Ministry of Higher Education
and Scientific Research
Al-Muthanna University
Civil Engineering Department

Traffic Engineering II

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Lecture No.1

4th May 2020

Traffic flow characteristics

- **Hourly volume**: can be defined as the number of vehicles that pass a certain point on a highway during one hour.

$$q = \ measured\ in\ (\frac{veh}{hr})\ \ or\ \ (vph)$$

- **Flow rate**: can be defined as the number of vehicles that pass a certain point on a highway during a period time (usually 5, 10, 15 minutes).

$$q = \frac{N}{T}\ \ measured\ in\ (\frac{veh}{hr})\ \ or\ \ (vph)$$

- **Peak Hour Flow (PHF)**: is the ratio of total hourly volume to the peak flow rate within an hour.

$$PHF = \frac{Hourly\ volume}{Peak\ flow\ rate\ within\ the\ hour}$$

$$PHF = \frac{No.\ of\ observed\ veh\ in\ an\ hour}{Max\ flow\ rate\ in\ peak\ hour}$$

Examples

Time	No. of Veh.	Flow Rate
9:00 – 9:15	500	2000
9:15 – 9:30	600	2400
9:30 – 9:45	550	2200
9:45 – 10:00	500	2000
Hourly volume (vph)		**2150**
Peak Flow Rate (vph)		**2400**
PHF		**0.896**

Time	No. of Veh.	Flow Rate
9:00 – 9:10	350	2100
9:10 – 9:20	400	2400
9:20 – 9:30	450	2700
9:30 – 9:40	350	2100
9:40 – 9:50	320	1920
9:50 – 10:00	280	1680
Hourly volume (vph)		**2150**
Peak Flow Rate (vph)		**2700**
PHF		**0.796**

➢ **Density**: can be described as the number of vehicles that occupying a given length of a highway.

$$D = \frac{N}{L} \qquad measured\ in\ (\frac{veh}{mile})\ or\ \left(\frac{veh}{km}\right)$$

➢ **Spacing or distance headway**: can be measured between the front bumpers of two successive vehicles as shown in Figure 1 below.

$$S = \frac{1000}{D} \qquad measured\ in\ (\frac{m}{veh})$$

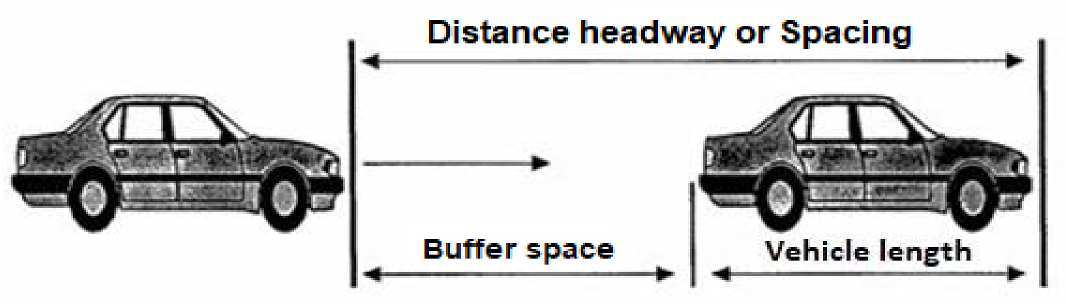


Figure 1: Illustration of distance headway (spacing)

Example

About 60 vehicles per lane have been observed on 0.5 km length motorway. If the motorway consists of 3 lanes, determine the traffic density and space between successive vehicles.

$$Density\ on\ one\ lane = \frac{60}{0.5} = 120\ veh/km$$

$$Density\ on\ 3\ lanes = 3 * 120 = 360\ veh/km$$

$$Space = \frac{1000}{120} = 8.33\ m/veh$$

➢ **Speed**: is the distance travelled by a vehicle during a unit time. It can be divided into:

- **Space Mean Speed (SMS)**: which represents the average travel speed of a vehicle on a highway section that used in the calculation of the levels of service. Also, it can be defined as Average Travel Speed (ATS).

$$SMS = v_s = \frac{N}{\sum_1^N \frac{1}{v_i}} = \frac{N * L}{\sum_1^n t_i}\ measured\ in\ (\frac{mile}{hr} = mph)\ or\ \left(\frac{km}{hr} = kph\right)$$

- **Time Mean Speed (TMS)**: which is the arithmetic mean of speeds of vehicles passing a certain point on a highway during a unit time.

$$TMS = \frac{\sum_1^N v_{t_i}}{N} \; measured\ in\ \left(\frac{mile}{hr} = mph\right)\ or\ \left(\frac{km}{hr} = kph\right)$$

Example

Calculate density, space, time mean speed and space mean speed for the four cars A,B,C & D on 500 m length roadway. The speeds were obtained by photography as shown in table below.

Cars	Speed (kph)
A	25
B	30
C	45
D	55

$$\boldsymbol{Density = \frac{4 * 1000}{500} = 8\ veh/km}$$

$$\boldsymbol{Space = \frac{1000}{8} = 125\ m/veh}$$

$$\boldsymbol{TMS = \frac{25 + 30 + 45 + 55}{4} = 38.75\ kph}$$

$$\boldsymbol{SMS = \frac{4}{\left(\frac{1}{25} + \frac{1}{30} + \frac{1}{45} + \frac{1}{55}\right)} = 35.40\ kph}$$

Fundamental relationships between speed, flow and density of highway traffic stream:

$$\boldsymbol{Flow\ (q) = Space\ Mean\ Speed\ (SMS) * Density\ (D)}$$

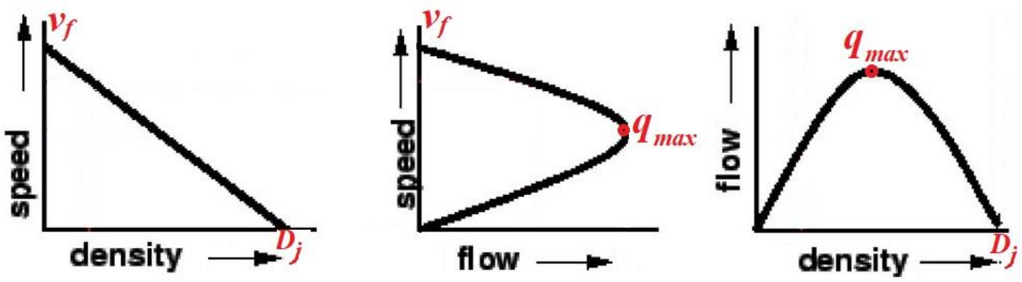


Figure 2: Fundamental relationships diagram

If the relationship between any two of these characteristics are known then the remaining variables can be obtained. It is often assumed that **speed** and **density** are connected by a linear relationship with free-flow speed $\boldsymbol{v_f}$ at zero density and zero speed at jam density $\boldsymbol{D_j}$.

$$q_{max} \qquad at \qquad v_s = \frac{v_f}{2} \qquad and \qquad D = \frac{D_j}{2}$$

The three relationships are shown in Figure 2. It can be seen that the maximum flow occurs when the density is half of the jam density and when the speed is half of the free-flow speed.

The speed – density relationship is of the form:

$$v_s = v_f\left(1 - \frac{D}{D_j}\right) \quad or \quad D = D_j\left(1 - \frac{v_s}{v_f}\right) \quad then$$

Since $\boldsymbol{Flow\ (q) = v_s * D}$ so that

$$Flow\ (q) = v_s\, D_j - \left(\frac{D_j}{v_f}\right) v_s^2$$

$$for\ obtaining\ max\ flow$$

$$\frac{dq}{dv_s} = 0 \qquad resulting\ in$$

$$D_j - \left(\frac{D_j}{v_f}\right) * 2v_s = 0 \quad then$$

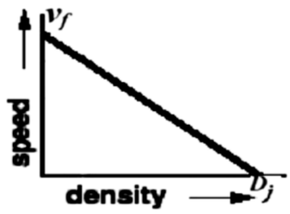
$$v_s = \frac{v_f}{2} \qquad finally, the\ max\ flow\ can\ be\ founded\ as$$

$$q_{max} = \frac{D_j\, v_f}{2} - \frac{D_j}{v_f}\left(\frac{v_f}{2}\right)^2 = \frac{D_j\, v_f}{4}$$

Example

The relationship between speed and density for a given section on a highway was founded to be:

$$v = 50 - \frac{D}{3}$$



1. Calculate the jam density.
2. Determine the free speed.
3. Compute the max flow.

The jam density occurs at $v = 0$ $\quad then \quad D_j = 150\ veh/km$

The free speed occurs at $D = 0$ $\quad then \quad v_f = 50\ kph$

$$q_{max} = \frac{D_j\, v_f}{4} = \frac{150 * 50}{4} = 1875\ vph$$

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Capacity and Service Volume

- **Capacity:** can be defined as the maximum traffic volume which produces comfortable free condition (see Table 1).
- **Service volume**: is the number of vehicles that can pass over a given section of lane or roadway during a given period time.

Table 1: Typical capacities for highway systems

Facility	Definition	Capacity in passenger cars
Freeways and expressways away from ramps and weaving sections	Per lane of freeway per hour	2400
Two-lane highway	Total in both directions, per hour	2800
Urban signalized intersection	Total per lane for through movement per hour of continuous green (urban areas with population over 250,000)	1900
Small town or rural signalized intersection	Total per lane for through movement per hour of continuous green	1750
Modern roundabout	Total per approach lane without any conflicting traffic in circle, depending on roundabout geometry and configuration	1400–1600

- **Traffic Flow Regime**: is an operation threshold that describes the capacity over a certain road section (in either one or both directions) as shown in Figure 3. It can be described as follows:

    1. *Undersaturated or uncongested flow*: is a traffic condition in which the arrival flow rate is lower than the capacity or the service flow rate at a point or uniform segment of a lane or roadway.
    2. *Saturated or uniform flow*: is a traffic condition in which the arrival flow rate is equal to the capacity or the service flow rate at a lane or roadway section.
    3. *Oversaturated or congested flow*: is a traffic flow condition in which the arrival flow rate is greater than the capacity. Congested flow is often caused by a downstream bottleneck and results in queuing upstream of the bottleneck or choke point.

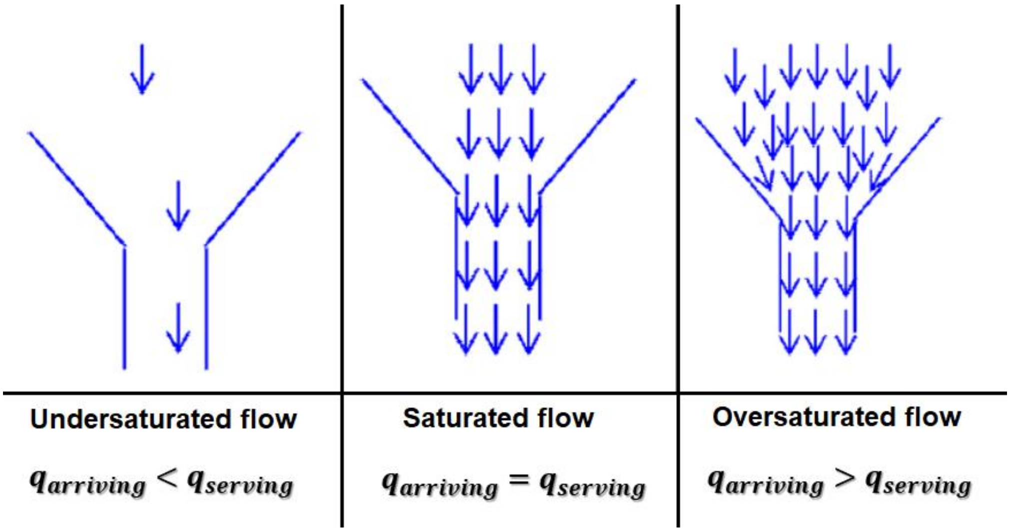

Figure 3: Illustration of flow regimes

- **Level of Service**: is a measure of the quality of flow along a highway. In the HCM (2010), there are six defined levels of service, designated as shown in Figure 4.

Figure 4: Illustration of six LOS on a highway

Table 6.1 LOS Criteria for Basic Freeway Segments

Criterion	LOS A	B	C	D	E
			FFS = 75 mi/h		
Maximum density (pc/mi/ln)	11	18	26	35	45
Average speed (mi/h)	75.0	74.8	70.6	62.2	53.3
Maximum *v/c*	0.34	0.56	0.76	0.90	1.00
Maximum service flow rate (pc/h/ln)	820	1350	1830	2170	2400
			FFS = 70 mi/h		
Maximum density (pc/mi/ln)	11	18	26	35	45
Average speed (mi/h)	70.0	70.0	68.2	61.5	53.3
Maximum *v/c*	0.32	0.53	0.74	0.90	1.00
Maximum service flow rate (pc/h/ln)	770	1260	1770	2150	2400
			FFS = 65 mi/h		
Maximum density (pc/mi/ln)	11	18	26	35	45
Average speed (mi/h)	65.0	65.0	64.6	59.7	52.2
Maximum *v/c*	0.30	0.50	0.71	0.89	1.00
Maximum service flow rate (pc/h/ln)	710	1170	1680	2090	2350
			FFS = 60 mi/h		
Maximum density (pc/mi/ln)	11	18	26	35	45
Average speed (mi/h)	60.0	60.0	60.0	57.6	51.1
Maximum *v/c*	0.29	0.47	0.68	0.88	1.00
Maximum service flow rate (pc/h/ln)	660	1080	1560	2020	2300
			FFS = 55 mi/h		
Maximum density (pc/mi/ln)	11	18	26	35	45
Average speed (mi/h)	55.0	55.0	55.0	54.7	50.0
Maximum *v/c*	0.27	0.44	0.64	0.85	1.00
Maximum service flow rate (pc/h/ln)	600	990	1430	1910	2250

Criterion	LOS A	B	C	D	E
			FFS = 120 km/h		
Maximum density (pc/km/ln)	7	11	16	22	28
Average speed (km/h)	120.0	120.0	114.6	99.6	85.7
Maximum *v/c*	0.35	0.55	0.77	0.92	1.00
Maximum service flow rate (pc/h/ln)	840	1320	1840	2200	2400
			FFS = 110 km/h		
Maximum density (pc/km/ln)	7	11	16	22	28
Average speed (km/h)	110.0	110.0	108.5	97.2	83.9
Maximum *v/c*	0.33	0.51	0.74	0.91	1.00
Maximum service flow rate (pc/h/ln)	770	1210	1740	2135	2350
			FFS = 100 km/h		
Maximum density (pc/km/ln)	7	11	16	22	28
Average speed (km/h)	100.0	100.0	100.0	93.8	82.1
Maximum *v/c*	0.30	0.48	0.70	0.90	1.00
Maximum service flow rate (pc/h/ln)	700	1100	1600	2065	2300
			FFS = 90 km/h		
Maximum density (pc/km/ln)	7	11	16	22	28
Average speed (km/h)	90.0	90.0	90.0	89.1	80.4
Maximum *v/c*	0.28	0.44	0.64	0.87	1.00
Maximum service flow rate (pc/h/ln)	630	990	1440	1955	2250

Table 6.11 LOS Criteria for Multilane Highways

Criterion	LOS A	B	C	D	E
			FFS = 60 mi/h		
Maximum density (pc/mi/ln)	11	18	26	35	40
Average speed (mi/h)	60.0	60.0	59.4	56.7	55.0
Maximum *v/c*	0.30	0.49	0.70	0.90	1.00
Maximum service flow rate (pc/h/ln)	660	1080	1550	1980	2200
			FFS = 55 mi/h		
Maximum density (pc/mi/ln)	11	18	26	35	41
Average speed (mi/h)	55.0	55.0	54.9	52.9	51.2
Maximum *v/c*	0.29	0.47	0.68	0.88	1.00
Maximum service flow rate (pc/h/ln)	600	990	1430	1850	2100
			FFS = 50 mi/h		
Maximum density (pc/mi/ln)	11	18	26	35	43
Average speed (mi/h)	50.0	50.0	50.0	48.9	47.5
Maximum *v/c*	0.28	0.45	0.65	0.86	1.00
Maximum service flow rate (pc/h/ln)	550	900	1300	1710	2000
			FFS = 45 mi/h		
Maximum density (pc/mi/ln)	11	18	26	35	45
Average speed (mi/h)	45.0	45.0	45.0	44.4	42.2
Maximum *v/c*	0.26	0.43	0.62	0.82	1.00
Maximum service flow rate (pc/h/ln)	490	810	1170	1550	1900

Criterion	LOS A	B	C	D	E
			FFS = 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 *v/c*	0.32	0.50	0.72	0.92	1.00
Maximum service flow rate (pc/h/ln)	700	1100	1575	2015	2200
			FFS = 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
			FFS = 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
			FFS = 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

Factors affecting LOS of the road

There are several factors must be considered in evaluation the LOS of the road:

1. Speed and travel time
2. Freedom to manoeuvre
3. Safety and accidents
4. Traffic interruption or restriction
5. Driving comfortable and convenience
6. Economy

Therefore, two parameters have been selected in measuring the LOS of the road:

- Average travel speed or SMS on urban roads, and operation speed on rural roads.
- The ratio of demand volume to capacity (*v/c*)

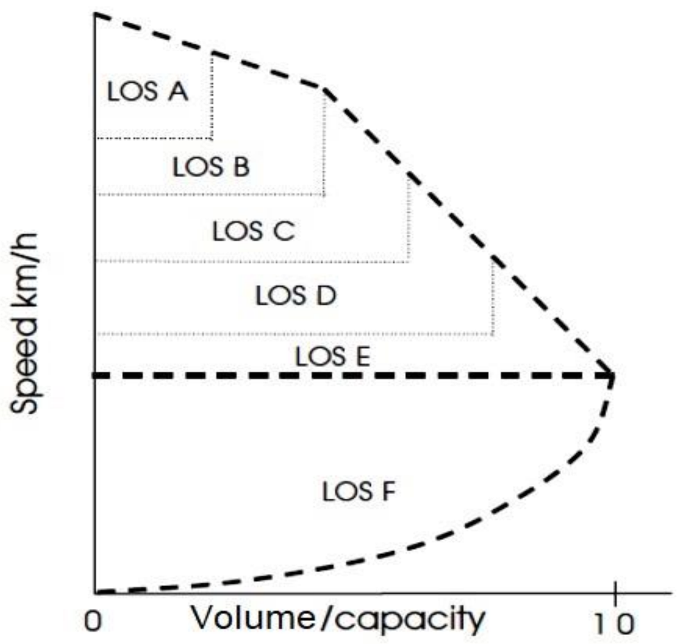

Figure 5: Relationship between the SMS speed and *v/c* ratio

Estimation of LOS for a highway

The basic relationship governing analysis of a given highway section is as follows:

$$SF = C * \frac{v}{c} * N * f_w * f_b * f_{HV}$$

Where,

SF = Service Flow Rate under prevailing conditions in total (vph) for one direction. It can be determined as $SF = \frac{Hourly\ volume}{PHF}$

C = Capacity of highway lane under ideal conditions in (pcu/hr/lane) or (pcphpln).

v/c = maximum allowable *v/c* ratio

N = Number of lanes in one direction

f_w, f_b, f_{HV} are adjustment factors for restricted lane width, non-regular driving behaviour and presence of heavy good vehicles, respectively.

