐ Design (N) ☐ Design (v_p) ☐ Planning (LOS) ☐ Planning (N) ☐ Planning (v_p)

Flow Inputs

Volume, V	1,500 veh/h	Peak-hour factor, PHF	0.90
Annual avg. daily traffic, AADT	veh/day	% Trucks and buses, P_T	6
Peak-hour proportion of AADT, K		% RVs, P_R	0
Peak-hour direction proportion, D		General terrain	
DDHV = AADT * K * D	veh/h	☐ Level ☐ Rolling ☐ Mountainous	
Driver type		Grade: Length 1.830 km	Up/Down 4 %
☒ Commuter/Weekday ☐ Recreational/Weekend		Number of lanes	2

Calculate Flow Adjustments

f_p	1.00	E_R	
E_T	1.5/3.0	$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$	0.971/0.893

Speed Inputs

Lane width, LW	3.6 m
Total lateral clearance, TLC	> 3.6 m
Access points, A	6/0 A/km
Median type, M	☐ Undivided ☒ Divided
FFS (measured)	km/h
Base free-flow Speed, BFFS	84.0/74.0 km/h

Calculate Speed Adjustments and FFS

f_{LW}	km/h
f_{LC}	km/h
f_A	4.0/0.0 km/h
f_M	km/h
FFS = BFFS – f_{LW} – f_{LC} – f_A – f_M	80.0/74.0 km/h

Operational, Planning (LOS); Design, Planning (v_p)

Operational (LOS) or Planning (LOS)

$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$	858/933 pc/h/ln
S	80.0/74.0 km/h
D = v_p /S	10.7/12.6 pc/km/ln
LOS	B/C

Design (v_p) or Planning (v_p)

LOS	
v_p	pc/h/ln
V = v_p * PHF * N * f_{HV} * f_p	veh/h
S	km/h
D = v_p/S	pc/km/ln

Design, Planning (N)

Design (N) or Planning (N) 1st Iteration

N	assumed
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$	pc/h/ln
LOS	

Design (N) or Planning (N) 2nd Iteration

N	assumed
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$	pc/h/ln
LOS	
S	km/h
D = v_p /S	pc/km/ln

Glossary

- N - Number of lanes
- V - Hourly volume
- v_p - Flow rate
- LOS - Level of service
- DDHV - Directional design-hour volume
- S - Speed
- D - Density
- FFS - Free-flow speed
- BFFS - Base free-flow speed

Factor Location

- E_T - Exhibit 21-8, 21-9, 21-11
- E_R - Exhibit 21-8, 21-10
- f_p - Page 21-11
- LOS, S, FFS, v_p - Exhibit 21-2, 21-3
- f_{LW} - Exhibit 21-4
- f_{LC} - Exhibit 21-5
- f_M - Exhibit 21-6
- f_A - Exhibit 21-7

EXAMPLE PROBLEM 3

The Highway A new 3.2-km segment of multilane highway with right-of-way width of 27.4 m.

The Question What is the cross section required to meet the design criterion of LOS D? What is the expected travel speed for passenger cars?

The Facts

√ 60,000 annual average daily traffic,
√ 80.0-km/h speed limit,
√ Peak-hour volume is 10 percent of daily traffic,
√ Peak-hour traffic has 55/45 directional split,
√ Rolling terrain,
√ 5 percent trucks,
√ 6 access points/km, and
√ 0.90 peak-hour factor.

