From Eq. (3-6) n = \frac{1}{k} \frac{u_s}{m^2} \frac{g}{k_w} \frac{p}{u_s} \text{ where } n = \text{ number of lanes} \text{, } u_s = \text{ speed of sound} \text{, } m = \text{ mass of air} \text{, } k_w = \text{ wave number} \text{, and } p = \text{ pressure}

The mean free speed (u_f): It is the maximum speed that can be obtained on the highway.

Assume velocity 50 feet per second to find the mean free speed of the highway. This speed should be used to determine the maximum capacity of the highway.

Determine the maximum speed of the vehicles for this capacity. Let it be 75 mph. 

The required max. speed of the vehicles is 75 mph.
Traffic capacity

The capacity: It is the ability of a highway to accommodate traffic or the flow which produces minimum acceptable journey speed or the maximum traffic volume for comfortable free-flow conditions.

Definition (Traffic capacity): the maximum number of vehicles that can pass over a given section of a lane or roadway in one direction (or in both directions for a two-lane highway) during a given time period (one hour) under prevailing roadway and traffic conditions.

6. maximum

6. No. of vehs. passenger cars/hr 3(x5/hr) مدة السير / الأسباب خلال 16 ساعة. كل سار.

6. one direction versus two directions نشاطات ذات دقة تأثير الإعداد الإقتصادي لا تكون له أي تأثير على الطرق الإعدادية الإنتاجية بصرف النظر عن الإSTALL/STAY WhatsApp 6 لعبة ماشية نظرية وظيفية تم تقسيمها بين اللاعبين نظرية Game 6. lane high one direction two lane high
4. **Capacity**

A given time period (veh/hr)

- Capacity
- Peak hr factor (PHF)

```
PHF = \frac{\text{hourly volume}}{\text{shorter period volume}} / \text{No. of periods in an hr}
```

**Example:**

- Hourly vol. = 1500 veh
- Highest 5 min vol. = 150

\[
\text{PHF} = \frac{1500}{150 \times 12} = 0.83
\]

5. **Prevailing roadway and traffic conditions**

- Physical features of the highway, which do not change unless the geometric design of the highway changes: lane width, lateral clearance, shoulders, auxiliary lanes.

6. **Factors affecting capacity**

- Traffic conditions, which are determined by the composition of the traffic:
  - Vehicles of Buses
  - Variation of traffic
  - Traffic interaction

- Ambient conditions which include visibility, road surface conditions, temperature and wind, pavement, grades.
speed - flow relationship

Theoretical relationship: K = congestion

free flow
limiting capacity
normal conditions
unstable conditions

The basic relationships between speed and volume (flow) are:
1. The speed decreases on each road as the flow increases, indicating that the measurements lie within the zone of normal conditions.
   - Traffic flow at any particular speed increases as the roads get wider.

- 2.5 km/hr
- 7.3
- 4.0
- 6.7
- 3.1
- 2.0
- 1.8
- 1.3
- 0.9
- 0.8
- 0.4
- 0.3
- 0.2
- 0.1
- 0.0

flows: veh/hr

roads in rural area
(3.9 km/hr)
(8.1 km/hr)
(12.3 km/hr)
(16.5 km/hr)
(20.7 km/hr)
(24.9 km/hr)
(29.1 km/hr)
(33.3 km/hr)
(37.5 km/hr)
(41.7 km/hr)
(45.9 km/hr)
(50.1 km/hr)
(54.3 km/hr)
(58.5 km/hr)
(62.7 km/hr)
(66.9 km/hr)
(71.1 km/hr)
(75.3 km/hr)
(79.5 km/hr)
(83.7 km/hr)
(87.9 km/hr)
(92.1 km/hr)
(96.3 km/hr)
(100.5 km/hr)
(104.7 km/hr)
(108.9 km/hr)
(113.1 km/hr)
(117.3 km/hr)
(121.5 km/hr)
(125.7 km/hr)
(129.9 km/hr)
(134.1 km/hr)
(138.3 km/hr)
(142.5 km/hr)
(146.7 km/hr)
(150.9 km/hr)
(155.1 km/hr)
(159.3 km/hr)
(163.5 km/hr)
(167.7 km/hr)
(171.9 km/hr)
(176.1 km/hr)
(180.3 km/hr)
(184.5 km/hr)
(188.7 km/hr)
(192.9 km/hr)
(197.1 km/hr)
(201.3 km/hr)
(205.5 km/hr)
(209.7 km/hr)
(213.9 km/hr)
(218.1 km/hr)
(222.3 km/hr)
(226.5 km/hr)
(230.7 km/hr)
(234.9 km/hr)
(239.1 km/hr)
(243.3 km/hr)
(247.5 km/hr)
(251.7 km/hr)
(255.9 km/hr)
(260.1 km/hr)
(264.3 km/hr)
(268.5 km/hr)
(272.7 km/hr)
(276.9 km/hr)
(281.1 km/hr)
(285.3 km/hr)
(289.5 km/hr)
(293.7 km/hr)
(297.9 km/hr)
(302.1 km/hr)
(306.3 km/hr)
(310.5 km/hr)
(314.7 km/hr)
(318.9 km/hr)
(323.1 km/hr)
(327.3 km/hr)
(331.5 km/hr)
(335.7 km/hr)
(339.9 km/hr)
(344.1 km/hr)
(348.3 km/hr)
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(356.7 km/hr)
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(365.1 km/hr)
(369.3 km/hr)
(373.5 km/hr)
(377.7 km/hr)
(381.9 km/hr)
(386.1 km/hr)
(389.3 km/hr)
(393.5 km/hr)
(397.7 km/hr)
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(417.7 km/hr)
(421.9 km/hr)
(426.1 km/hr)
(429.3 km/hr)
(433.5 km/hr)
(437.7 km/hr)
(441.9 km/hr)
(446.1 km/hr)
(449.3 km/hr)
(453.5 km/hr)
(457.7 km/hr)
(461.9 km/hr)
(466.1 km/hr)
(469.3 km/hr)
(473.5 km/hr)
(477.7 km/hr)
(481.9 km/hr)
(486.1 km/hr)
(489.3 km/hr)
(493.5 km/hr)
(497.7 km/hr)
(501.9 km/hr)
(506.1 km/hr)
(509.3 km/hr)
(513.5 km/hr)
(517.7 km/hr)
(521.9 km/hr)
(525.1 km/hr)
(529.3 km/hr)
(533.5 km/hr)
(537.7 km/hr)
(541.9 km/hr)
(546.1 km/hr)
(549.3 km/hr)
(553.5 km/hr)
(557.7 km/hr)
(561.9 km/hr)
(566.1 km/hr)
(569.3 km/hr)
(573.5 km/hr)
(577.7 km/hr)
(581.9 km/hr)
(586.1 km/hr)
(589.3 km/hr)
(593.5 km/hr)
(597.7 km/hr)
(601.9 km/hr)
(606.1 km/hr)
(609.3 km/hr)
(613.5 km/hr)
(617.7 km/hr)
(621.9 km/hr)
(626.1 km/hr)
(629.3 km/hr)
(633.5 km/hr)
(637.7 km/hr)
(641.9 km/hr)
(646.1 km/hr)
(649.3 km/hr)
(653.5 km/hr)
(657.7 km/hr)
(661.9 km/hr)
(666.1 km/hr)
(669.3 km/hr)
(673.5 km/hr)
(677.7 km/hr)
(681.9 km/hr)
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(689.3 km/hr)
(693.5 km/hr)
(697.7 km/hr)
(701.9 km/hr)
(706.1 km/hr)
(709.3 km/hr)
(713.5 km/hr)
(717.7 km/hr)
(721.9 km/hr)
(726.1 km/hr)
(729.3 km/hr)
(733.5 km/hr)
(737.7 km/hr)
(741.9 km/hr)
(746.1 km/hr)
(749.3 km/hr)
(753.5 km/hr)
(757.7 km/hr)
(761.9 km/hr)
(766.1 km/hr)
(769.3 km/hr)
(773.5 km/hr)
(777.7 km/hr)
(781.9 km/hr)
(786.1 km/hr)
(789.3 km/hr)
(793.5 km/hr)
(797.7 km/hr)
(801.9 km/hr)
(806.1 km/hr)
(809.3 km/hr)
(813.5 km/hr)
(817.7 km/hr)
(821.9 km/hr)
(826.1 km/hr)
(829.3 km/hr)
(833.5 km/hr)
(837.7 km/hr)
(841.9 km/hr)
(846.1 km/hr)
(849.3 km/hr)
(853.5 km/hr)
(857.7 km/hr)
(861.9 km/hr)
(866.1 km/hr)
(869.3 km/hr)
(873.5 km/hr)
(877.7 km/hr)
(881.9 km/hr)
(886.1 km/hr)
(889.3 km/hr)
(893.5 km/hr)
(897.7 km/hr)
(901.9 km/hr)
(906.1 km/hr)
(909.3 km/hr)
(913.5 km/hr)
(917.7 km/hr)
(921.9 km/hr)
(926.1 km/hr)
(929.3 km/hr)
(933.5 km/hr)
(937.7 km/hr)
(941.9 km/hr)
(946.1 km/hr)
(949.3 km/hr)
(953.5 km/hr)
(957.7 km/hr)
(961.9 km/hr)
(966.1 km/hr)
The capacity of any road to carry traffic is a function not only of its width but also of the speed/flow relationship. More vehs. can be accommodated at a lower level of service with a higher (V/C) ratio.

