JCE 3460
Transportation Engineering

Introduction to Traffic Flow

Topic Areas

- Traffic Flow
  - Highway Capacity Analysis
  - Freeway Analysis
  - Intersection Capacity Analysis

- MUTCD
  - Principles
  - Signs
  - Pavement Markings
  - Design Considerations
  - Traffic Signal Warrants

- Transportation Studies
  - Safety
  - Financing
Ice Breaker

Basic Flow Fundamentals
Defining Traffic Flow

- **ADT (Average Daily Traffic)**
  - Total volume occurring in a continuous 24 hour period
  - Does not indicate traffic variations

- **AADT (Average Annual Daily Traffic)**
  - Total traffic year calculated to represent a single day value
  - More accurately represents actual/average traffic levels

- **DHV (Design Hourly Volume)**
  - Typically 30th highest hourly volume for year
  - Hourly Volume Graphs (Rural versus Urban)

- **K factor**
  - Percentage of daily traffic occurring during the peak hour
  - 8 – 12% in urban, 12 – 18% in rural areas

- **D (Directional factor)**
  - Percentage of traffic traveling in the heavier direction
  - 60 – 80% in rural/suburban, almost 50% in CBD areas

- **Peak Hour Factor (PHF)**

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Design Hourly Volume

EXHIBIT 8-8. RANKED HOURLY VOLUMES

- Recreation Access Route MN 169
- Main Rural Route I-35
- Urban Circumferential Freeway I-94
- Urban Radial Freeway I-35E

Source: Minnesota Department of Transportation.
K-Factor

- Analysis Hourly Volume/AADT
- Normally 8-12%
- Typically
  - the higher the volumes, the lower the K-factor
  - the more urbanized the area, the lower the K-factor
Peak Hour Factor (PHF)

- Typically, facilities are designed for peak 15 minute flow interval
- 15 minute flows are accounted for through the Peak Hour Factor (PHF)
- Definition:
  \[
  \text{PHF} = \frac{\text{Hourly Volume}}{\text{Peak 15 Minute Volume} \times 4}
  \]
- Typically range from 0.75-0.98
- What is the highest possible value?
- What is the lowest possible value?

Capacity

“…the maximum hourly rate at which persons or vehicles can be expected to traverse a point or a uniform section of a lane or roadway given a time period under prevailing roadway, traffic, and control conditions.”

(HCM 2000, p 2-2)
What Factors Determine Capacity?

- Facility Type
  - Freeway, Rural Highway, Multimodal
- Roadway Conditions
  - Weather, Pavement Condition, Geometric Design
- Traffic Conditions
  - Vehicle Mix (e.g., Trucks, RV, SUV, Taxis)
  - Driver Mix (e.g., Commuter versus Recreational Traffic, Geographic location)
- Traffic Control
  - Traffic Signals, ITS, Work Zones

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Sample Speed/Flow Curve

EXHIBIT 22-7: SPEED-FLOW CURVES FOR DIFFERENT WEATHER CONDITIONS

Note:
ITS = 75 mi/h (base conditions).
Demand

“...the principal measure of the amount of traffic using a given facility. Demand relates to vehicles arriving; volume relates to vehicles discharging. If there is no queue, demand is equivalent to the traffic volume at a given point on the roadway.” (HCM 2000, p 2-2)

How to Measure Demand?

- Uncongested Facilities
- Congested Facilities
  - Latent Demand and System Impacts
- Route 40 Improvements at Missouri River Crossing
  - WB bridge - 32’ wide, built 1935, 2 lanes to 3
  - EB bridge is 48’ wide, built 1985, 3 lanes to 4
  - Volumes increased by 30% almost overnight
- It is difficult to measure demand in a congested network. Travel Demand Modeling (TDM) techniques have been developed to study demand in urban areas.
Performance Measures

- Speed
- Density
- Delay
- Level of Service

Speed (HCM 7-3)

- **Average running speed** — A traffic stream measure based on the observation of vehicle travel times traversing a section of highway of known length. It is the length of the segment divided by the average running time of vehicles to traverse the segment. Running time includes only time that vehicles are in motion.

- **Average travel speed / Space mean speed** — A traffic stream measure based on travel time observed on a known length of highway. It is the length of the segment divided by the average travel time of vehicles traversing the segment, including all stopped delay times. It is also a space mean speed. It is called a space mean speed because the average travel time weights the average to the time each vehicle spends in the defined roadway segment or space.

- **Time mean speed** — The arithmetic average of speeds of vehicles observed passing a point on a highway; also referred to as the average spot speed. The individual speeds of vehicles passing a point are recorded and averaged arithmetically.