Comments

√ This solution assumes that the given AADT is for the design year and that the other factors, although current, are accepted as representative of expected design year conditions.
√ Assume base FFS to be 8.0 km/h greater than the posted speed.
BFFS = 80.0 + 8.0 = 88.0 km/h

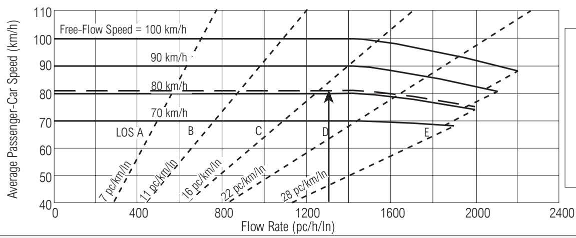
Steps

1. Convert AADT to design-hour volume.	DDHV = AADT * K * D DDHV = 60,000 * 0.10 * 0.55 = 3,300 veh/h
2. Find f_{HV} (use Exhibit 21-8). $f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$	$f_{HV} = \frac{1}{1 + 0.05(2.5 - 1) + 0} = 0.930$
3. Compute free-flow speed (use Exhibits 21-4, 21-5, 21-6, and 21-7).	$FFS = BFFS - f_{LW} - f_{LC} - f_A - f_M$ FFS = 88.0 – 0.0 – 0.0 – 4.0 – 0.0 = 84.0 km/h
4. Determine maximum v_p (use Exhibit 21-3).	v_p = 1,775 pc/h/ln
5. Determine minimum N required.	$N = \frac{V}{PHF * v_p * f_{HV} * f_p}$ $N = \frac{3,300}{0.90 * 1,775 * 0.930 * 1.00} = 2.2$ (use 3)
6. Compute v_p using minimum N required.	$v_p = \frac{3,300}{0.90 * 3 * 0.930 * 1.00} = 1,314$ pc/h/ln
7. Determine if base conditions will fit within available right-of-way with a 3.6-m median to accommodate left-turn bays in the future.	Median width = 3.6 m Lane width = 3.6 m Lateral clearance (shoulder) = 1.8 m Total required width = 3.6 + 6 * 3.6 + 2 * 1.8 = 28.8 m (greater than available width)
8. Assume different design to fit available right-of-way. Use 1.8-m median and do not use shoulder at median.	Median width = 1.8 m (raised) Lane width = 3.6 m Lateral clearance (shoulder) = 1.8 m Total required width = 1.8 + 21.6 + 2 * 1.8 = 27.0 m (fits within available 27.4 m)
9. Compute FFS (use Exhibits 21-4, 21-5, 21-6, and 21-7).	FFS = 88.0 – 0.0 – 0.0 – 4.0 – 0.0 = 84.0 km/h
10. Determine LOS (use Exhibit 21-3).	LOS D

The Results A six-lane highway with lane widths of 3.6 m, a 1.8-m median, and lateral clearances of 1.8 m on the right will meet the operational objective of LOS D during the peak-hour period. The passenger-car speed of 84.0 km/h and density of 15.6 pc/km/ln are computed.

Example Problem 3

MULTILANE HIGHWAYS WORKSHEET



Application	Input	Output
Operational (LOS)	FFS, N, v_p	LOS, S, D
Design (N)	FFS, LOS, v_p	N, S, D
Design (v_p)	FFS, LOS, N	v_p, S, D
Planning (LOS)	FFS, N, AADT	LOS, S, D
Planning (N)	FFS, LOS, AADT	N, S, D
Planning (v_p)	FFS, LOS, N	v_p, S, D

General Information

Analyst	JMYE
Agency or Company	EHI
Date Performed	5/16/99
Analysis Time Period	PM

Site Information

Highway/Direction of Travel	US 6 (N/E)
From/To	31st to 156th St.
Jurisdiction	M. County
Analysis Year	1999

❑ Operational (LOS) ❑ Design (N) ❑ Design (v_p) ❑ Planning (LOS) ☒ Planning (N) ❑ Planning (v_p)

Flow Inputs

Volume, V	____ veh/h	Peak-hour factor, PHF	0.90
Annual avg. daily traffic, AADT	60,000 veh/day	% Trucks and buses, P_T	5
Peak-hour proportion of AADT, K	0.10	% RVs, P_R	0
Peak-hour direction proportion, D	55/45	General terrain	
DDHV = AADT * K * D	3300 veh/h	❑ Level ☒ Rolling ❑ Mountainous	
Driver type		Grade: Length ____ km	Up/Down ____ %
☒ Commuter/Weekday ❑ Recreational/Weekend		Number of lanes	____