Capacities for modern highway are in past Ogilby or 113 vehs./miles.
level of service

العوامل التي تؤثر على حالة المرور تحدد مستوى service level A توجد بسمة عند المركبات خلال دارة عالية 1 أو 1.5 دقيقة زمنية للمرور. هذا هو الوضع الافتراضي (Flow rate), حيث نستخدم (Flow rate).

![Diagram](image)

<table>
<thead>
<tr>
<th>Level of Service</th>
<th>Flow Rate</th>
<th>Speed (km/hr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Free</td>
<td>45+</td>
</tr>
<tr>
<td>B</td>
<td>Stable</td>
<td>90-95</td>
</tr>
<tr>
<td>C</td>
<td>Stable</td>
<td>80-90</td>
</tr>
<tr>
<td>D</td>
<td>Stable but approaching unstable</td>
<td>65-80</td>
</tr>
<tr>
<td>E</td>
<td>Unstable</td>
<td>50-65</td>
</tr>
<tr>
<td>F</td>
<td>Forced</td>
<td>Under 50</td>
</tr>
</tbody>
</table>

definition: level of service is a term used to classify the varying conditions of traffic flow that take place on a highway. It is a qualitative measure of the effect of a number of factors which include:

- Speed and travel time
- Traffic interruptions
- Freedom to maneuver
- Safety
- Driving comfort and convenience
- Economy considered from point of view of each operating cost.

margin: 0.5 cm

العوامل التي تؤثر على حالة المرور تحدد مستوى service level A توجد بسمة عند المركبات خلال دارة عالية 1 أو 1.5 دقيقة زمنية للمرور. هذا هو الوضع الافتراضي (Flow rate), حيث نستخدم (Flow rate).
level A: Free flow, speed controlled by drivers' desires; speed limits, or physical road way conditions.
  max vol 2 lanes one direction = 1400 pcu/hr
  S = 97 km/hr

level B: Stable flow; operating speeds begining to be restricted; little or no restrictions on maneuverability by other vehicles;  max vol 2 lanes one direction = 2000 pcu/hr
  S = 88 km/hr

level C: Stable flow; speeds and maneuverability more closely restricted.
  max vol 2 lanes one direction = 2750 pcu/hr
  S = 70 km/hr

level D: Approaches unstable flows; tolerable speeds can be maintained but temporary restrictions to flow cause substantial drops in speeds; Little freedom to maneuver, comfort and convenience low.
  Vol 2 lanes one direction = 3600 pcu/hr
  S = 54 km/hr

level E: Volumes near capacity; speed typically in neighborhood of 30mph (48 km/hr); flow unstable; stoppages of momentary duration. Ability to maneuver severely limited; max vol. 2 lane one direction = 3000 pcu/hr
  S = 48-64 km/hr

level F: Forced flow, low operating speeds, volumes below capacity; queues formed.
  Vol: 0 - Capacity or 2000
  S = stop = 48 km/hr

<table>
<thead>
<tr>
<th>Level of Service</th>
<th>Capacity pcu/hr</th>
<th>Speed km/hr</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1400</td>
<td>&gt;97</td>
</tr>
<tr>
<td>B</td>
<td>2000</td>
<td>&gt;88</td>
</tr>
<tr>
<td>C</td>
<td>2750</td>
<td>&gt;80</td>
</tr>
<tr>
<td>D</td>
<td>3600</td>
<td>&gt;64</td>
</tr>
<tr>
<td>E</td>
<td>3650</td>
<td>48-64</td>
</tr>
<tr>
<td>F</td>
<td>0-2000</td>
<td>0-48</td>
</tr>
</tbody>
</table>

For multilane rural with controlled access highway; 2 Lanes - One direction
### Level of Service Characteristics by Highway Type

<table>
<thead>
<tr>
<th>Level of Service</th>
<th>Two Lane</th>
<th>Multilane Rural Without Access Control</th>
<th>Controlled Access Highway</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Operating Speed</td>
<td>Service Volume</td>
<td>Operating Speed</td>
</tr>
<tr>
<td><strong>A</strong> Free flow</td>
<td>km/hr</td>
<td>≥ 97</td>
<td>400 pass. vehicles per hr., two-way</td>
</tr>
<tr>
<td><strong>B</strong> Stable flow</td>
<td>≥ 80</td>
<td>900 pass. vehs. per hr., two-lane</td>
<td>≥ 88</td>
</tr>
<tr>
<td><strong>C</strong> Stable flow</td>
<td>≥ 64</td>
<td>1400 pass. veh. per hr., two-lane</td>
<td>≥ 72</td>
</tr>
<tr>
<td><strong>D</strong> Approaching unstable flow</td>
<td>≥ 56</td>
<td>1700 pass. veh. per hr., two-lane</td>
<td>≥ 56</td>
</tr>
<tr>
<td><strong>E</strong> Unstable flow</td>
<td>≥ 48</td>
<td>2000 pass. veh. per hr. per lane</td>
<td>≥ 48</td>
</tr>
<tr>
<td><strong>F</strong> Forced, congested flow</td>
<td>&lt; 48</td>
<td>under 2000 pass vehs. per hr., two way</td>
<td>&lt; 48</td>
</tr>
</tbody>
</table>
Intersections

An intersection is the general area where two or more highways join or cross. It is the focal point of conflicts and congestion.