- **Free-flow speed** — The average speed of vehicles on a given facility, measured under low-volume conditions, when drivers tend to drive at their desired speed and are not constrained by control delay.
Delay

- Several Types of Delay
  - Control
  - Geometric
  - Total
- Expressed as seconds per vehicle
- Critical parameter in interrupted flow facilities (e.g., traffic signals)

Level of Service

- Traffic Service Levels A-F
  - A is “free flow” conditions
  - E is capacity conditions
  - F is breakdown conditions
  - C, D are USUALLY design criteria
- Criteria to determine is function of facility type
  - Freeways – Density
  - Intersections - Delay
Freeways

Traffic Flow

- Traffic flow is a rate
  - typically expressed in vehicles per hour (vph)
- Traffic volume is a number
  - vehicles that pass by a point in a given period of time
- Traffic flow is usually expressed as vph, but is usually expressed from a 15 minute volume through the use of a PHF
Traffic Density (HCM, p7-4)

- The number of vehicles occupying a section of roadway
- Usually averaged over time
- Usually expressed as vehicles or passenger cars per mile.
- Direct measurement of density in the field is difficult.
  Density can be computed from the average travel speed and flow rate.

\[ D = \frac{v}{s} \]

where

- \( v \) = flow rate (veh/h),
- \( s \) = average travel speed (mi/h), and
- \( D \) = density (veh/mi).

Headway and Spacing
(HCM, p7-4)

- Spacing is the **distance** between successive vehicles, measured from the same point on each vehicle
- Headway is the **time** between successive vehicles, also measured from the same point on each vehicle
Relationships

- Definitions
  - Speed (distance/time)
  - Density (vehicles/distance)
  - Spacing (distance/vehicle)
  - Headway (time/vehicle)
  - Flow (vehicles/time)

- Relationships
  - Spacing and Density are Inverse
  - Headway and Flow are Inverse
  - Speed * Density = Flow

Simple Car Following Theory

- If you remember from physics…
  \[ x = v_0 t + \frac{1}{2} at^2 \]
  - \( x \) = distance traveled during acceleration
  - \( v_0 \) = initial velocity
  - \( t \) = time
  - \( a \) = acceleration

  \[ x = \frac{(v^2 - v_0^2)}{2a} \]
  - \( v \) = final velocity
Applied to a Simple Car Following Model

\[
s = \left( v_{t\text{pr}} + v_f^2/2a_f + x_o + L \right) - v_l^2/2a_l
\]

**Resulting Speed/Flow Curve**

Theoretical Speed / Flow Curve

![Theoretical Speed / Flow Curve Diagram](image-url)
Field Speed / Flow Plots

2-lane German Freeway

5-lane US Freeway

Empirically Derived Speed Flow Curve (HCM, p13-3)
Basic Freeway Segments

HCM Applications

- What is a Freeway?
  - divided highway with full control of access
  - two or more lanes for the exclusive use of traffic in each direction
  - no signalized or stop-controlled at-grade intersections
  - direct access to and from adjacent property is not permitted
  - access to and from the freeway is limited to ramp locations
  - opposing directions of flow are continuously separated by a raised barrier, an at-grade median, or a continuous raised median

- Areas of Freeway Analysis
  - Basic Freeway Segments - Chapter 23
  - Freeway Weaving - Chapter 24
  - Ramps and Ramp Junctions - Chapter 25
  - Freeway Facilities - Chapter 22
Basic Freeway Segments

FLOW CHARACTERISTICS

- **Under saturated flow**
  - unaffected by upstream or downstream conditions
  - generally defined within a speed range of 55 to 75 mi/h at low to moderate flow rates and a range of 45 to 60 mi/h at high flow rates

- **Queue discharge flow**
  - traffic flow that has just passed through a bottleneck and is accelerating back to the FFS
  - relatively stable as long as the effects of another bottleneck downstream are not present
  - generally defined within a narrow range of 2,000 to 2,300 pc/h/ln, with speeds typically ranging from 35 mi/h up to the FFS of the freeway segment
  - depending on horizontal and vertical alignments, queue discharge flow usually accelerates back to the FFS of the facility within 0.5 to 1 mi downstream from the bottleneck
  - the queue discharge flow rate from the bottleneck is lower than the maximum flows observed before breakdown. A typical value for this drop in flow rate is approximately 5 percent

- **Oversaturated flow**
  - traffic flow that is influenced by the effects of a downstream bottleneck
  - traffic flow can vary over a broad range of flows and speeds depending on the severity of the bottleneck
  - queues may extend several thousand feet upstream from the bottleneck
Calculation Procedures

Base (Optimal) Conditions

- 12 ft lane widths
- 6 ft right-shoulder lateral clearance
- 2 ft median lateral clearance
- passenger cars only
- 5 or more lanes (in urban areas only)
- 2 mi or greater interchange spacing
- Level terrain (2 percent maximum grades)
- regular user driver population
- free-flow speed (FFS) of 70 mi/h or greater
Calculation of Free Flow Speed

\[ \text{FFS} = \text{BFFS} - f_L - f_C - f_N - f_D \]

- **FFS** = free-flow speed
- **BFFS** = base free-flow speed; 70 mi/h (urban) or 75 mi/h (rural)
- \( f_L \) = lane width adjustment; Exhibit 23-4
- \( f_C \) = right-shoulder lateral clearance adjustment; Exhibit 23-5
- \( f_N \) = number of lanes adjustment; Exhibit 23-6 (does NOT apply in rural areas)
- \( f_D \) = interchange density adjustment; Exhibit 23-7

Lane Width and Lateral Clearance

- Standard Freeway
  - Lane Width = 12 feet
  - Shoulder Width = 10 feet
Calculation of Free Flow Speed

\[ FFS = BFFS - f_{LW} - f_{LC} - f_N - f_ID \]