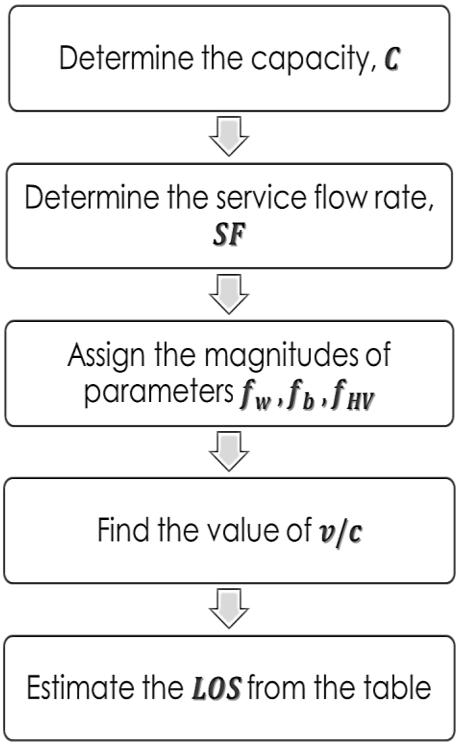

Figure 6: Steps to estimate or evaluate the LOS

Example 1

An existing 4 lanes highway (with 80 kph) in urban area serve a peak hour demand of 2400 vph with 10% trucks and 3% buses. The PHF= 0.9. All drivers are regular users, lane capacity 2000 pcu/hr/ln. what is the LOS of this highway?

$$Hourly\ volume = 2400\ (0.10 * 3 + 0.03 * 2 + 0.87 * 1) = 2952\ pcu/hr$$

$$SF = \frac{Hourly\ volume}{PHF} = \frac{2952}{0.9} = 3280\ pcu/hr$$

$$SF = C * \frac{V}{C} * N * f_w * f_b * f_{HV}$$

$$3280 = 2000 * \frac{V}{C} * 2 * 1 * 1 * 1$$

$$\frac{v}{c} = 0.82 \quad \longrightarrow \quad LOS = D$$

Example 2

An urban freeway with 120 kph, when designed was expected to carry a directional design hourly volume of 4000 vph. About 10% are trucks and 15% are buses with PHF=0.89. If the LOS=C is desired on this freeway, how many lanes would be required without capacity restriction if lane capacity is 2000 pcu/hr/ln? Assume any required data suitably.

$$Hourly\ volume = 4000\ (0.10 * 3 + 0.15 * 2 + 0.75 * 1) = 5400\ pcu/hr$$

$$SF = \frac{Hourly\ volume}{PHF} = \frac{5400}{0.89} = 6068\ pcu/hr$$

$$SF = C * \frac{v}{c} * N * f_w * f_b * f_{HV}$$

$$6068 = 2000 * 0.77 * N * 1 * 1 * 1$$

$$N = 3.94 \approx 4\ lanes$$

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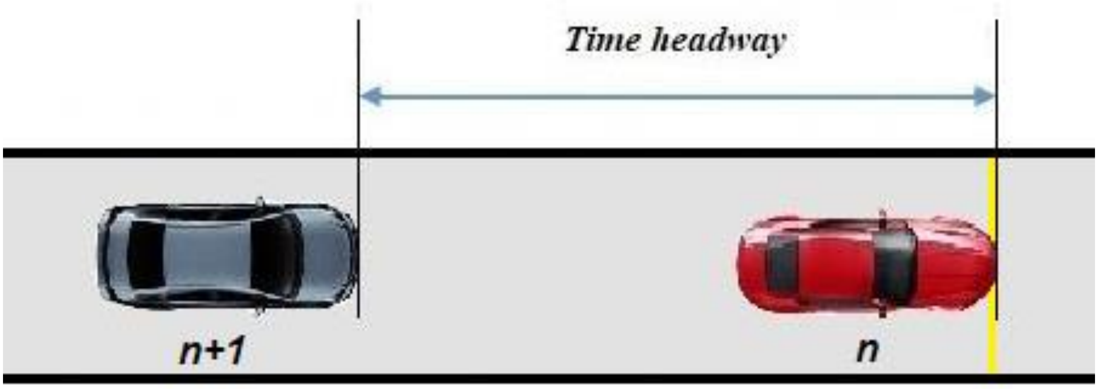
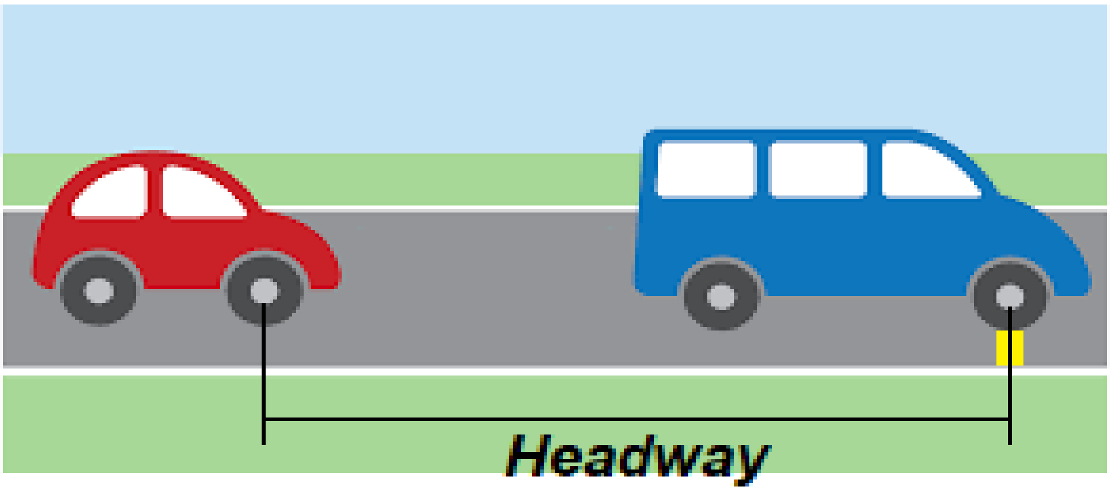
Lecture No.3

Time headway in traffic flow

- **Headway:** It can be defined as the time gap between two successive vehicles passing over a certain point or a reference line on a road section. See Figure below.



- Another definition of headway that it may be the interval between the passage of two vehicles usually measured on the basis of front wheels as shown below.



It can be measures as $$Time\ Headway = T_{n+1} - T_n$$

- The time headway is one of the surrogate safety measures that plays a significant role in traffic safety and performance.

- The allowable following headway with the leading vehicle should not be less than **2 seconds** distance since it gives an indication of the probability of two or more vehicles involved in a tailgating collision or rear-end collision.

- Time headway is affected by several factors such as traffic composition, traffic flow, reaction time of the driver, and braking distance.

- Several statistical formulas of time headway distributions can be applied to represent the vehicles' arrival based on the traffic flow rates:
  1) **Poisson Distribution** : which is appropriate to describe the truly random arrival of vehicles at a certain point on a road.

$$\boldsymbol{P(arriving\ n\ vehicle\ at\ t) = \frac{(qt)^n e^{-qt}}{n!}}$$

  2) Negative Exponential Distribution : which gives the probability of discrete event occurring with a specific time interval.

$$\boldsymbol{P(headway \geq t) = e^{-qt}}$$

Fitting the expected distribution to the observed data

After applying the distribution formula to obtain the calculated frequencies, it is necessary to compare these frequencies with the observed data. This is usually made by using the Chi-square test. This test measures the differences between the expected and observed data for each class as shown in the following equation:

$$\boldsymbol{Calculated\ Chi - square = \frac{(observed - expected)^2}{expected}}$$

Then, a comparison between the calculated Chi-square and theoretical Chi-square will be made for deciding on goodness of distribution fit. The method of calculation of each distribution are explained in the next sections.

1) Counting distribution: Poisson distribution

Before fitting the distribution with the observed data, it can be rewrite the Poisson's equation as shown below to simplify the calculation:

$$\boldsymbol{P(arriving\ n\ vehicle\ at\ t) = \frac{(m)^n e^{-m}}{n!} \qquad where \qquad m = qt}$$

Now when a Poisson distribution is to be fitted to the observed data, the parameter $\boldsymbol{m}$ is computed as:

$$\boldsymbol{m = \frac{total\ number\ of\ observed\ vehicles}{total\ length\ of\ time\ period} \qquad then \qquad P(0) = e^{-m}}$$

$$\boldsymbol{\frac{P(n)}{P(n-1)} = \frac{\frac{m^{(n)}}{n!}\ e^{-m}}{\frac{m^{(n-1)}}{(n-1)!}\ e^{-m}} = \frac{m}{n} \qquad then,\ Poisson\ equation\ becomes:}$$

$$\boldsymbol{P(n) = \frac{m}{n} P(n-1) \qquad Thus}$$

$$\boldsymbol{P(0) = e^{-m} \qquad\qquad P(2) = \frac{m}{2} P(1)}$$

$$\boldsymbol{P(1) = mP(0) \qquad\qquad P(3) = \frac{m}{3} P(2)}$$

Example 1

On Vere street in London, the tabulated observation were made. Is it possible to use Poisson distribution to represent the data?

Count of vehicles per 10 seconds	Observed frequency	Total observed vehicles	Poisson Equation $P(n) = \frac{m}{n} P(n-1)$	Expected frequency	Chi-square value
0	94	0	$e^{-0.616} = 0.54$	0.54*180 = 97.2	0.105
1	63	63	$0.616 * 0.54 = 0.332$	0.332*180 = 59.8	0.171
2	21	42	$\frac{0.616}{2} * 0.332 = 0.103$	0.103*180 = 18.5	0.338
3	2	6	$\frac{0.616}{3} * 0.103 = 0.021$	0.021*180 = 3.8	1.389
>3	0	0	0.004	0.004*180 = 0.7	
Total	180	111	1.000	180	2.003

Since there were 111 vehicles in 180 (10 sec) periods, then:

$$m = \frac{111}{180} = 0.616 \quad \text{and} \quad Hourly\ flow = \frac{111}{180*10/3600} = 222\ vph$$

$$Calculated\ Chi-square = \frac{(observed - expected)^2}{expected} = \frac{(94-97.2)^2}{97.2} = 0.105$$

Then, the final calculated Chi-square is 2.003 which will be compared with the tabulated Chi-square that will be obtained from the Table. After merging the classes of frequencies less than 5 observations, the degree of freedom can be calculated as :

$d.f. = No.\ of\ classes - No.of\ parameters = 4 - 3 = 1$

Next, enter the Chi-squre table with d.f $= 1$ and significance level of 5%, the Tabulated Chi-square is equal to 3.84. Since the Tabulated Chi-square is greater than the Calculated Chi-square (3.84 > 2.003), the observed data can be represented by Poisson distribution.

Table of Chi-square values under different levels of significance and degrees of freedom.

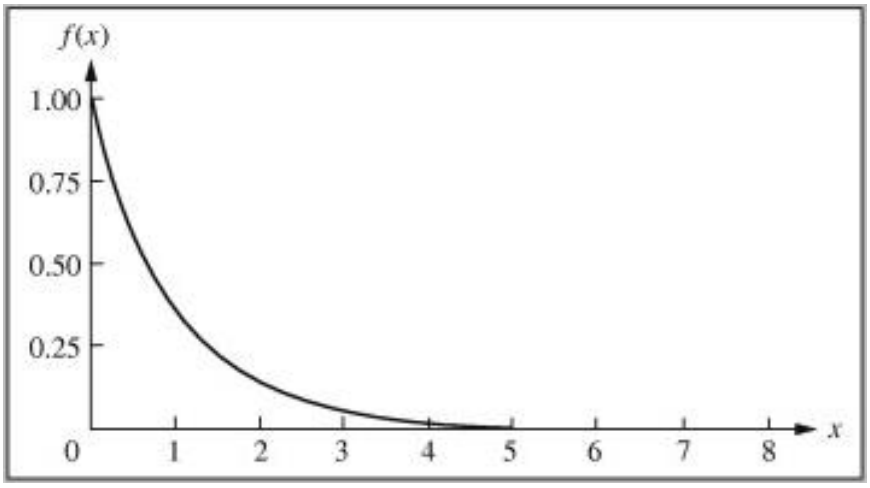
d.f.	$\chi^2_{.25}$	$\chi^2_{.10}$	$\chi^2_{.05}$	$\chi^2_{.025}$	$\chi^2_{.010}$	$\chi^2_{.005}$	$\chi^2_{.001}$
1	1.32	2.71	3.84	5.02	6.63	7.88	10.8
2	2.77	4.61	5.99	7.38	9.21	10.6	13.8
3	4.11	6.25	7.81	9.35	11.3	12.8	16.3
4	5.39	7.78	9.49	11.1	13.3	14.9	18.5
5	6.63	9.24	11.1	12.8	15.1	16.7	20.5
6	7.84	10.6	12.6	14.4	16.8	18.5	22.5
7	9.04	12.0	14.1	16.0	18.5	20.3	24.3
8	10.2	13.4	15.5	17.5	20.1	22.0	26.1
9	11.4	14.7	16.9	19.0	21.7	23.6	27.9
10	12.5	16.0	18.3	20.5	23.2	25.2	29.6
11	13.7	17.3	19.7	21.9	24.7	26.8	31.3
12	14.8	18.5	21.0	23.3	26.2	28.3	32.9
13	16.0	19.8	22.4	24.7	27.7	29.8	34.5
14	17.1	21.1	23.7	26.1	29.1	31.3	36.1
15	18.2	22.3	25.0	27.5	30.6	32.8	37.7
16	19.4	23.5	26.3	28.8	32.0	34.3	39.3
17	20.5	24.8	27.6	30.2	33.4	35.7	40.8
18	21.6	26.0	28.9	31.5	34.8	37.2	42.3
19	22.7	27.2	30.1	32.9	36.2	38.6	32.8
20	23.8	28.4	31.4	34.2	37.6	40.0	45.3
21	24.9	29.6	32.7	35.5	38.9	41.4	46.8
22	26.0	30.8	33.9	36.8	40.3	42.8	48.3
23	27.1	32.0	35.2	38.1	41.6	44.2	49.7
24	28.2	33.2	36.4	39.4	32.0	45.6	51.2
25	29.3	34.4	37.7	40.6	44.3	46.9	52.6
26	30.4	35.6	38.9	41.9	45.6	48.3	54.1
27	31.5	36.7	40.1	43.2	47.0	49.6	55.5
28	32.6	37.9	41.3	44.5	48.3	51.0	56.9
29	33.7	39.1	42.6	45.7	49.6	52.3	58.3
30	34.8	40.3	43.8	47.0	50.9	53.7	59.7
40	45.6	51.8	55.8	59.3	63.7	66.8	73.4
50	56.3	63.2	67.5	71.4	76.2	79.5	86.7
60	67.0	74.4	79.1	83.3	88.4	92.0	99.6
70	77.6	85.5	90.5	95.0	100	104	112
80	88.1	96.6	102	107	112	116	125
90	98.6	108	113	118	124	128	137
100	109	118	124	130	136	140	149

2) Interval distribution: Negative exponential distribution (NED)

Another traffic characteristics of importance is the time between events, i.e. the headway between arrival of vehicles:

$\boldsymbol{P(h \geq t) = e^{-qt}}$ where q is flow rate that can be computed as the reciprocal of the mean headway.

From this relationship, it may be seen that (under condition of random flow) the number of headway greater than any given value will be distributed according to an exponential curve as follows:



Example 1

Observation of headways between vehicles in single traffic stream were measured and classified into 3 seconds interval. Show the goodness of NED to observed data.

Headway in seconds	0-3	3-6	6-9	9-12	12-15	15-18	Total
Observed data	37	36	26	11	9	5	124

Headway interval in sec	Observed frequency	Cumu. Expected frequency = e^{-qt}	Relative Expected frequency	Expected frequency	Chi-square value
0-3	37	$e^{-0.169*0} = 1.0$	1.0 - 0.6 = 0.4	49.6	3.20
3-6	36	$e^{-0.169*3} = 0.6$	0.6 - 0.36 = 0.24	29.7	1.33
6-9	26	$e^{-0.169*6} = 0.36$	0.36 - 0.22 = 0.14	17.4	4.25
9-12	11	$e^{-0.169*9} = 0.22$	0.22 - 0.13 = 0.09	11.2	0.004
12-15	9	$e^{-0.169*12} = 0.13$	0.13 – 0.08 = 0.05	6.2	1.26
15-18	5	$e^{-0.169*15} = 0.08$	0.08 – 0.0 = 0.08	9.9	2.42
Total	124		1.00	124	12.464

$$Total\ time = 1.5(37) + 4.5(36) + 7.5(26) + 10.5(11) + 13.5(9) + 16.5(5) = 732\ sec$$

$$q = \frac{total\ observed\ events\ (vehicles)}{Total\ time} = \frac{124}{732} = 0.169\ \ veh/sec$$

$$Calculated\ Chi - square = \frac{(observed - expected)^2}{expected} = \frac{(37 - 49.6)^2}{49.6} = 3.20$$

Then, the final calculated Chi-square is 11.45 which will be compared with the tabulated Chi-square that will be obtained from the Table.

$$Degree\ of\ freedom = No.\ of\ classes - No. of\ parameters = 6 - 2 = 4$$

Enter the following table with d.f = 4 and significance level of 5% the Tabulated Chi-square is equal to 9.49. Since the Tabulated Chi-square is lower than the Calculated Chi-square (9.49 < 12.46), the observed data cannot be represented by Negative Exponential Distribution.

Homework No.1

Q1/ The number of road accidents per day were reported in 100 consecutive days as shown in table below. Check whether the distribution of these accidents can be considered as random.

No. of accidents	0	1	2	3	4	5	6
No. of days	**19**	**26**	**26**	**15**	**9**	**4**	**1**

Q2/ The following are tabulated headway in a single lane of traffic stream. Test the data for goodness fitting to the NE distribution.

Headway in sec	0-1	1-2	2-3	3-4	4-5	5-6	6-7	7-8
Observed data	**19**	**67**	**58**	**29**	**26**	**14**	**17**	**7**

Solutions

Q1/

No. of accidents	0	1	2	3	4	5	6	Total
No. of days (Observed frequency)	**19**	**26**	**26**	**15**	**9**	**4**	**1**	**100**
Total observed accidents	0	26	52	45	36	20	6	**185**
Poisson Equation P(n)	0.158	0.292	0.269	0.167	0.077	0.028	0.009	**1.00**
Expected frequency	15.8	29.2	26.9	16.7	7.7	2.8	0.9	**100**
Chi-square value	0.65	0.35	0.03	0.173	0.22	0.457		**1.88**

There were 185 accidents within 100 days , $\boldsymbol{m = \frac{185}{100} = 1.85}$

Combine classes which have a frequency less than 5

The calculated Chi-square = 1.88 Degree of freedom = 6 – 3 = 3 then,

The tabulated Chi-square = 7.81

Since the tabulated Chi-square is greater than the calculated Chi-square (7.81 > 1.88), then the observed data can be represented by Poisson distribution.

Q2/

Headway in sec	0-1	1-2	2-3	3-4	4-5	5-6	6-7	7-8	Total
Observed freq.	**19**	**67**	**58**	**29**	**26**	**14**	**17**	**7**	**237**
Total time	9.5	100.5	145	101.5	117	77	110.5	52.5	**713.5**
Cumu. Expected frequency = e^{-qt}	1.0	0.717	0.515	0.369	0.276	0.199	0.136	0.098	
Relative freq.	0.283	0.202	0.146	0.093	0.077	0.063	0.038	0.098	**1.00**
Expected freq	67.07	47.87	34.60	22.04	18.25	14.93	9.01	23.23	**237**
Chi-square value	34.45	7.64	15.83	2.20	3.29	0.058	2.76	11.34	**77.57**

$\boldsymbol{Total\ time = 713.5\ sec}$ **then** $\boldsymbol{q = \frac{total\ observed\ events\ (vehicles)}{Total\ time} = \frac{237}{713.5} = 0.332\ \ veh/sec}$

The calculated Chi-square = 77.57 **Degree of freedom = 8 – 2 = 6** **then,**

The tabulated Chi-square = 12.6

Since the tabulated Chi-square is less than the calculated Chi-square (12.6 < 77.57), the observed data cannot be represented by Negative Exponential Distribution.

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Traffic Engineering II

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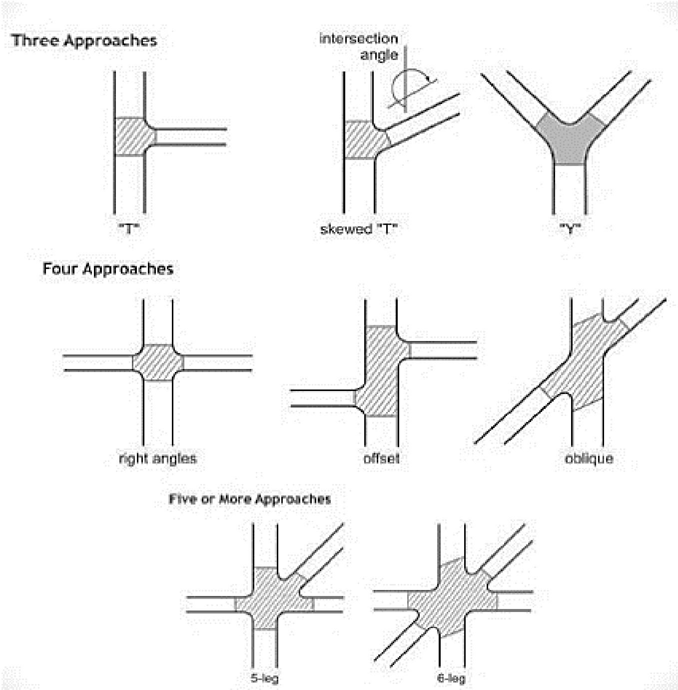
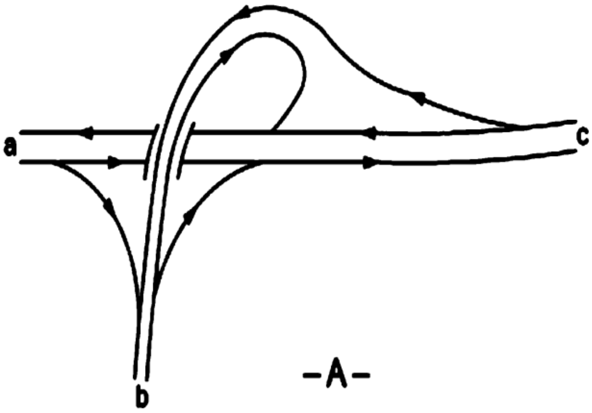
Lecture No.4

Intersections Design and Control

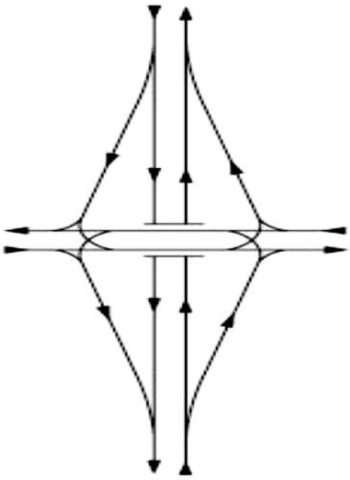
- **Intersection** is an area shared by two or more roads. This area is designated including side facilities for traffic movement for the vehicles to turn to different directions to reach their desired destinations.
- The main objectives of a signalised intersection design are:
  1) To reduce delay and severity of potential accidents.
  2) To improve the intersection performance and give priority to specific road users.
- Therefore, a properly designed approach should include a sufficient number of lanes (with the allowable speed limit) to serve the traffic flow and achieve the above objectives.
- The common intersections involve through or cross traffic movements on one or more of highways as well as involve turning movements between these highways.
- Intersections can be classified depending on the number of crossed or joined approaches:
  1) **At grade Intersections:** two or more roads intersect at a same level such as models in figure below:



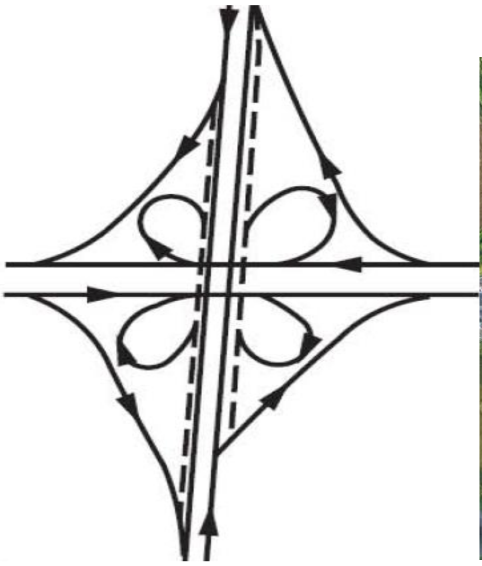
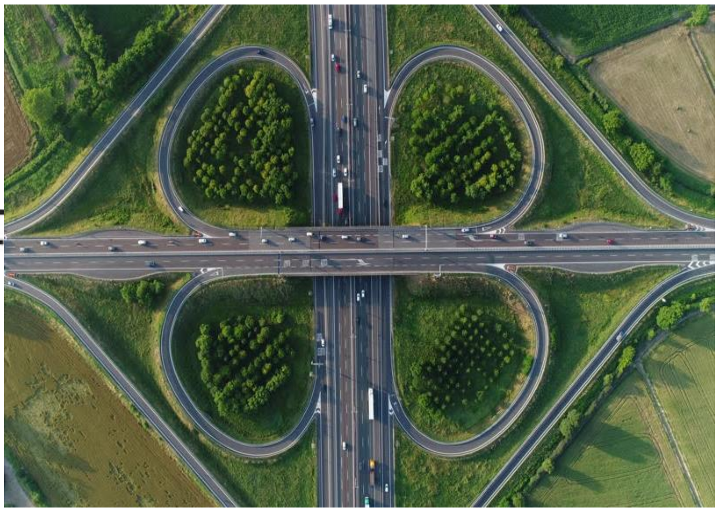
2) **Grade-separated Intersections:** known as 'Interchange' which is one road crosses others at a different level. See figures below:



-A- The trumpet type



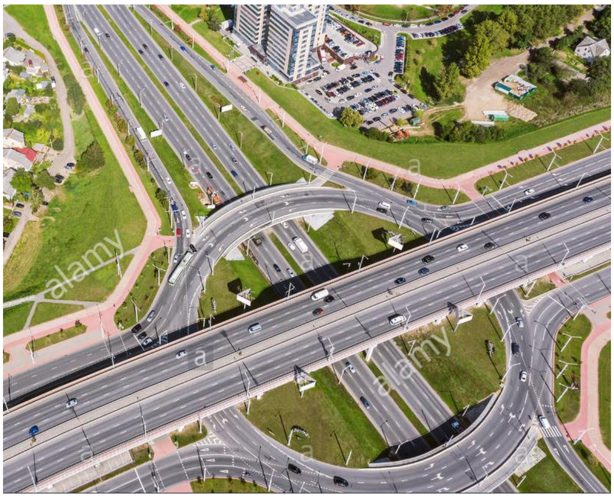

-B- The diamond type

-C- The cloverleaf type

3) **Roundabout:** is a circular intersection that provides a circular traffic pattern with significant reduction in the crossing conflict points. See figures below:



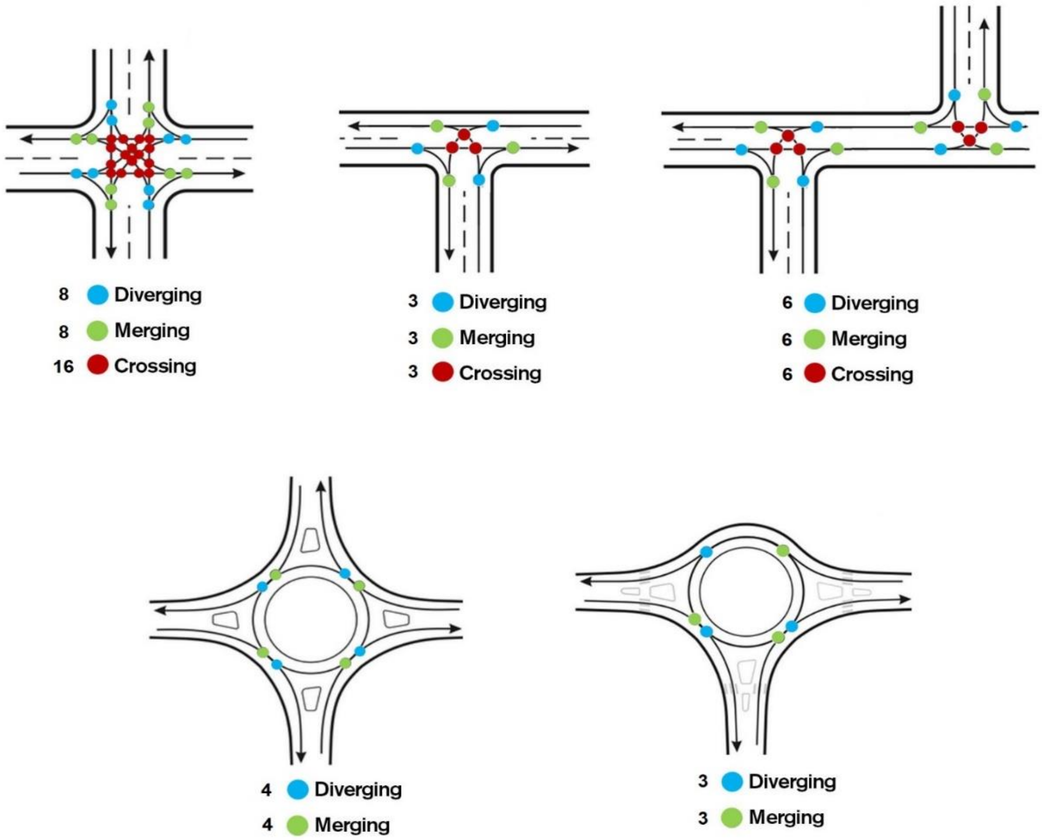



Factors considering in the selection of an intersection type:

1. Traffic volume
2. Design speed
3. Delay
4. Pedestrians movement
5. Cost and availability of land
6. Traffic accidents data

Conflict points:

Intersections have many conflict points (see figure below for different types of intersections) which might result in traffic accidents such as rear-end and right-angle collisions.

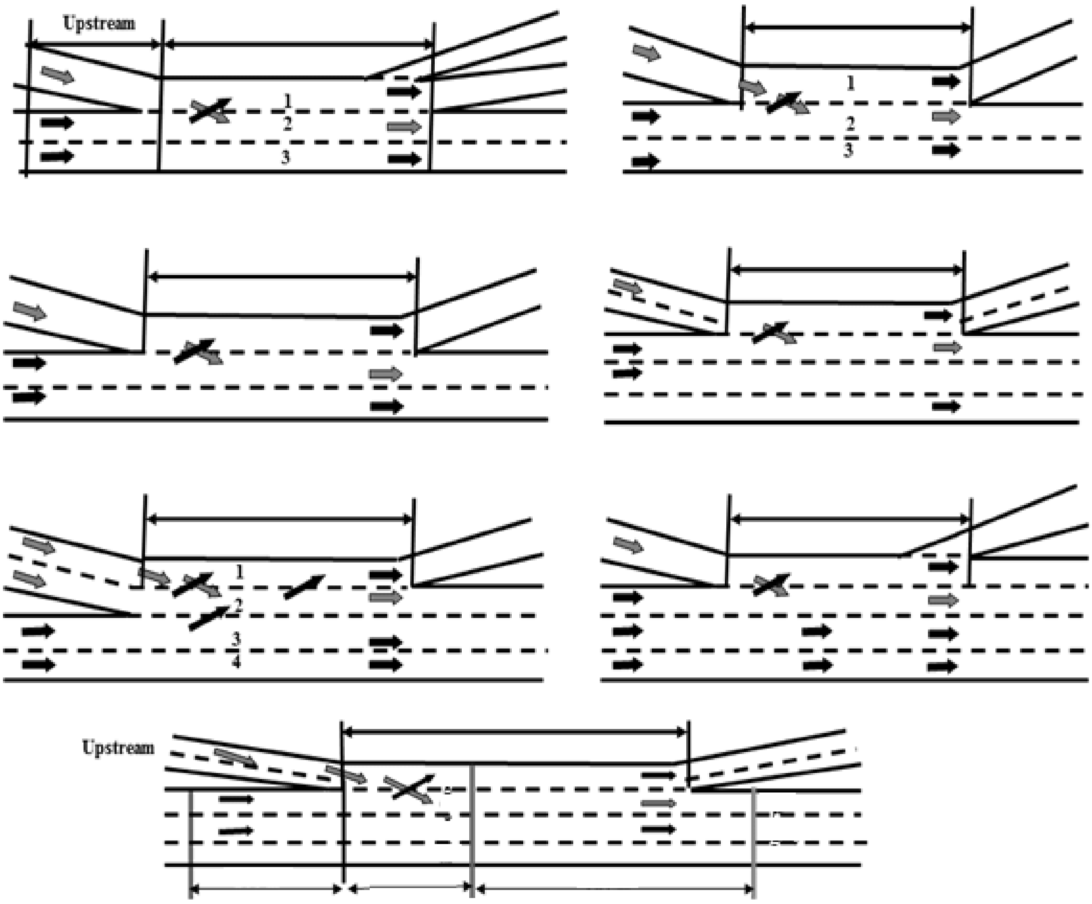


Merging movement: means that two separated traffic streams join.

Diverging movement: means that combined traffic streams separate.

Crossing movement: means that two or more traffic streams cross each other at a specific point.

Also, weaving movement can be notice along the highway with many merging and diverging movements as shown in figure below:

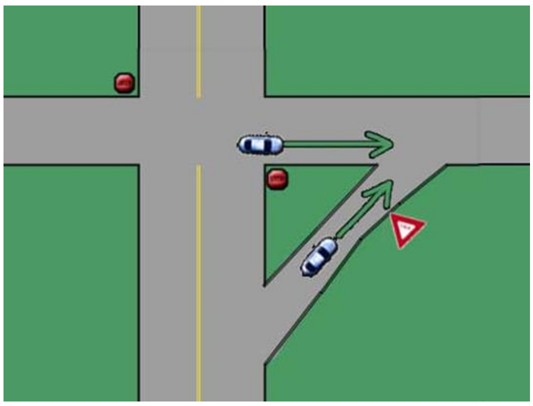


Weaving movements on different sections of highways

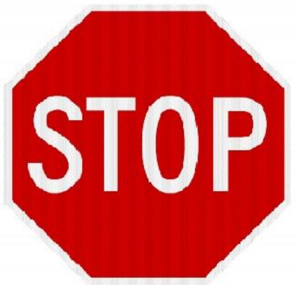
Intersection Control

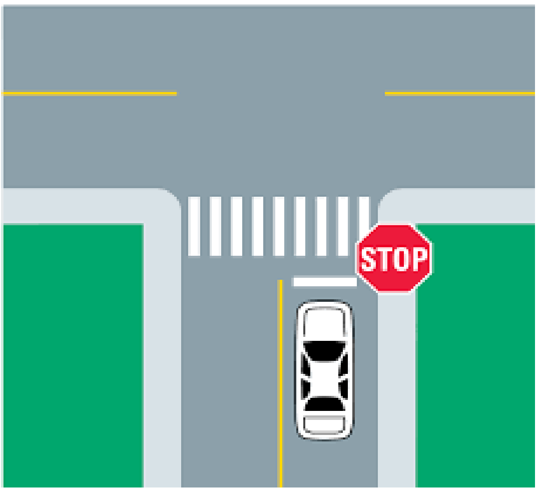
1) <u>Traffic Signs</u>

The **YIELD** or **GIVE WAY** sign indicates that merging drivers must prepare to stop if necessary to let a driver on another approach proceed.



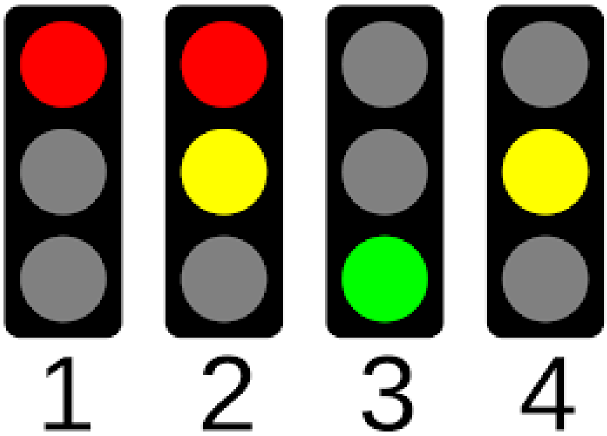
The **STOP** sign is designed to notify drivers that they must come to a complete stop and make sure the intersection is safely clear of vehicles and pedestrians.



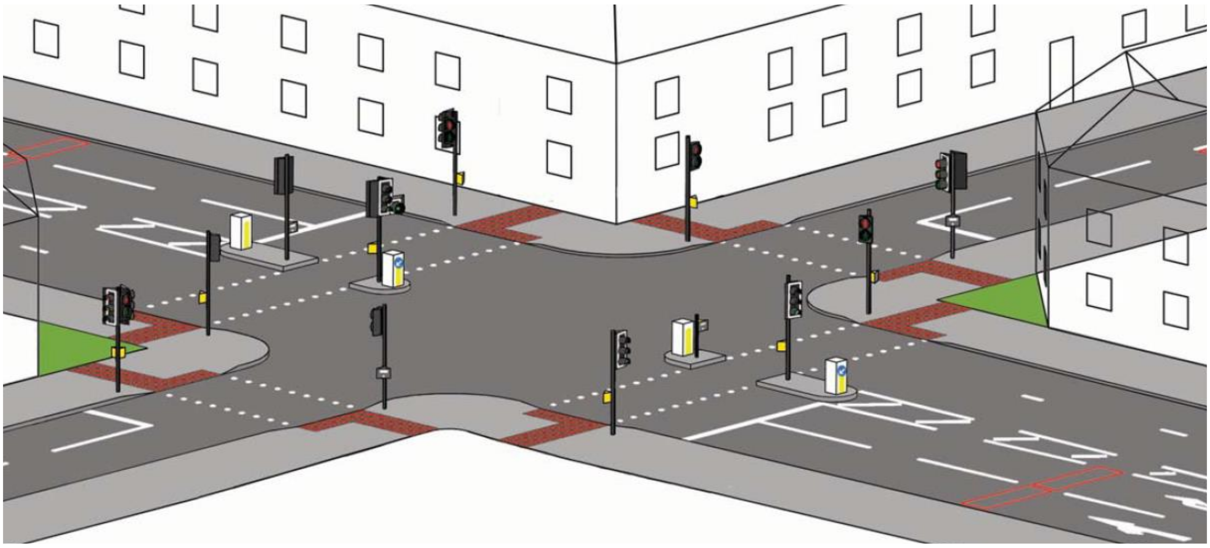


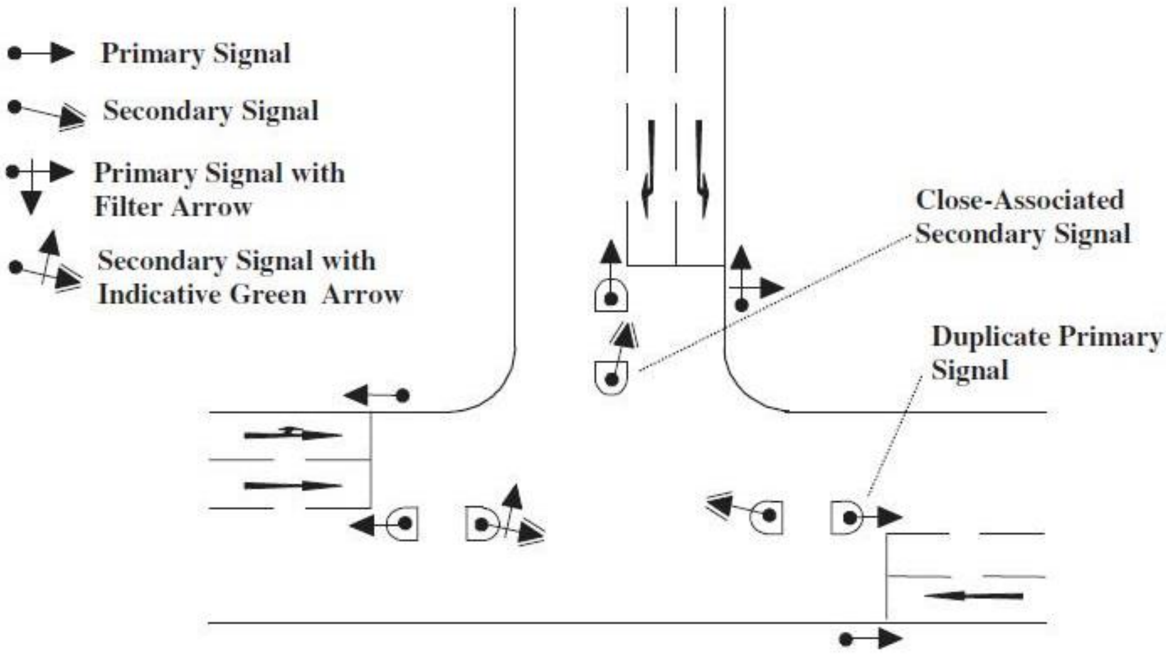
2) <u>Traffic light Signals</u>

A traffic light signal is designed to minimise the delays and control interaction between road users in the intersection area.



- **Red** interval applies for stopped condition including all-red period.
- **Red-Amber** interval or Ready-to-Go usually sets as 2 seconds.
- **Green** interval applies for traffic flow crossing the stopline.
- **Amber** interval which means Ready-to-Stop or Go if it is safe, usually sets as 3 seconds.



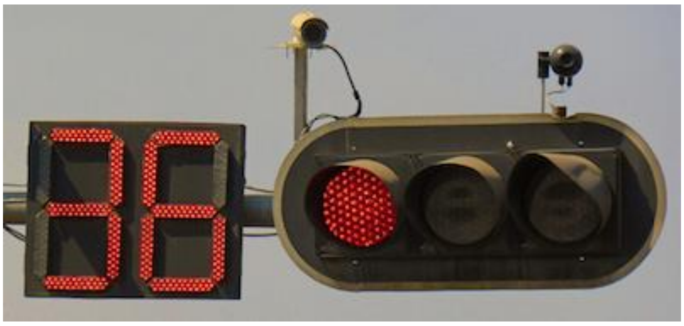


At least, 2 signals (one primary and one secondary) should be visible from each approach, and stopline. The distance between the stopline and secondary signal should not exceed 50 metres. Where separate signalling of turning movement is employed, this advice applies to approach lane(s) associated with each turning movement, and a signal post can then display information applicable to more than one turning movement.

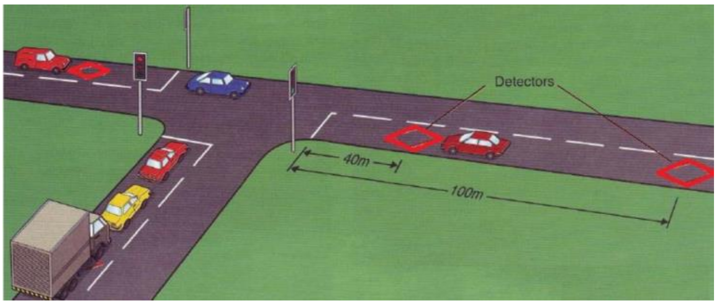
Types of Traffic Signals

The traffic light system can be operated by one of the following modes:

- **Fixed Time signals (FT):** are designed to provide a maximum cycle time length for a fixed time signal sequence and the length of effective green for an intersection.



- **Vehicle Actuated signals (VA):** are designed with the aid of vehicle detection method such as loop detectors. This technique provides an extension in the green time to reduce delay and increase intersection capacity.



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Lecture No.5

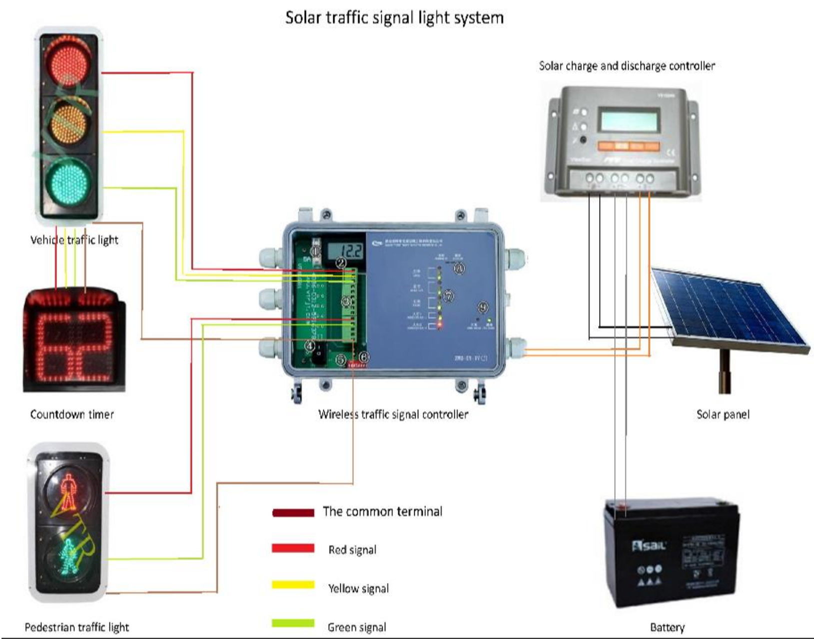
Signalized Intersections

- **A traffic light signal** is designed to minimize delays and control interaction between road users in the intersection area.
- **Factors considering in the installation of traffic light signals:**
  1) Vehicular and pedestrian flows.
  2) Number of approaches.
  3) Capacity of each approach.
  4) Design speed.
  5) Delay.
  6) Cost.
  7) Accidents records.