- Intersections cause of a large proportion of traffic delays.
- They cause accidents.

Highway intersection design objectives: Due to the congestion, proper intersection control is employed to accomplish the following objectives:

1. Increase intersection capacity: (i.e) parking restrictions in the vicinity (o.e) of the intersection, right turn reduce conflict point and the use of traffic signals.

2. Reduction and prevention of accidents: most of the accidents occur at intersections. (about 1/3 of total accidents occur in urban area and 25% of total occur in rural area).

3. Protection of major streets: provide more continuous movement along major streets at a greater speed & increase safety.
In the selection of intersection type, the following factors should be considered:

1. Traffic volume & delay design speeds
2. Pedestrian movement
3. Cost and availability of land
4. The accident record of the existing intersection

Types of Intersection Control

A. Priority intersections
B. Signalised intersections
C. Rotary intersections
D. Grade-separated intersections

A. Priority intersections: The universal adoption of the "give way to traffic on the left" rule at roundabouts together with the use of "give ways" and "stop" control at junctions has considerably increased the no. of occasions at which a driver has to merge or cross a major road traffic stream making use of gaps or lags in one or more conflicting streams.

B. Signalised intersections: There are two types
   1. Veh. actuated signals
   2. Fixed time signals

C. Rotary intersections: Chapter 20 Page 182
   R.J. Salter 1976 "highway traffic design & analysis"

D. Grade-separated intersections - which are interchanges
Intersection types

Four types of intersections

A - Three-leg intersection

B - Four-leg intersections

C - Multi-leg intersections

D - Rotary intersection
Intersection Characteristics

1. Maneuvers (Turning movement)
   Four types of maneuvers are possible:

   A - Diverging
     - Right
     - Left
     - Mutual
     - Multiple

   B - Merging
     - Right
     - Left
     - Mutual
     - Multiple

   C - Crossing
     - Direct Right
     - Direct Left
     - Oblique
     - Opposed

   D - Weaving, accomplished by merging maneuver followed by a diverging maneuver
2. Conflicts

A conflict arises whenever the paths followed by vehicles diverge, merge, or cross.

- Diverging conflicts
- Merging conflicts
- Through-flow crossing conflicts
- Turning-flow crossing conflicts

The number of conflicts that can be expected at different intersections is shown in Fig.

4-leg intersection single-lane approach
No signal control
Traffic signals -

Traffic signals tell the motorists to stop and go. As we know, the traffic signals are used in grade separated intersections (grade separated intersection).

Traffic signals are used to control traffic at intersections. To be effective, any traffic signal control must fulfill four basic requirements:

1. **Meaning:**
   - Red: stop
   - Green: go

2. **Attention:**
   - Black, black or black and yellow back board

3. **Respect:**
   - Each light signal is accompanied by the sound of different sounds.

4. **Time for response:**
   - At code (MART) these signals are used.

   - **Red:** stop, but advise "prepare to stop."
   - **Green:** go, if safe to do so.
   - **Amber:** stop, unless unsafe to do so.
Vehicle actuated signals

1. Vehicle actuated signals

Vehicle-controlled: Whenever a vehicle leaves the roadway, the red signal is transmitted to the controller. The controller is responsible for activating the signal when vehicles enter the area.

2. Fixed time signals

3. Detector pad - installed under the roadway, it detects when vehicles enter or leave the area.

4. Loop detectors - embedded in the roadway, they detect when vehicles enter or leave the area based on the change in the magnetic field.
Definitions for vehicle actuated signals

Minimum green period - The shortest period of right away, long enough for a stationary veh at the stop line to start up and clear the intersection

Vehicle extension period - With loop detectors a 1.5 sec extension to the minimum green time is made for the crossing of each of the loops. With pad detectors, a fixed, location specific, time is added to the min green time for each registered demand up to the maximum green period.

Maximum green period: Starts at the beginning of the green period if vehs are waiting on the other approach, or from the time of the first demand on that approach.
Intergreen period: the time between one approach losing the green and another approach obtaining it. A four seconds intergreen is normal but if it is necessary to clear traffic from the intersection it is possible to vary this by incorporating an all-red period.

(2) Fixed time signals

Fixed time signals: a kaiul. actuated signals or max. green period

Turning traffic

Turning traffic is straightforward traffic signal

Traffic signal controlled junctions

(a) late start
(b) early cut-off... from which is evolved the all-red period
Fig. (2-5) : Accumulation of Parking Vehicles by Facility Type (33).
(4) Intergreen period: the time between one approach losing the green and another approach obtaining it. A four seconds intergreen is normal but if it is necessary to clear traffic from the intersection it is possible to vary this by incorporating an all Red period.

(2) Fixed time signals

A fixed time signal is a veh. actuated signal but has a max. green period.

(3) Turning traffic

A straightforward traffic signal is a signal for the movement of vehicles from one side of the road to another. Turning traffic is the movement of vehicles from one lane to another. Turning traffic can be controlled by means of traffic signals. Turning traffic signals are used to control the flow of vehicles at intersections. Turning traffic signals are classified into three types: left turn, right turn, and both directions.

(a) late start
(b) early cut-off. From which is evolved the
(c) all red period.
Traffic signal operation allowing for heavy left-turning movement by late start or early cut-off.

In cases where the flow is not free and traffic congestion occurs, a cut-off is made in the early phase to allow for late start.
Fig. (2-4): Accumulation of Vehicles by Purpose in a Multi-Storey Car parks in Coventry (18).

What do you suggest to apply for a small signalised intersection, a late start or early cut off of the opposing phase method and why?
early cut-off or a late start of the opposing phase is employed when the number of left turning vehicles is not sufficient to justify the provision of a left turning phase, but where left-turning vehicles have difficulty in completing the traffic movement.

Storage space for left turning vehicles is mostly needed in early cut-off method rather than in late start method.

\[
\text{late start} \quad \text{early cut-off}
\]
Fig. (2-3) : Pattern of Journey Purposes for Cars
Arriving Hourly in Lone.
The 'phase' $E_{W}$ is the sum of the two flows $Y_{w}$ and $Y_{e}$:

$$E_{W} = Y_{w} + Y_{e}$$

The maximum value of $E_{W}$ is $Y_{\max}$. If $E_{W}$ is greater than $Y_{\max}$, then $E_{W}$ is the sum of the two flows.

Let $Y_{w} + Y_{e} > Y_{w}$, then $Y_{e}$ is the value for the first stage and $Y_{w}$ is the value for the second stage.

Let $Y_{w} > Y_{w} + Y_{e}$, then the green flow that $w$ requires should be divided in proportion to the $y$ values of streams $w$ and $e$.