- **FFS** = free-flow speed
- **BFFS** = base free-flow speed; 70 mi/h (urban) or 75 mi/h (rural)
- **f_{LW}** = lane width adjustment; Exhibit 23-4
- **f_{LC}** = right-shoulder lateral clearance adjustment; Exhibit 23-5
- **f_N** = number of lanes adjustment; Exhibit 23-6 (does NOT apply in rural areas)
- **f_ID** = interchange density adjustment; Exhibit 23-7

### Exhibit 23-4. Adjustments for Lane Width

<table>
<thead>
<tr>
<th>Lane Width (ft)</th>
<th>Reduction in Free-Flow Speed, ( f_{LW} ) (mi/h)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>0.0</td>
</tr>
<tr>
<td>11</td>
<td>1.9</td>
</tr>
<tr>
<td>10</td>
<td>6.6</td>
</tr>
</tbody>
</table>

### Exhibit 23-5. Adjustments for Right-Shoulder Lateral Clearance

<table>
<thead>
<tr>
<th>Number of Lanes</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>( \geq 5 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>0.0</td>
<td>0.4</td>
<td>0.2</td>
<td>0.1</td>
</tr>
<tr>
<td>4</td>
<td>1.2</td>
<td>0.8</td>
<td>0.4</td>
<td>0.2</td>
</tr>
<tr>
<td>3</td>
<td>1.8</td>
<td>1.2</td>
<td>0.6</td>
<td>0.3</td>
</tr>
<tr>
<td>2</td>
<td>2.4</td>
<td>1.6</td>
<td>0.8</td>
<td>0.4</td>
</tr>
<tr>
<td>1</td>
<td>3.0</td>
<td>2.0</td>
<td>1.0</td>
<td>0.6</td>
</tr>
<tr>
<td>0</td>
<td>3.6</td>
<td>2.4</td>
<td>1.2</td>
<td>0.6</td>
</tr>
</tbody>
</table>
Number of Lanes

- As the number of lanes increases, so does the opportunity for drivers to position themselves to avoid slower moving traffic.
- In typical freeway driving, traffic tends to be distributed across lanes according to speed. Traffic in the median lane typically moves faster than in the right lane.
- A four-lane freeway (two lanes in each direction) provides less opportunity for drivers to move around slower traffic than does a freeway with 6, 8, or 10 lanes.
- Decreased maneuverability tends to reduce the average speed of vehicles.
- Factor DOES NOT APPLY in rural areas.
- Both basic and auxiliary lanes should be considered (weaving).
- HOV lanes should not be included.

Calculation of Free Flow Speed

\[ FFS = BFFS - f_{LW} - f_{LC} - f_{N} - f_{ID} \]

- **FFS =** free-flow speed
- **BFFS =** base free-flow speed; 70 mi/h (urban) or 75 mi/h (rural)
- **\( f_{LW} \) =** lane width adjustment; Exhibit 23-4
- **\( f_{LC} \) =** right-shoulder lateral clearance adjustment; Exhibit 23-5
- **\( f_{N} \) =** number of lanes adjustment; Exhibit 23-6 (does NOT apply in rural areas)
- **\( f_{ID} \) =** interchange density adjustment; Exhibit 23-7

<table>
<thead>
<tr>
<th>Number of Lanes (One Direction)</th>
<th>Reduction in Free-Flow Speed, ( f_{N} ) (mi/h)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \geq 5 )</td>
<td>0.0</td>
</tr>
<tr>
<td>4</td>
<td>1.5</td>
</tr>
<tr>
<td>3</td>
<td>3.0</td>
</tr>
<tr>
<td>2</td>
<td>4.5</td>
</tr>
</tbody>
</table>

Note: For all rural freeway segments, \( f_{N} \) is 0.0.
Interchange Density

- Freeway segments with closely spaced interchanges, such as those in heavily developed urban areas, operate at lower FFS than suburban or rural freeways where interchanges are less frequent.
- The merging and weaving associated with interchanges affect the speed of traffic. Speeds generally decrease with increasing frequency of interchanges.
- The ideal average interchange spacing over a reasonably long section of freeway (5 to 6 mi) is 2 mi or greater.
- The minimum average interchange spacing considered possible over a substantial length of freeway is 0.5 mi.
- FHWA usually requires 1 mi spacing between interchanges in urban areas.

Calculation of Free Flow Speed

\[ \text{FFS} = \text{BFFS} - f_{\text{LW}} - f_{\text{LC}} - f_{\text{N}} - f_{\text{ID}} \]

- **FFS** = free-flow speed
- **BFFS** = base free-flow speed; 70 mi/h (urban) or 75 mi/h (rural)
- **f_{\text{LW}}** = lane width adjustment; Exhibit 23-4
- **f_{\text{LC}}** = right-shoulder lateral clearance adjustment; Exhibit 23-5
- **f_{\text{N}}** = number of lanes adjustment; Exhibit 23-6 (does NOT apply in rural areas)
- **f_{\text{ID}}** = interchange density adjustment; Exhibit 23-7

| Exhibit 23-7. Adjustments for Interchange Density |
|---------------------------------|------------------|
| Interchanges per Mile          | Reduction in Free-Flow Speed, \( f_{\text{ID}} \) (mi/h) |
| 0.50                           | 0.0              |
| 0.75                           | 1.3              |
| 1.00                           | 2.5              |
| 1.25                           | 3.7              |
| 1.50                           | 5.0              |
| 1.75                           | 6.3              |
| 2.00                           | 7.5              |
Free Flow Speed Example