Calculate Flow Adjustments

f_p	1.00	E_R	0
E_T	2.5	$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$	0.930

Speed Inputs

Lane width, LW	3.6/3.6 m
Total lateral clearance, TLC	3.6 m
Access points, A	6 A/km
Median type, M	❑ Undivided ☒ Divided
FFS (measured)	____ km/h
Base free-flow Speed, BFFS	88.0 km/h

Calculate Speed Adjustments and FFS

f_{LW}	0.0/0.0 km/h
f_{LC}	0.0/0.0 km/h
f_A	4.0/4.0 km/h
f_M	0.0 km/h
FFS = BFFS – f_{LW} – f_{LC} – f_A – f_M	84.0/84.0 km/h

Operational, Planning (LOS); Design, Planning (v_p)

Operational (LOS) or Planning (LOS)

$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$	____ pc/h/ln
S	____ km/h
D = v_p /S	____ pc/km/ln
LOS	____

Design (v_p) or Planning (v_p)

LOS	____
v_p	____ pc/h/ln
V = v_p * PHF * N * f_{HV} * f_p	____ veh/h
S	____ km/h
D = v_p/S	____ pc/km/ln

Design, Planning (N)

Design (N) or Planning (N) 1st Iteration

N	2.2 assumed
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$	1,775 pc/h/ln
LOS	D

Design (N) or Planning (N) 2nd Iteration

N	3 assumed
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$	1,314 pc/h/ln
LOS	C
S	84.0 km/h
D = v_p /S	15.6 pc/km/ln

Glossary

N - Number of lanes
V - Hourly volume
v_p - Flow rate
LOS - Level of service
DDHV - Directional design-hour volume
S - Speed
D - Density
FFS - Free-flow speed
BFFS - Base free-flow speed

Factor Location

E_T - Exhibit 21-8, 21-9, 21-11
E_R - Exhibit 21-8, 21-10
f_p - Page 21-11
LOS, S, FFS, v_p - Exhibit 21-2, 21-3
f_{LW} - Exhibit 21-4
f_{LC} - Exhibit 21-5
f_M - Exhibit 21-6
f_A - Exhibit 21-7

EXAMPLE PROBLEM 4

The Highway A 4.0-km segment of a six-lane highway in a growing urban area to be improved.

The Question What is the estimated LOS for the existing and improved roadway? How much additional traffic can be added and still maintain the improved LOS?

The Facts

√ 1,400 pc/h/ln flow rate,
√ Free-flow travel time is 180 s, and
√ Improved free-flow travel time is 150 s.

Comments

This problem involves upgrading the design of a substandard section of multilane highway. The substandard highway has a measured FFS of 80 km/h. It is proposed to upgrade the design to a 96-km/h FFS through wider shoulders, widening the lanes to current standards, straightening the alignment on a few critical curves, restricting access to fronting properties and constructing a median.

Outline of Solution Using given peak-hour volume and FFS, determine LOS for improved and for current conditions. Determine additional volume that can be accommodated while still maintaining the improved LOS.

Steps

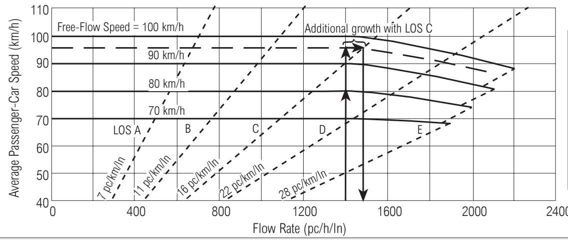
1. Determine LOS and speed of existing highway (use Exhibit 21-3).	v_p = 1,400 pc/h/ln, S = 80.0 km/h, LOS D
2. Determine maximum allowable flow at improved LOS and FFS (use Exhibit 21-3).	v_p = 1,400 pc/h/ln, FFS = 96.0 km/h, LOS C, Speed = 96.0 km/h
3. Compute additional volume.	Additional volume = 1,536 – 1,400 = 136 pc/h/ln

The Results

- Currently LOS D,
- Improved LOS C,
- Additional volume = 136 pc/h/ln,
- Speed = 96.0 km/h, and
- Density = 14.6 pc/km/ln.

