TRANSPORTATION ENGINEERING

Asst.Prof.Dr. Mohammed Qadir Ismael

References:

1) "Principles of Highway Engineering and Traffic Analysis" by "Fred L. Mannering and Scott S. Washburn", Fifth Edition, 2013.
2) "Traffic and Highway Engineering" by "Nicholas J. Garber and Lester A. Hoel", Fourth Edition, 2010.
3) "Transportation Infrastructure Engineering" by "Lester A. Hoel, Nicholas J. Garber and Adel W. Sadek" International Student Edition, 2008.
4) 'Highways: The Location, Design, Construction and Maintenance of Pavements" by "C. A. O'Flaherty", Fourth Edition, 2007.
5) "Highway Engineering" by "Paul H. Wright and Karen K. Dixon", Seventh Edition, 2004.
6) 'Pavement Analysis and Design" by "Yang H. Huang", Second Edition, 2004.
7) AASHTO. A Policy on Geometric Design of Highways and Streets, American Association State Highway and Transportation Officials, Washington, D.C.: 2011.
8) American Association of State Highways and Transportation Officials (1993), AASHTO Guide for Design of Pavement Structure,AASHTO, Washington, D.C.
9) ASTM (2009), American Society for Testing and Materials, vol. 04:03.
10) TRB. Highway Capacity Manual, Transportation Research Board, Washington D.C. 2010.
11) SCRB, (2003), "Standard Specification for Roads and Bridges." Republic of Iraq, Ministry of Housing and Construction.
12) SCRB, (2005), "Highway Design Manual" Republic of Iraq, Ministry of Housing and Construction.

Syllabus:

1) Functional Classification of Highways
2) Location Surveys
3) Design Control and Criteria
4) Elements of Geometric Design:
    - decision sight distance,
    - passing sight distance,
    - stopping sight distance,
    - vertical alignment,
    - crest curve,
    - sag curve,
    - horizontal alignment,
    - transition curve ,
    - superelevation runoff,
    - widening.
5) Cross-Section Elements
    - travel lanes (numbers and width) ,
    - roadway cross slope,
    - types of roadway surfaces,
    - shoulders and sidewalk,
    - curb and gutter,
    - medians,
    - highway roadside
    - right-of way,
    - vertical clearance.
6) Intersections
    - types of at-grade intersections,
        - three-leg intersections,
        - four-leg intersections ,
    - channelization at intersections,
    - speed change lanes.

7) Interchanges

- justification of interchanges
    - elimination of bottlenecks
    - elimination of hazards
    - road-user benefits
- design classification
- types of interchanges
    - three-leg interchanges
    - four-leg interchanges
    - diamond interchanges
    - split-diamond interchanges
    - cloverleaf interchanges
    - partial cloverleaf ramp arrangements
    - directional interchanges
- selection of interchanges
- ramps in interchanges

8) Earthwork Quantities

9) Highway Paving Materials

- asphalt concrete
- desirable properties of asphalt cement
- asphalt types and testing
- prime and tack coats
- fractional components of asphalt cement
- rheological behavior
- aggregate and mineral filler
- job-mix formula
- asphalt mix design by Marshall test

10) Flexible Pavement Design

- layers of flexible pavement
- AASHTO design method for flexible pavements
- traffic loads

- subgrade support for flexible pavements
- flexible-pavement material
- structural numbers for flexible pavements
- determination of course thicknesses

11) Rigid Pavement

- subbase for a rigid pavement
- types of concrete pavements
    - jointed reinforced concrete pavement
    - continuously reinforced concrete pavement
- reinforcing steel for concrete pavement
- reinforced concrete pavement slabs
- tie bars
- load-transfer devices
- joints in concrete pavement
    - transverse expansion joints
    - longitudinal joints
    - construction joints
- joint sealing

Iraqi Highway Network:

Year	1987	1997	2004	2011
Population (million)	17	22	25	33
Vehicles	910,000	1,250,000	2,000,000	3,501,183

Rank	Country	Road length (km) (paved & unpaved)	Major Highway / Expressway length (km)	Date of information
#1	United States	6,506,204	75,238	2008
#2	China	4,237,500	96,200	2011
#3	India	4,020,000	21,181	2013
#15	Turkey	426,906	2,080	2009
#24	Saudi Arabia	221,372	3,891	2006
#30	Iran	172,927	1,429	2006
#144	Jordan	7,891		2009
#71	Egypt	65,050		2009
#83	Iraq	45,550	1,083	2002
#151	Kuwait	5,749		2004
#221	Tuvalu	8		2002

Rank	Country	Capita per motor vehicle
# 1	United States	1.30
# 10	Japan	1.84
# 23	Qatar	2.64
# 26	Saudi Arabia	2.97
# 70	Iran	5.0
#93	Turkey	6.94
#129	Iraq	20
#131	Egypt	22.22
#186	Bangladesh	333

Some Bridges across Tigris River in Baghdad:

Year	Bridge	Length	Cost (ID)	Company
1939	Al-Shuhada	5span/219 m	250,000	British
1940	Al-Ahrar	7span/303 m	300,000	British
1951	Al-Sarafiya	7span/450 m	3,000,000	British
1957	Al-Jumhuriya	8span/390 m	1,570,000	Germany
1982	Al-Jadiriya	29span/1276 m	30,000,000	Germany

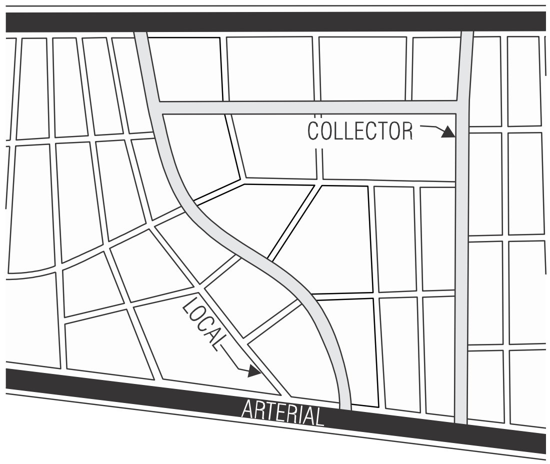
Highway Engineering deals with:

1. provisions for meeting public needs for highways;
2. environmental impact of highways;
3. planning, design, construction, maintenance, and rehabilitation of highways;
4. access to and exit from highways;
5. economics and financing of highway construction;
6. traffic control; and
7. safety of those using or affected by the use of highways.

1- <u>Functional Classification of Highways</u>

Highways are grouped in accordance with the type of service they provide; that is, the type of travel associated with the road. They are classified in accordance with functional characteristics. These characteristics are based on the location of the road, such as urban or rural; width of the road, such as single lane or multilane; and the type of service the road is to provide, such as local access or travel between cities.

AASHTO defines an urban area as “those places within boundaries set by the responsible State and local officials having a population of 5,000 or more.” Furthermore, the AASHTO Policy defines an *urbanized area* as one with a population of 50,000 and over and a small *urban area* as one with a population between 5000 and 50,000. *Rural areas* are defined as areas falling outside the definition of urban areas.



There are two primary categories of service provided by roadways and roadway systems, these are: Accessibility and Mobility

“Accessibility” refers to the direct connection to abutting lands and land uses provided by roadways.

“Mobility” refers to the through movement of people, goods, and vehicles from Point A to Point B in the system.

Arterials are surface facilities that are designed primarily for through movement but permit some access to abutting lands.

Local streets are designed to provide access to abutting land uses with through movement only a minor function, if provided at all.

The *collector* is an intermediate category between arterials and local streets.

A **freeway** is a divided highway with fully controlled access. Access to a freeway is made *without* use of at-grade intersections.
The freeway (*limited-access facility)* provides for 100% through movement, or mobility. No direct access to abutting land uses is permitted.

Properties of principle arterials:

- High design speed (≤ 130 kph)
- High level of service (≈ B)
- Long distance (connecting cities)
- Full control of access

Properties of minor arterials

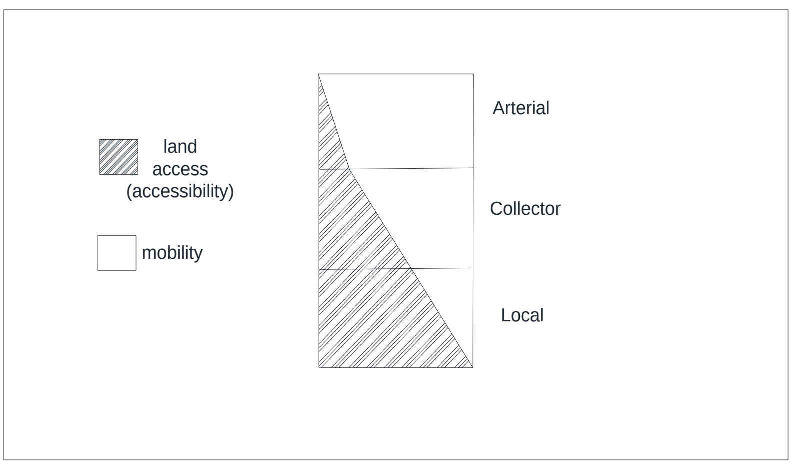
- Moderate design speed (≤ 110 kph)
- Level of service (B - C)
- Partial control of access

Properties of collectors (major or minor)