Then the $y$ value for the first stage is

$$Y_{1st} = \frac{Y_{e} \cdot Y_{w}}{Y_{e} + Y_{w}}$$

and for the second stage

$$Y_{2nd} = \frac{Y_{w} \cdot Y_{w}}{Y_{e} + Y_{w}}$$

The same procedure may be used with a later-start for the opposing flow.
4. Other signal operations

by deploying a family of algorithms to detect vehicles. This process
involves the following steps:

1. Selective vehicle detection
2. Non-vehicle detection
3. Other signal operations

- Deployment of a family of algorithms to detect vehicles.
- Non-vehicle detection.
- Other signal operations.

The deployment of these detection algorithms helps to enhance
the accuracy and reliability of signal detection.

Standard time: Throughout the day, the time of
standard time is observed. In this regard, the
standard time is marked by the clock that is
in sync with the time server. These clocks
are synchronized with the time server.

The time is marked by clocks that are
in sync with the time server. These clocks
are synchronized with the time server.

The time is marked by clocks that are
in sync with the time server. These clocks
are synchronized with the time server.
Sometimes there are more than two major traffic conflicts and then it is necessary to employ more than two phases in the traffic control system.

Like where at normal cross roads there is a heavy left turning movement on one of the approaches.

Traffic movement in a 3 phase system for heavy left turning movement on one of the approaches.

Example:
Traffic signals are to be installed at a T-junction where there is a heavy left turning movement and a normal right turning movement from the minor road.

(a) how many phases would be required for conventional signal control?

(b) Illustrate the traffic movements in the system.

Solution (b)

Selected

Solve (a) The correct no. of phases for conventional signal control at this intersection is two; there is no need to provide a third phase to cater for the heavy left-turning movement for the minor road because it does not conflict with any other flow at a T-junction.
3. Calculate traffic flow in pcu for each approach:

- private cars, taxis, light good vehs. (conversion to pcu)
  - 1.00
- heavy goods vehs. (HGVs)
  - 1.75
- public transport vehs. (PSVs)
  - 2.25
- motor cycle
  - 0.33
- pedal cycle
  - 0.20

4. Calculate saturation flow (S), which is the max. flow that can pass through an intersection from one approach without impedance by signals.

For width of the approach $>$ 5.5 m

$$ S = 525 W \text{ pcu/hr} $$

For $W \leq 5.5 \text{ m}$, a non-linear relationship is used as in this table:

<table>
<thead>
<tr>
<th>$W$ (m)</th>
<th>3</th>
<th>3.5</th>
<th>4</th>
<th>4.5</th>
<th>5</th>
<th>5.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S$ (pcu/hr)</td>
<td>1850</td>
<td>1875</td>
<td>1975</td>
<td>2175</td>
<td>2250</td>
<td>2900</td>
</tr>
</tbody>
</table>

These numbers must be increased for gradients.

- $S = 1.035$ for downhill gradient
- $S = 0.975$ for uphill gradient

Definition of gradient of the approach: the average slope between the stop line and a point on the approach 61 m before it.
where \( r \) = turning radius in meters = 12.16 m

for good environment: dual carriageway approaches, no noticeable pedestrian interference, no parked vehicles, no interferences to traffic flow from left-turning vehs., good visibility, adequate turning radii

\[ S = \frac{1800}{1 + 1.52I} \text{ pcn/hr for single file streams} \]
\[ S = \frac{3000}{1 + 1.52I} \text{ pcn/hr for double file streams} \]

\[ 17.69 \approx 2700 \text{ pcn/hr} \]

for poor environment: low average speed, interference from standing vehs. and left-turning vehs., poor visibility, poor alignment

\[ S = \frac{120}{S_{\text{standard}}} \]

\[ S = \frac{85}{S_{\text{standard}}} \]

The effect of roadway and environmental factors on the capacity of a traffic signal approach:

Saturation flow must be amended for the effect of:

1. Gradient: +3% for each 1% downhill gradient, -3% uphill.

2. Environment:
   +20% \( S \) for good environment
   -15% \( S \) for poor environment
example: What is the saturation flow of an approach with good environmental conditions with a continuous uphill gradient of 2% where all vehs. discharge straight across the intersection and where the approach width is 10.5 m

Solution

\[
\text{for } W > 5.5 \text{ m } \quad S = 525 \times W
\]

\[
: \quad S = 525 \times 10.5
\]
\[
= 5512.5 \text{ pcu/hr}
\]

for good environmental conditions the saturation flow will increased by 20%

\[
: \quad S = 5512.5 \times (1+0.2)
\]
\[
= 6615 \text{ pcu/hr}
\]

for the uphill gradient the S will decreased 3% for each 1% uphill gradient

\[
: \quad S = 6615 \times (1- \frac{3}{100} \times 2)
\]
\[
= 6218.1 \text{ pcu/hr}
\]

For 3% decrease:

\[
S = 525 \times 10.5 \times 1.2 \times (1- 0.03 \times 2)
\]
\[
= 525 \times 10.5 \times 1.2 \times 0.94
\]
\[
= 6218.1 \text{ pcu/hr}
\]
5. Assume intergreen period (IG), which is the period between one phase losing right of way and the next phase gaining right of way.

Min IG = 45

Amber period = 3 Sec
red/amber = 2 Sec

In Iraq:

<table>
<thead>
<tr>
<th>G</th>
<th>AAAA</th>
<th>Red</th>
<th>A</th>
</tr>
</thead>
</table>
| IG = 4.5 Sec

<table>
<thead>
<tr>
<th>R</th>
<th>G</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>G</th>
<th>AAAA</th>
<th>Red</th>
</tr>
</thead>
</table>
| IG = 6 Sec

<table>
<thead>
<tr>
<th>Red</th>
<th>G</th>
</tr>
</thead>
</table>

R = Red
G = Green
A = Amber
R/A = Red/Amber signal used in U.K.

<table>
<thead>
<tr>
<th>Red</th>
<th>Green</th>
<th>Amber</th>
</tr>
</thead>
<tbody>
<tr>
<td>All Red, IG</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

U.K:

<table>
<thead>
<tr>
<th>Red</th>
<th>Red/Amber</th>
<th>Green</th>
<th>Amber</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 Sec</td>
<td>3 Sec</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
6 - Calculate lost time, which is total time during a full cycle which is not effectively used for veh. movement in one or other direction.