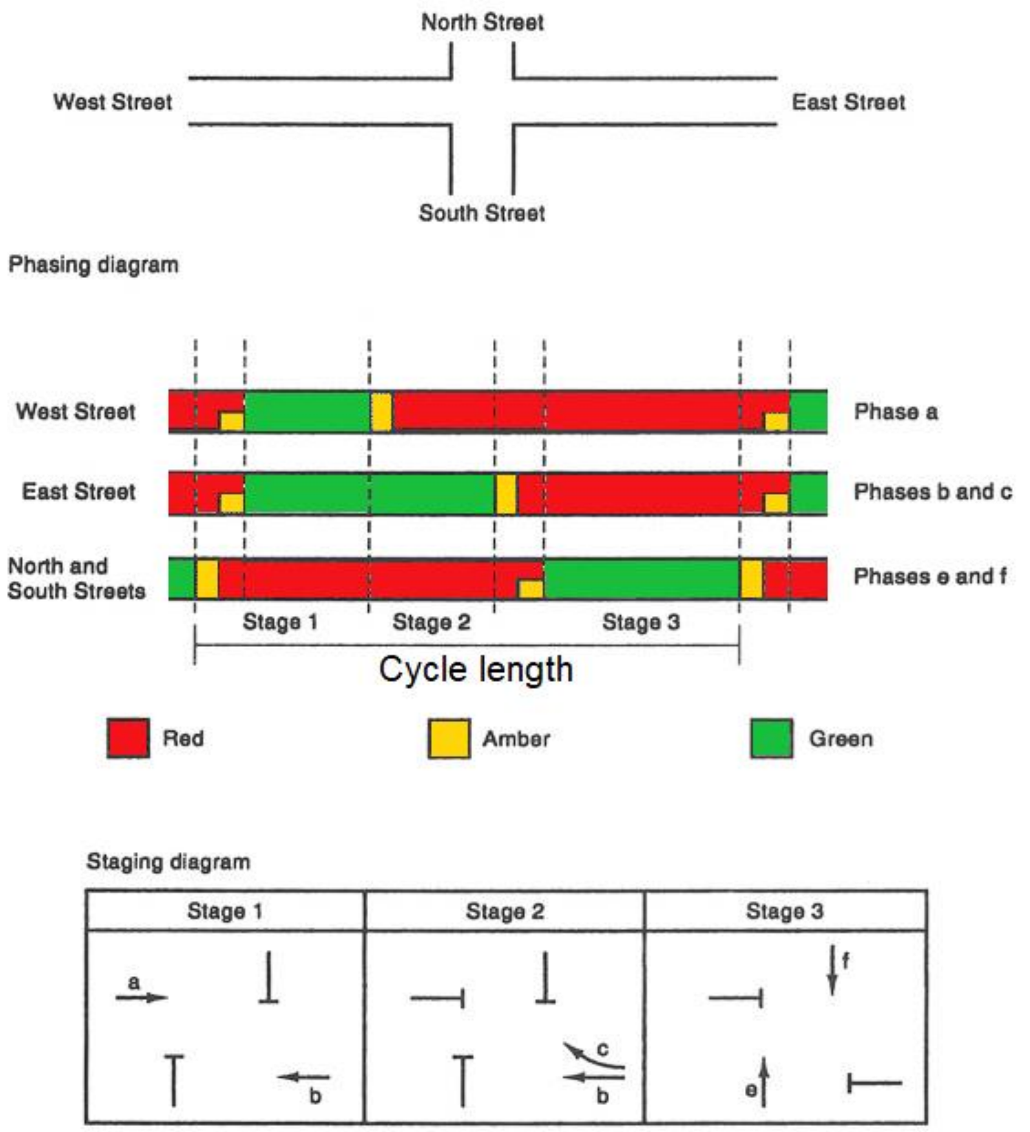
- Therefore, accidents and capacity perspectives are very important for traffic engineers to study the intersection area for urban design purposes.

The principles of traffic signals timing

- **Control system device:** it controls the changes of signal lamps colours. It gives the right-of-way to different approaches at certain times:



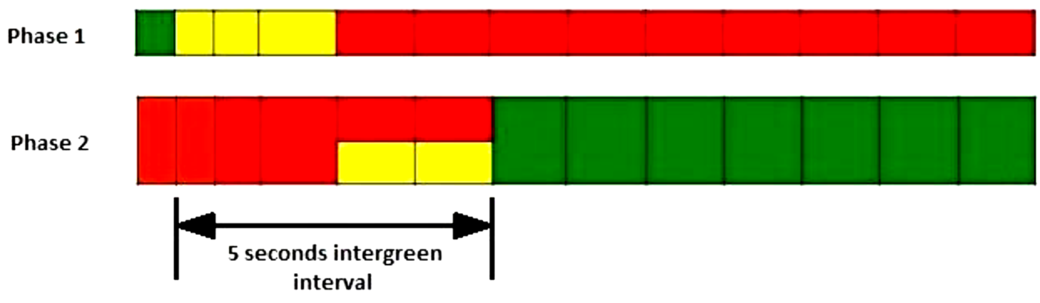
- **Cycle length:** a period in seconds that requires for completing a sequence of signal aspects.
- **Phase:** it is a part of cycle that assigns to a single traffic stream or more.
- **Stage:** describes that part of the cycle during which a certain set of phases receive green aspect.
- **Interval:** Any part within the cycle length that indicates a certain signal length such as red interval, amber interval or green interval.

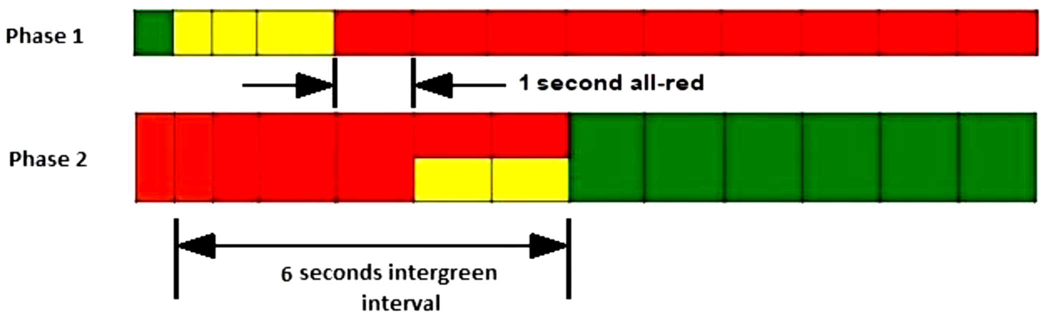


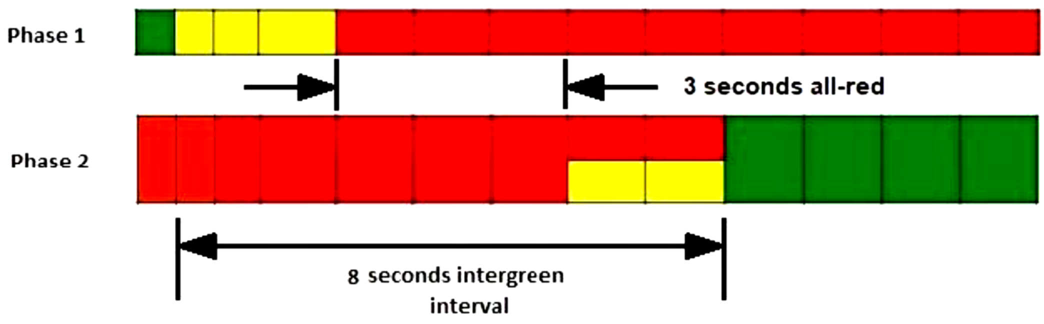
With full phasing specification (streams a to e)
Phasing is denoted by letters, staging by numbers

➢ **All-red interval:** it is displayed to all approaches. Used to ensure that the intersection area is clear from pedestrians and vehicles before the onset of green.

➢ **Intergreen interval:** it is also known as clearance period and is defined as the time between the end of the green time of a traffic flow and the beginning of the green time of another conflicting traffic.

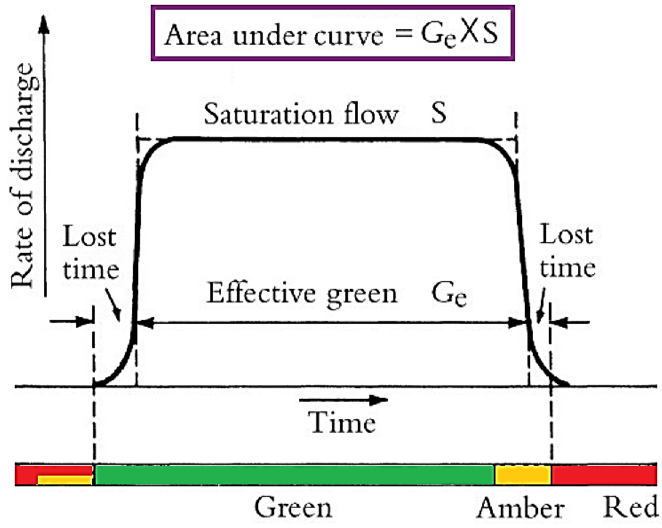




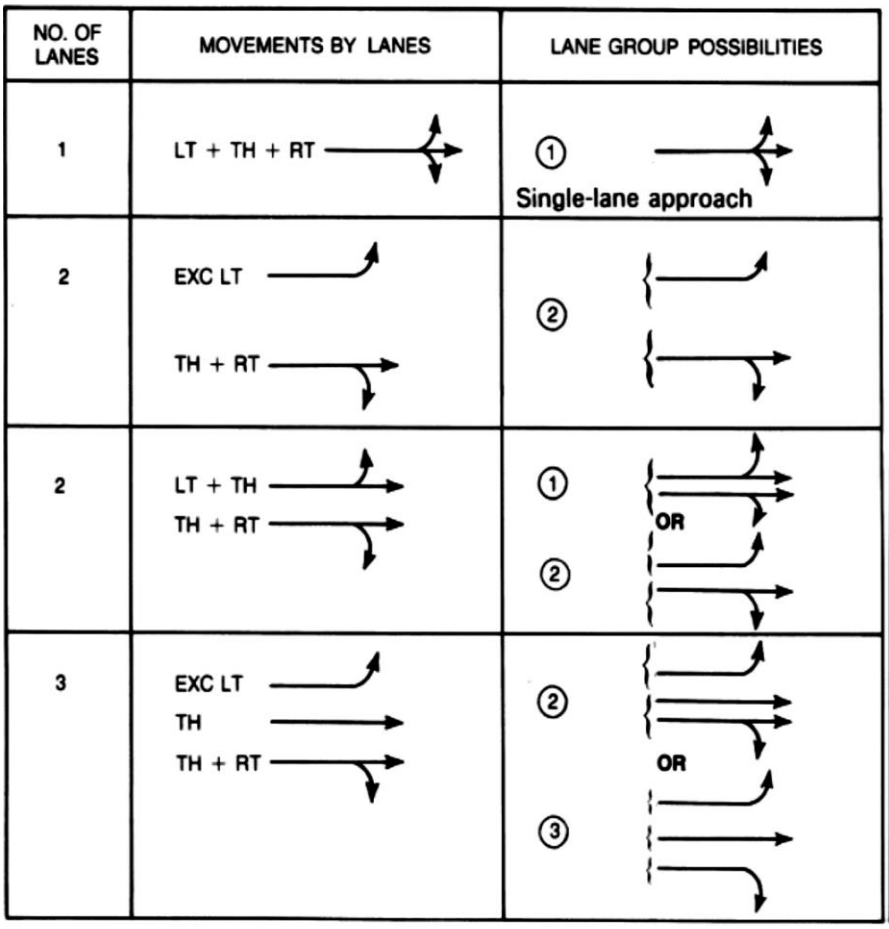


➢ **Lost time :** is the time that is spent by a driver of a first vehicle in the queue to start his movement when the signal turns from red to green.

➢ **Effective green:** is the time during which a given traffic movement or set of movements may proceed at saturation flow rate.



- **Saturation flow:** it is the maximum flow that can be discharged from a traffic lane when there is a continuous green and a continuous queue on the approach.
- **Lane group:** it consists of one or more lanes on an approach and giving the same green phase for its traffic stream.



NO. OF LANES	MOVEMENTS BY LANES	LANE GROUP POSSIBILITIES
1	LT + TH + RT	1 Single-lane approach
2	EXC LT TH + RT	2
2	LT + TH TH + RT	1 OR 2
3	EXC LT TH TH + RT	2 OR 3

Traffic signals timing

➢ <u>Step (1) Determination of Saturation Flow Rate</u>

Saturation flow represents the highest flow that can cross the stopline when there is a continuous green signal aspect and a continuous vehicles' queue on the approach. According to HCM 2010, Saturation flow rate for each lane group is obtained by **Webster's equation** as follows:

$$s = s_o\, f_w\, f_{HV}\, f_g\, f_p\, f_{bb}\, f_a\, f_{LU}\, f_{LT}\, f_{RT}\, f_{Lpb}\, f_{Rpb}$$

where

s = saturation flow rate for subject lane group, expressed as a total for all lanes in lane group (veh/h);

s_o = base saturation flow rate per lane (pc/h/ln); **Usually = 1900 pc/hr/ln**

Other correction factors or adjustment factors for correcting saturation flow rate can be found in Table below:

Factor	Formula	Definition of Variables	Notes
Lane width	$f_W = 0.96$ if $W < 10.0$ $f_W = 1.00$ if $W \geq 10.0 - 12.9$ $f_W = 1.04$ if $W > 12.9$	W = lane width (ft)	
Heavy vehicles	$f_{HV} = \dfrac{100}{100 + \%HV(E_T - 1)}$	% HV = % heavy vehicles for lane group volume	$E_T = 2.0$ pc/HV
Grade	$f_g = 1 - \dfrac{\%G}{200}$	% G = % grade on a lane group approach	$-6 \leq \%\ G \leq +10$ Negative is downhill
Parking	$f_p = \dfrac{N - 0.1 - \dfrac{18N_m}{3600}}{N}$	N = number of lanes in lane group N_m = number of parking maneuvers/h	$0 \leq N_m \leq 180$ $f_p \geq 0.050$ $f_p = 1.000$ for no parking
Bus blockage	$f_{bb} = \dfrac{N - \dfrac{14.4N_B}{3600}}{N}$	N = number of lanes in lane group N_B = number of buses stopping/h	$0 \leq N_B \leq 250$ $f_{bb} \geq 0.050$
Type of area	$f_a = 0.90$ in CBD $f_a = 1.00$ in all other areas		

Factor	Formula	Definition of Variables	Notes
Lane utilization	$f_{LU} = v_g/(v_{g1}N)$	v_g = unadjusted demand flow rate for the lane group, veh/h v_{g1} = unadjusted demand flow rate on the single lane in the lane group with the highest volume N = number of lanes in the lane group	
Left turns	Protected phasing: Exclusive lane: $f_{LT} = 0.95$ Shared lane: $f_{LT} = \frac{1}{1.0 + 0.05P_{LT}}$	P_{LT} = proportion of LTs in lane group	
Right turns	Exclusive lane: $f_{RT} = 0.85$ Shared lane: $f_{RT} = 1.0 - (0.15)P_{RT}$ Single lane: $f_{RT} = 1.0 - (0.135)P_{RT}$	P_{RT} = proportion of RTs in lane group	$f_{RT} \geq 0.050$
Pedestrian-bicycle blockage	LT adjustment: $f_{Lpb} = 1.0 - P_{LT}(1 - A_{pbT})(1 - P_{LTA})$ RT adjustment: $f_{Rpb} = 1.0 - P_{RT}(1 - A_{pbT})(1 - P_{RTA})$	P_{LT} = proportion of LTs in lane group A_{pbT} = permitted phase adjustment P_{LTA} = proportion of LT protected green over total LT green P_{RT} = proportion of RTs in lane group P_{RTA} = proportion of RT protected green over total RT green	

➢ Step (2) Calculation of Equivalent Hourly Flow

This can be estimated by dividing the peak hour volume by the peak hour factor. For example:

Time	*Volume*
6:00–6:15 p.m.	375
6:15–6:30 p.m.	380
6:30–6:45 p.m.	412
6:45–7:00 p.m.	390

$$q = \frac{Hourly\ volume\ during\ peak\ hour}{PHF}$$

$Peak\ flow\ rate\ of\ 15\ min = 412\ veh$

$Hourly\ volume\ during\ peak\ hour = 375 + 380 + 412 + 390 = 1557\ vph$

$$PHF = \frac{Hourly\ volume\ during\ peak\ hour}{4 * Peak\ flow\ rate\ of\ 15\ min} = \frac{1557}{4 * 412} = 0.945$$

Then;

$$q = \frac{Hourly\ volume\ during\ peak\ hour}{PHF} = \frac{1557}{0.945} = 1648\ vph$$

- **<u>Step (3) Calculation of the Optimum Cycle Time Length</u>**

Webster's equation can be applied to obtain the optimum cycle time length (***C_o***) as follows:

$$C_o = \frac{1.5\ L + 5}{1 - \ Y}$$

L: is the total lost time per cycle (sec)

L = phase lost time * number of phases or L = ∑ phases lost times

Y: is the sum of highest y of the approaches within the phase

Y = ∑ max(y) for each phase

y: is the ratio of equivalent flow to saturation flow (y=q/S)

- **<u>Step (4) Calculation of Green Time</u>**

Total Effective Green Time for <u>whole cycle time</u> $G_E = C_o - L$

Effective Green Time <u>for each phase</u> can be calculated as:

$$g_{e]\,phase} = G_E * \frac{y_{max}\ of\ this\ phase}{Y}$$

Actual Green Time <u>for each phase</u> can be calculated as:

$$g_{a]\,phase} = g_{e]\,phase} - amber + phase\ lost\ time$$

Actual Red Time for each phase can be calculated as:

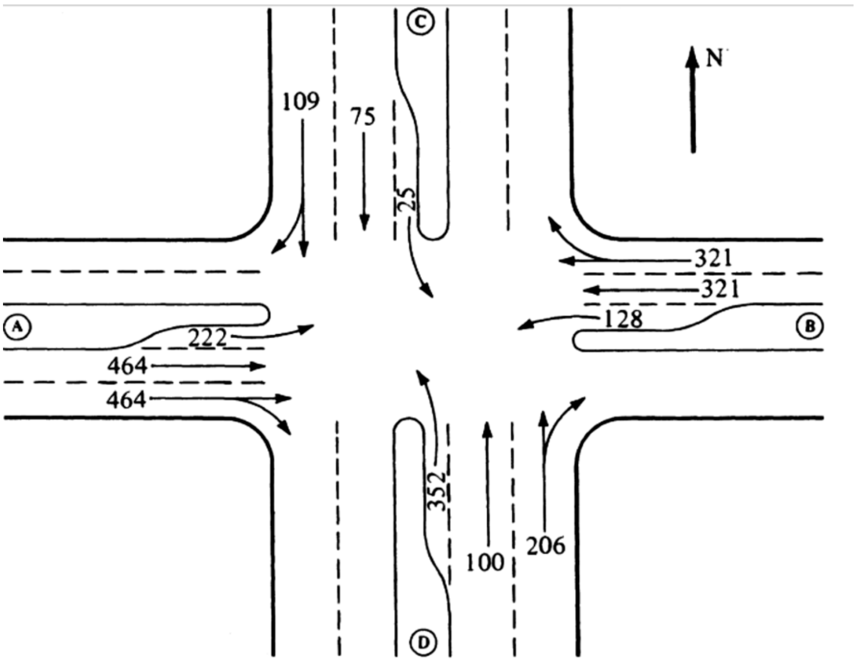
$$R_{a]\,phase} = C_o - g_{a]\,phase} - amber - Red_{amber} - All\,red\ (if\ exist)$$

The $g_{a]\,phase}$, $g_{e]\,phase}$ and $R_{a]\,phase}$ intervals will be apply to all lane groups on the approach. Therefore, the approaches (phases) will have different red and green intervals.

General Example:

The following figure shows peak-hour volumes for an intersection. Using the Webster method, determine a suitable signal timing for the intersection using the four-phase system shown below. Use amber interval of 3 seconds and the saturation flow given in the table.

Phase	*Lane Group*	*Saturation Flow*
A	①	1615
	②	3700
B	①	3700
	②	1615
C	①	1615
	②	3700
D	①	1615
	②	3700

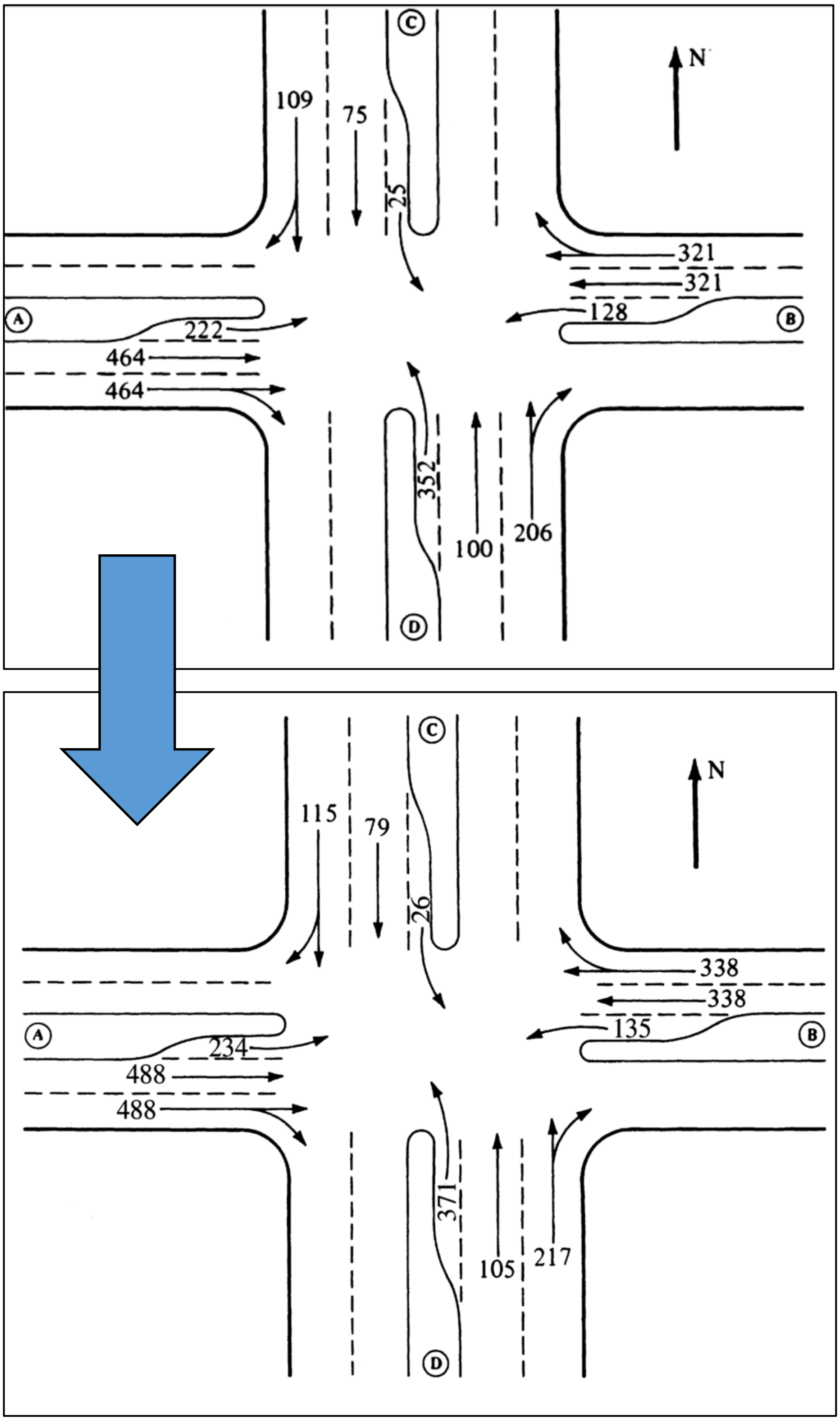


PHF = 0.95
Pedestrian volume is negligible.