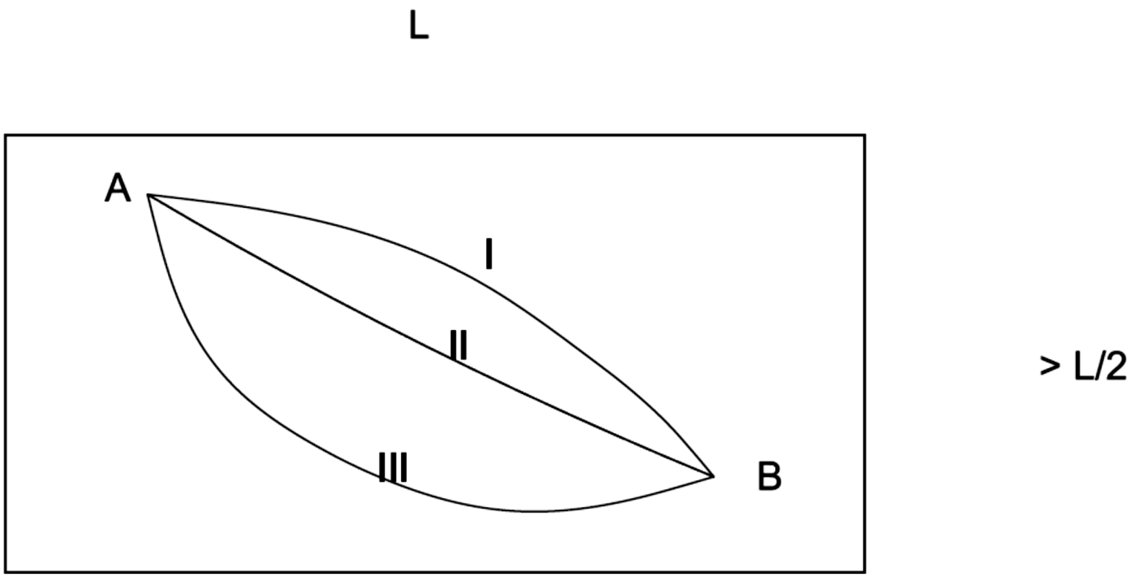
- Intermediate design speed (≤ 80 kph)
- Level of service (C - D)

Properties of local roads and streets: (road – rural area) while (street–urban area)

- Design speed (20 – 40 kph)
- Low level of service (D - E)
- Serving a butting area



2- Location Surveys



I- Reconnaissance survey :

- Collection of available maps, arial photograph for the area.
- Selection of the best route

  Based on:

- Design standards
- Earthwork and grading cost
- Drainage structure (bridge or culvert)
- Availability of construction materials
- Traffic services

II- Preliminary survey :

- Evaluation of the alternative route and the selection of the best route.
- Locate on the paper the CL of the proposed route (P-line).

III- Final location survey:

Locating the final CL of the route on the ground and preparing design drawing as well as an estimation of construction cost.

- Detail survey
- Soil investigation
- Alignment design (vertical, horizontal)
- Cross section design
- Pavement structural design
- Preparing Bill of Quantity (BoQ)

3- <u>Design Control and Criteria</u>

- Vehicle characteristics
- Traffic characteristics

<u>Vehicle characteristics:</u>

The physical characteristics of vehicles and the proportions of various size vehicles using the highway are important control factors in highway design. To obtain such control factors, all vehicles are grouped in general classes, and representative types within each class are selected as the "design vehicle".
The most important vehicle features for highway design are:

- Dimensions and minimum turning radius as a control for highway geometric design.
- Weight and axle loading as a control for pavement structural design.

Vehicle classification;

1- Passenger car (PC): vehicles that have four tires touching the pavement. (including pick-up and mini bus).

2- Heavy vehicle (HV): vehicles that have more than four tires touching the pavement like trucks and buses.

DESIGN VEHICLE FOR GEOMETRIC DESIGN

A "design vehicle" is a selected motor vehicle which is used for the determination of design controls.

For the purpose of geometric design the general class groups are:

Passenger cars (P)
Single unit trucks and small buses (SU)
Large buses (BUS)
Truck combinations (WB)

The classification of most basic commercial vehicle types in regular operation, designated by code based on axle arrangement, is shown in Figure (5). The first digit indicates the number of axles of the truck or truck-tractor. The letter "S" indicates a semitrailer, and the digit immediately fol1owing an "S" indicates the number of axles of the semitrailer. Any digit other than the first in a combination when not preceded by an "S", indicates a trailer and the number of its axles.

For example, a 2-S2 combination is a two-axle truck-tractor with a tandem-axle semitrailer. A 3-S1-2 combination is a three-axle truck-tractor with tandem rear axles, a semitrailer with a single axle, and a trailer with two axles.

The maximum allowable axle-loads, gross weights including a load for commercial vehicles in regular operation in Iraq, are shown in Figure (6) while the vehicles dimensions are listed in table (5).

Figure (5): Commercial Vehicle Types:

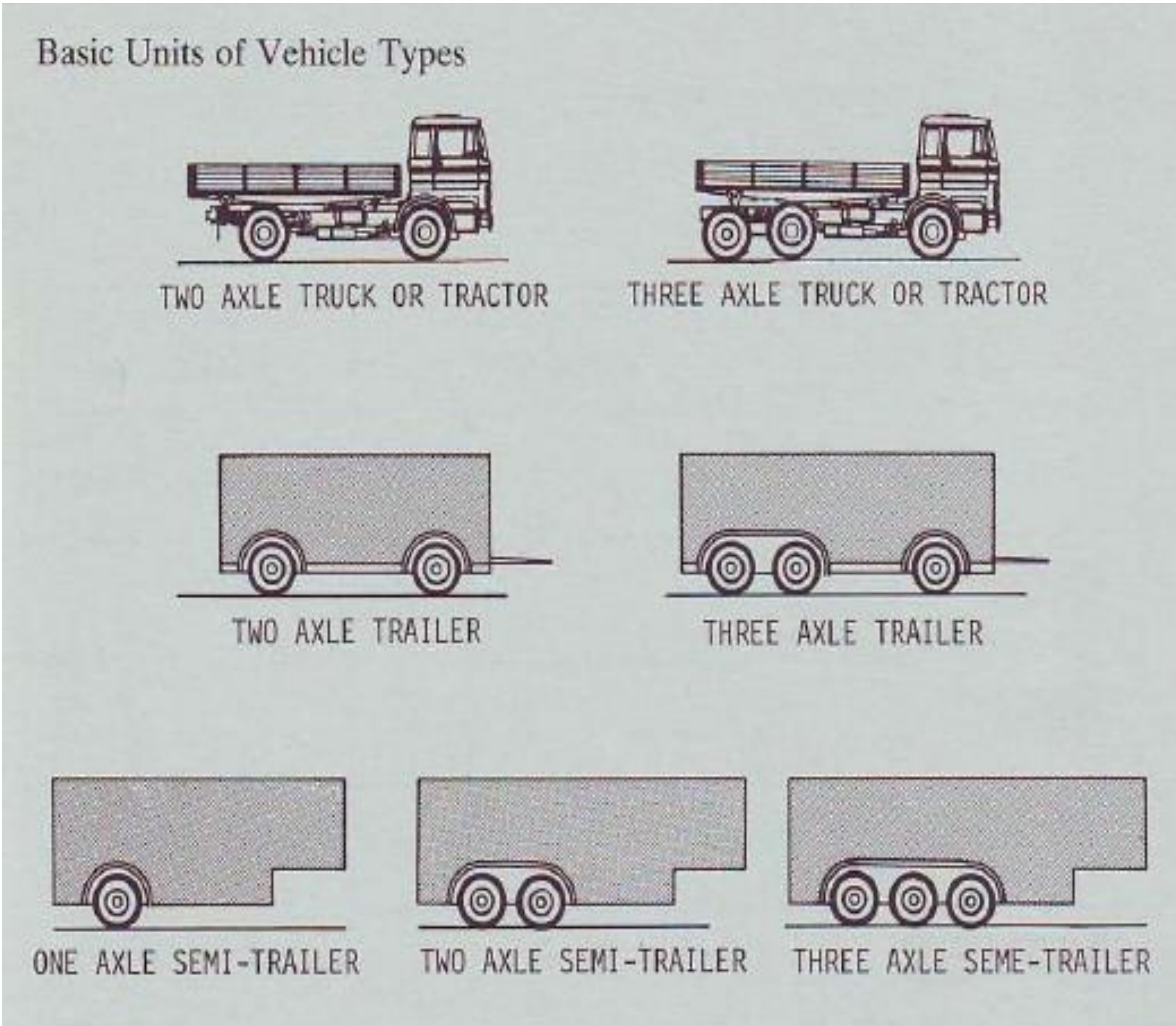


Figure (5): Commercial Vehicle Types:

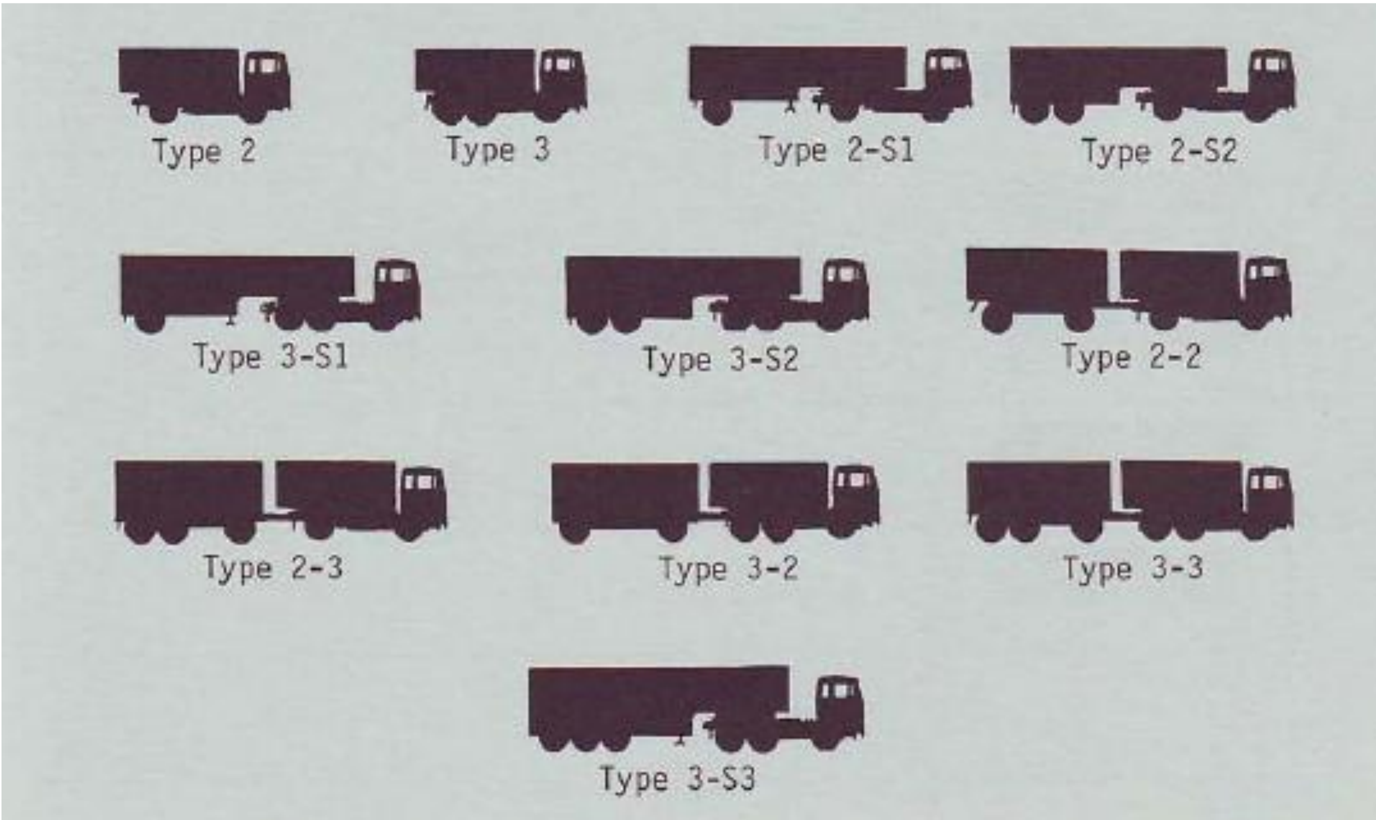


Table (5): Vehicles dimensions:

Vehicle type	Width(m)	Length(m)	Height(m)	Min. turning radius(m)
Passenger car	2.1	5.8	1.3	7.3
Single unit truck	2.55	9.2	4.05	12.8
Tractor trailer	2.6	31	4.1	13.7

Figure (6): Legal Axle and Gross Weights Permitted on Motor Vehicles in Regular Operation in Iraq

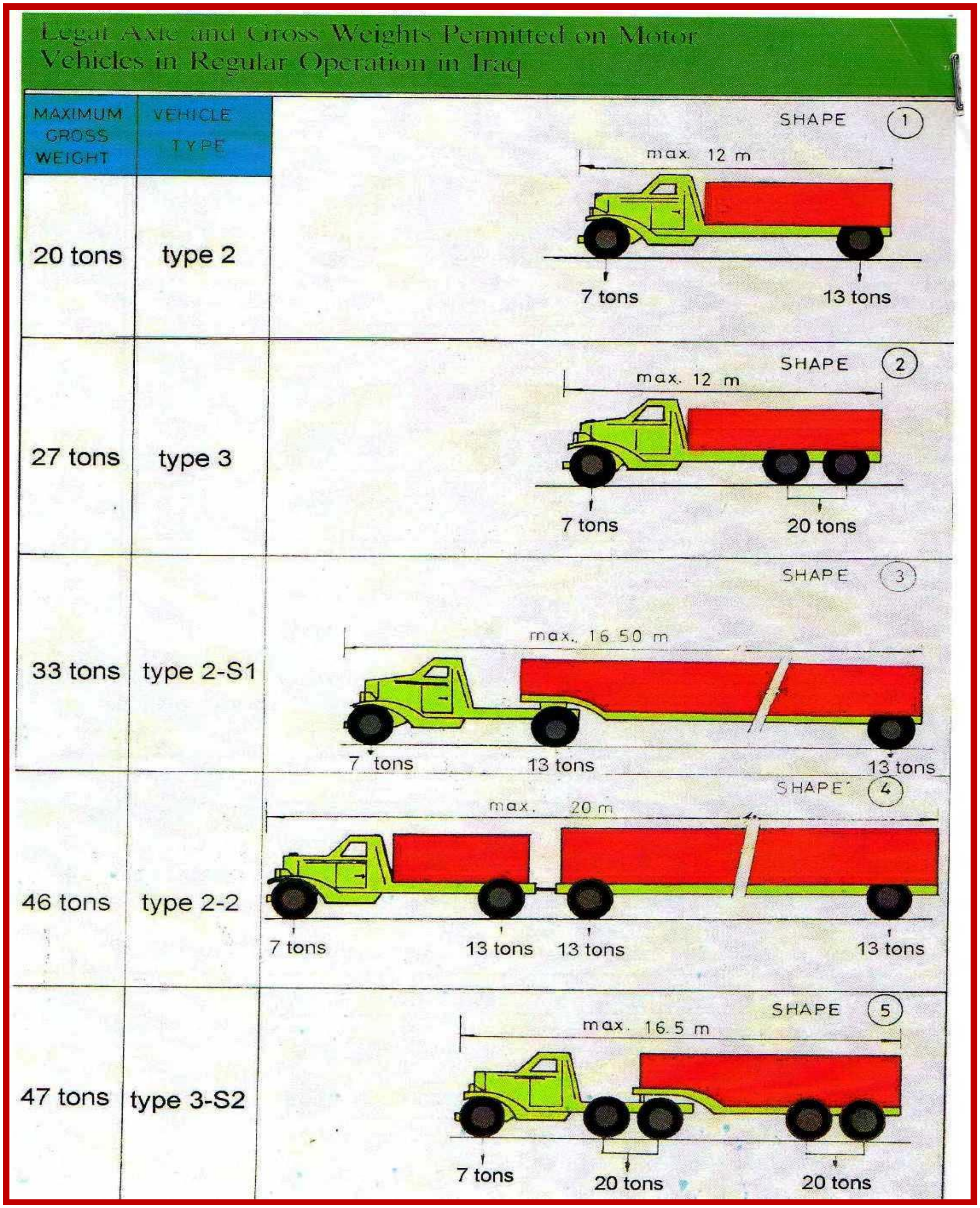
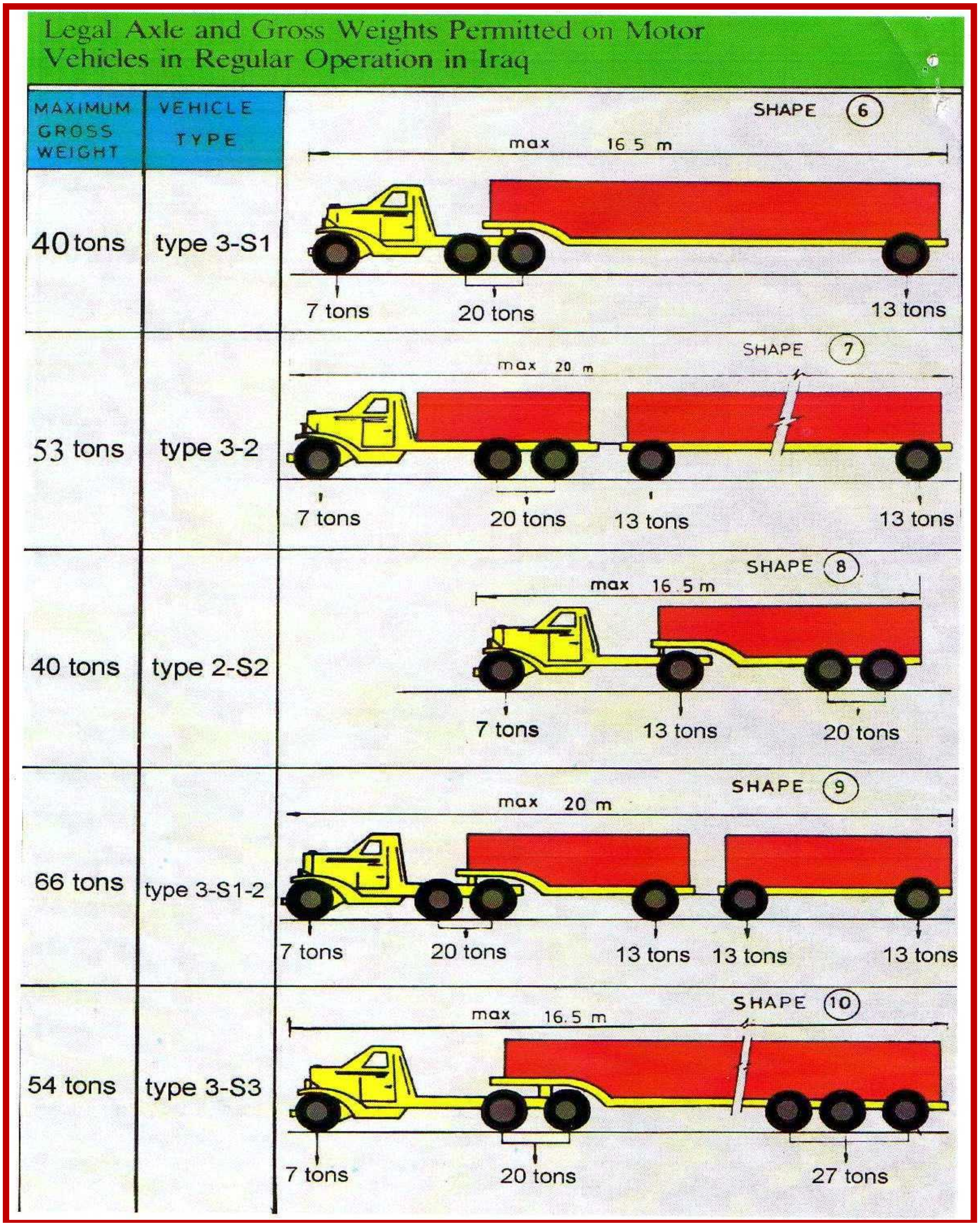


Figure (6): Legal Axle and Gross Weights Permitted on Motor Vehicles in Regular Operation in Iraq



Passenger Car Units

Vehicles of different types have different road space requirements and different effects on the capacity of highway and intersection because of variations in size and performance.
The overall effect on traffic operations by any vehicle can be expressed in terms of the effect of the basic unit - usually a passenger car. Equivalency between the Passenger Car Unit (PCU) and other vehicles grouped in classes is determined by conversion factors expressing the relative effect of different types of vehicles on traffic flow.
The passenger car unit equivalents for heavy vehicles under different topographic conditions are shown in Table (6).