Example Problem 4

MULTILANE HIGHWAYS WORKSHEET



Application	Input	Output
Operational (LOS)	FFS, N, v_p	LOS, S, D
Design (N)	FFS, LOS, v_p	N, S, D
Design (v_p)	FFS, LOS, N	v_p, S, D
Planning (LOS)	FFS, N, AADT	LOS, S, D
Planning (N)	FFS, LOS, AADT	N, S, D
Planning (v_p)	FFS, LOS, N	v_p, S, D

General Information

Analyst	JMYE
Agency or Company	EHI
Date Performed	5/16/99
Analysis Time Period	PM

Site Information

Highway/Direction of Travel	Georgia Dr.
From/To	Meno to Woodstock
Jurisdiction	M. County
Analysis Year	1999

☐ Operational (LOS) ☐ Design (N) ☒ Design (v_p) ☐ Planning (LOS) ☐ Planning (N) ☐ Planning (v_p)

Flow Inputs

Volume, V	____ veh/h	Peak-hour factor, PHF	____
Annual avg. daily traffic, AADT	____ veh/day	% Trucks and buses, P_T	____
Peak-hour proportion of AADT, K	____	% RVs, P_R	____
Peak-hour direction proportion, D	____	General terrain	
DDHV = AADT * K * D	____ veh/h	☒ Level ☐ Rolling ☐ Mountainous	
Driver type		Grade: Length ____ km	Up/Down ____ %
☐ Commuter/Weekday ☐ Recreational/Weekend		Number of lanes	3

Calculate Flow Adjustments

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

Speed Inputs

Lane width, LW	____ m
Total lateral clearance, TLC	____ m
Access points, A	____ A/km
Median type, M	☐ Undivided ☒ Divided
FFS (measured)	96.0 km/h
Base free-flow Speed, BFFS	____ km/h

Calculate Speed Adjustments and FFS

f_{LW}	____ km/h
f_{LC}	____ km/h
f_A	____ km/h
f_M	____ km/h
FFS = BFFS − f_{LW} − f_{LC} − f_A − f_M	____ km/h

Operational, Planning (LOS); Design, Planning (v_p)

Operational (LOS) or Planning (LOS)

$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$	____ pc/h/ln
S	____ km/h
D = v_p /S	____ pc/km/ln
LOS	____

Design (v_p) or Planning (v_p)

LOS	____
v_p	1,536 - 1,400 = 136 pc/h/ln
V = v_p * PHF * N * f_{HV} * f_p	____ veh/h
S	96.0 km/h
D = v_p/S	14.6 pc/km/ln

Design, Planning (N)

Design (N) or Planning (N) 1st Iteration

N	____ assumed
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$	____ pc/h/ln
LOS	____

Design (N) or Planning (N) 2nd Iteration

N	____ assumed
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$	____ pc/h/ln
LOS	____
S	____ km/h
D = v_p /S	____ pc/km/ln

Glossary

- N - Number of lanes
- V - Hourly volume
- v_p - Flow rate
- LOS - Level of service
- DDHV - Directional design-hour volume
- S - Speed
- D - Density
- FFS - Free-flow speed
- BFFS - Base free-flow speed

Factor Location

- E_T - Exhibit 21-8, 21-9, 21-11
- E_R - Exhibit 21-8, 21-10
- f_p - Page 21-11
- LOS, S, FFS, v_p - Exhibit 21-2, 21-3
- f_{LW} - Exhibit 21-4
- f_{LC} - Exhibit 21-5
- f_M - Exhibit 21-6
- f_A - Exhibit 21-7

EXAMPLE PROBLEM 5

The Highway New suburban facility under planning with an opening-day AADT forecast of 42,000 veh/day.