Note: The influence of heavy vehicles and turning movements and all other factors that affect the saturation flow have already been considered.

Step (1) Saturation Flow Rates (S) are given in the example.

Step (2) Equivalent hourly flow (q) can be calculated by dividing the peak hour volume by the PHF = 0.95

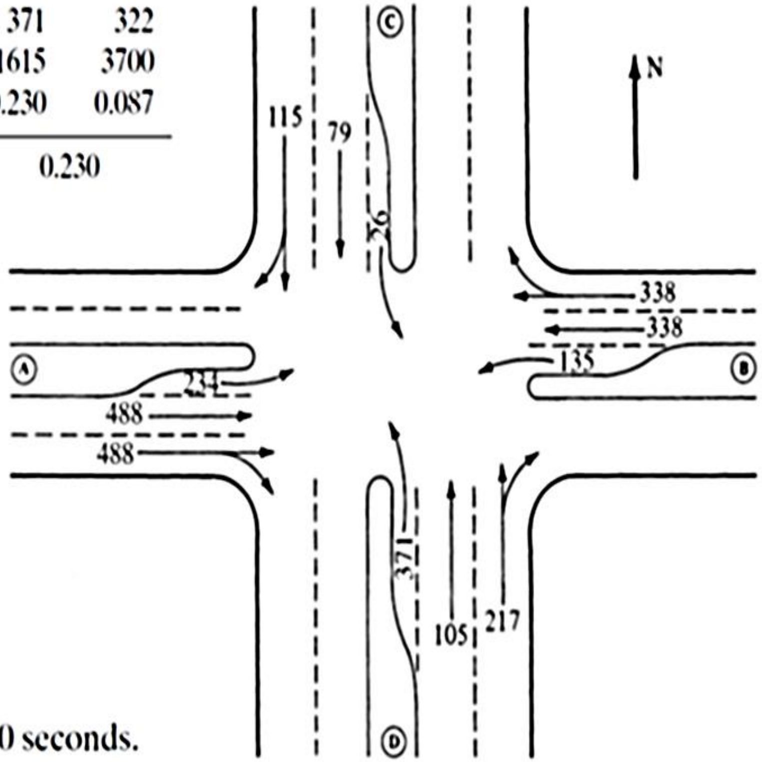


Step (3) Calculation of Cycle length:

Compute the total lost time. Since there is not an all-red phase that is, $R = 0$ and there are four phases.

$L = \sum \ell_i = 4 \times 3.5 = 14$ sec (assuming lost time per phase is 3.5 sec)

Lane Group	*Phase A (EB)* 1	2	*Phase B (WB)* 1	2	*Phase C (SB)* 1	2	*Phase D (NB)* 1	2
q_{ij}	234	976	135	676	26	194	371	322
S_j	1615	3700	1615	3700	1615	3700	1615	3700
q_{ij}/S_j	0.145	0.264	0.084	0.183	0.016	0.052	0.230	0.087
Y_i	0.264		0.183		0.052		0.230	



Determine Y_i and $\sum Y_i$.

$$\sum Y_1 = (0.264 + 0.183 + 0.052 + 0.230) = 0.729$$

Determine the optimum cycle length

$$C_o = \frac{1.5L + 5}{1 - \sum_{i=1}^{\phi} Y_i}$$

$$= \frac{(1.5 \times 14) + 5}{1 - 0.729} = 95.9 \text{ sec}$$

Use 100 seconds as cycle lengths are usually multiples of 5 or 10 seconds.

Step (4) Calculation of Effective and actual green intervals:

	Phase A (EB)		Phase B (WB)		Phase C (SB)		Phase D (NB)	
Lane Group	1	2	1	2	1	2	1	2
q_{ij}	234	976	135	676	26	194	371	322
S_j	1615	3700	1615	3700	1615	3700	1615	3700
q_{ij}/S_j	0.145	0.264	0.084	0.183	0.016	0.052	0.230	0.087
Y_i	0.264		0.183		0.052		0.230	

$$\Sigma Y_1 = (0.264 + 0.183 + 0.052 + 0.230) = 0.729$$

Find the total effective green time.

$$G_{te} = C - L = (100 - 14) = 86 \text{ sec}$$

Effective time for phase *i* is obtained as

$$G_{ei} = \frac{Y_i}{Y_1 + Y_2 + \ldots + Y_n} G_{te} = \frac{Y_i}{0.264 + 0.183 + 0.052 + 0.230} \times 86 = \frac{Y_i}{0.729} \times 86$$

Yellow time = 3.0 sec; the actual green time G_{ai} for each phase is obtained as

$$G_{ai} = G_{ei} + \ell_i - 3.0$$

Actual green time for Phase A

$$(G_{aA}) = \frac{0.264}{0.729} \times 86 + 3.5 - 3.0 \approx 32 \text{ sec}$$

Actual green time for Phase B

$$(G_{aB}) = \frac{0.183}{0.729} \times 86 + 3.5 - 3.0 \approx 22 \text{ sec}$$

Actual green time for Phase C

$$(G_{aC}) = \frac{0.052}{0.729} \times 86 + 3.5 - 3.0 \approx 7 \text{ sec}$$

Actual green time for Phase D

$$(G_{aD}) = \frac{0.23}{0.729} \times 86 + 3.5 - 3.0 \approx 27 \text{ sec}$$

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Traffic Engineering II

Dr. Noorance Al-Mukarm

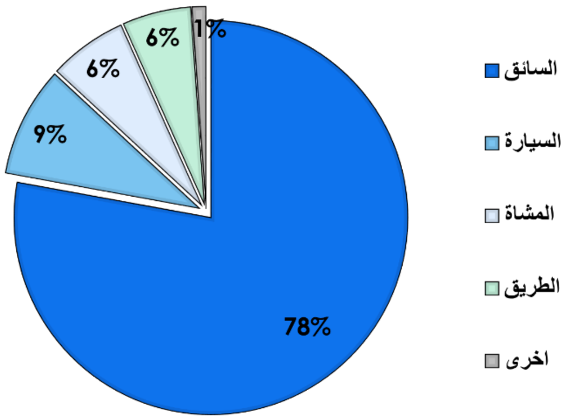
Lecture No.6

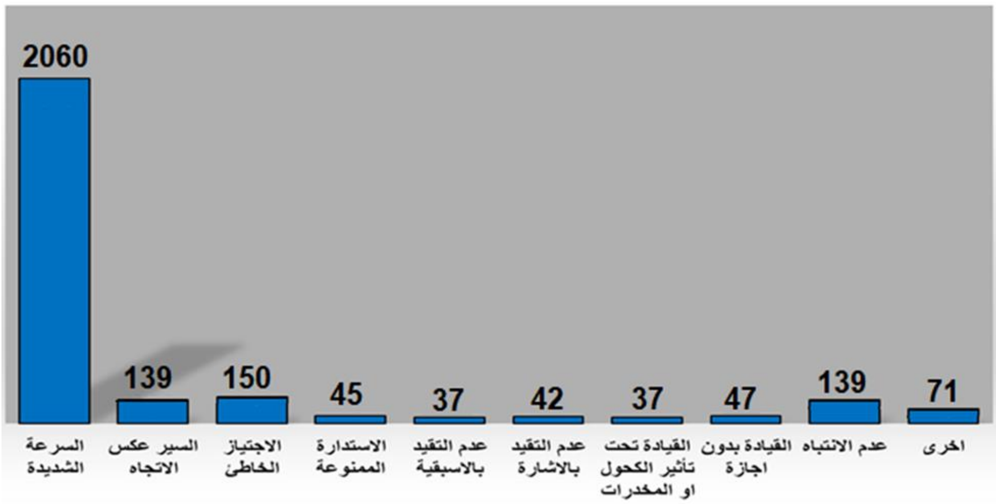
Traffic Accidents Data and Analysis

- **An Accident** is a rare, random, multi-factor event always preceded by a situation in which one or more road users have failed to cope with their environment.

- **Traffic accidents lead to:**
  1) Death.
  2) Disabilities from injuries.
  3) Property damage.

- **Traffic accidents are caused due to:**
  1) Road user behaviour.
  2) Vehicles.
  3) Road conditions.
  4) Road design.
  5) Environment factors.
  6) Other causes.





The Objectives of Accident Studies

- To study the causes of accidents and suggest corrective measures at potential location.
- To evaluate existing design.
- To compute the financial losses incurred.
- To support the proposed design and provide economic justification to the improvement suggested by the traffic engineer.
- To carry out before and after studies and to demonstrate the improvement in the problem.

Accidents Statistics

Accidents database provides a useful information from the available accidents records that reported by the police.

مديرية مرور (   )
قاطع مرور (   )

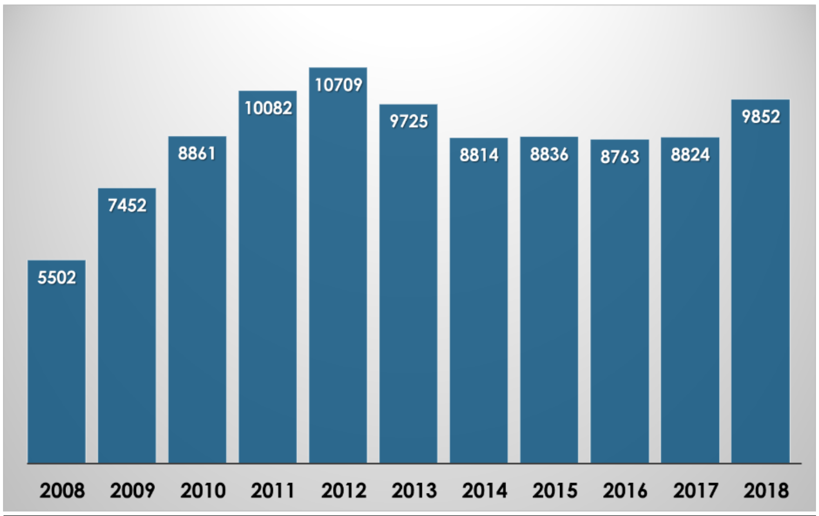
محضر كشف حادث مروري

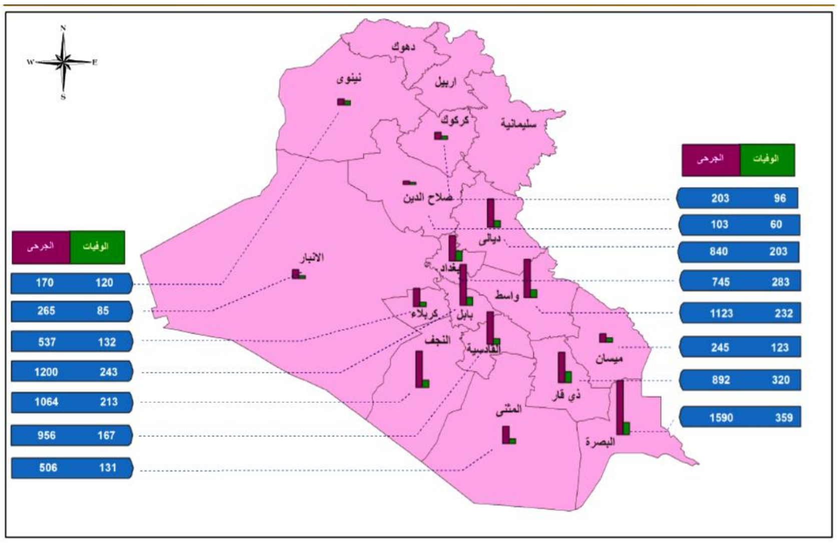
ساعة ويوم وتاريخ الحادث المروري					
مركز الشرطة التابع له الحادث					
نوع الحادث	دهس	اصطدام	انقلاب	اخرى:	
مكان الحادث	موقع وممر الحادث	نوع الطريق	حالة الطريق	حالة الجو	موقع الحادث
	داخل المدينة	مبلط	يابس	صحو	على خط العبور
	خارج المدينة	ترابي	مبلل	ممطر	خارج خط العبور
	ممر واحد	حصوي	مزحلق	ضباب	لايوجد خط عبور
	ممرين	سريع	مغطى بالثلج	غبار	
	اكثر من ممرين	نفق		غائم	
	ممر واحد متواجه			ثلوج	
	ممرين متواجهين				
ارقام المركبات ونوعها ولونها واسماء سائقيها	(1) رقم المركبة الاولى ونوعها ولونها: (2) رقم المركبة الثانية ونوعها ولونها: (3) رقم المركبة الثالثة ونوعها ولونها:			اسم سائقها: اسم سائقها: اسم سائقها:	
الاضرار بالاشخاص واسمائهم (وفيات او جرحى)	وفيات : (1)		(2)		(3)
	جرحى : (1)		(2)		(3)
مضبوطات الحادث					
اضرار المركبات الظاهرية					
اضرار الطريق وملحقاته					
اسباب وقوع الحادث والمقصرين					

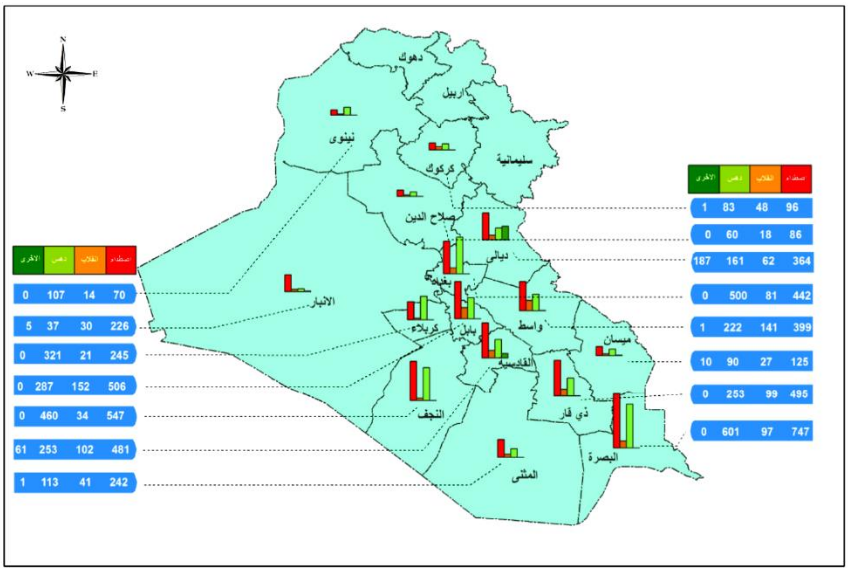
الملاحظات:
1- ضع علامة (✓) في الفراغ الموجود امام الحقل المناسب.
2- الاضرار بالمركبات: تحطم كلي، تحطم جزئي، اضرار بسيطة مع ذكر مكان الضرر.
3- الاضرار بالاشخاص: وفاة، جرح بليغ، جرح بسيط، مع ذكر الاسماء وهوياتهم.
4- اسباب الحادث: السرعة الشديدة، عدم الالتزام بالاشارة المرورية ،عدم الانتباه، رعونة، سكر، مرض، عدم توفر شروط المتانة في المركبة، اخرى تذكر.
5- مضبوطات الحادث وامتعة المصابين وغيرها تذكر.

It includes the following information: time and date of accident, location, type of accident, type of highway, weather condition, casualties' details, vehicle details, damages and description of the accident occurred.

Traffic Accidents in Iraq



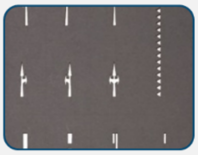




Analysis of Accidents Data

Driver and pedestrian
- Age and gender group
- Psychological test results

Vehicle
- Speed, size, defects
- Location, severity, property damage



Road environment
- Road works, lighting, signs, signals, layout
- Skidding, rutting, cracking, dry, wet, snow

Determination of Accidents rate:

- **Accident rate per kilometer:** is defined as the number of accidents of all types per km of each highway and street classification.

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

where, R = total accident rate per km for one year, A = total number of accident occurring in one year, L = length of control section in kms

- **Accident involvement rate:** is defined as the numbers of drivers of vehicles with certain characteristics who were involved in accidents per 100 million vehicle-kms of travel.

$$R = \frac{N \times 100000000}{V}$$

where, R = accident involvement per 100 million vehicle-kms of travel, N = total number of drivers of vehicles involved in accidents during the period of investigation and V = vehicle-kms of travel on road section during the period of investigation

- **Death rate based on population:** is defined as the number of traffic fatalities per 100,000 populations. This rate reflects the accident exposure for entire area.

$$R = \frac{B \times 100000}{P}$$

where, R = death rate per 100,000 population, B = total number of traffic death in one year and P = population of area

- **Death rate based on registration:** is defined as the number of traffic fatalities per 10,000 vehicles registered.

$$R = \frac{B \times 10000}{M}$$

where, R = death rate per 10,000 vehicles registered, B = total number of traffic death in one year and M = number of motor vehicles registered in the area

- **Accident Rate based on vehicle-kms of travel:** is defined as the number of accidents per 100 million vehicle km of travel. The true exposure to accident is nearly approximated by the miles of travel of the motor vehicle than the population or registration.

$$R = \frac{C \times 100000000}{V}$$

where, R = accident rate per 100 million vehicle kms of travel, C = number of total accidents in one year and V = vehicle kms of travel in one year

Example

The Motor vehicle consumption in a city is 5.082 million liters, there were 3114 motor vehicle fatalities, 355,799 motor vehicle injuries, 6,721,049 motor vehicle registrations and an estimated population of 18,190,238. Kilometer of travel per liter of fuel is 12.42 km/liter. Calculate registration death rate, population death rate and accident rate per vehicle km.

Solution

Solution Approximate vehicle kms of travel = Total consumption o fuel $\times$ kilometer of travel per liter of fuel $=5.08 \times 10^9 \times 12.42 = 63.1 \times 10^9$ km.

1. Registration death rate can be obtained from the equation

$$R = \frac{B \times 10,000}{M}$$

Here, R is the death rate per 10,000 vehicles registered, B (Motor vehicle fatalities) is 3114, M (Motor vehicle registered) is 6.72×10^6. Hence,

$$R = \frac{3114 \times 10000}{6.72 \times 10^6} = 4.63$$

2. Population Death Rate can be obtained from the equation.

$$R = \frac{B \times 100,000}{P}$$

Here, R is the death rate per 100,000 population, B (Motor vehicle fatalities) is 3114, P (Estimated population) is$= 18.2 \times 10^6$.

$$R = \frac{3114 \times 100000}{18.2 \times 10^6} = 17.1$$

3. Accident rate per vehicle kms of travel can be obtained from the equation below as:

$$R = \frac{C \times 100,000,000}{V}$$

Here, R is the accident rate per 100 million vehicle kms of travel, C (total accident same as vehicle fatalities) is 3114, V (vehicle kms of travel) is 63.1×10^9.

$$R = \frac{3114 \times 100 \times 10^6}{63.1 \times 10^9} = 4.93$$

Safety Measures (EEE)

Engineering measures:

Providing routine maintenance to the road and installed signals in order to ensure that they remain working consistently within the design specifications.

Enforcement measures:

Developing safety equipment's to prevent road traffic accidents and reduce the number of injuries in future. For example, speed control and red light camera.