Lost time = lost time during the intergreen period (all red periods) + starting and stopping losses (5 + 5) losses

\[ L = zn + R \]

where

\[ \frac{5 + 5}{2} = 2.5 \text{ sec} \]

\[ L = \text{Lost time (sec)} \]

\[ n = \text{No. of phases} \]

\[ R = \text{total all red or red/red-amber times (sec)} \]

or

\[ L = \frac{5}{2} (IG - 1) \]

\[ IG = \text{Intergreen period sec} \]

7 - Calculate \( Y = y_{max} \)

\[ Y = \frac{y_{max} + y_{max}}{2} \]

The greater \( Y \) value from at a normal four-way junction, the north & south approaches added to the greater of the two values from the other pair of approaches.

where

\[ Y = \frac{q}{s} \]

\[ \text{Traffic flow on the approach} \]

\[ \text{saturation flow} \]

8 - Calculate the optimum cycle time \( (C_0) \) for an intersection

\[ C_0 = \frac{1.5L + 5}{1 - Y} \]

\[ \text{min } C_0 = 25 \text{ sec} \]

\[ \text{max } C_0 = 120 \text{ sec} \]

This optimum cycle time gives the least average delay to all vehicles using the intersection.
9. Calculate the total effective green time \( g \)

\[ g = C_o - L \]

and then effective green time for the phases

\[ g_{\text{phase}} = \frac{y_{\text{max}}}{Y} \times g \]

\[ \frac{g_1}{g_2} = \frac{y_1}{y_2} \]

\[ g_{\text{phase2}} = \frac{y_{\text{max2}}}{Y} \times g \quad \text{etc.} \]

10. Calculate the actual green time for individual phases \( G_1, G_2 \ldots \) etc.

\[ G_1 = g_1 - \text{Amber period} + \left( \frac{5 + 5 \text{ losses}}{2} \right) \text{ sec} \]

\[ . \quad G_1 = g_1 - 1 \]

actual green time = effective green time - 1

![Diagram showing saturation flow, effective green time, and lost time with annotations for green and amber phases. Variation in discharge across the stop line is also indicated.](image-url)
effective green time

× area under the curve: represent the no. of vehs. that cross the stop line during the green period.

× The height of the rectangle is equal to the saturation flow and the base of the rectangle is the effective green time.

× The time intervals between the commencement of green and the commencement of the effective green, and termination of effective green and the termination of the amber period are referred to as the lost time due to starting delays.

× Lost time due to starting delays = 2 Secs

1. Effective green time = actual green time + 3 Secs (amber period) - 25 (lost time)

\[ g = G - 1 \]
Delays at traffic signals

The delay in sec per veh due to traffic signal is equal

d = 0.9 \left( \frac{S(G - G')}{2G(G - G')} + \frac{1800 G' Co}{G(S(G - G'))} \right)^2

where

- \( d \) = delay/veh (sec)
- \( C \) = cycle time (sec)
- \( G \) = green time (sec)
- \( G' \) = flow (veh/hr)
- \( S \) = saturation flow (veh/hr)

The pcu factor will be the overall pcu volume divided by the overall volume in (veh/hr) = (1.10 to 1.35)

The time is equal 15 s (veh/hr) or 8640 s/hr (veh/hr)
Example: A control signal for this function.

Note: Use two phase system
IG period = 4 sec for each phase

\[ \begin{align*}
N & \quad w = 8m \\
400 & \quad \downarrow \\
650 & \quad \downarrow \\
w \quad w = 8m & \rightarrow 200 \\
140 & \quad \uparrow \quad \rightarrow \quad 200 \\
700 & \quad \downarrow \\
180 & \quad \uparrow \\
352 & \quad \leftarrow \quad \rightarrow \quad 750 \\
w \quad w = 8m & \rightarrow 200 \\
140 & \quad \downarrow \\
525 & \quad \downarrow \\
w \quad w = 6m & =
\end{align*} \]

Solution

\[ L = z (IG-1) = (4-1) x (4-1) = 6 \text{ sec} \]

The two phases will be N-S + E-W

\[ \begin{align*}
q_N &= 200 + 400 + 100 \times 1.75 = 775 \text{ pcw/hr} \\
q_S &= 200 + 50 + 352 \times 1.75 = 866 \text{ pcw/hr} \\
q_E &= 700 + 400 + 180 \times 1.75 = 1415 \text{ pcw/hr} \\
q_W &= 200 + 140 + 650 \times 1.75 = 1477.5 \text{ pcw/hr} \\
S &= 525 \text{ W for } w > 5.5 \text{ m} \\
S_N &= 525 \times 8 = 4200 \text{ pcw/hr} \\
S_S &= 525 \times 8 = 4200 \\
S_W &= 525 \times 8 = 4200 \\
S_E &= 525 \times 6 = 3150
\end{align*} \]
<table>
<thead>
<tr>
<th>Phase I</th>
<th>Phase II</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>$g$ (pcw/hr)</td>
<td>775</td>
</tr>
<tr>
<td>$S$ (pcw/hr)</td>
<td>866</td>
</tr>
<tr>
<td>$y = g/S$</td>
<td>0.185</td>
</tr>
<tr>
<td>$y_{max}$</td>
<td>0.206</td>
</tr>
<tr>
<td>$Y = 2y_{max}$</td>
<td></td>
</tr>
</tbody>
</table>

\[ C_0 = \frac{1.5 \times 6 + 5}{1 - y} = \frac{1.5 \times 6 + 5}{1 - 0.655} = 40.5 \Rightarrow 41 \text{ sec} \]
\[ \text{Total effective green time} = C_0 - L \]
\[ = 41 - 6 = 35 \text{ sec} \]
\[ g_{N-S} = \frac{0.206 \times 35}{0.655} = 11 \text{ sec} \Rightarrow G_1 = 11 - 1 = 10 \text{ sec} \]
\[ g_{E-W} = \frac{0.449 \times 35}{0.655} = 24 \text{ sec} \Rightarrow G_1 = 24 - 1 = 23 \text{ sec} \]

The time setting of the two phases.

The diagrams depict the flow at different points, showing the directions and concentrations at various locations.
d = 0.9 \left[ \frac{S (G - G_0)^2}{2G_0 (S - G)} + \frac{1800 \beta G_0^2}{G S (G S - G_0)} \right]

If the composition of the traffic in all approaches is 5% HGV, 13% buses and 82% passenger car and the equivalents are 1 for passenger car, 1.75 for HGV and 2.25 for buses.

The pce factor = 0.82 \times 1 + 0.05 \times 1.75 + 0.13 \times 2.25 
= 1.2

<table>
<thead>
<tr>
<th>$q$ (veh/hr)</th>
<th>$N$</th>
<th>$S$</th>
<th>$E$</th>
<th>$S$</th>
</tr>
</thead>
<tbody>
<tr>
<td>646</td>
<td>722</td>
<td>1179</td>
<td>1231</td>
<td></td>
</tr>
<tr>
<td>3500</td>
<td>3500</td>
<td>2625</td>
<td>3500</td>
<td></td>
</tr>
<tr>
<td>G</td>
<td>10</td>
<td>10</td>
<td>23</td>
<td>23</td>
</tr>
<tr>
<td>Co</td>
<td>41</td>
<td>41</td>
<td>41</td>
<td>41</td>
</tr>
</tbody>
</table>

d = 0.9 \left( 14.37 + 6.56 \right)

(see) 18.8
example for early cutoff and late start facilities

The following hourly flows (table below) and saturation flows relate to an intersection to be controlled by two-phase signals incorporating a late-start feature. Minimum intergreen periods are employed and starting delays are 2 s, for each green plus amber period.