The Question For opening-day volumes, how many lanes will be needed to provide LOS C during the peak hour?

The Facts

√ 42,000 veh/day,
√ Rolling terrain, and
√ 10 percent trucks.

Comments

√ Several input variables are not given. Reasonable default values based on the traffic engineer's knowledge of local conditions are selected as 10 percent trucks, 0 percent RVs, lane width of 3.6 m, undivided highway, K = 0.10, directional split of 60/40, BFFS = 90.0 km/h, access-point density of 4 access points/km, PHF = 0.90, and shoulder width of 1.8 m.
√ Assume commuter traffic (f_p = 1.00).

Outline of Solution Using the multilane highways worksheet (Appendix A), determine required lane configuration.

Steps

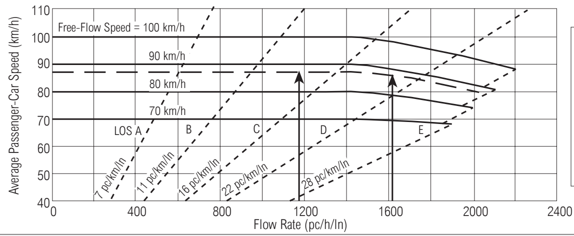
1. Convert AADT to directional design-hour volume (DDHV).	DDHV = AADT * K * D DDHV = 42,000 * 0.10 * 0.60 DDHV = 2,520 veh/h
2. Find f_{HV} (use Exhibit 21-8).	$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$ $f_{HV} = \frac{1}{1 + 0.1(2.5 - 1) + 0} = 0.870$
3. Compute free-flow speed (use Exhibits 21-4, 21-5, 21-6, and 21-7).	$FFS = BFFS - f_{LW} - f_{LC} - f_A - f_M$ FFS = 90.0 – 0.0 – 0.0 – 2.7 – 2.6 = 84.7 km/h
4. Assume four-lane highway and compute v_p (use Equation 21-3).	$v_p = \frac{2,520}{0.90 * 0.870 * 2 * 1.00} = 1,609$ pc/h/ln
5. Determine LOS (use Exhibit 21-3).	LOS D
6. Assume six-lane highway and compute v_p (use Equation 21-3).	$v_p = \frac{2,520}{0.90 * 0.870 * 3 * 1.00} = 1,073$ pc/h/ln
7. Determine LOS (use Exhibit 21-3).	LOS C

The Results

- A 6-lane freeway is needed,
- LOS C,
- Speed = 84.7 km/h, and
- D = 12.7 pc/km/h.

Example Problem 5

MULTILANE HIGHWAYS WORKSHEET



Application	Input	Output
Operational (LOS)	FFS, N, v_p	LOS, S, D
Design (N)	FFS, LOS, v_p	N, S, D
Design (v_p)	FFS, LOS, N	v_p, S, D
Planning (LOS)	FFS, N, AADT	LOS, S, D
Planning (N)	FFS, LOS, AADT	N, S, D
Planning (v_p)	FFS, LOS, N	v_p, S, D

General Information

Analyst	JMYE
Agency or Company	EHI
Date Performed	5/16/99
Analysis Time Period	PM

Site Information

Highway/Direction of Travel	
From/To	
Jurisdiction	M. County
Analysis Year	1999

☐ Operational (LOS) ☐ Design (N) ☐ Design (v_p) ☐ Planning (LOS) ☒ Planning (N) ☐ Planning (v_p)