Table (6): Equivalency factor for passenger car (E_T)

	Terrain		
	Flat(level)	Rolling	Mountain
HV	1.5	2.5	4.0

Example: For design hourly volume equal to 300 vph consisting of 10 percent heavy vehicle, determine the hourly volume expressed by passenger car unit if the type of terrain is level.

Solution:

A) From table (6), $E_T = 1.5$

$$\begin{aligned} \text{Volume(pcph)} &= 300\ (\ HV*E_T + PC * 1) \\ &= 300\ (0.1 * 1.5 + 0.9 *1) \\ &= 315 \text{ pcph} \end{aligned}$$

B) $$f_{HV} = \frac{1}{1+P_T(E_T-1)}$$

$$f_{HV} = \frac{1}{1+0.1(1.5-1)} = 0.95 \ \rightarrow \text{vol}=\frac{300}{0.95} = 315 \text{ pcph}$$

Traffic Stream Parameters

Traffic stream parameters represent the engineer's quantitative measure for understanding and describing traffic flow. Traffic stream parameters fall into two broad categories: macroscopic parameters, which characterize the traffic stream as a whole, and microscopic parameters, which characterize the behavior of individual vehicles in the traffic stream with respect to each other.
The three macroscopic parameters that describe traffic stream are volume or rate of flow, speed, and density.

Volume and Flow

Volume is simply the number of vehicles that pass a given point on the roadway or a given lane or direction of a highway in a specified period of time. The unit of volume is simply vehicles, although it is often expressed as annual, daily, hourly peak and off-peak. The subsequent sections explain the range of commonly used daily volumes, hourly volumes, and subhourly volumes.

Daily Volumes. Daily volumes are frequently used as the basis for highway planning, for general trend observations, as well as for traffic volume projections. Four daily volume parameters are widely used: average annual daily traffic (AADT), average annual weekday traffic (AAWT), average daily traffic (ADT), and average weekday traffic (AWT).

• AADT is the average 24-hour traffic volume at a given location over a full year, that is, the total number of vehicles passing the site in a year divided by 365. AADT is normally obtained from permanent counting stations, typically bidirectional flow data rather than lane-specific flow data.

• AAWT is the average 24-hour traffic volume occurring on weekdays over a full year. AAWT is normally obtained by dividing the total weekday traffic for the year by the annual weekdays (usually 260 days). This volume is of particular importance since weekend traffic is usually low; thus, the average higher weekday volume over 365 days would hide the impact of the weekday traffic.

• ADT is the average 24-hour traffic volume at a given location for a period of time less than a year (e.g., summer, six months, a season, a month, a week). ADT is valid only for the period of time over which it was measured.

• AWT is the average 24-hour traffic volume occurring on weekdays at a given location for a period of time less than a year, such as a month or a season.

The unit describing all these volumes is vehicles per day (veh/ day). Daily volumes are often not differentiated per lane or direction but rather are given as totals for an entire facility at a particular location.

Hourly Volumes

Hourly volumes are designed to reflect the variation of traffic over the different time period of a day. They are also used to identify single hour or period of highest volume in a day occurring during the morning and evening commute, that is, rush hours. The single hour of the day corresponding to the highest hourly volume is referred to as peak hour. The peak hour traffic volume is a critical input in the design and operational analysis of transportation facilities.
The peak hour volume is usually a directional traffic, that is, the direction of flows is separated. Highway design as well as other operations analysis, such as signal design, must adequately serve the peak-hour flow corresponding to the peak direction.
Peak hour volumes can sometimes be estimated from AADT, as follows:

$$\boxed{\text{DDHV} = \text{AADT} \times K \times D}$$

Where:
DDHV = directional design hourly volume (veh/ hr)
AADT = average annual daily traffic (24 hours) (veh/day)
K = factor for proportion of daily traffic occurring at peak hour
D = factor for proportion of traffic in peak direction.

K and D values vary depending on the regional characteristics of the design facilities, namely, rural versus urban versus suburban. K often represents the AADT proportions occurring during the thirtieth or fiftieth highest peak hour of the year. If the 365 peak hour volumes of the year at a given location are listed in descending order, the 30th peak hour is 30th on the list and represents a volume that is exceeded in only 29 hours of the year. For rural facilities, the 30th peak hour may have a significantly lower volume than the worst hour of the year, as critical peaks may occur only infrequently. In such cases, it is not considered economically feasible to invest large amounts of capital in providing additional capacity that will be used in only 29 hours of the year.

Subhourly Volumes

Subhourly volumes represent traffic variation within the peak hour, For example, a volume of 200 vehicles observed over a 15-minute period may be expressed as a rate of 200 X 4 = 800 vehicles/hour, even though 800 vehicles would not be observed if the full hour were counted. The 800 vehicles/hour becomes a rate of flow that exists for a 15-minute interval.

The peak-hour factor (PHF) is calculated to relate the peak flow rate to hourly volumes.
This relationship is estimated as follows:

$$\text{PHF} = \frac{V}{4 \times V_{15}}$$

where
PHF = peak hour factor
V = peak hour volume (veh/ hr)
$V15$ = volume for peak 15-min period (veh)

The PHF describes trip-generation characteristics. When PHF is known, it can be used to convert a peak-hour volume to an estimated peak rate of flow within an hour:

$$v = \frac{V}{\text{PHF}}$$

Where
v = peak rate of flow within hour (veh/ hr)
V = peak hourly volume (veh/ hr)
PHF = peak hour factor

The maximum possible value for the *PHF* is 1.00, which (occurs when the volume in each interval is constant. For 15-minute periods, each would have a volume of exactly one quarter of the full hour volume. This indicates a condition in which there is virtually no variation of flow within the hour. The minimum value occurs when the entire hourly volume occurs in a single 15-minute interval. In this case, the *PHF* becomes 0.25, and represents the most extreme case of volume variation within the hour. In practical terms, the *PHF* generally varies between a low of 0.70 for rural and sparsely developed areas to 0.98 in dense urban areas.

Example:

1,000 vehicles counted over a 15-minute interval could be expressed as 1,000 veh/0.25 h = 4,000 veh/h. The rate of flow of 4,000 veh/h is valid for the 15-minute period in which the volume of 1,000 vehs was observed. Table (7) illustrates the difference between volumes and rates of flow. The full hourly volume is the sum of the four 15- minute volume observations, or 4,200 veh/h. The rate of flow for each 15-minute interval is the volume observed for that interval divided by the 0.25 hours over which it was observed. In the worst period of time, 5:30-5:45 PM, the rate of flow is 4,800 veh/h. This is a flow *rate*, not a volume. The actual volume for the hour is only 4,200 veh/h.

Table(7): Illustration of Volumes and Rates of Flow

Time Interval	Volume for Time Interval (vehs)	Rate of Flow for Time Interval (vehs/h)
5:00–5:15 PM	1,000	1,000/0.25 = 4,000
5:15–5:30 PM	1,100	1,100/0.25 = 4,400
5:30–5:45 PM	1,200	1,200/0.25 = 4,800
5:45–6:00 PM	900	900/0.25 = 3,600
5:00–6:00 PM	Σ = 4,200	

Speed

The speed of a vehicle is defined as the distance it travels per unit of time. It is the inverse of the time taken by a vehicle to traverse a given distance. Most of the time, each vehicle on the roadway will have a speed that is somewhat different from the speed of the vehicles around it. In quantifying the traffic stream, the average speed of the traffic is the significant variable. The average speed, called the space mean speed, can be found by averaging the individual speeds of all of the vehicles in the study area.

Space Mean versus Time Mean Speed

Two different ways of calculating the average speed of a set of vehicles are reported, namely the space mean speed and the time mean speed. This difference in computing the average speed leads to two different values with different physical significance. While the time mean speed (TMS) is defined as the average speed of all vehicles passing a point on a highway over a specified time period, the space mean speed (SMS) is defined as the average speed of all vehicles occupying a given section of a highway over a specified time period. TMS is a point measure and SMS is a measure relating to a length of highway or lane. TMS and SMS may be computed from a series of measured travel times over a measured distance. TMS takes the arithmetic mean of the observation. It is computed as:

$$\text{TMS} = \frac{\sum \frac{d}{t_i}}{n}$$

SMS could be calculated by taking the harmonic mean of speeds measured at a point over time. It is computed by dividing the distance by an average travel time, as shown below:

$$\text{SMS} = \frac{d}{\sum \frac{t_i}{n}} = \frac{nd}{\sum t_i}$$

where
TMS = time mean speed (mps or kph)
SMS = space mean speed (mps or kph)
d = distance traversed (m or km)
n = number of travel times observed
t_i = travel time for the *i*th vehicles (sec or hr)

The time mean speed is always higher than the space mean speed. The difference between these speeds tends to decrease as the absolute values of speeds increase. It has been shown from field data that the relationship between time mean speed and space mean speed can be given as:

$$\bar{u}_t = \bar{u}_s + \frac{\sigma^2}{\bar{u}_s}$$

Density

Density is the number of vehicles present on a given length of roadway or lane. Normally, density is reported in terms of vehicles per kilometer or per mile. High densities indicate that individual vehicles are very close to each other, while low densities imply greater distances between vehicles. Density is a difficult parameter to measure directly in the field. Direct measurements of density can be obtained through aerial photography, which is an expensive method, or it can be estimated from the density, flow, and speed relationship.