- Interchange Under Construction
  - 3 Lanes direction
  - 11’ Lanes
  - 2' Right Shoulders
  - 2 Interchanges in 3 Miles
  - Urban Area (BFFS = 70 MPH)

- FFS = BFFS – $f_{LW} – f_{LC} – f_N – f_{ID}$

Calculation of Flow Rate

\[ v_p = \frac{V}{PHF \times N \times f_{HV} \times f_p} \]

where
- $v_p$ = 15-min passenger-car equivalent flow rate (pc/h/ln),
- $V$ = hourly volume (veh/h),
- $PHF$ = peak-hour factor,
- $N$ = number of lanes,
- $f_{HV}$ = heavy-vehicle adjustment factor, and
- $f_p$ = driver population factor.
Heavy Vehicles

- Heavy Vehicles induce frequent gaps of excessive length both in front of and behind themselves.
- The speed of vehicles in adjacent lanes and their spacing may be affected by these generally slower-moving large vehicles.
- Physical space taken up by a large vehicle is typically two to three times greater in terms of length than that taken up by a typical passenger car.

Calculation of Flow Rate

\[ v_p = \frac{V}{PHF \cdot N \cdot f_{HV} \cdot f_p} \]

where

- \( v_p \) = 15-min passenger-car equivalent flow rate (pc/h/ln),
- \( V \) = hourly volume (veh/h),
- \( PHF \) = peak-hour factor,
- \( N \) = number of lanes,
- \( f_{HV} \) = heavy-vehicle adjustment factor, and
- \( f_p \) = driver population factor.

\[ f_{HV} = \frac{1}{1 + \frac{P_T(E_T - 1)}{P_R(E_R - 1)}} \]  (23-3)

where

- \( E_T, E_R \) = passenger-car equivalents for trucks/buses and recreational vehicles (RVs) in the traffic stream, respectively;
- \( P_T, P_R \) = proportion of trucks/buses and RVs in the traffic stream, respectively; and
- \( f_{HV} \) = heavy-vehicle adjustment factor.
Heavy Vehicle Factor

\[ f_{HV} = \frac{1}{\tau + P_T(E_T - 1) + P_R(E_R - 1)} \]  

(23-3)

where

- \( E_T, E_R \) = passenger-car equivalents for trucks/buses and recreational vehicles (RVs) in the traffic stream, respectively;
- \( P_T, P_R \) = proportion of trucks/buses and RVs in the traffic stream, respectively;
- \( f_{HV} \) = heavy-vehicle adjustment factor.

### Exhibit 23-9. Passenger-Car Equivalents on Extended Freeway Segments

<table>
<thead>
<tr>
<th>Factor</th>
<th>Level</th>
<th>Rolling</th>
<th>Mountainous</th>
</tr>
</thead>
<tbody>
<tr>
<td>( E_T ) (trucks and buses)</td>
<td>1.5</td>
<td>2.5</td>
<td>4.5</td>
</tr>
<tr>
<td>( E_R ) (RVs)</td>
<td>1.2</td>
<td>2.0</td>
<td>4.0</td>
</tr>
</tbody>
</table>

Level Terrain - heavy vehicles maintain the same speed as passenger cars
Rolling Terrain - heavy vehicles reduce speeds below passenger cars
Mountainous Terrain - heavy vehicles operate at crawl speeds

### Exhibit 23-3. Passenger-Car Equivalents for Trucks and Buses on Uplinks

<table>
<thead>
<tr>
<th>Upgrade (%)</th>
<th>Length (mi)</th>
<th>Percentage of Trains and Buses</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 2</td>
<td>All</td>
<td>1.5  1.5  1.5  1.5  1.5  1.5  1.5  1.5</td>
</tr>
<tr>
<td>&gt; 2-3</td>
<td>All</td>
<td>1.5  1.5  1.5  1.5  1.5  1.5  1.5  1.5</td>
</tr>
<tr>
<td>&gt; 3-4</td>
<td>All</td>
<td>1.5  1.5  1.5  1.5  1.5  1.5  1.5  1.5</td>
</tr>
<tr>
<td>&gt; 4-5</td>
<td>All</td>
<td>1.5  1.5  1.5  1.5  1.5  1.5  1.5  1.5</td>
</tr>
<tr>
<td>&gt; 5-6</td>
<td>All</td>
<td>1.5  1.5  1.5  1.5  1.5  1.5  1.5  1.5</td>
</tr>
<tr>
<td>&gt; 6</td>
<td>All</td>
<td>1.5  1.5  1.5  1.5  1.5  1.5  1.5  1.5</td>
</tr>
</tbody>
</table>

24
Driver Population

- Three Primary Driver Tasks
  - Control involves the driver’s interaction with the vehicle in terms of speed and direction (accelerating, braking, and steering)
  - Guidance refers to maintaining a safe path and keeping the vehicle in the proper lane.
  - Navigation means planning and executing a trip

- Studies have noted that non-commuter driver populations do not display the same characteristics as regular commuters. For recreational traffic, capacities have been observed to be as much as 10 to 15 percent lower than for commuter traffic traveling on the same segment, but FFS does not appear to be similarly affected.