Education measures:

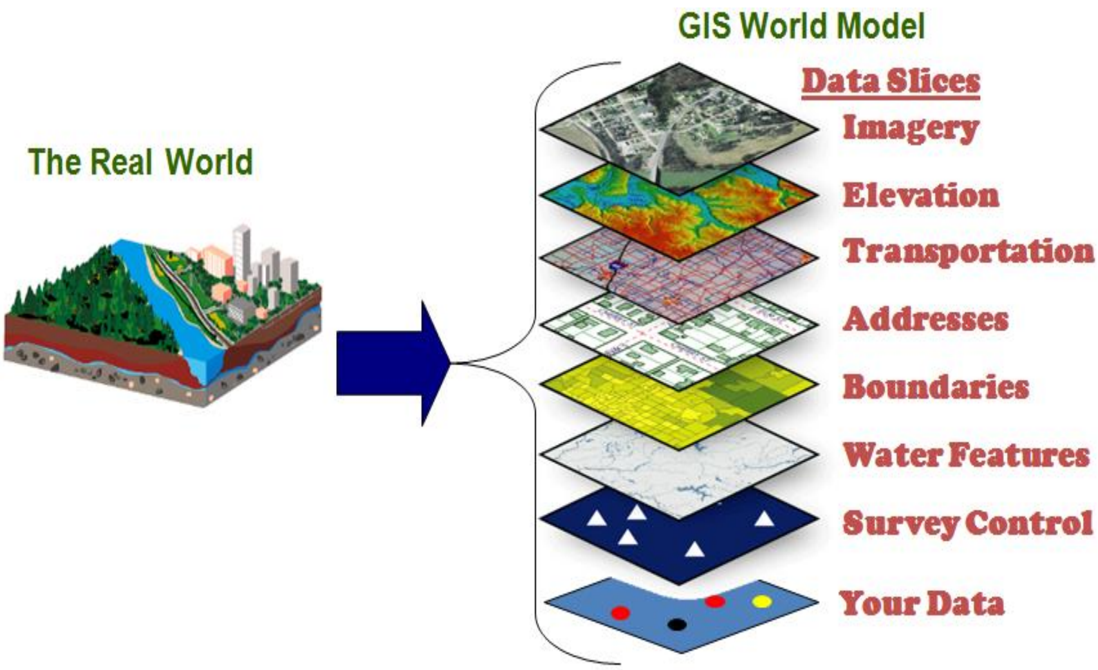
Improving drivers, pedestrian, passengers and children skills by a series of educational methods during training courses and workshops.

Intelligent Transport Systems (ITS)

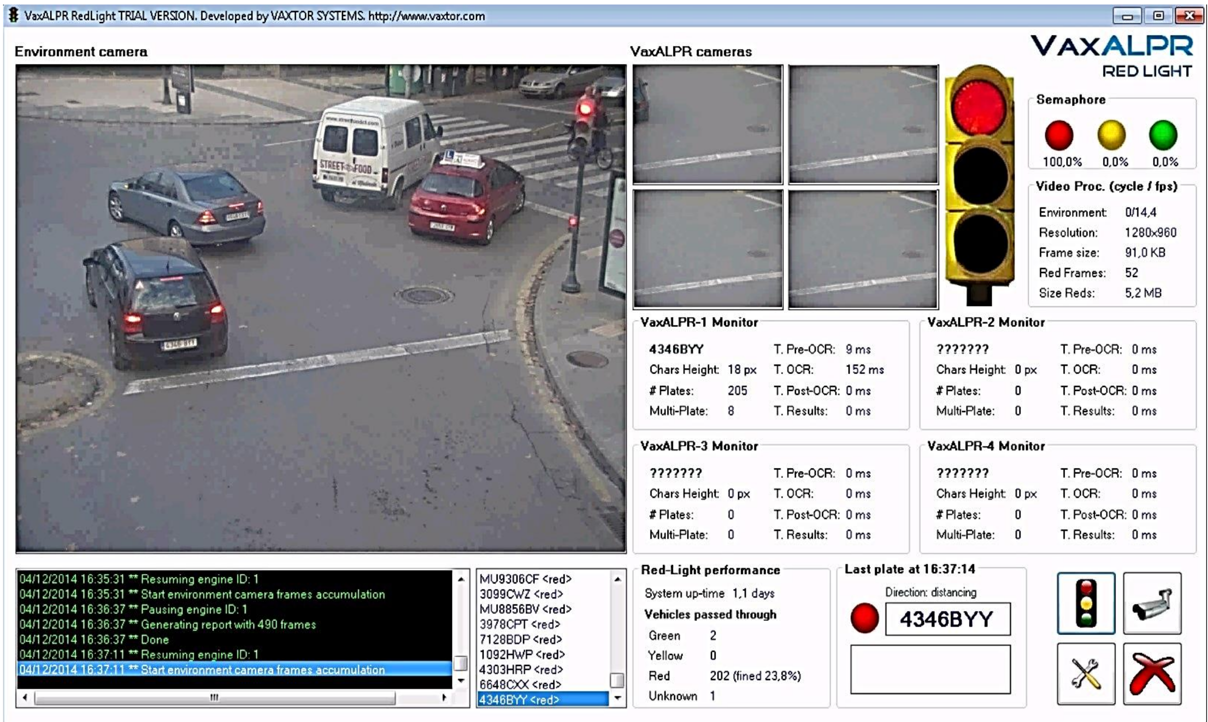
- These technological systems are useful for transport planning and providing information and communication technology.
- They can be used all together to manage transport operations across all modes.
- ITS brings value added features to the existing infrastructure.
- Examples of ITS:
  - Traffic control
  - Vehicle warning system
  - Route and journey information
  - Booking and consignment processes

Data Capture → **Processing** → **Transmission** → **Reception**

Examples of ITS



GIS: Geographic Information System



Red light camera system

Examples of ITS

Glowing path for pedestrians and cyclists

Oyster Card

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Lecture No.7

<u>Parking Area and Survey</u>

➢ **Parking** is a space or area where vehicles can be left.

➢ **Parking is designed for the following points:**

1) To minimise the effects of on street parking upon road safety and congestion.
2) To help maintain the vitality of town centres by reducing dependence on the car, particularly in town centres.
3) To implement innovative methods of parking management such as: car washing, electric charging equipment, electronic pay on site ticket machines … etc.
4) To ensure that car parking provision and enforcement processes.
5) To review and improve where necessary, over a 5 year period, the physical condition of all Council car parks to ensure they are fit for purpose and meet the expectations of the users.

➢ **Parking signs:**

Parking area marking

- Design a recognized spaces for car parking and people with disabilities.
- Number each parking space.
- Separate each space for vehicle safety and providing pedestrian walkway.
- Separate car parking spaces on the road to avoid reducing capacity.

Several requirements should be taken into consideration in designing and planning for a car park

- The drivers
- Shopkeepers
- Public transport operators
- Commercial vehicles drivers
- Through traffic
- Car park operators
- Traffic engineers

Types of Parking

There are two types: On-street and Off-street.

On-street

Off-street

On-street parking is an area occupied by vehicles alongside or parallel to the road kerb.

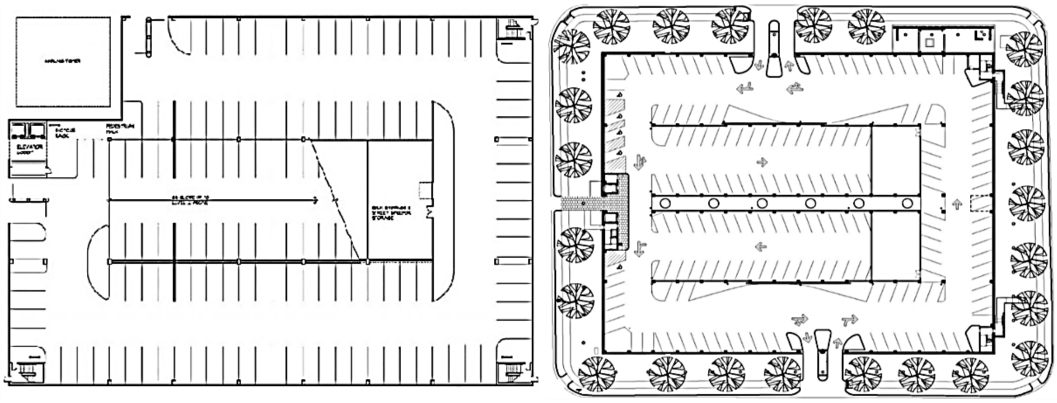
- Selection of street with no restriction on parking.
- Selection of street with parking prohibited at a certain time.
- Officially authorized parking spaces controlled by a form within a specific time.
- Officially authorized parking spaces controlled by residents permit and other categories.

Off-street parking is an area occupied by vehicles and available as either a surface car park or a multi-store building.

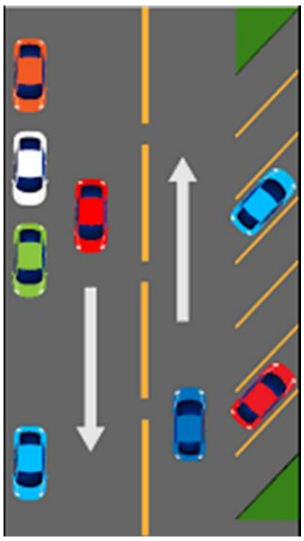
- ➢ Parking spaces available to all people: free or on payment, designated for park and ride, public or private owned.
- ➢ Parking spaces available to a specific people: hotel, hospital, museum, mall, airport, university… etc.

Parking space

It can be defined as the area or bay occupied by one vehicle. So, each parked vehicle has a space or bay. Two types of parking space can be shown in Figure below:



Parallel   Angle



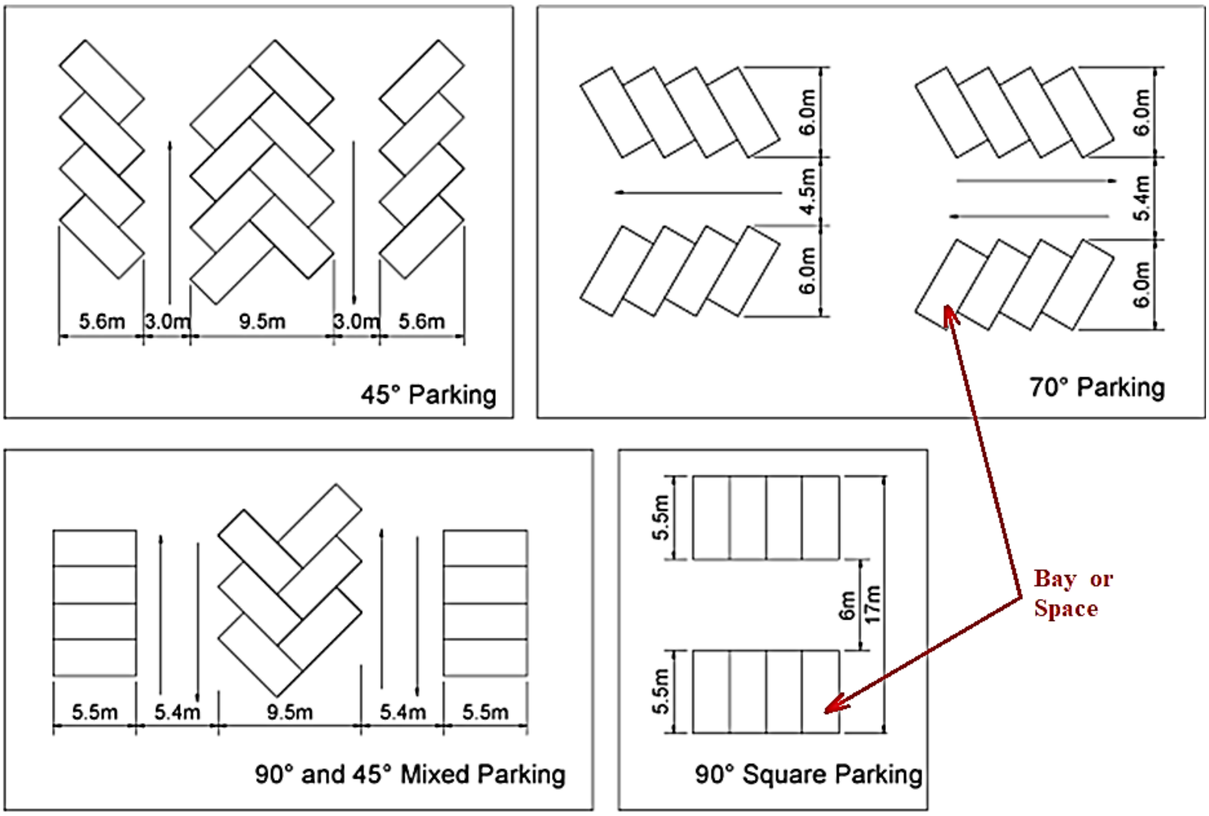
The advantage of angle parking are:

1) More convenient for motorists and drivers.
2) More parked cars in the same space length.

The disadvantage of angle parking are:

1) Cause higher accident rate.
2) Reduce the road capacity.

Layout of parking areas



Examples of typical parking layouts

Parking Survey

- **Parking accumulation**: it is defined as the number of vehicles parked at a given instant of time. Normally this is expressed by accumulation curve. Accumulation curve is the graph obtained by plotting the number of bays or spaces occupied with respect to time.

- **Parking index**: it is also called **occupancy** or **efficiency**. It is defined as the ratio of number of bays occupied in a time duration to the total space available.

- **Parking load**: it can be obtained by simply multiplying the number of vehicles occupying the parking area at each time interval with the time interval. It is expressed as vehicle hours.

<u>Numerical Example</u>

From an in-out survey conducted for a parking area consisting of <u>40</u> bays, the initial count was found to be <u>25</u> . The number of vehicle coming in and out of the parking lot for a time interval of 5 minutes is as shown in the table 1. Find the accumulation, total parking load, average occupancy and efficiency of the parking lot.

In-out parking survey

Time (min)	In	Out	Accumulation (parked vehicles)	Occupancy %	Parking load (veh.hrs)
5	3	2	26	65	130
10	2	4	24	60	120
15	4	2	26	65	130
20	5	4	27	67.5	135
25	7	3	31	77.5	155
30	8	2	37	92.5	185
35	2	7	32	80	160
40	4	2	34	85	170
45	6	4	36	90	180
50	4	1	39	97.5	195
55	3	3	39	97.5	195
60	2	5	36	90	180
Total					1935

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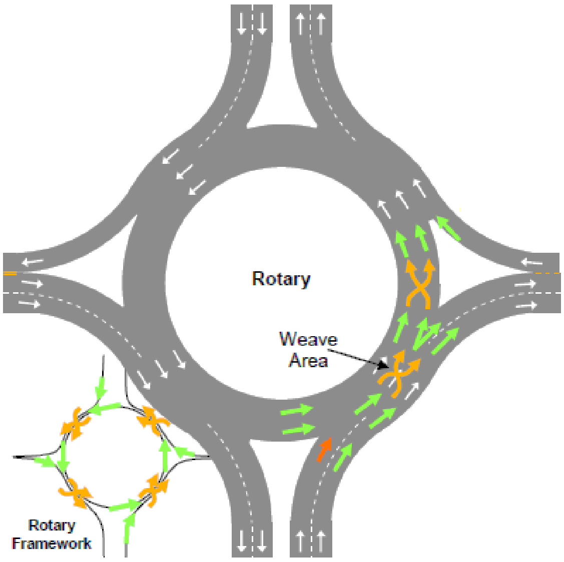
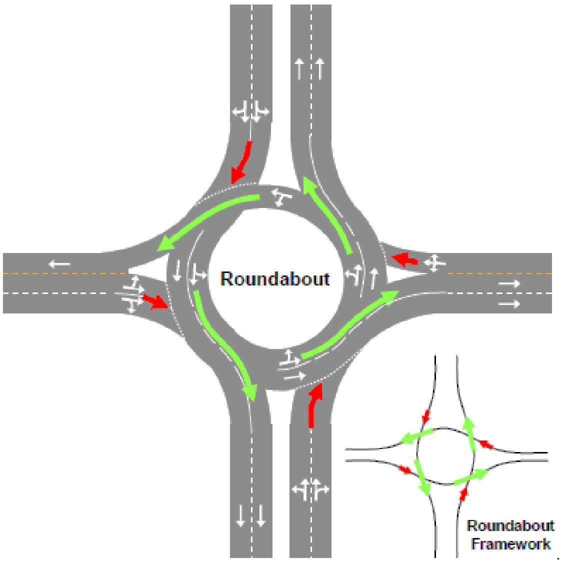
Lecture No.8

Roundabouts and Rotaries

- **Roundabout or rotary intersection** is a special form of at-grade intersections laid out for the movement of traffic in one direction round a central traffic island.
- **It is designed for the following points:**

1) To reduce the number of conflicts as much as possible.
2) To force the traffic to move in anti-clockwise direction and then weave out of the rotary to the desired direction.





Rotary	Modern Roundabout
It is typical to enter a rotary alongside traffic that is circulating in the inside lanes, like a freeway cloverleaf loop entrance where the ramp entrance lane continues under or over a bridge to the next exit.	Entering traffic must always yield to ALL traffic in the roundabout, regardless of which lane they are in, just like crossing a one-way road.
No intersections occur in a rotary, only adding and dropping of lanes. The right lane usually does not need to yield, but must find a gap to change lanes. The left entry lane must merge or yield before entering.	A roundabout is a series of "crossing intersections" where traffic entering the roundabout must **yield** the right of way to all traffic from the left.
The circle is usually not striped, though multiple vehicles may travel side by side. Lane changes occur after you have entered the circle.	The circle is striped as a spiral. Never change lanes in a roundabout. Choose your lane before entering, just like at a standard intersection.
Entering drivers who wish to circulate must change lanes while circulating and weave with vehicles trying to exit.	No lane changes occur within a roundabout. Except for vehicles that are turning right, entering a roundabout is a "crossing" movement.
A rotary is typically large, with entry speeds of 40 mph or higher.	A roundabout is generally small; speeds are rarely more than 25 mph.
Rotaries work well at low volumes, but very poorly under heavy traffic conditions. Most were designed in the 1940's or earlier.	Roundabouts are able to handle heavy traffic and are used for efficiency and safety. Roundabouts were developed in the 1960's.
Entry may be controlled by yield signs, merge signs, or no signs at all.	Entry is always controlled by yield signs for maximum efficiency.

➢ Advantages:

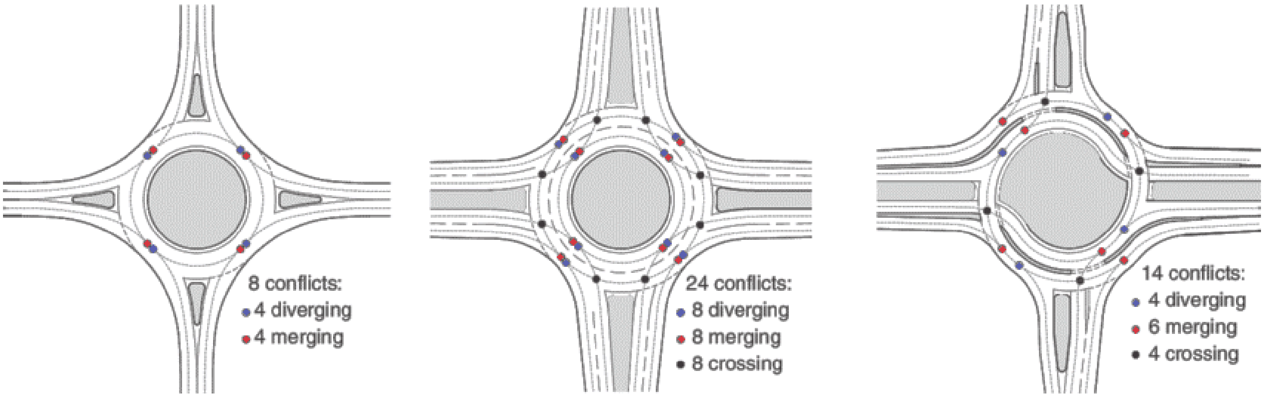
1) Traffic flow is regulated to only one direction of movement, thus eliminating severe conflicts between crossing movements.
2) All the vehicles entering are gently forced to reduce the speed and continue to move at slower speed. Thus, more of the vehicles need to be stopped.
3) Because of lower speed of negotiation and elimination of severe conflicts, accidents and their severity are much less than those occur in intersections.
4) They are self-governing and do not need practically any control by police or traffic signals.
5) They are ideally suited for moderate traffic, especially with irregular geometry, or intersections with more than three or four approaches.

➢ Disadvantages:

1) All the vehicles are forced to slow down and negotiate the intersection. Therefore the cumulative delay will be much higher than channelized intersection.
2) Even when there is relatively low traffic, the vehicles are forced to reduce their speed.
3) They require an area of relatively at land making them costly at urban areas.
4) Since the vehicles are not stopping, and the vehicles accelerate at the exits, they are not suitable when there is high pedestrian movements.

➢ Conflict Points

1) **Diverging:** It is a traffic operation when the vehicles moving in one direction is separated into different streams according to their destinations.
2) **Merging:** It is the opposite of diverging, when traffic streams coming from various places and going to a common destination are joined together into a single stream it is referred to as merging.
3) **Weaving:** It is the combined movement of both the merging and diverging movements in the same direction.
4) **Crossing:** It is the crossing of different streams resulting in a conflict.