<table>
<thead>
<tr>
<th>Approach</th>
<th>Flow (pcw/hr)</th>
<th>S (pcw/hr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>west, straight ahead</td>
<td>400</td>
<td>1900</td>
</tr>
<tr>
<td>and right turning</td>
<td></td>
<td>1600</td>
</tr>
<tr>
<td>west, left turning</td>
<td>200</td>
<td>1900</td>
</tr>
<tr>
<td>east all movements</td>
<td>700</td>
<td>1900</td>
</tr>
<tr>
<td>north</td>
<td>500</td>
<td>1900</td>
</tr>
<tr>
<td>south</td>
<td>600</td>
<td>1900</td>
</tr>
</tbody>
</table>

IS the period that left-turning vehicles from the west approach require to complete their turning movement without obstruction from the straight ahead flow on east approach at 6 s, 11 s, or 16 s.

Solution

<table>
<thead>
<tr>
<th></th>
<th>N</th>
<th>S</th>
<th>E</th>
<th>W</th>
</tr>
</thead>
<tbody>
<tr>
<td>Q</td>
<td>500</td>
<td>600</td>
<td>700</td>
<td>400</td>
</tr>
<tr>
<td>S</td>
<td>1900</td>
<td>1900</td>
<td>1900</td>
<td>1900</td>
</tr>
<tr>
<td>y_max</td>
<td>0.26</td>
<td>0.32</td>
<td>0.37</td>
<td>0.210</td>
</tr>
<tr>
<td>y_max</td>
<td></td>
<td>0.32</td>
<td></td>
<td>0.50</td>
</tr>
</tbody>
</table>
| Min T (G) = 6 sec
| L = 2n + R = 2 x 2 + (4+1) = 6 sec
| or L = 5 (G+1) = (4+1) + (4+1) = 6 sec
| G0 = 1.5L + 5 1 - y = 1.5 x 6 + 5 1 - 0.82 = 78 sec
| Total effective green time = 78 - 6 = 72 sec

GNS = 0.32 x 72 = 28  G = 27
GFW = 0.5 x 72 = 44  G = 43
G1 = 0.27 x 43 = 32 sec
GFW = 0.5 x 43 = 41 sec
For early cut off:

...
Example: Design a typical good junction shown below with a 5 secs intergreen period on one phase and a 6 secs intergreen on the other, use 2 phase assuming that turning traffic is negligible.

**Solution**

\[ R = (5-3) + (5-3) = 2 + 3 = 5 \]  
\[ J_{sec} \text{ = Amps} \]

\[ L = 2N + R \]
\[ 2 \times 2 + (2+3) \]
\[ = 9 \text{ sec} \]

Or \[ L = 2 \times (I G - 1) \]
\[ = (5-1) + (6-1) \]
\[ = 9 \text{ sec} \]

<table>
<thead>
<tr>
<th></th>
<th>Approach</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>N</td>
<td>S</td>
<td>E</td>
<td>W</td>
</tr>
<tr>
<td><strong>y</strong> (pcw)</td>
<td>650</td>
<td>600</td>
<td>1400</td>
<td>1500</td>
</tr>
<tr>
<td><strong>s</strong> (pcw)</td>
<td>1850</td>
<td>1400</td>
<td>3833</td>
<td>3833</td>
</tr>
<tr>
<td>y</td>
<td>0.351</td>
<td>0.316</td>
<td>0.365</td>
<td>0.391</td>
</tr>
<tr>
<td>y_{max}</td>
<td>0.351</td>
<td></td>
<td>0.391</td>
<td></td>
</tr>
<tr>
<td>y = 2y_{max}</td>
<td></td>
<td></td>
<td>0.742</td>
<td></td>
</tr>
<tr>
<td>C_{o}</td>
<td>\frac{1.5 \times 9 + 5}{1 - 0.742} = 71.7 \text{ say 72 sec}</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total effective green time = C_{o} - L = 72 - 9 = 63 sec</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>g_{NS}</td>
<td>\frac{y_{NS} \times \text{total effective green time}}{y_{max}} = \frac{0.351 \times 63}{0.742} = 30 \text{ sec}</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>g_{EW}</td>
<td>\frac{0.391 \times 63}{0.742} = 33 \text{ sec}</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Actual green time ( g_{NS} = 9 - 1 = 30 - 1 = 29 \text{ sec} )</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( g_{EW} = 33 - 1 = 32 \text{ sec} )</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Example 2: Design a 3-phase traffic signal for the intersection shown.

IG for each N-S phases = 4 sec

IG for EW = 6 sec

There is a heavy left turning traffic from north and south.

Solution

Chose phase N, EW, S

<table>
<thead>
<tr>
<th></th>
<th>N</th>
<th>E</th>
<th>W</th>
<th>S</th>
</tr>
</thead>
<tbody>
<tr>
<td>Q</td>
<td>1100</td>
<td>400</td>
<td>300</td>
<td>1100</td>
</tr>
<tr>
<td>S</td>
<td>3675</td>
<td>1900</td>
<td>1900</td>
<td>3675</td>
</tr>
<tr>
<td>y</td>
<td>0.3</td>
<td>0.21</td>
<td>0.15</td>
<td>0.3</td>
</tr>
<tr>
<td>ymax</td>
<td>0.3</td>
<td>0.21</td>
<td>0.3</td>
<td></td>
</tr>
<tr>
<td>y</td>
<td></td>
<td></td>
<td></td>
<td>0.81</td>
</tr>
</tbody>
</table>

\[ C_0 = \frac{1.5 \times 11 + 5}{1 - 0.81} = 113 \text{ sec} \]

Total effective green time = 113 - 11 = 102 sec

\[ G_N = \frac{0.3}{0.81} \times 102 = 38 \text{ sec} \]

\[ G_{EW} = \frac{0.21}{0.81} \times 102 = 26 \text{ sec} \]

\[ G_s = 38 - 1 = 37 \text{ sec} \]

\[ G_{EW} = 26 - 1 = 25 \text{ sec} \]
Design hour traffic flows at 4 legs 3 phase intersection are given in the table below. The intergreen period of 5 secs, and the starting delay of 28 secs. per green intervals are used with fixed amber period of 3 secs. The setting of the phases as follows:

Phase (1) East and West for straight and right turn movements

Phase (2) East and West, left turn traffic where they have their own lanes.

Phase (3) North and South, all direction movements of traffic
\[
\begin{align*}
G_1 &= \frac{0.332}{0.715} \times 69 \approx 32^\circ \quad \Rightarrow \quad G_1 = 3 - \frac{3}{4} (6 + 5) = 1.2 - 1.5 = 1.5 \ \text{sec} \\
G_{11} &= \frac{0.172}{0.715} \times 69 \approx 17\% \\
G_{111} &= \frac{0.21}{0.715} \times 69 \approx 20\% \\
G_{III} &= 20 - 1 = 19\% \ \text{sec}
\end{align*}
\]

\( b \)
\[
G_{III} = 200 \ \text{sec} \quad s = 3832.5 \ \text{pcau/hr}
\]
\[
\text{max no. & pcau} = \frac{20 \times 3832.5}{3600} = 21.3 \ \text{pcau}
\]

Since all vehs. are passenger cars.