Calculation of Flow Rate

\[ v_p = \frac{V}{PHF \cdot N \cdot f_{HV} \cdot f_p} \]

where

- \( v_p \) = 15-min passenger-car equivalent flow rate (pcv/ln/h),
- \( V \) = hourly volume (veh/h),
- \( PHF \) = peak-hour factor,
- \( N \) = number of lanes,
- \( f_{HV} \) = heavy-vehicle adjustment factor, and
- \( f_p \) = driver population factor.

- \( f_p \) values range from 0.85 to 1.00
- 1.00 reflects commuter traffic
- Comparative field studies of commuter and recreational traffic flow and speeds are recommended to determine lower values
Flow Rate Example

- Boone Bridge Traffic
  - 9500 vph
  - 5 lanes
  - PHF = 0.95
  - 12% Trucks
  - 0.2% RVs
  - Level Terrain
  - Urban / Local Users

- What if we were on a 6% uphill grade for 0.55 miles?

Look up LOS: Calculate Density

\[
D = \frac{V_p}{S}
\]

where

- \( D \) = density (pcu/mi/ln)
- \( V_p \) = flow rate (pcu/h/ln), and
- \( S \) = average passenger-car speed (mi/h)

<table>
<thead>
<tr>
<th>LOS</th>
<th>Density Range (pcu/mi/ln)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0–11</td>
</tr>
<tr>
<td>B</td>
<td>11–18</td>
</tr>
<tr>
<td>C</td>
<td>18–26</td>
</tr>
<tr>
<td>D</td>
<td>26–35</td>
</tr>
<tr>
<td>E</td>
<td>35–45</td>
</tr>
<tr>
<td>F</td>
<td>&gt; 45</td>
</tr>
</tbody>
</table>
Example Problem

EXAMPLE PROBLEM 1

The Freeway  Existing four-lane freeway, rural area, very restricted geometry, rolling
terrain, 70-mph speed limit.

The Question  What is the LOS during the peak hour?

The Facts

- Two lanes in each direction,
- 11-ft lane width,
- 2-ft lateral clearance,
- Commuter traffic,
- 2,000-veh/h peak-hour volume
  (one direction),
- 5 percent trucks,
- 0.92 PHF,
- 1 interchange per mile, and
- Rolling terrain.

Comments

- Assume 0 percent buses and RVs since none are indicated.
- Assume BFFS of 75 mph for rural areas.
- Assume that the number of lanes does not affect free-flow speed, since the
  freeway is in a rural area.
- Assume $T = 1.00$ for commuter traffic.
Example Problem 2

The Freeway  New suburban freeway is being designed.
The Question  How many lanes are needed to provide LOS D during the peak hour?
The Facts
✓  4,000 veh/h (one direction), ✓  0.85 PHF,
✓  Level terrain, ✓  1.50 interchanges per mile,
✓  15 percent trucks, ✓  3 percent RVs, and
✓  12-ft lane width, ✓  6-ft lateral clearance.
Comments
✓  Assume commuter traffic. Thus, \( I_s = 1.00 \).
✓  Assume BFFS of 70 m/h.
✓  Assume that the number of lanes affects free-flow speed, since the freeway is
being designed in a suburban area.

Basic Freeway Segment Methodology

Limitations
- HOV, truck, and climbing lanes
- Extended bridge and tunnel segments
- Toll plaza segments
- FFS below 55 mph or above 75 mph
- v/c greater than 1
- Downstream blockages (Over Saturation)
- Posted speed limits / Police enforcement
- ITS
  - Advanced traveler information systems (ATIS)
  - Ramp metering
Freeway Weaving

Weaving Segment

- Definition
  - crossing of two or more traffic streams
  - traveling in the same general direction
  - along a significant length of highway
  - without traffic control devices (with the exception of guide signs)
- Weaving segments are formed when an on-ramp is closely followed by an off-ramp and the two are joined by an auxiliary lane.
- If a one lane on-ramp is closely followed by a one-lane off-ramp and the two are not connected by an auxiliary lane, the merge and diverge movements are considered separately using procedures for the analysis of ramp terminals.
- Weaving segments require intense lane-changing maneuvers. Traffic in a weaving segment is subject to turbulence in excess of that normally present on basic freeway segments.
Weaving Segment

Important Factors for Weaving Segments

- All Basic Freeway Segment Factors
  - Need to convert vph to pcph
  - Need to know (or calculate) FFS
- Weaving Type (A, B, C)
- Weaving Volumes
- Weaving Length
Weaving Segment Calculations

- Initial Information
  - Weaving Volumes and Characteristics
- Space Mean Speed
  - Weaving and non-weaving vehicles
- Proportional Use of Lanes
  - Unconstrained versus constrained operations
- Average Density
- LOS Determination
- Weaving Capacity

Initial Information

- Weaving Volumes
  - Same Factors as Basic Freeway Segments
- Maximum Weaving Length is 2500 feet
- Weaving Type
  - A, B, C
Type “A” Weave

- All weaving vehicles must make one lane change to complete their maneuver successfully

![Type A Weaving Segments]

Type “B” Weave

- One weaving movement can be made without making any lane changes
- The other weaving movement requires at most one lane change

![Type B Weaving Segments]
Type “C” Weave

- One weaving movement may be made without making a lane change, and
- The other weaving movement requires two or more lane changes.