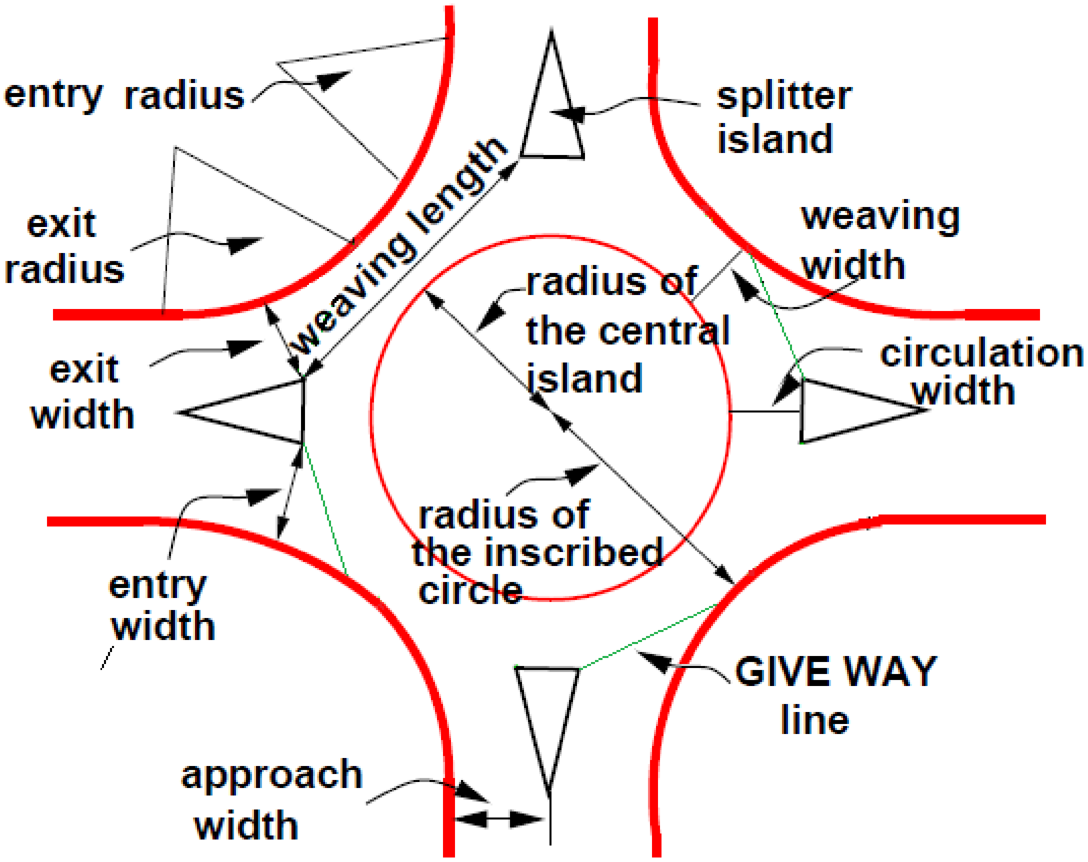
➢ Design Considerations

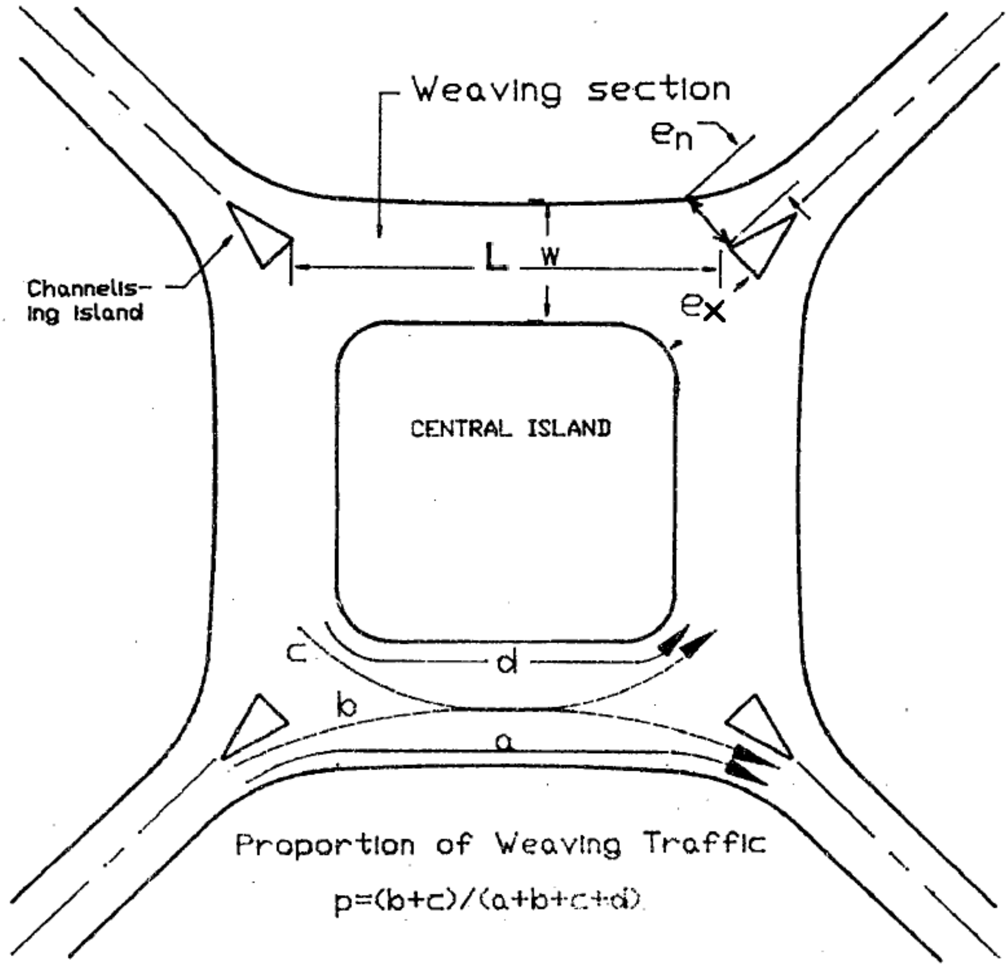
1) Traffic entering from all the four approaches are relatively equal.

2) Maximum capacity is around (3000 vph) and minimum capacity is approximately (500 vph).

3) It is beneficial when the left-turn percentage of traffic is very high; typically it is suitable for more than 30%.

4) It is beneficial when there are more than four approaches, or if there is no suitable lanes available for left-turn traffic.





▶ $\boldsymbol{Width\ of\ weaving\ section\ W_w = e + 3.5}$

▶ $\boldsymbol{Capcity = \dfrac{280 \times W_w \times \left[1 + \frac{e}{W_w}\right] \times \left[1 - \frac{p}{3}\right]}{\left[1 + \frac{W_w}{L}\right]}}$

▶ $\boldsymbol{e = \frac{en+ex}{2}}$, $\boldsymbol{p = \frac{b+c}{a+b+c+d}}$, $\boldsymbol{L = 4 \times W_w}$

en and ex represent entery and exit width, respectively.

b and c represent the weaving traffic movements.

a and d represent the non − weaving traffic movements.

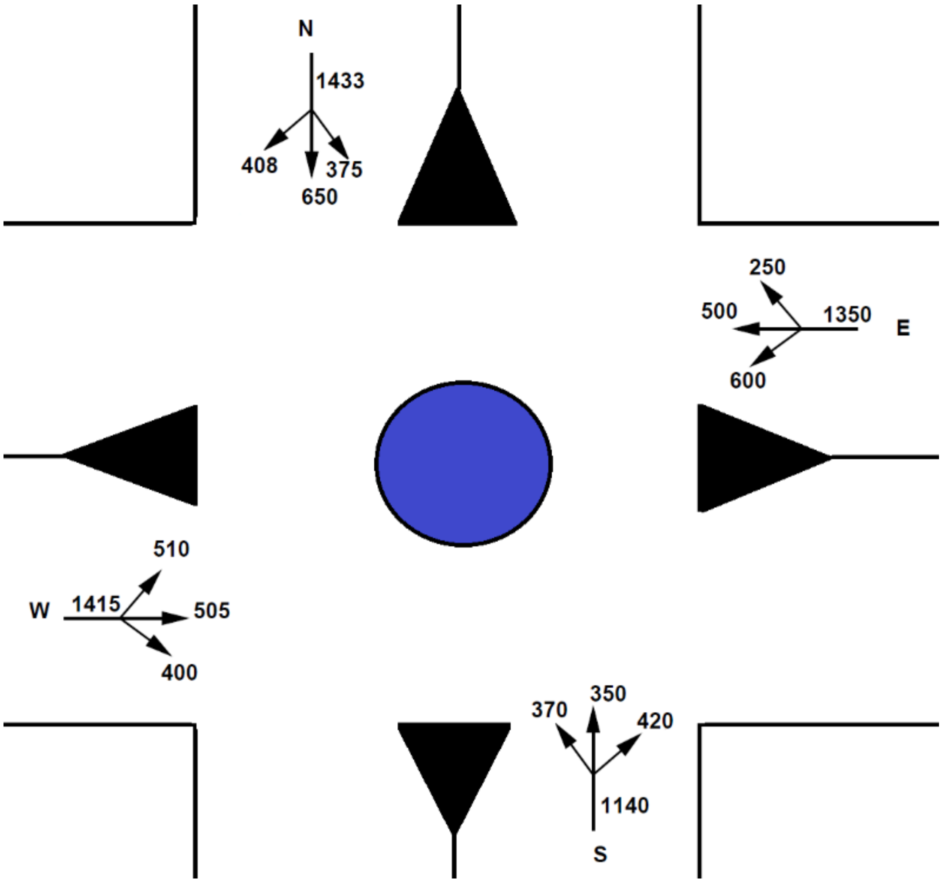
p represents the max proportion weaving traffic to non − weaving traffic.

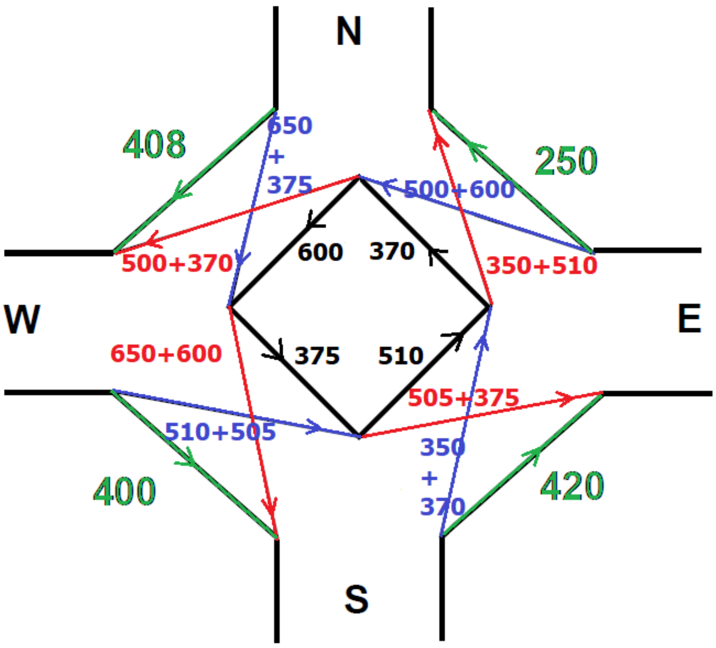
L represents the length of weaving section.

W_w represents the width of weaving section.

Numerical example:

The width of a carriageway approaching an intersection is given as 17 m. The entry and exit width at the rotary is 10 m. The traffic approaching the intersection from the four sides is shown in the Figure below. Find the capacity of the rotary using the given data.





- $Width\ of\ weaving\ section\ W_w = \frac{10+10}{2} + 3.5 = 13.5\ m$
- $Weaving\ length\ L = 4 \times W_w = 54\ m$
- $Determine\ the\ proportion\ of\ weaving\ traffic\ to\ non-weaving\ traffic\ as\ follows$:

$$p_{EN} = \frac{500+600+350+510}{500+600+350+510+250+370} = 0.76$$

$$p_{NW} = \frac{650+375+500+370}{650+375+500+370+408+600} = 0.65$$

$$p_{WS} = \frac{510+505+650+600}{510+505+650+600+400+375} = 0.75$$

$$p_{SE} = \frac{350+370+505+375}{350+370+505+375+420+510} = 0.63$$

- $Min\ Capacity = \frac{280 \times 13.5 \times \left[1+\frac{10}{13.5}\right] \times \left[1-\frac{0.76}{3}\right]}{\left[1+\frac{13.5}{54}\right]} = 3929\ vph$

- $Max\ Capacity = \frac{280 \times 13.5 \times \left[1+\frac{10}{13.5}\right] \times \left[1-\frac{0.63}{3}\right]}{\left[1+\frac{13.5}{54}\right]} = 4157\ vph$

Ministry of Higher Education
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Al-Muthanna University
Civil Engineering Department

Traffic Engineering II

Dr. Noorance Al-Mukarm

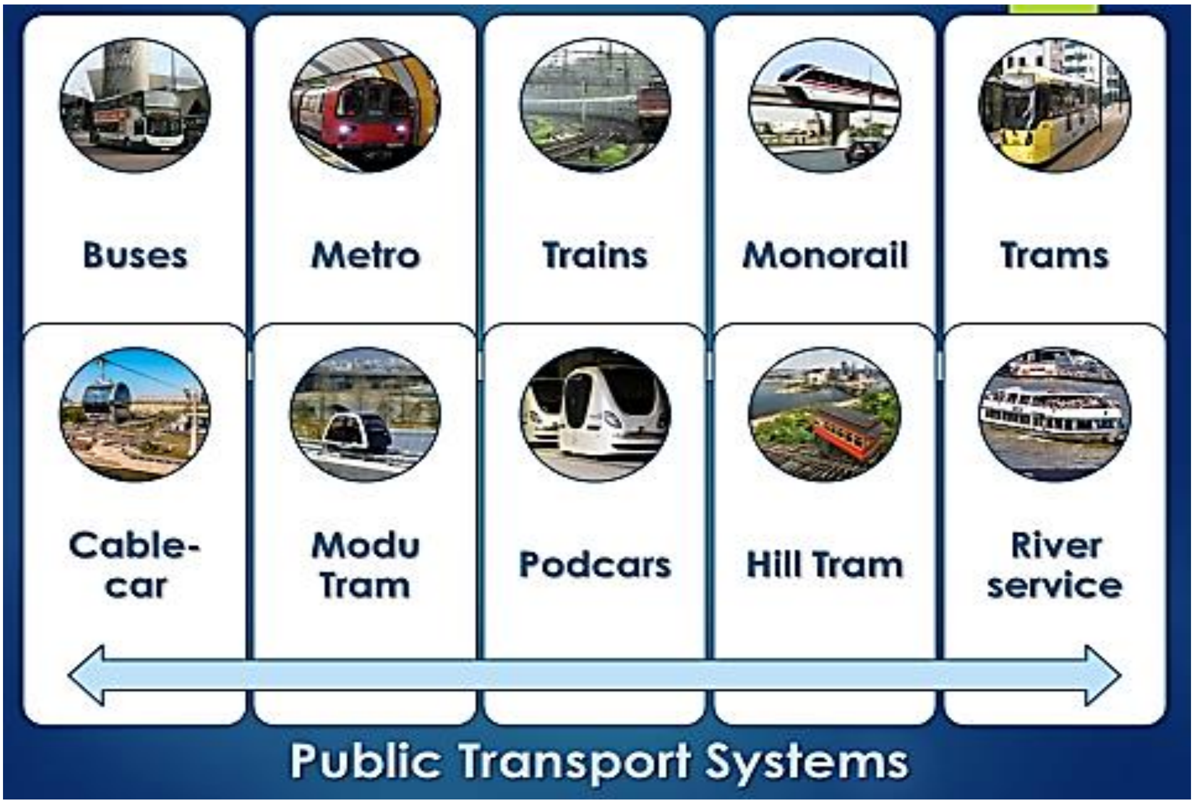
Lecture No.9

Public Transport

- **Public Transport** is a system of transport for passengers by group travel systems available for use by the general public. Typically managed on a schedule, operated on established routes, and that charge a posted fee for each trip.
- **Objectives:**

1) Operational:
   - Maximize efficiency of vehicle and crew time
   - Maximize reliability and regularity of services
   - Ensure control of services
   - Ensure efficiency of ‘layover’ or non-service time
   - Maximize patronage
2) Passengers:
   - Create easy access to stops and services.
   - Minimize car dependency and interchange penalties
   - Reduce journey times (competitiveness versus car)
   - Ensure regularity and reliability

Different Modes of Public Transport



A number of considerations should be taken into account:

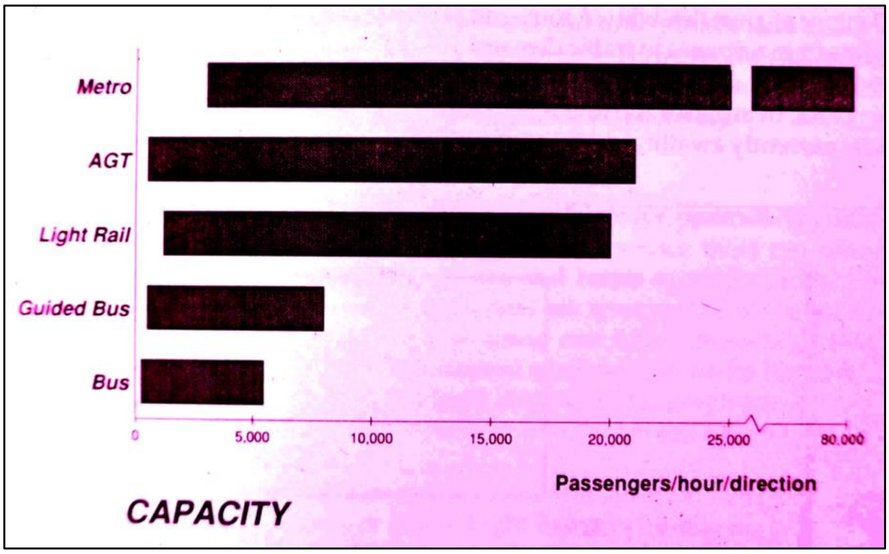
- Demand patterns
- Land use patterns
- Trips generation
- Future developments (residential, retail, commercial, industrial, education, health, recreation)
- Existing transport facilities
- Performance of existing system
- Accessibility for mobility impaired
- Modal split (walk, cycle, public/private, bus/rail)
- Existing problems (e.g. congestion, environmental impacts, safety)
- Opportunities:
    - Planning objectives
    - Capacities
    - Stop locations

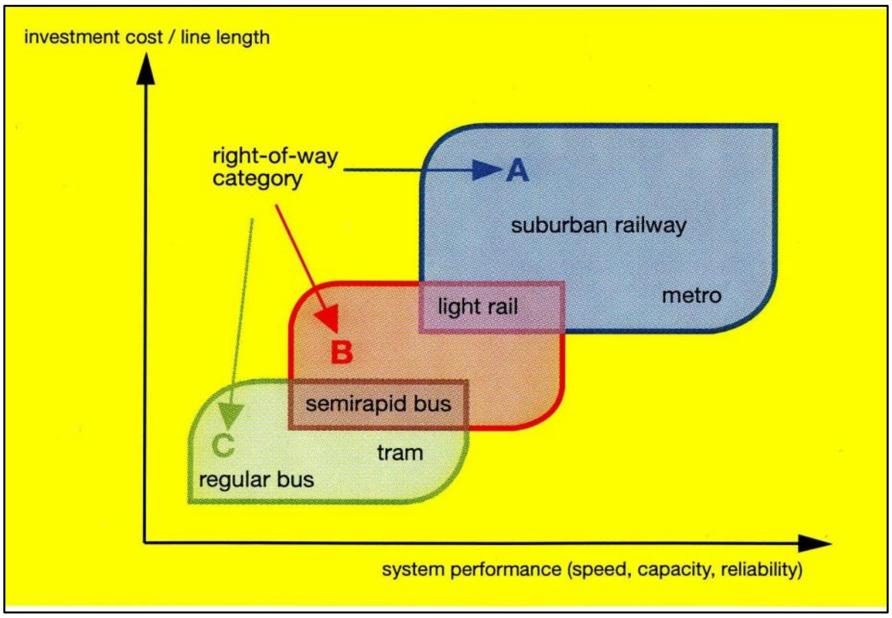
Costs:

- Capital costs
    - Infrastructure (track, stops/stations, power supply, structures, tunnels, other civil engineering works)
    - Rolling stock
    - Service diversions
- Land costs
- Legal costs (TWA, Planning approvals, PFI… etc)
- Operating costs
- Maintenance costs
- Third party costs
- Revenues
- Non-quantifiable costs
- Environmental costs
- Risk transfer costs

Benefits:

- Time savings
- Accident savings
- Operating cost savings
- Maintenance cost savings
- Renewal cost savings
- Decongestion benefits
- Environmental benefits
- Land use/planning gain benefits/regeneration
- Socio-economic benefits





Buses

bustimes from 30th April 2018

City Centre • Kingswells 14

MONDAY TO FRIDAY

Service No.	14	14	14	14	14	14	14	14	14		14	14	14	14	14	14	14	14		14
Upperkirkgate	-	0630	-	0730	-	0815	0835	0905	0932		1432	1532	-	1640	1705	1747	1812	1900		2200
Union Street at Adelphi	-	0633	-	0733	-	0818	0838	0908	0935		1435	1535	-	1643	1708	1750	1815	1903		2203
Aberdeen Union Square	0533	▼	0700	▼	0803	▼	▼	▼	▼		▼	▼	1557	▼	▼	▼	▼	▼		▼
Holburn Junction	0538	0638	0705	0738	0808	0824	0844	0913	0940	then hourly until	1440	1541	1606	1649	1714	1756	1820	1908	then hourly until	2208
Westburn Harcourt Road	0544	0644	0711	0744	0814	0831	0851	0919	0946		1446	1548	1613	1656	1721	1803	1826	1914		2214
Foresterhill Hospital	▼	▼	▼	▼	▼	▼	0854	▼	▼		▼	▼	▼	▼	▼	▼	▼	▼		▼
Summerhill Shapinsay Square	0548	0648	0715	0749	0819	0836	-	0923	0950		1450	1553	1618	1701	1726	1808	1830	1918		2218
Kingswells Fairley Road	0554	0654	0721	0756	0826	0843	-	0929	0956		1456	1600	1625	1708	1733	1815	1836	1924		2224
Kingswells Medical Centre	0557	0657	0724	0759	0829	0846	-	0932	0959		1459	1603	1628	1711	1736	1818	1839	1927		2227

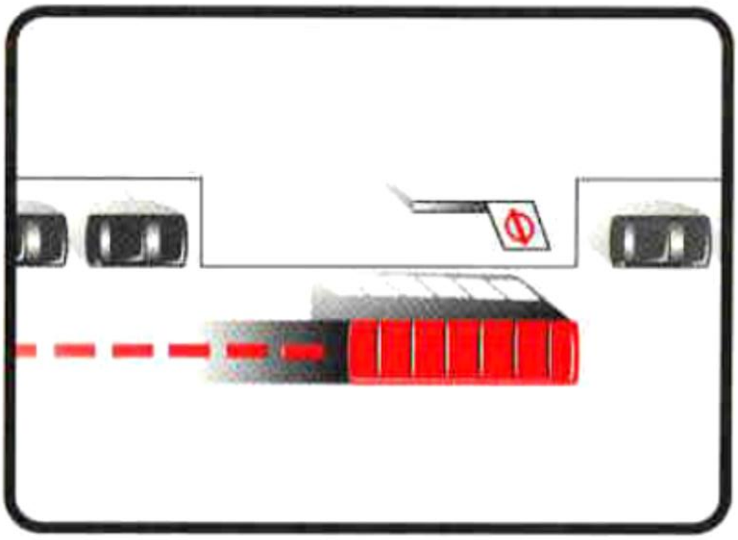
SATURDAY

Service No.	14	14	14	14		14	14		14	14	14	14	14	14		14
Upperkirkgate	-	0630	-	0730		0930	1030		1530	-	1630	1700	1730	1800		2200
Union Street at Adelphi	-	0633	-	0733		0933	1033		1533	-	1633	1703	1733	1803		2203
Aberdeen Union Square	0533	▼	0703	▼		▼	▼		▼	1600	▼	▼	▼	▼		▼
Holburn Junction	0538	0638	0708	0738	then every 30 mins until	0938	1038	then hourly until	1538	1608	1638	1708	1738	1808	then hourly until	2208
Westburn Harcourt Road	0544	0644	0714	0744		0944	1044		1544	1614	1644	1714	1744	1814		2214
Summerhill Shapinsay Square	0548	0648	0718	0748		0948	1048		1548	1618	1648	1718	1748	1818		2218
Kingswells Fairley Road	0554	0654	0724	0754		0954	1054		1554	1624	1654	1724	1754	1824		2224
Kingswells Medical Centre	0557	0657	0728	0757		0957	1057		1557	1627	1657	1727	1757	1827		2227

NO SUNDAY SERVICE

Bus Stop Borders with No Street Parking

Bus Stop Borders with On-Street Parking



Tram Station

Train Station

Ministry of Higher Education
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Al-Muthanna University
Civil Engineering Department

Traffic Engineering II

Dr. Noorance Al-Mukarm

Lecture No.10

Numerical Examples

Q1/ The number of road accidents per day were reported in 100 consecutive days as shown in table below. Check whether the distribution of these accidents can be considered as random.