\( \text{No. of vehs.} = 21.3 \ \text{veh.} \)
Flow and directions in pcv/kr

<table>
<thead>
<tr>
<th></th>
<th>left</th>
<th>straight</th>
<th>right</th>
<th>width</th>
</tr>
</thead>
<tbody>
<tr>
<td>North approach</td>
<td>100</td>
<td>445</td>
<td>178</td>
<td>2 x 3.65 m</td>
</tr>
<tr>
<td>South approach</td>
<td>68</td>
<td>390</td>
<td>111</td>
<td>2 x 3.65 m</td>
</tr>
<tr>
<td>East approach</td>
<td>209</td>
<td>460</td>
<td>60</td>
<td>2 x 3.5 m</td>
</tr>
<tr>
<td>West approach</td>
<td>275</td>
<td>550</td>
<td>75</td>
<td>2 x 3.5 m</td>
</tr>
</tbody>
</table>

a) Determine the cycle time that gives the minimum delay over all the intersection and the actual green periods per phase.

b) Estimate the max. No. of vehe that can pass the stop lines during the time of phase (s).

\[ L = \frac{2(16-1)}{1-y} = \frac{(5-1) \times 3}{1-0.715} = 12 \text{ sec} \]

\[ C_0 = \frac{1.5L + 5}{1-Y} = \frac{1.5 \times 12 + 5}{1-0.715} = 80.4 \Rightarrow 81 \text{ sec} \]

Total effective green time = 81 - 12 - 60 sec
Parking surveys

The effect of parking

The effect of parked vehs on effective road width

Parking
Parking requirements

1. The Car driver(s)
2. Shop keepers
3. Public transport operators
4. Commercial vehicle drivers
5. Through traffic drivers
6. Car park operators
7. The traffic engineer:

- **The inventory of parking space**

  - **On-street Parking**
  - **Off-street Parking**

  a. Officially authorized metered parking spaces
  b. Officially authorized parking spaces with some form of time control
  c. Officially authorized parking spaces controlled by residents' permits of various types.
3. Officially authorized but uncontrolled parking spaces

4. Sections of street with no control or restriction on parking

5. Sections of street with parking prohibited at certain times

6. Public-owned or private-owned

7. Parks designated for park-and-ride systems

8. Parking space available to the general public or on payment

9. Ground- or multi-storey

10. No restrictions (on parking)

11. Public-owned or private-owned

12. Parks designated for park-and-ride systems

a. Parking space available to the general public or on payment

b. Parking space available only to specific people

1. Small and large PNR parks

2. Spaces within park reserved for visitors and/or commercial vehicles and those for employees

3. Spaces for offices and/or factories and spaces for shoppers

4. Reserve spaces for employees, etc.
Turn over

Turnover

\[ T = \frac{\text{No. of different veh. parked}}{\text{No. of Parking spaces}} \]

\[ = \frac{30}{14} = 2.15 \]

Space-hour demand = parked veh. hours

طلب ساعات اليدارات بالساعات

\[ 2 \frac{1}{2} \text{ hr} = 2.5 \text{ hr} \]

الوقت 2.5 ساعة لكل اليدارات متوفر ذا في البارك

المماذ
Park-veh-hours = 47½
Peak duration = 1-2 hrs or 47½

Space = No. of vechs / No. of vechs

Parking Index = No. of cars parked / no. of spaces available x 100
Parking control has three underlying purposes not all of which are appropriate in all circumstances:

a) to make the best use of the existing road capacity
   reducing obstructions to movement
b) to accommodate parking demand as efficiently as possible
c) to restrain travel demand, particularly in congested areas, to a level that can be accommodated

Parking
- on-street parking
- off-street parking
  - surface car parks
  - multi-story car parks

On-street parking

On-street parking can be alongside and parallel to the kerb, at right angles to or diagonally on to the kerb, or in the centre of the road.
For the angle parking:

**Advantages**
- More convenient for the motorist.
- Require less manoeuvring into and out of the space.

**Disadvantages**
- Cause a higher accident rate.

- Unusual area in the parking spaces more than on the parallel parking.

- Reduce the road capacity more than parallel parking.

- An angle parked car will project into a street by 4.5 m while in parallel 2 m only.

- Its worth in the centre of a wide, very lightly trafficked road or possibly in a city square.

---

**Diagram:**

- Angle parking: 6 cars in 17.5 m of kerb length but taking up 4.5 m of road width.

- Parallel parking: 4 cars in 22 m of kerb length and only taking up 2 m of road width.

---

Where to control:
- There is no need to control parking along quiet residential streets.

- There is little doubt that parking on busy shopping streets should be controlled in some way if not prohibited.

- In larger town there may be a case for parking control to persuade car commuters to switch to bus travel.

- In small town it may be more necessary to get parked cars off the main street altogether in order to ease the passage of through traffic.
on-street parking control methods

There are three basic ways of controlling on-street parking

1. **by time restriction**

   - Space: Long with parking prohibited
   - Capacity: Area wide with parking permitted

   **Advantages**
   1. More cars can be parked in a given length than with meters
   2. Negligible cost of introduction
   3. No unsightly street furniture (no visual intrusion)

   **Disadvantages**
   1. Enforcement is difficult
   2. Cost of enforcement is not recouped (revenue only from fines)
**by parking meters**

length of metered parking bays = 6 m

The idea of using meters is to ensure that the parking spaces are used for their intended purpose and to manage the flow of traffic. The main advantages are:

1. Clear indication of parking space and adequate size for nearly all vehicles.
2. Readily enforceable with minimum staff.
4. Flexible:
   -ジュルブンナルアラル
5. Ease of temporary prohibition.
6. Universally understood.
7. Encourage quick turnover.

The disadvantages include:

1. Initial cost of installation.
2. Continuing cost of wardens on regular patrols to prevent meter-fraud.
3. Need for regular revenue collection.
4. Susceptibility to vandalism and theft.
5. Unsightly street furniture (visual intrusion).

**Price per unit time**
Day Parking disc or Other display Card.

Although many of these systems are employed in urban areas, the following advantages are given:

- **Advantages**
  1. Negligible cost of introduction
  2. No unsightly street furniture
  3. Flexibility
  4. More cars can be parked in a given length of road
  5. Visual indication of arrival and departure times

**Disadvantages**

- Continuing cost of wardens on regular patrol to check disc settings
- No revenue with a reduced number of vehicles
- Visiting motorists may not understand system

Off-street parking can be classified by availability and controllability:

- Publicly available, publicly controlled
- Publicly available, privately controlled
- Private - restricted availability

Through trip car parking
- Trips to work by car
- Shopping trips by car
- Delivery trips by car
- All waste, recycling or public transport trips

Restrain

Discourage

where to impose restraint

neighborhood
small town
large town
shopping centers
destinations
off-street parking control methods

Three basic methods of controlling

1. **control by space** - the ultimate method
   - Where using the manoeuvre, the motorists have difficulty
     parking

2. **control by time** - in a town centre
   - After certain time, motorists have difficulty
     parking

3. **control by area** - in a town centre
   - After certain time, motorists have difficulty
     parking
Parking Design

Assessing parking demand

From the relationships between the number of car trips and trips per vehicle (vehicle travel demand) [1] and the number of parking spaces (parking demand) [2], we can determine the parking demand. This can be done by simply dividing the total number of vehicles by the total number of parking spaces.

Let's assume an unrestrained demand.