What type of weave is this?
What type of weave is this?

Interchanges

- Grade separations provide greatest amount of operational efficiency at intersections.
- Two general types:
  - Systems interchange - connect controlled access highways;
  - Service interchange - connect a higher functional class road to a lower functional class road.
General Interchange Types

Traffic Signal Fundamentals
Agenda

- Flow Attributes
  - Saturation Flow, Lost Time
  - Clearance Intervals
- Phasing Schemes
- Splits
  - Queuing Theory
  - Capacity, Delay, and Queue Length
- Offset and Coordination

Major MOE

- Delay
- v/c (degree of saturation)
- % Stops
- Queue Length
  - 50%
  - 95%
Types of Traffic Signal Systems

- **Fixed Time**
  - Low Cost / Easy to Maintain
  - $150,000 per Signal
  - NOT Responsive to Changing Traffic Demands

- **Actuated**
  - $200,000 - $300,000 per Signal
  - Requires traffic detection
    - Loop detectors
    - Video detection
  - Fully Responsive to Traffic Demands at Intersection

- **Semi Actuated**
  - Responsive to Side-Street Demands at Intersection

- **Coordinated**
  - Requires Communication Between Controllers

- **Adaptive**
  - Requires Master Optimization and Detailed Algorithms
  - Intensive Traffic Detection and Communication
  - Responsive to Traffic Demands of Traffic Signal Systems

Major Timing Parameters

- **Cycle Length**
  - Time required to display complete sequence

- **Phasing**
  - Control turning movements

- **Splits**
  - Time allocated to a given movement relative to cycle

- **Controller Type**
  - Pre-timed, Actuated, Semi-actuated, Adaptive

- **Coordination/Offset**
  - Start of cycle at one intersection relative to start of cycle at adjacent intersection
Traffic Signal Cycle

Traffic Signal Coordination
Other Key Parameters

- Effective Green / Effective Red
- Saturation Flow Rate
- Lost Time = G+Y-g
  - Clearance Interval
  - Start-up Delay
  - Green Extension
- Time Space Diagram
- Capacity of the lane group = sat flow *g/C
among actual green, lost time, extension of effective green, and effective green is shown in Exhibit 10-10. When $t_1 = 2$ and $e = 2$ (typical), then $t_2 = \gamma_e$.

\[ t_2 = t_1 + t_2 = t_1 + \gamma_e = \gamma_e \]

(10-1)

**EXHIBIT 10-10. RELATIONSHIP AMONG ACTUAL GREEN, LOST-TIME ELEMENTS, EXTENSION OF EFFECTIVE GREEN, AND EFFECTIVE GREEN**

As shown in Exhibit 10-10, the lost time for the movement is deducted from the beginning of the actual green phase. Thus, a small portion of $G_0$ becomes part of the effective red, $t_1$. This portion is equal to the lost time for the movement, $t_1$. Because all of the lost time for the movement is deducted at the beginning of the green, effective green can be assumed to run through the end of the yellow-plus-all-red change and clearance interval, $Y_e$. Thus, for any given movement, effective green time is computed by Equation 10-2 and effective red time by Equation 10-3.

\[ g = G_0 + \gamma_e - t_1 \]  

(10-2)

\[ r = R_1 + t_2 \]  

(10-3)

---

**Start-up Lost time**

![Graph showing headway vs. vehicle in queue with saturation headway indicated.](image-url)
### Calculating Saturation Flow Rate

A line of vehicles at a signalized intersection begins to move with the initiation of the green signal. The following values represent the headway between the first 7 vehicles as they cross the intersection.

<table>
<thead>
<tr>
<th>Headway (sec)</th>
<th>Vehicle</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.5</td>
<td>2.9</td>
<td>2.5</td>
<td>2.2</td>
<td>2.0</td>
<td>2.0</td>
<td>2.0</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

What is the saturation flow rate and start-up delay for this approach?
Yellow and All Red
Clearance Times

- Yellow Times
  - Dilemma Zone Prevention
  - Typical Guidelines
    - 25 and lower = 3 seconds
    - 30-40 = 3-4 seconds
    - 40-50 = 4-5 seconds
    - 55 and greater = 5 seconds

- Red – Clearance Interval
  - Intersection Width and Speeds

Driver Dilemma Zone
### Driver Dilemma Zone at Various Speeds and Yellow Intervals

<table>
<thead>
<tr>
<th>Vehicle Speed (mph)</th>
<th>Stopping Distance &quot;Xs&quot; (ft)</th>
<th>Clearance Distance &quot;Xc&quot; (ft)</th>
<th>Dilemma Zone (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>3.2 Sec.</td>
<td>4.0 Sec.</td>
<td>5.0 Sec.</td>
</tr>
<tr>
<td>20</td>
<td>73</td>
<td>65</td>
<td>88</td>
</tr>
<tr>
<td>25</td>
<td>104</td>
<td>81</td>
<td>110</td>
</tr>
<tr>
<td>30</td>
<td>141</td>
<td>97</td>
<td>132</td>
</tr>
<tr>
<td>35</td>
<td>184</td>
<td>113</td>
<td>154</td>
</tr>
<tr>
<td>40</td>
<td>232</td>
<td>176</td>
<td>235</td>
</tr>
<tr>
<td>45</td>
<td>285</td>
<td>198</td>
<td>264</td>
</tr>
<tr>
<td>50</td>
<td>344</td>
<td>220</td>
<td>293</td>
</tr>
<tr>
<td>55</td>
<td>408</td>
<td>242</td>
<td>323</td>
</tr>
<tr>
<td>60</td>
<td>477</td>
<td>264</td>
<td>352</td>
</tr>
</tbody>
</table>