Time Headway

Time headway (h_t) is the difference between the time the front of a vehicle arrives at a point on the highway and the time the front of the next vehicle arrives at that same point. Time headway is usually expressed in seconds. The *average* headway in a lane is directly related to the rate of flow:

$$v = \frac{3{,}600}{h_a}$$

where:
v = rate of flow, veh/h/ln
h_a = average headway in the lane, s

Space Headways

Space headway (h_s) is the distance between the front of a vehicle and the front of the following vehicle and is usually expressed in meter.
The *average* spacing in a traffic lane can be directly related to the density of the lane:

$$D = \frac{1000}{hs}$$

where:
D = density, veh/km/ln
h_s = *average spacing between vehicles in the lane, m*

Gap: is the headway in a major stream, which is evaluated by a vehicle driver in a minor stream who wishes to merge into the major stream. It is expressed either in units of time (time gap) or in units of distance (space gap).

Time lag is the difference between the time a vehicle that merges into a main traffic stream reaches a point on the highway in the area of merge and the time a vehicle in the main stream reaches the same point.

Space lag is the difference, at an instant of time, between the distance a merging vehicle is away from a reference point in the area of merge and the distance a vehicle in the main stream is away from the same point.

Flow, Speed, Density Relationship

Speed, flow, and density are all related to each other and are fundamental for measuring the operating performance and level of service of transportation facilities, such as freeway sections. Under uninterrupted flow conditions, speed, density, and flow are all related by the following equation:

$$\text{Flow} = \text{Density} \times \text{Speed}: v = S \times D$$

where
v = flow (veh/ hr)
S = space mean (average running) speed (kph)
D = density (veh/km)

The general form of relationships between speed, density, and flow is illustrated in Figure (7), also known as the fundamental diagrams of traffic flow. The relationship between speed and density is consistently decreasing. As density increases, speed decreases. This diagram as well as the above formula shows that flow is zero under two different conditions:

• When density is zero: thus, there is no vehicle on the road
• When speed is zero: vehicles are at complete stop because of traffic congestion.

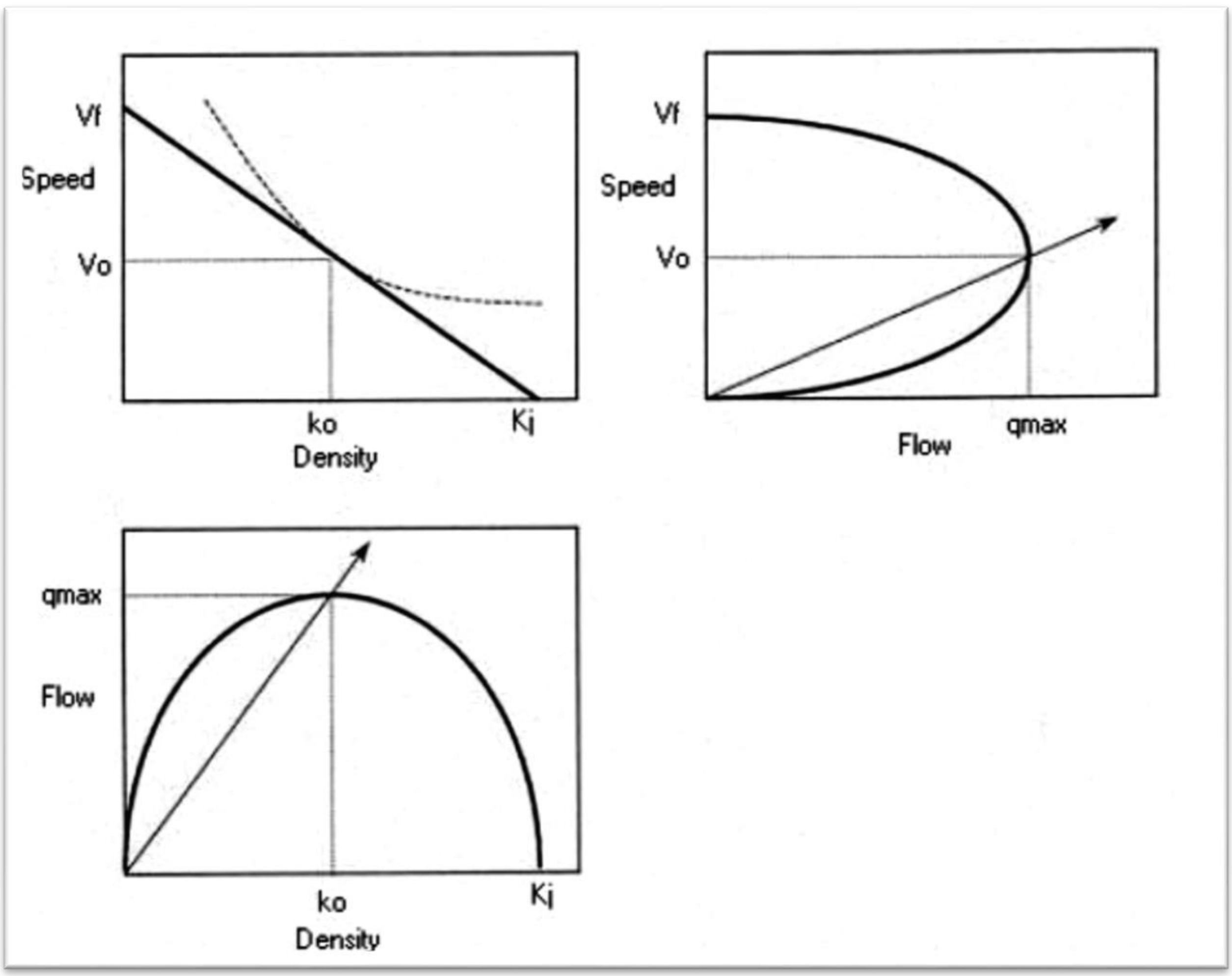


Figure (7): Fundamental flow-speed-density diagram.

Greenshields Model

The first steady-state speed-density model was introduced by Greenshields, who proposed a linear relationship between speed and density as follows:

$$u = u_f - \left(\frac{u_f}{k_j}\right) \times k$$

where
u = speed at any time
u_f = free-flow speed
k = density at that instant
k_j = maximum density

In these equations, as the flow increases, density increases and the speed decreases. At optimum density, flow becomes maximum (q_m) at $u = u_f /2$ and $k = k_j /2$.

Greenberg Model

A second early model was suggested by Greenberg (1959), showing a logarithmic relationship as follows:

$$u = c \ln (k/k_j)$$

where
u = speed at any time
c = a constant (optimum speed)
k = density at that instant
k_j = maximum density

Flow-density relationship

$$q = u_f k - \frac{u_f}{k_j} k^2$$

where
q = flow at any time
u_f = free-flow speed
k = density at that instant
k_j = maximum density

It can be concluded from this equation that maximum flow occurs at density equal to ($k_j/2$).

Flow-speed relationship

$$\boxed{q = k_j\, u - \frac{k_j}{u_f}\, u^2}$$

where
q = flow at any time
u = speed at any time
u_f = free-flow speed
k_j = maximum density

It can be concluded from this equation that maximum flow occurs at speed equal to ($u_f/2$).

Since q=k u , therefore $q_{max} = \frac{k_j\, u_f}{4}$

Capacity and Level of Service

Capacity is a measure of the demand that a highway can potentially service.
Level of service is a qualitative measure of the highway's operating conditions under a given demand within a traffic stream and their perception by motorists and/or passengers. Thus, LOS intends to relate the quality of traffic service to given volumes (or flow rates) of traffic. The parameters selected to define LOS for each facility type are called measures of effectiveness (MOE). These parameters can be based on various criteria, such as travel times, speeds, total delay, probability of delay, comfort, and safety.

The *Highway Capacity Manual* defines six levels of service, designated A through F, with A being the highest level of service and F the lowest.

Table (8) specify the operational condition and operating speed for the level of service category.

Table (8): operational condition and operating speed for LOS

LOS	Description operational condition	Operating speed* (kph)
A	Free flow	96
B	Stable flow	88
C	Stable flow with restriction	72
D	Approaching unstable flow	56
E	Unstable flow	48
F	Forced flow (stop-go condition)	$<$ 48

* operating speed: it is the speed at which drivers are observed to operate their vehicles during free flow condition. It can be determined as the 85th percentile value from the cumulative frequency curve of observed speed. In Iraq this speed can be assumed to be 56-88 kph.

Design capacity (design service flow rate):

The maximum hourly rate at which vehicles can be expected to pass a point or section of a lane or roadway during one hour under prevailing roadway and traffic condition for a designated level of service.

The design capacity for different level of service is presented in table (9) while table (10) specifies the desired LOS for various terrain.

Table (9)

LOS	Design capacity (pcphpl)
A	660
B	1080
C	1550
D	1980
E	2200

Table (10)

Highway	Level	Rolling	Mountain
Principle arterial	B	B	C
Minor arterial	B	B	C
Collector	C	C	D
Local	D	D	D

Traffic forecast

The design of new highways or of improvements to existing highways should not be based on current traffic volumes, but on the future traffic expected to use the facilities.

Components of Future Traffic

1. Current Traffic: Existing and Attracted

Current traffic is the volume of traffic that would use a new or improved highway if it were open to traffic, i.e. traffic already using the route plus traffic transferring to the new highway from less attractive routes.

2. Normal Traffic Growth

Normal traffic growth is the increase of current traffic due to general increase in number and usage of motor vehicles.

3. Generated Traffic

Generated traffic consists of motor vehicle trips that would not have been made if the new facility had not been provided.

The future traffic is calculated by multiplying the present traffic volume with traffic projection factor which can be determined from the following equation:

$$TPF = (1+r)^{n+x}$$

Where:

r = annual rate of traffic increase (0 – 10 %) , default value= 6%

x = construction period in years (2-4 yrs)

n = design life in years (20-50 yrs)

Example:

r= 6% , x=2 years, n= 20 years

TPF= $(1+0.06)^{22}$ = 3.6

Example:

It is proposed to design a minor arterial highway within an urban rolling area to serve an anticipated current daily volume of 10000 pcpd. Find the required number of lanes for this highway.

Solution:

For a minor arterial highway within rolling area, the selected design LOS obtained from table (10) is B, consequently, the design capacity= 1080 pcphpl.