No. of accidents	0	1	2	3	4	5	6	Total
No. of days (Observed freq.)	19	26	26	15	9	4	1	100
Total observed accidents	0	26	52	45	36	20	6	185
Poisson Equation *P(n)*	0.158	0.292	0.269	0.167	0.077	0.028	0.009	1.00
Expected frequency	15.8	29.2	26.9	16.7	7.7	2.8	0.9	100
Chi-square value	0.65	0.35	0.03	0.173	0.22	0.457		1.88

There were 185 accidents within 100 days , $m = \frac{185}{100} = 1.85$

Combine classes which have a frequency less than 5

The calculated Chi-square = 1.88 Degree of freedom = 6 – 3 = 3 then,

The tabulated Chi-square = 7.81

Since the tabulated Chi-square is greater than the calculated Chi-square (7.81 > 1.88), then the observed data can be represented by Poisson distribution.

Q2/ The following are tabulated headway in a single lane of traffic stream. Test the data for goodness fitting to the NE distribution.

Headway in sec	0-1	1-2	2-3	3-4	4-5	5-6	6-7	7-8	Total
Observed freq.	19	67	58	29	26	14	17	7	237
Total time	9.5	100.5	145	101.5	117	77	110.5	52.5	713.5
Cumu. Expected frequency = e^{-qt}	1.0	0.717	0.515	0.369	0.276	0.199	0.136	0.098	----
Relative freq.	0.283	0.202	0.146	0.093	0.077	0.063	0.038	0.098	1.00
Expected freq	67.07	47.87	34.60	22.04	18.25	14.93	9.01	23.23	237
Chi-square value	34.45	7.64	15.83	2.20	3.29	0.058	2.76	11.34	77.57

$\boldsymbol{Total\ time = 713.5\ sec}$ **then** $q = \frac{total\ observed\ events\ (vehicles)}{Total\ time} = \frac{237}{713.5} = 0.332\ veh/sec$

The calculated Chi-square = 77.57 Degree of freedom = 8 – 2 = 6 then,

The tabulated Chi-square = 12.6

Since the tabulated Chi-square is less than the calculated Chi-square (12.6 < 77.57), the observed data cannot be represented by Negative Exponential Distribution.

Q3/ For the geometric and traffic characteristics shown below, determine a suitable signal phasing system and phase lengths for the intersection using the Webster method. Show a detailed layout of the phasing system and the intersection geometry used.

Approach	North	South	East	West
Peak-hour approach volume:				
Left turn	133	73	168	134
Through movement	420	373	563	516
Right turn	140	135	169	178
PHF	0.95	0.95	0.95	0.95

Assume the following saturation flows:

Through lane 1600 veh/ln/h

Right lane 1400 veh/ln/h

Left lane 1000 veh/ln/h

Step (1) Calculate the equivalent hourly flow:

Approach	North	South	East	West
Peak-hour approach volume				
Left turn (L)	133/0.95=140	77	177	141
Through movement (TH)	442	393	593	543
Right turn (R)	147	142	178	187

Step (2) Assign lane group: assume have *one-left turn lane* & *one through+right lane*:

Approach	North	South	East	West
Peak-hour approach volume				
Left turn (L)	140	77	177	141
Through+Right (TH+R)	589	535	771	730

<u>Step (3)</u> Assign 4 phase scheme, saturation flow and determine *Y value*

Phase scheme	N-S (L)	N-S (TH+R)	E-W (L)	E-W (TH+R)
q	140	589	177	771
S	1000	3000	1000	3000
Y=q/S	0.14	0.196	0.177	0.257

$\sum Y = 0.77$

<u>Step (4)</u> Calculate the total lost time for the whole cycle length:

Assume lost time per phase (l) is equal to *3.5 sec*, then:

The total lost time L = No. of phases × lost time per phase = 4 × 3.5 = 14 sec

<u>Step (5)</u> Calculate the cycle length:

$$C_o = \frac{1.5\,L + 5}{1 - \sum Y} = \frac{(1.5 \times 14) + 5}{1 - 0.77} = 113.044 \approx 120\ sec$$

<u>Step (6)</u> Calculate the actual green, red, amber and amber-red timings:

Total effective green for the whole cycle $G_{t.eff} = C_o - L = 120 - 14 = 106$ sec

Phase scheme	N-S (L)	N-S (TH+R)	E-W (L)	E-W (TH+R)
Y	0.14	0.196	0.177	0.257
Yellow (amber)	3.0	3.0	3.0	3.0
Red-amber	2.0	2.0	2.0	2.0
$G_{eff} = G_{t.eff} \times (Y/\sum Y)$	19.272	26.98	24.366	35.379
$G_{act} = G_{eff} + l - amber$	19.772	27.48	24.866	35.879
$Red = C_o - G_{act} - amber - (Red\text{-}amber)$	95.228	87.52	90.134	79.121

Q4/ Repeat Q3 with the same traffic data and determine a suitable phase length, knowing that the all-red time per phase is 1.5 seconds.

<u>Solution</u>:

- **Steps (1) to (3) are the same.**
- **In Step (4) the lost time per phase l = 3.5 +1.5 = 5 sec, then the total lost time for the whole cycle L = 4 × 5 = 20 seconds.**
- **Complete the Steps (5) & (6).**

Q5/ The following are sample gross accidents statistics for 2000 km urban highway in 2018, determine accident and death rates for the given data:

Fatalities	Fatal accidents	Injury accidents	Involved drivers	Vehicle-km travelled	Registered vehicles	Population area
75	60	300	4100	15,000,000,000	100,000	300,000

$$Accident\ rate\ per\ km = \frac{total\ accidents\ (fatal + injury)}{length\ of\ highway\ section} = \frac{60 + 300}{2000} = 0.18$$

$$Accident\ rate\ per\ veh - km = \frac{total\ accidents \times 10^8}{vehicle - km\ travelled} = \frac{(60 + 300) \times 10^8}{15 \times 10^9} = 2.4$$

$$Accident\ involvement\ rate = \frac{total\ no.\ of\ drivers\ involving\ in\ accidents \times 10^8}{vehicle - km\ travelled}$$

$$= \frac{4100 \times 10^8}{15 \times 10^9} = 27.33$$

$$Death\ rate\ per\ population\ = \frac{fatalities \times 10^5}{population} = \frac{75 \times 10^5}{300,000} = 25.0$$

$$Death\ rate\ per\ registered\ vehicle\ = \frac{fatalities \times 10,000}{registered\ veh.} = \frac{75 \times 10^4}{100,000} = 7.5$$

Q6/ Consider the following data for suburban community in 2015, compute all accident and death rates:

Fatalities	All accidents	Involved drivers	Vehicle-km travelled	Registered vehicles	Population area	Section length
15	600	3400	12,000,000,000	350,000	500,000	1800

$$Accident\ rate\ per\ km = \frac{total\ accidents}{length\ of\ highway\ section} = \frac{600}{1800} = 0.33$$

$$Accident\ rate\ per\ veh - km = \frac{total\ accidents \times 10^8}{vehicle - km\ travelled} = \frac{600 \times 10^8}{12 \times 10^9} = 5.0$$

$$Accident\ involvement\ rate = \frac{total\ no.\ of\ drivers\ involving\ in\ accidents \times 10^8}{vehicle - km\ travelled}$$

$$= \frac{3400 \times 10^8}{12 \times 10^9} = 28.33$$

$$Death\ rate\ per\ population\ = \frac{fatalities \times 10^5}{population} = \frac{15 \times 10^5}{500,000} = 3.0$$

$$Death\ rate\ per\ registered\ vehicle\ = \frac{fatalities \times 10,000}{registered\ veh.} = \frac{15 \times 10^4}{350,000} = 0.43$$

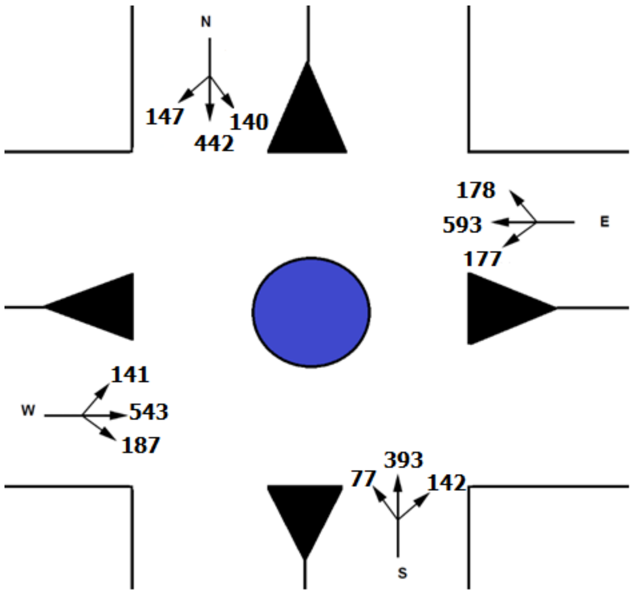
Q7/ Complete the following parking study table shown below. Determine the initial parking area, giving that the total number of bays is 60:

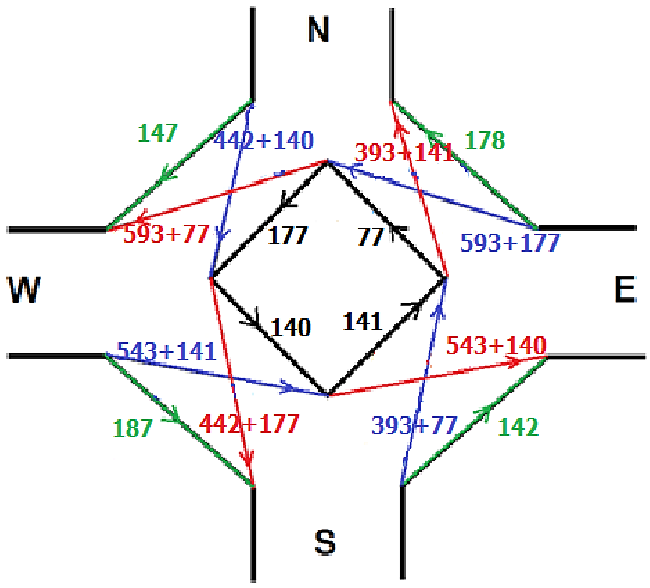
Time (min)	In	Out	Accumulation	Occupancy	Parking load
10	3	0	38	63.33	380
20	4	2	40	66.67	400
30	0/1/2/..	0/1/2/..	40	66.67	400
40	3	1	42	70.00	420
50	6	1	47	78.33	470
60	4	1	50	83.33	500

The initial parking area = 38 – 3 = 35 parked vehicles

Q8/ The width of a carriageway approaching an intersection is given as 17 m. The entry and exit width at the rotary is 10 m. The traffic approaching the rotary intersection from the four sides is given in the table below. Find the capacity of rotary intersection.

Approach	North	South	East	West
Left turn	140	77	177	141
Through movement	442	393	593	543
Right turn	147	142	178	187





- $\boldsymbol{Width\ of\ weaving\ section\ W_w = \frac{10+10}{2} + 3.5 = 13.5\ m}$
- $\boldsymbol{Weaving\ length\ L = 4 \times W_w = 54\ m}$
- $\boldsymbol{Determine\ the\ proportion\ of\ weaving\ traffic\ to\ non-weaving\ traffic\ as\ follows}$:

$$p_{EN} = \frac{393 + 141 + 593 + 177}{393 + 141 + 593 + 177 + 77 + 178} = \frac{1304}{1559} = 0.836$$

$$p_{NW} = \frac{442 + 140 + 593 + 77}{442 + 140 + 593 + 77 + 177 + 147} = \frac{1252}{1576} = 0.794$$

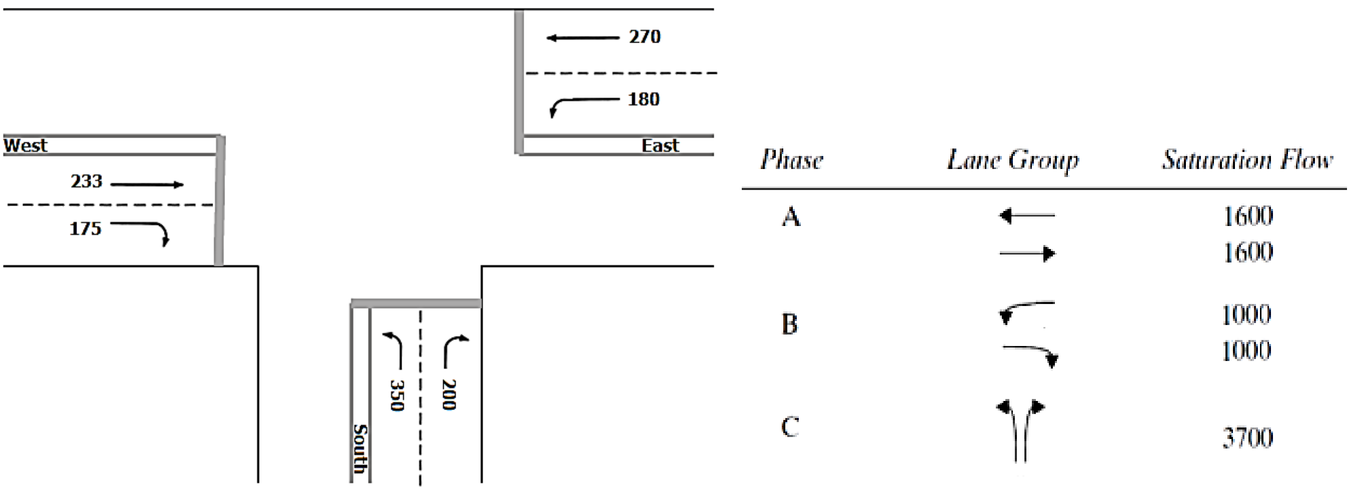
$$p_{WS} = \frac{543 + 141 + 442 + 177}{543 + 141 + 442 + 177 + 140 + 187} = \frac{1303}{1630} = 0.799$$

$$p_{SE} = \frac{543 + 140 + 393 + 77}{543 + 140 + 393 + 77 + 142 + 141} = \frac{1153}{1436} = 0.803$$

- $Min\ Capacity = \frac{280 \times 13.5 \times \left[1 + \frac{10}{13.5}\right] \times \left[1 - \frac{0.836}{3}\right]}{\left[1 + \frac{13.5}{54}\right]} = 3796\ vph$

- $Max\ Capacity = \frac{280 \times 13.5 \times \left[1 + \frac{10}{13.5}\right] \times \left[1 - \frac{0.794}{3}\right]}{\left[1 + \frac{13.5}{54}\right]} = 3869\ vph$

Q9/ Design a 3 phases traffic signal timings for T-intersection as shown in Figure 1. Given that PHF= 0.90 and the lost time for each phase is 4.5 seconds.



<u>Step (1)</u> Calculate the equivalent hourly flow:

Approach	South	East	West
Peak-hour approach volume			
Right turn (R)	200/0.9=223	0	195
Left turn (L)	389	200	0
Through movement (TH)	0	300	259

Step (2) Assign 3 phases scheme, saturation flow and determine *Y value*

Phase scheme	A		B		C
q	300	259	195	200	612
S	1600	1600	1000	1000	3700
Y=q/S	0.188	0.162	0.195	0.200	0.165
Y_{max}	0.188		0.200		0.165

$\sum Y_{max} = 0.553$

Step (3) Calculate the total lost time for the whole cycle length:

Assume lost time per phase (*l*) is equal to *4.5 sec*, then:

The total lost time L = No. of phases × lost time per phase = 3 × 4.5 = 13.5 sec

Step (4) Calculate the cycle length:

$$C_o = \frac{1.5\,L + 5}{1 - \sum Y} = \frac{(1.5 \times 13.5) + 5}{1 - 0.553} = 56.48 \approx 60\ sec$$

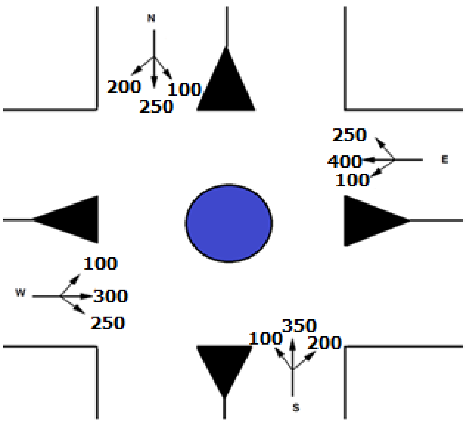
Step (5) Calculate the actual green, red, amber and amber-red timings:

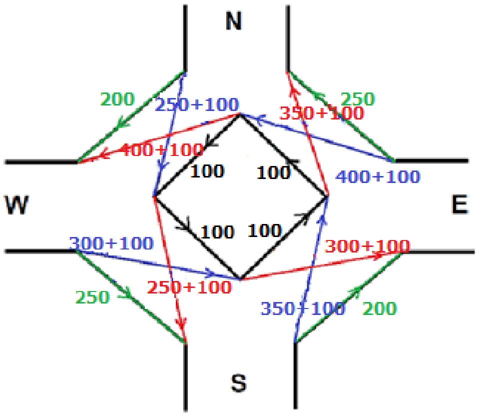
Total effective green for the whole cycle $G_{t.eff} = C_o - L = 60 - 13.5 = 46.5\ sec$

Phase scheme	A	B	C
Y	0.188	0.200	0.165
Yellow (amber)	3.0	3.0	3.0
Red-amber	2.0	2.0	2.0
$G_{eff} = G_{t.eff} \times (Y/\sum Y)$	15.8	16.8	13.87
$G_{act} = G_{eff} + l - amber$	17.3	18.3	15.37
$Red = C_o - G_{act} - amber - (Red\text{-}amber)$	37.7	36.7	39.7

Q10/ The width of a carriageway approaching an intersection is given as 15 m. The entry and exit width at the rotary is 9 m. The traffic approaching the rotary intersection from the four sides is given in the table below. Find the maximum and minimum capacity of rotary intersection.

Approach	North	South	East	West
Left turn	100	100	100	100
Through movement	250	350	400	300
Right turn	200	200	250	250





- $Width\ of\ weaving\ section\ W_w = \frac{9+9}{2} + 3.5 = 12.5\ m$

- $Weaving\ length\ L = 4 \times W_w = \ 4 \times 12.5 = 50\ m$

- $Determine\ the\ proportion\ of\ weaving\ traffic\ to\ non-$
  $weaving\ traffic\ as\ follows$:

$$p_{EN} = \frac{350 + 100 + 400 + 100}{350 + 100 + 400 + 100 + 250 + 100} = \frac{950}{1300} = 0.73$$

$$p_{NW} = \frac{250 + 100 + 400 + 100}{250 + 100 + 400 + 100 + 200 + 100} = \frac{850}{1150} = 0.74$$

$$p_{WS} = \frac{250 + 100 + 300 + 100}{250 + 100 + 300 + 100 + 250 + 100} = \frac{750}{1100} = 0.68$$

$$p_{SE} = \frac{300 + 100 + 350 + 100}{300 + 100 + 350 + 100 + 200 + 100} = \frac{850}{1150} = 0.74$$

- $Min\ Capacity = \frac{280 \times W_w \times \left[1 + \frac{e}{W_w}\right] \times \left[1 - \frac{p}{3}\right]}{\left[1 + \frac{W_w}{L}\right]} = \frac{280 \times 12.5 \times \left[1 + \frac{9}{12.5}\right] \times \left[1 - \frac{0.74}{3}\right]}{\left[1 + \frac{12.5}{50}\right]} = 3627\ vph$

- $Max\ Capacity = \frac{280 \times 12.5 \times \left[1 + \frac{9}{12.5}\right] \times \left[1 - \frac{0.68}{3}\right]}{\left[1 + \frac{12.5}{50}\right]} = 3725\ vph$