Flow Inputs

Volume, V	veh/h
Annual avg. daily traffic, AADT	42,000 veh/day
Peak-hour proportion of AADT, K	0.10
Peak-hour direction proportion, D	0.60
DDHV = AADT * K * D	2520 veh/h
Driver type	☒ Commuter/Weekday ☐ Recreational/Weekend
Peak-hour factor, PHF	0.90
% Trucks and buses, P_T	10
% RVs, P_R	0
General terrain	☐ Level ☒ Rolling ☐ Mountainous
Grade: Length ___ km	Up/Down ___ %
Number of lanes	

Calculate Flow Adjustments

f_p	1.00
E_T	2.5
E_R	
$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$	0.870

Speed Inputs

Lane width, LW	3.6 m
Total lateral clearance, TLC	3.6 m
Access points, A	4 A/km
Median type, M	☒ Undivided ☐ Divided
FFS (measured)	km/h
Base free-flow Speed, BFFS	90 km/h

Calculate Speed Adjustments and FFS

f_{LW}	0.0 km/h
f_{LC}	0.0 km/h
f_A	2.7 km/h
f_M	2.6 km/h
$FFS = BFFS - f_{LW} - f_{LC} - f_A - f_M$	84.7 km/h

Operational, Planning (LOS); Design, Planning (v_p)

Operational (LOS) or Planning (LOS)

$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$	pc/h/ln
S	km/h
$D = v_p / S$	pc/km/ln
LOS	

Design (v_p) or Planning (v_p)

LOS	
v_p	pc/h/ln
$V = v_p * PHF * N * f_{HV} * f_p$	veh/h
S	km/h
$D = v_p/S$	pc/km/ln

Design, Planning (N)

Design (N) or Planning (N) 1st Iteration

N	2 assumed
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$	1,609 pc/h/ln
LOS	D

Design (N) or Planning (N) 2nd Iteration

N	3 assumed
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$	1,073 pc/h/ln
LOS	C
S	84.7 km/h
$D = v_p / S$	12.7 pc/km/ln

Glossary

- N - Number of lanes
- V - Hourly volume
- v_p - Flow rate
- LOS - Level of service
- DDHV - Directional design-hour volume
- S - Speed
- D - Density
- FFS - Free-flow speed
- BFFS - Base free-flow speed

Factor Location

- E_T - Exhibit 21-8, 21-9, 21-11
- E_R - Exhibit 21-8, 21-10
- f_p - Page 21-11
- LOS, S, FFS, v_p - Exhibit 21-2, 21-3
- f_{LW} - Exhibit 21-4
- f_{LC} - Exhibit 21-5
- f_M - Exhibit 21-6
- f_A - Exhibit 21-7

V. REFERENCES

1. Reilly, W., D. Harwood, J. Schoen, and M. Holling. *Capacity and LOS Procedures for Rural and Urban Multilane Highways*. NCHRP Project 3-33, Final Report, JHK & Associates, Tucson, Ariz., May 1990.

2. Hool, J. N., S. Maghsoodloo, A. D. Veren, and D. D. Brown. Analysis of Selective Enforcement Strategy Effects on Rural Alabama Traffic Speeds. In *Transportation Research Record 910*, TRB, National Research Council, Washington, D.C., 1983, pp. 74–81.
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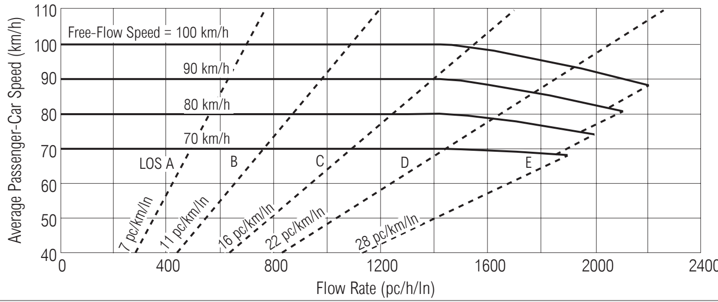
6. *Manual of Traffic Engineering Studies*. Institute of Transportation Engineers, Arlington, Va., 1976.
7. Webster, N., and L. Elefteriadou. *A Simulation Study of Truck Passenger Car Equivalents (PCE) on Basic Freeway Sections*. Transportation Research B33, Washington, D.C., 1999, pp. 323–336.