Future vol.= current vol. * TPF

Assume TPF= 3.6 $\rightarrow$ F.V= 10000 *3.6= 36000 pcpd

DHV/both direction= 0.12 * 36000=4320 pcph

DHV/one direction= 4320 *0.8= 3456 pcph/one direction

No. of lanes/ one direction= 3456/1080= 3.2 , use 4 lanes

Total no. of lanes for both direction= 4 *2=8 lanes

Example:

A multilane principal arterial is being designed through a rolling rural area. The current daily volume= 8000 vpd in both direction with 20 % truck, peak hourly factor= 90%, directional distribution factor = 60-40. How many lanes are required?

Solution:

For a principle arterial highway within rolling area, the selected design LOS obtained from table (10) is B, consequently, the design capacity= 1080 pcphpl.

From table (6), for rolling area, E_T=3.0

Current volume= 8000 [0.2 *3.0 + 0.8 * 1.0]= 11200 pcpd

F.V= 11200 *3.6= 40320 pcpd

DHV/both direction= 0.15 *40320 = 6048 pcph

Hourly flow rate= vol/ PHF= 6048/0.9= 6720 pcph

Directional hourly flow rate= 6720 * 0.6 = 4032 pcph/one direction

No. of lanes /one direction= 4032/1080 = 3.73, use 4 lanes

Total no. of lanes = 4 *2= 8 lanes

4- <u>Elements of Geometric Design:</u>

The design of highways necessitates the determination of specific design elements which include:

1. the number of lanes,
2. lane width,
3. median type (if any) and width,
4. length of acceleration and deceleration lanes for on- and off-ramps,
5. need for truck climbing lanes for steep grades,
6. curve radii required for vehicle turning,
7. the alignment required to provide adequate stopping and passing sight distances.

The most of geometric features depends primarily on available sight distance.

<u>Sight distance.</u>

Sight distance is defined as the length of carriageway that the driver can see in both the horizontal and vertical planes. Three types of sight distance are detailed: stopping distance ,passing(overtaking) distance and decision distance.

1. *Stopping sight distance*

This is defined as the minimum sight distance required by the driver in order to be able to stop the car before it hits an object on the highway. It is of primary importance to the safe working of a highway. This type of sight distance should be provided for all types of highway facilities.

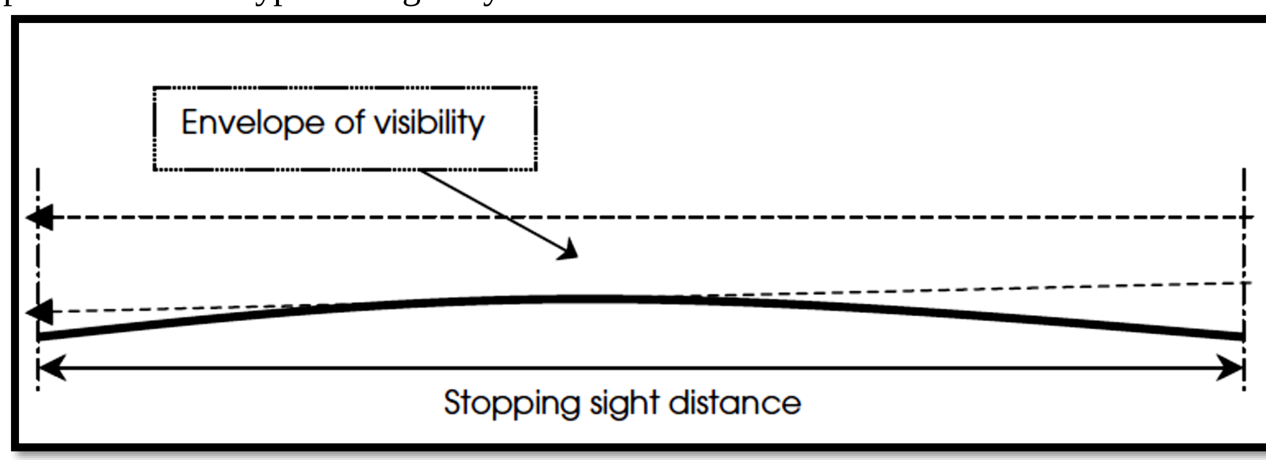


Figure (8): stopping sight distance

The stopping sight distance (SSD) can be subdivided into three constituent parts:

1. The perception distance :

length of highway travelled while driver perceives hazard
(t=1.5 sec for rural area and t=0.5 sec for urban area).

2. The reaction distance :

   length of highway travelled during the period of time taken by the driver to apply the brakes and for the brakes to function.
   (t=1.0 sec)

The combined perception and reaction time, *t*, can vary widely depending on the driver. However, in the Iraq, a value of 1.5 seconds is taken for urban area and 2.5 second for rural area, **when the designated area is not specified, the value of 2.5 second will be used for safe and comfortable design**.

The length of highway travelled during the perception-reaction time (lag distance, D_{lag}) is calculated from the formula:

$$\text{Perception-reaction distance } (D_{lag}, m) = 0.278 \times u \times t$$

where
u = initial speed (km/hr)
t = combined perception and reaction time (sec)

3. The braking distance :

length of highway travelled while the vehicle actually comes to a halt.

$$D_{braking} = \frac{u^2}{254(f_b \pm G)}$$

So that, the total SSD will be:

$$SSD = 0.278\, u \times t + \frac{u^2}{254(f_b \pm G)}$$

Where:

u= design speed (initial speed), kph

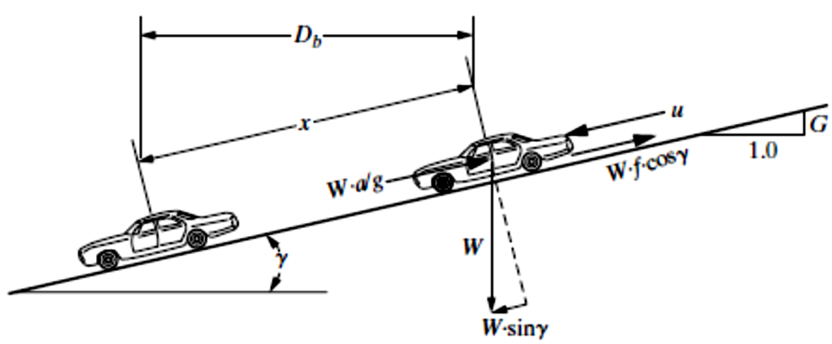
t= perception – reaction time, 2.5 sec (rural) and 1.5 sec (urban)

f_b= braking coefficient of friction (forward friction),

0.55 – 0.62 dry pavement condition
0.28- 0.40 wet pavement condition
0.10 muddy pavement condition
0.05 icy pavement condition

G= longitudinal grade (0.0 – 0.09)

Consider a vehicle traveling downhill with an initial velocity of *u,* in km/h, as shown in Figure (9). Let
W = weight of the vehicle
f = coefficient of friction between the tires and the road pavement
γ = angle between the grade and the horizontal
a = deceleration of the vehicle when the brakes are applied
D_b = horizontal component of distance traveled during braking



W = weight of vehicle
f = coefficient of friction
g = acceleration of gravity
a = vehicle acceleration
u = speed when brakes applied
D_b = braking distance
γ = angle of incline
G = tan γ (% grade/100)
x = distance traveled by the vehicle along the road during braking

Figure (9): Forces Acting on a Vehicle Braking on a Downgrade

Frictional force on the vehicle $= Wf \cos \gamma$

The force acting on the vehicle due to deceleration is $W\,a/g$, where g is acceleration due to gravity. The component of the weight of the vehicle is $W \sin \gamma$. Substituting into $\Sigma f = ma$, we obtain:

$$W \sin \gamma - Wf \cos \gamma = \frac{Wa}{g}$$

The deceleration that brings the vehicle to a stationary position can be found in terms of the initial velocity $\boldsymbol{u}$ **as** $\boldsymbol{a = -u^2/2x}$ (assuming uniform deceleration), where x is the distance traveled in the plane of the grade during braking.

$$W \sin \gamma - Wf \cos \gamma = -\frac{Wu^2}{2gx}$$

However, $\boldsymbol{D_b = x \cos \gamma}$, and we therefore obtain:

$$\frac{Wu^2}{2gD_b} \cos \gamma = Wf \cos \gamma - W \sin \gamma$$

Giving:

$$\frac{u^2}{2gD_b} = f - \tan \gamma$$

and

$$D_b = \frac{u^2}{2g(f - \tan \gamma)}$$

Note, however, that $\tan \gamma$ is the grade G of the incline (that is, percent of grade/100)

$$D_b = \frac{u^2}{2g(f - G)}$$

If g is taken as 9.81 m /sec^2 and u is expressed in km/h

$$D_b = \frac{u^2}{254(f - G)}$$

2. *Passing(overtaking) distance*

The passing sight distance is the minimum sight distance required on a two-lane, two-way highway that will permit a driver to complete a passing maneuver without colliding with an opposing vehicle and without cutting off the passed vehicle.
In order to determine the minimum passing sight distance, certain assumptions have to be made regarding the movement of the passing vehicle during a passing maneuver:

1. The vehicle being passed (impeder) is traveling at a uniform speed.
2. The speed of the passing vehicle is reduced and is behind the impeder as the passing section is entered.
3. On arrival at a passing section, sometime elapses during which the driver decides whether to undertake the passing maneuver.
4. If the decision is made to pass, the passing vehicle is accelerated during the passing maneuver, and the average passing speed is about **16 kph** more than the speed of the impeder vehicle.
5. A suitable clearance exists between the passing vehicle and any opposing vehicle when the passing vehicle reenters the right lane.

These assumptions have been used by AASHTO to develop a minimum passing sight distance requirement for two-lane, two-way highways.

The minimum passing sight distance is the total of four components as shown in Figure (10).