### Signal Phasing - (by direction)

![Signal Phasing Diagram]
Standard 8 Phase Sequence

Protected Only Phasing

- Opposing Speeds Higher than 40 mph
- Sight Distance Issues
- More than 2 Opposing Lanes
- Dual Left Turn Lanes
Lane Group Capacity
V/C Ratio

\[ c_i = \frac{g_i}{C} \]  \hspace{1cm} (16-6)

where

- \( c_i \) = capacity of lane group i (veh/h),
- \( g_i \) = saturation flow rate for lane group i (veh/h), and
- \( \frac{g_i}{C} \) = effective green ratio for lane group i.

V/C Ratio

The ratio of flow rate to capacity (v/c), often called the volume to capacity ratio, is given the symbol X in intersection analysis. It is typically referred to as degree of saturation. For a given lane group i, \( X_i \) is computed using Equation 16-7.

\[ X_i = \left( \frac{v_i}{c_i} \right) = \frac{v_i}{\frac{g_i}{C}} = \frac{v_i}{\frac{g_i}{C}} \]  \hspace{1cm} (16-7)

where

- \( X_i \) = (v/c)i = ratio for lane group i,
- \( v_i \) = actual or projected demand flow rate for lane group i (veh/h),
- \( g_i \) = saturation flow rate for lane group i (veh/h),
- \( \frac{g_i}{C} \) = effective green time for lane group i (s), and
- \( C \) = cycle length (s).

Cycle Length

- **Minimum Cycle Length**
  - **Minimum Green Times**
    - Minimum Perception Reaction and Start-up Time (5-7 Seconds)
  - Minimum g/C for Capacity
    - \( G_{max} = v/s^C \)
    - \( C_{max} = \text{sum} (G_{max}) + \text{Lost time} \)
      - (100% Saturation of Signal)
  - Minimum Pedestrian Clearance Times
  - Clearance Intervals

- **Optimal Cycle Length**
  - Longer cycle lengths offer higher capacities
  - Critical Lane Groups
  - Longer cycle lengths result in higher delays
  - Delay Equation
Optimal Cycle Length

\[ C_o = \frac{1.5L + 5}{1 - \sum (q/s)} \]

Minimum Green - Pedestrians

\[
G_p = \begin{cases} 
3.2 + \frac{L}{S_p} \left( \frac{2.7 N_{ped}}{W_E} \right) & \text{for } W_E > 10 \text{ ft} \\
3.2 + \frac{L}{S_p} \left( 0.27 N_{ped} \right) & \text{for } W_E \leq 10 \text{ ft}
\end{cases}
\]

where

- \( G_p \) = minimum green time (s),
- \( L \) = crosswalk length (ft),
- \( S_p \) = average speed of pedestrians (ft/s),
- \( W_E \) = effective crosswalk width (ft),
- 3.2 = pedestrian start-up time (s), and
- \( N_{ped} \) = number of pedestrians crossing during an interval (p).

It is assumed that the 15th-percentile walking speed of pedestrians crossing a street is 4.0 ft/s in this computation. This value is intended to accommodate crossing pedestrians who walk at speeds slower than the average. Where local policy uses different criteria for estimating minimum pedestrian crossing requirements, these criteria should be used in lieu of Equation 16-2.
2-Stage Crossings

- Allocates intersection capacity to conflicting movements
- Directly entered on pretimed controllers
- Implicitly selected for actuated controllers through:
  - Maximum green times
  - Minimum green times

Figure 61. This refuge island enables two-stage pedestrian crossings.

Figure 58. Issues associated with intersections with a wide median.

Split
Continuum Model

- How do you find?
  - Average Delay per Vehicle
  - Maximum Length of Queue
  - Percentage of Vehicles Stopped
  - Percentage of the Cycle with a Queue
HCM Traffic Signal Analysis

- Overarching Characteristics
  - Each Approach (Lane Group) Calculated Separately
  - Intersection is Analyzed by Approaches and Aggregated
  - Basic Signal Timing Plan must be Assumed
  - HCM DOES NOT Optimize Traffic Signal Timing Plans
  - Results in an Iterative Process
    - Show Example
  - Other Software Packages Optimize Traffic Signal Timing Plans
    - e.g., SYNCHRO, TRANSYT 7F, PASSER
    - Show Example

- Basic Modals
  - Input Modal
  - Flow Rate Adjustments
  - Saturation Flow Rate Calculations
  - Capacity Calculations
  - MOE Calculations
EXHIBIT 16-1. SIGNALIZED INTERSECTION METHODOLOGY

- Input Parameters
  - Geometric
  - Traffic
  - Signal

- Lane Grouping and Demand
  - Lane grouping
  - PHF
  - RTOR

- Saturation Flow Rate
  - Basic equation
  - Adjustment factors

- Capacity and v/c
  - Capacity
  - v/c

- Performance Measures
  - Delay
  - Progression adjustment
  - LOS
  - Back of queue