Now, we can see how many parking spaces are needed for each type of building. For example:

- Shopping centers: 1 car space per 20-120 m² of gross floor space
- Offices: 1 car space per 30-100 m² of gross floor space
- Hotels: 1 car space per 4-5 bedrooms
- Cinemas: 3-10 seats
- Theatres: 1 car space per 3-10 seats
- Restaurants: 1 car space per 3-10 seats

The total parking spaces as a function of the total floor area by purpose.

There are some factors that need to be considered when determining the size, location, and number of parking spaces required for different types of buildings. These factors include:

1. In small towns, new parking spaces should ideally be situated within about 400 m of the service that the parkers wish to visit.

2. In urban areas, the parking demand is generally higher due to the higher number of trips per day.
To locate the ideal site for a new park the demands in various small areas can be multiplied by their distance from a fixed point (as in calculating moments), and hence determining the "parking centroid" -- the ideal location.

\[
\text{Parking centroid} = \frac{20 \times 100 + 20 \times 100}{20 + 20} = \frac{4000}{40} = 100
\]

... etc...

2. The need to ensure commercial viability

The choice will usually depend on land costs.
Parking can also be differentiated by who actually parks the car— the driver himself or a car park attendant.

Mechanical or ramped parks

- Invariably operated with attendant
- Can be built at lower floor levels. Ceiling heights are not an extravagance.
- Cars parked much closer together.
- The car automatically lifts and moves, sideways, into a predetermined spot.
- There is no need for walls in the park.

The major disadvantage of mechanical car stacking is the possibility of mechanical failure and consequent inability to extract parked cars.

- Fig 10-1 Types of multistorey car park

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Design principles

Proto-structural comments

1. Split level: Level floors used for parking connected by short ramps.

2. Parking ramp: Sloping floors which are used for parking.

3. Warped slab: Level floors used.
Layout of parking space

Cars are preferably parked in marked stalls, minimum size for public use: \(4.8 \times 2.4 \text{ m}\) depth marked \(2.5 \text{ m} \) only

- min. width of one-way driveway = 6 m which parking stalls abut at right angles
- each driveway serve two rows of stalls one on each side
- min. bin width (stall-driveway-stall) = 15.6 m
  
  2 parallel bins
  
  About 25 m² per car including circulation space

- 70 layout of stalls used in multistorey car park
- 45° layout of stalls is advantage in surface car park

- In surface car parks, sacrifice some parking space to aid overall movement
Carpark entry and exit

The system is:

- Free entry, Variable time charge, Payment on exit

By entering the car park area, the vehicle is assigned a ticket which it must keep at all times. In case of damage, the vehicle is held responsible for the payment of any repair costs. Any breach of the rules will result in a fine of £500, payable immediately.

A bottle neck may be experienced during peak times, especially at weekends when car parks are at capacity. To avoid congestion, it is recommended that motorists arrive early to minimize queuing times.

Factors affecting the design of Interchange

1. Topography of the site
2. Traffic projection and character
3. Land availability
4. Impacts of the surrounding area
5. Overall environment
6. Economic viability
7. Financial constraints

Spacing of the interchange should be 8 km and not less than 3 km in rural area, 20 km in urban area not less than 20 km.
Functions of Interchange:

- Provide grade separation between two or more traffic arteries.
- To make possible the easy transfer from one artery to the other or between local street and freeway.

Interchange types:

A: Minor interchanges: They are between two roads of which at least one is not a freeway usually a freeway and a highway. Such as the cloverleaf, trumpet, Diamond (E.5.1h, E.5.1f, E.5.1a).

B: Major interchanges: They are between two freeways. Turning movement take place on direct ramps. 3 level interchange or 4 level interchange (E.5.2e, E.5.2d).
Diamond interchange

adapted to situations
where a Freeway crosses
a non-Freeway arterial

- simplest
- least costly

4 - one way sliproad

4 - one bridge

Freeway

Fig 9-18
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Partial clover leaf

- Left turn through opposing traffic on minor artery.
- Can take different forms to fit topography and traffic patterns.

Major

Minor

Interchange

Large area of land

Left turners must execute a 90° right turn and travel substantially greater than

Weaving
Cloverleaf interchange with collector-distributor roads

- can be applied to one or both through roadway if the costs of added land paving, and structures can be justified.

- Weaving and merging movement are separated.

If weaving volumes are large, the weaving section must be long. This distorts the shape and increases the land area required for a cloverleaf.

The collector-distributor road provides an opportunity for speed adjustment clear of the freeway.
A) Make one directional minor movement enter only.

- Right and left turn as directed.

- Enter on work side only.

- Move to freeway.

B) Make all turning movements.

- Right and left turn only.

- Enter on work side only.

- Move to freeway.
Y-shaped interchange

- One bridge
  - Major/Major
  - Major/Minor

\[ \begin{align*}
\text{Y for angle} &< 75^\circ \quad \text{or} \quad \text{or} \\
75^\circ - 105^\circ & \quad \text{or} \quad \text{or}
\end{align*} \]

T for angle between

- or trumpet

One bridge

Major/Major

Major/Minor

Traffic from lower to upper left must traverse a 270 turn

All other movements are accomplished with curvatures not much greater than 90°

- minor/major
  - considerable turning movement

Multi-way intersection

or Rotary

for low traffic only to pick up and discharge from several streets
advantages
- this type provides a relatively simple solution for traffic intersections with four or more approaches where speed and volumes are not high.

disadvantages
- large property requirements.
- the weaving sections limit the speed and capacity.
- the directional signing is difficult unless the diameter of the circle is large enough to provide adequate length in the weaving section.
Advantages
- Provides a relatively high-speed semi-direct movement for heavy turning volume of traffic.
- A single structure is required.
- No weaving.
- High capacity as all movements are free flow.
advantages

- all left turn conflicts eliminated in this simple structure design
- traffic signals are unnecessary.
- traffic movements are continuous and natural
- may be built in stages if necessary.

disadvantages

- long property requirements.
- weaving both on the freeway and the motor road may decrease motor capacity.
- weaving both on the freeway may create conflicts.
- weaving both on the freeway may require more effort to detect.
- insufficient length for deceleration from freeway speed to control speed of motor cars
- road safety features
advantages:
- High standard single ramp design of the structure.
- High standard single ramp design of the structure.
- Economical in principle and construction costs.
- Where the freeway is located, exit and entrance ramps assist the deceleration of exiting traffic and the acceleration of entering traffic.
- Single exit features a direct connection to the freeway.
- No need for spray painting the freeway under the structure.
- No weaving on the structure.

disadvantages:
- Lower capacity of the single ramp design of the turning movements.
- Difficulty of obtaining the clearances at the open throat ramp terminals especially where the minor road crosses over the freeway.
- Many points of conflict are created, increasing the accident potential of the design, unless signalized.
- Possibility of wrong way turning traffic from the freeway.
- Turning traffic from the freeway onto the minor road, storage lane treatment may be required.
- Little possibility of allowing for two-way traffic at the interchange but increased volumes may be handled by:
  (a) channelizing the other traffic
  (b) installing signals on the approach
  (c) providing two lane left turn
directional interchange
with right hand exits and entrances
F.5.2e directional interchange with right hand exits and entrances

- Used when freeways meet freeways and all traffic movements are heavy
- Used when land is expensive
- All left turns are direct (90°)