Where:

d_1 = distance traversed during perception-reaction time and during initial acceleration to the point where the passing vehicle just enters the left lane
d_2= distance traveled during the time the passing vehicle is traveling in the left lane
d_3 = distance between the passing vehicle and the opposing vehicle at the end of the passing maneuver
d_4 = distance moved by the opposing vehicle during two thirds of the time the passing vehicle is in the left lane (usually taken to be 2/3 $d2$)

The distance d_1 is obtained from the expression:

$$d_1 = 0.278\, t\,(u - m + \frac{at_1}{2})$$

where:

t_1 = time for initial maneuver **(= 4 sec)**
a = average acceleration rate (km/hr/sec)
 0.5 – 1.0 kphps for truck
 3.0 – 8.0 kphps for P-car
 16.0 – 24.0 kphps for sport car
u = average speed of passing vehicle (kph)
m = difference in speeds of passing and impeder vehicles

The distance d_2 is obtained from:

$$d_2 = 0.278\, u\, t_2$$

where:

t_2 = time passing vehicle is traveling in left lane **(=10 sec)**
u = average speed of passing vehicle (kph)

d_3 = 30-90 m

d_4 = 2/3 d_2

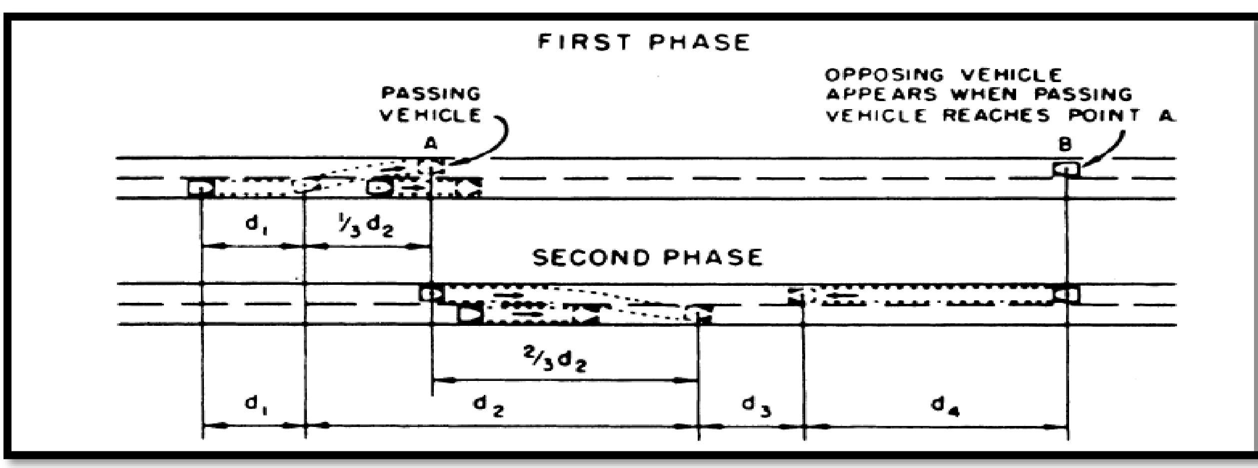


Figure (10): Elements of Total Passing Sight Distance on Two-Lane Highways

3. ***Decision sight distance***

It is defined by AASHTO as the "distance required for a driver to detect an unexpected or otherwise difficult-to-perceive information source or hazard in a roadway environment that may be visually cluttered, recognize the hazard of its threat potential, select an appropriate speed and path, and initiate and complete the required safety maneuvers safely and efficiently."

The decision sight distances depend on the type of maneuver required to avoid the hazard on the road, and also on whether the road is located in a rural or urban area. Different avoidance maneuvers, which can be used for design are listed below with the recommended time values

1. Avoidance Maneuver *A:* Stop on rural road, $t = 3.0$ s
2. Avoidance Maneuver *B:* Stop on urban road, $t = 9.1$ s
3. Avoidance Maneuver *C:* Speed/path/direction change on rural road, t varies between 10.2 and 11.2 s
4. Avoidance Maneuver *D:* Speed/path/direction change on suburban road, t varies between 12.1 and 12.9 s
5. Avoidance Maneuver *E:* Speed/path/direction change on urban road, t varies between 14.0 and 14.5 s

Principles of Highway Alignment

Referring to Figure (11), note that the horizontal alignment of a highway is referred to as the plan view, which is roughly equivalent to the perspective of an aerial photo of the highway. The vertical alignment is represented in a profile view, which gives the elevation of all points measured along the length of the highway.

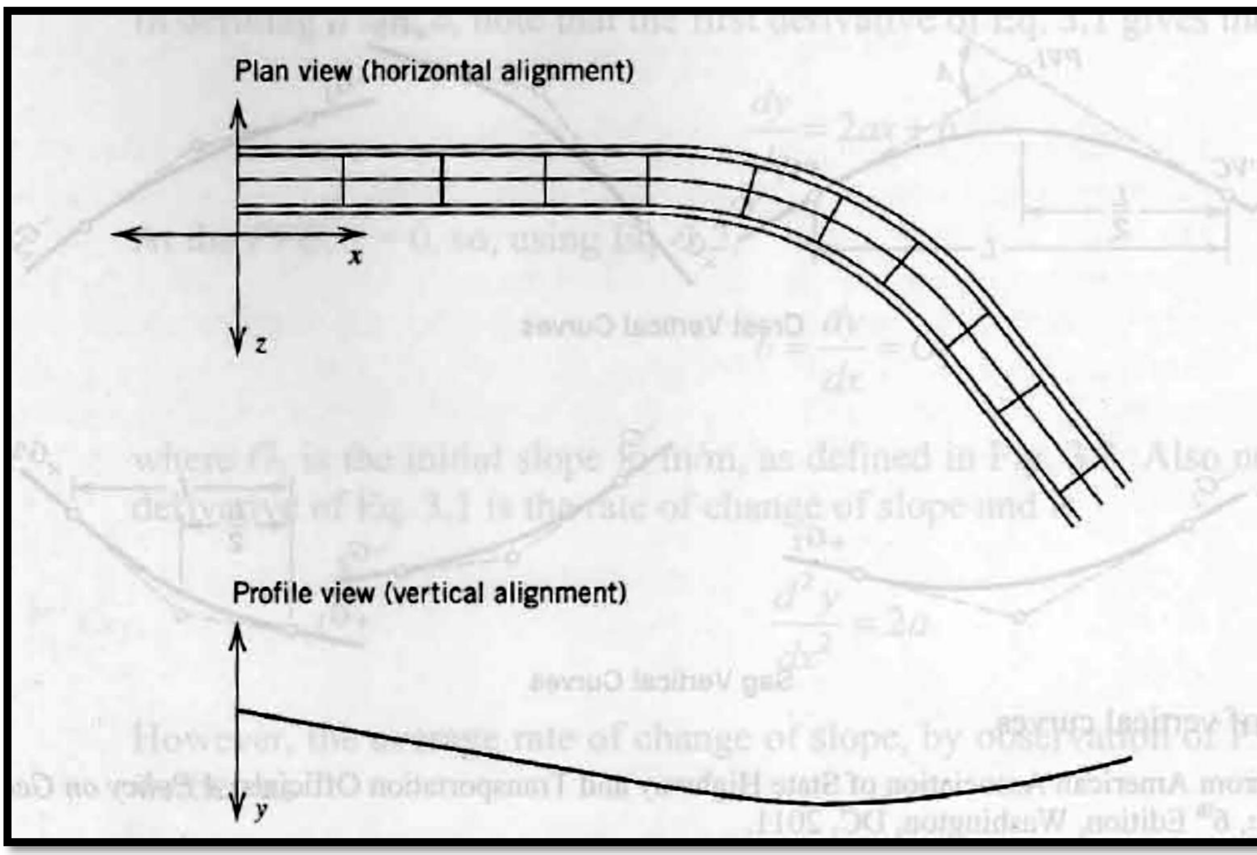


Figure (11): Highway alignment in two dimensional views

Vertical Alignment

Vertical alignment specifies the elevation of points along a roadway. The elevation of these roadway points is usually determined by the need to provide an acceptable level of driver safety, driver comfort, and proper drainage (from rainfall runoff).

Vertical curves can be broadly classified into crest vertical curves and sag vertical curves, as illustrated in Figure (12).

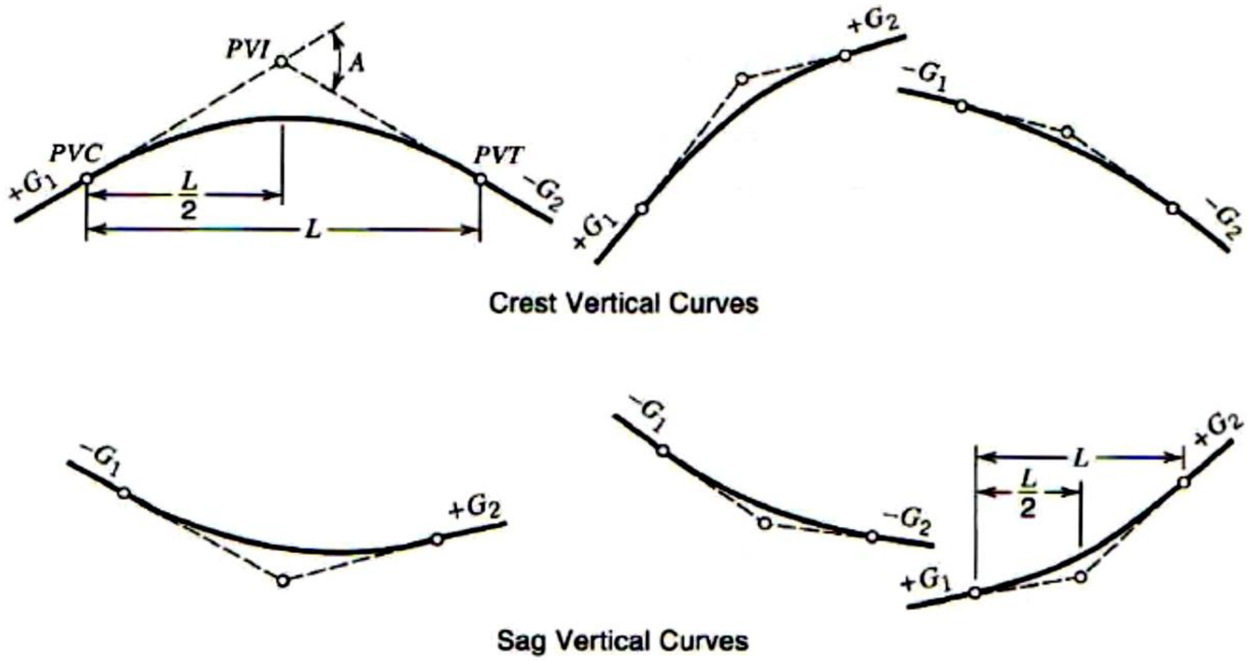


Figure (12): Types of vertical curves

G_1= initial roadway grade in percent or *m/m* (this grade is also referred to as the initial tangent grade, viewing Figure(12) from left to right),

G_2= final roadway (tangent) grade in percent or *m/m*

A = absolute value of the difference in grades (initial minus final, usually expressed in percent),

L= length of the curve in stations or m measured in a constant-elevation horizontal plane.