APPENDIX A. WORKSHEET

MULTILANE HIGHWAYS WORKSHEET

MULTILANE HIGHWAYS WORKSHEET



Application	Input	Output
Operational (LOS)	FFS, N, v_p	LOS, S, D
Design (N)	FFS, LOS, v_p	N, S, D
Design (v_p)	FFS, LOS, N	v_p, S, D
Planning (LOS)	FFS, N, AADT	LOS, S, D
Planning (N)	FFS, LOS, AADT	N, S, D
Planning (v_p)	FFS, LOS, N	v_p, S, D

General Information

Analyst __________________
Agency or Company __________________
Date Performed __________________
Analysis Time Period __________________

Site Information

Highway/Direction of Travel ________________
From/To ________________
Jurisdiction ________________
Analysis Year ________________

❑ Operational (LOS) ❑ Design (N) ❑ Design (v_p) ❑ Planning (LOS) ❑ Planning (N) ❑ Planning (v_p)

Flow Inputs

Volume, V __________________veh/h
Annual avg. daily traffic, AADT __________________veh/day
Peak-hour proportion of AADT, K __________________
Peak-hour direction proportion, D __________________
DDHV = AADT * K * D __________________veh/h
Driver type
❑ Commuter/Weekday ❑ Recreational/Weekend

Peak-hour factor, PHF ________________
% Trucks and buses, P_T ________________
% RVs, P_R ________________
General terrain
❑ Level ❑ Rolling ❑ Mountainous
Grade: Length _______km Up/Down _______%
Number of lanes ________________

Calculate Flow Adjustments

f_p __________________
E_T __________________

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

Speed Inputs

Lane width, LW __________________m
Total lateral clearance, TLC __________________m
Access points, A __________________A/km
Median type, M ❑ Undivided ❑ Divided
FFS (measured) __________________km/h
Base free-flow Speed, BFFS __________________km/h

Calculate Speed Adjustments and FFS

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

Operational, Planning (LOS); Design, Planning (v_p)

Operational (LOS) or Planning (LOS)

$v_p = \dfrac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$ __________________pc/h/ln
S __________________km/h
$D = v_p / S$ __________________pc/km/ln
LOS __________________

Design (v_p) or Planning (v_p)

LOS __________________
v_p __________________pc/h/ln
$V = v_p * PHF * N * f_{HV} * f_p$ __________________veh/h
S __________________km/h
$D = v_p/S$ __________________pc/km/ln

Design, Planning (N)

Design (N) or Planning (N) 1st Iteration

N ________________assumed
$v_p = \dfrac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$ ________________pc/h/ln
LOS ________________

Design (N) or Planning (N) 2nd Iteration

N ________________assumed
$v_p = \dfrac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$ ________________pc/h/ln
LOS ________________
S ________________km/h
$D = v_p / S$ ________________pc/km/ln

Glossary

N - Number of lanes
V - Hourly volume
v_p - Flow rate
LOS - Level of service
DDHV - Directional design-hour volume
S - Speed
D - Density
FFS - Free-flow speed
BFFS - Base free-flow speed

Factor Location

E_T - Exhibit 21-8, 21-9, 21-11
E_R - Exhibit 21-8, 21-10
f_p - Page 21-11
LOS, S, FFS, v_p - Exhibit 21-2, 21-3
f_{LW} - Exhibit 21-4
f_{LC} - Exhibit 21-5
f_M - Exhibit 21-6
f_A - Exhibit 21-7