PVC= point of the vertical curve (the initial point of the curve),
PVI= point of vertical intersection (intersection of initial and final grades), and
PVT=point of vertical tangent, which is the final point of the vertical curve (the point where the curve returns to the final grade or, equivalently, the final tangent).

Some additional properties of vertical curves can now be formalized. For example, offsets, which are vertical distances from the initial tangent to the curve, as illustrated in Figure(13), are extremely important in vertical curve design and construction.

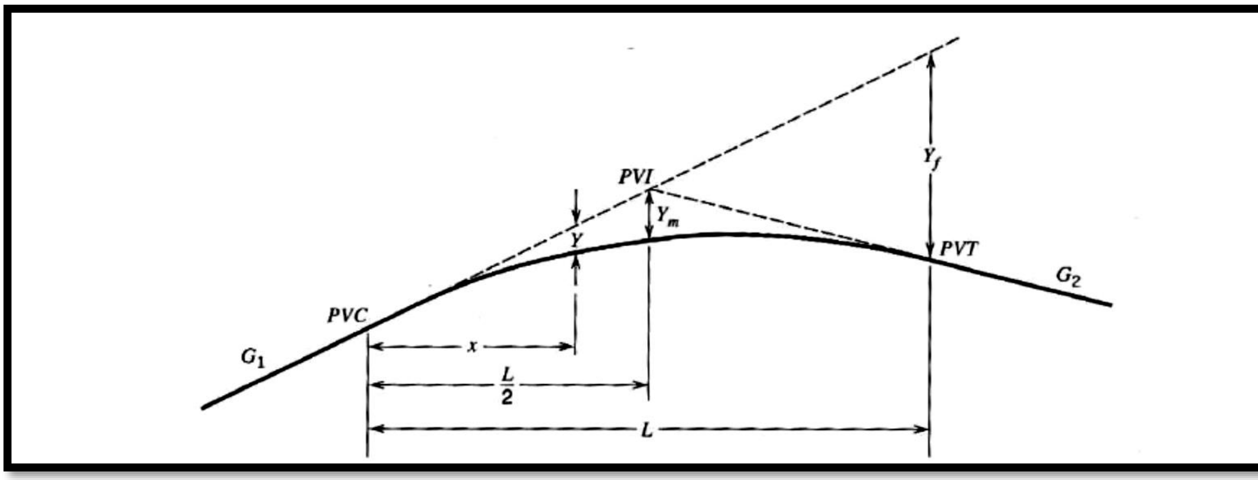


Figure (13) Offsets for equal-tangent vertical curves.

Y= offset at any distance x from the PVC in m,
Y_m= midcurve offset in m,
Y_f = offset at the end of the vertical curve in m,
x = distance from the *PVC* in m,
L = length of the curve in stations or m measured in a constant-elevation horizontal plane,

Another useful vertical curve property is one that gives the length of curve required to effect a **1 % change in slope**. Because the parabolic equation used for roadway elevations gives a constant rate of change of slope, it can be shown that the horizontal distance required to change the slope by 1 % is:

$$K = \frac{L}{A}$$

Vertical Curve Fundamentals

In connecting roadway grades (tangents) with an appropriate vertical curve, a mathematical relationship defining elevations at all points (or equivalently, stations) along the vertical curve is needed. A parabolic function bas been found suitable in this regard because, among other things, it provides a constant rate of change of slope and implies equal curve tangents. The general form of the parabolic equation, as applied to vertical curves, is

$$y = ax^2 + bx + c$$

where:

y = roadway elevation at distance x from the beginning of the vertical curve (the PVC) in m,

x = distance from the beginning of the vertical curve in stations or m,

a, b = coefficients, and

c = elevation of the PVC (because x = 0 corresponds to the PVC) in m.

In defining *a* and *b*, note that the first derivative of the previous equation. gives the slope and is

$$\frac{dy}{dx} = 2ax + b$$

At the *PVC*, *x* = 0, so,

$$b = \frac{dy}{dx} = G_1$$

where G_1, is the initial slope in *m/m*,. Also note that the second derivative of the equation is the rate of change of slope and is

$$\frac{d^2y}{dx^2} = 2a$$

However, the average rate of change of slope, can also be written as

$$\frac{d^2y}{dx^2} = \frac{G_2 - G_1}{L}$$

$$a = \frac{G_2 - G_1}{2L}$$

To determine the minimum length of curve for a required sight distance, the properties of a parabola for an equal tangent curve can be used to show that:

For $S < L$

$$L_m = \frac{AS^2}{200\left(\sqrt{H_1} + \sqrt{H_2}\right)^2}$$

For $S > L$

$$L_m = 2S - \frac{200\left(\sqrt{H_1} + \sqrt{H_2}\right)^2}{A}$$

Where:

L_m = minimum length of vertical curve in m,

A = absolute value of the difference in grades $|G_2 - G_1|$, expressed as a percentage.

For the sight distance required to provide adequate SSD, current AASHTO design guidelines [2011] use a driver eye height, ***H*₁ of 1080 mm** and a roadway object height, ***H*₂, of 600 mm** (the height of an object to be avoided by stopping before a collision). Substituting AASHTO guidelines for H_1 and H_2 and letting S = SSD gives:

For SSD < *L*

$$L_m = \frac{A \times \text{SSD}^2}{658}$$

For SSD > *L*

$$L_m = 2 \times \text{SSD} - \frac{658}{A}$$

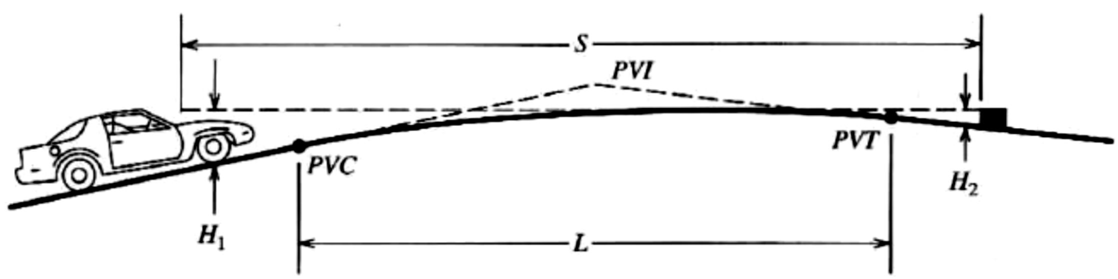


Figure (14): Stopping sight distance considerations for crest vertical curves

Sag Vertical Curve Design

Sag vertical curve design differs from crest vertical curve design in the sense that sight distance is governed by **nighttime conditions** because in daylight, sight distance on a sag vertical curve is unrestricted. Thus the critical concern for sag vertical curve design is the length of roadway illuminated by the vehicle headlights, which is a function of the height of the headlight above the roadway and the inclined angle of the headlight beam, relative to the horizontal plane of the car. The sag vertical curve sight distance design problem is illustrated in Figure(15).

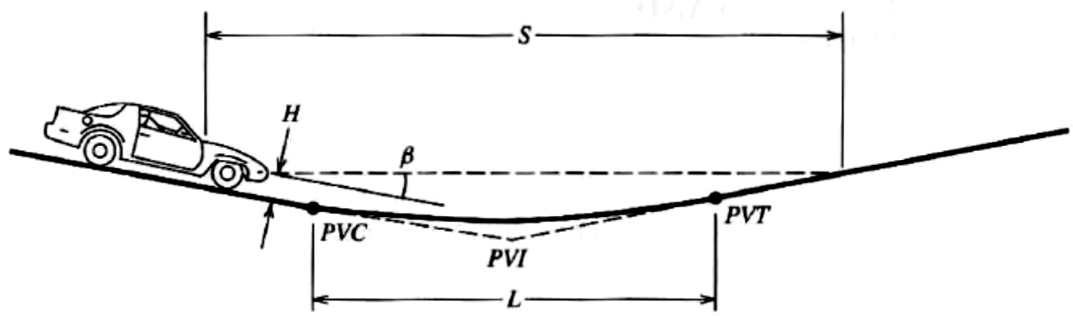


Figure (15): Stopping sight distance considerations for sag vertical curves

S= sight distance in m,
H= height of headlight in m,
β= inclined angle of headlight beam in degrees,
L= length of the curve in m
To determine the minimum length of curve for a required sight distance, the

properties of a parabola for an equal-tangent curve can be used to show that:

For $S < L$

$$L_m = \frac{AS^2}{200(H + S\tan\beta)}$$

For $S > L$

$$L_m = 2S - \frac{200(H + S\tan\beta)}{A}$$

For the sight distance required to provide adequate SSD, current AASHTO design guidelines [2011] use a **headlight height of 600 mm** and an **upward angle of one degree**. Substituting these design guidelines and S = SSD gives:

For SSD $< L$

$$L_m = \frac{A \times \mathrm{SSD}^2}{120 + 3.5 \times \mathrm{SSD}}$$

For SSD $> L$

$$L_m = 2 \times SSD - \frac{120 + 3.5 \times \mathrm{SSD}}{A}$$