EXHIBIT 16-3. INPUT DATA NEEDS FOR EACH ANALYSIS LANE GROUP

<table>
<thead>
<tr>
<th>Type of Condition</th>
<th>Parameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>Geometric conditions</td>
<td>Area type, N (m²)</td>
</tr>
<tr>
<td></td>
<td>Number of lanes, L (n)</td>
</tr>
<tr>
<td></td>
<td>Average lane width, w (ft)</td>
</tr>
<tr>
<td></td>
<td>Grade, G (%)</td>
</tr>
<tr>
<td></td>
<td>Existence of exclusive L1 or RT lanes</td>
</tr>
<tr>
<td></td>
<td>Length of storage bay, L2 or RT lane, L3 (ft)</td>
</tr>
<tr>
<td></td>
<td>Parking</td>
</tr>
<tr>
<td>Traffic conditions</td>
<td>Demand volume by movement, V (veh/h)</td>
</tr>
<tr>
<td></td>
<td>Base saturation flow rate, s₁ (pcu/h/lane)</td>
</tr>
<tr>
<td></td>
<td>Peak-hour factor, PHF</td>
</tr>
<tr>
<td></td>
<td>Percent heavy vehicles, MV (%)</td>
</tr>
<tr>
<td></td>
<td>Approach pedestrian flow rate, Vped (peds/h)</td>
</tr>
<tr>
<td></td>
<td>Local buses stopping at intersection, Nₘ (buses/h)</td>
</tr>
<tr>
<td></td>
<td>Parking activity, Nₚ (maneuvers/h)</td>
</tr>
<tr>
<td></td>
<td>Arrival type, AT</td>
</tr>
<tr>
<td></td>
<td>Proportion of vehicles arriving on green, P</td>
</tr>
<tr>
<td></td>
<td>Approach speed, S₀ (mph)</td>
</tr>
<tr>
<td>Signalization conditions</td>
<td>Cycle length, C (s)</td>
</tr>
<tr>
<td></td>
<td>Green time, G₁ (s)</td>
</tr>
<tr>
<td></td>
<td>Yellow-plus-all-red change-and-clearance interval (intermittent), Y (s)</td>
</tr>
<tr>
<td></td>
<td>Actuated or preeminent operation</td>
</tr>
<tr>
<td></td>
<td>Pedestrian push-button</td>
</tr>
<tr>
<td></td>
<td>Minimum pedestrian green, O₀ (s)</td>
</tr>
<tr>
<td></td>
<td>Phase plan</td>
</tr>
<tr>
<td></td>
<td>Analysis period, T (h)</td>
</tr>
</tbody>
</table>
Traffic Signal Coordination

**EXHIBIT 10-16. PROGRESSION QUALITY AND ARRIVAL TYPE**

<table>
<thead>
<tr>
<th>Progression Quality</th>
<th>Arrival Type</th>
<th>Conditions Under Which Arrival Type Is Likely To Occur</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very poor</td>
<td>1</td>
<td>Occurs for coordinated operation on two-way street where one direction of travel does not receive good progress. Signals are spaced less than 1,600 ft apart.</td>
</tr>
<tr>
<td>Favorable</td>
<td>2</td>
<td>A less extreme version of Arrival Type 1. Signals spaced at or more than 1,800 ft but less than 3,200 ft apart.</td>
</tr>
<tr>
<td>Random arrivals</td>
<td>3</td>
<td>Isolated signals spaced at or more than 3,200 ft apart (whether or not coordinated).</td>
</tr>
<tr>
<td>Favorable</td>
<td>4</td>
<td>Occurs for coordinated operation, often only in one direction on a two-way street. Signals are typically between 1,600 ft and 3,200 ft apart.</td>
</tr>
<tr>
<td>Highly favorable</td>
<td>5</td>
<td>Occurs for coordinated operation. More likely to occur with signals less than 1,600 ft apart.</td>
</tr>
<tr>
<td>Exceptional</td>
<td>6</td>
<td>Typical of one-way streets in dense networks and central business districts. Signal spacing is typically under 800 ft.</td>
</tr>
</tbody>
</table>

**DETERMINING SATURATION FLOW RATE**

A saturation flow rate for each lane group is computed according to Equation 16-4. The saturation flow rate is the flow in vehicles per hour that can be accommodated by the lane group assuming that the green phase were displayed 100 percent of the time (i.e., $g/C = 1.0$).

$$ s = s_o N f_{LV} f_{TH} f_{HR} f_{JC} f_{RT} f_{LB} f_{RB} \tag{16-4} $$

where

- $s_o$ = saturation flow rate for subject lane group, expressed as a total for all lanes in lane group (veh/h);
- $N$ = number of lanes in lane group;
- $f_{LV}$ = adjustment factor for lane width;
- $f_{TH}$ = adjustment factor for heavy vehicles in traffic stream;
- $f_{HR}$ = adjustment factor for approach grade;
- $f_{JC}$ = adjustment factor for existence of a parking lane and parking activity adjacent to lane group;
- $f_{RT}$ = adjustment factor for existence of a parking lane and parking activity adjacent to lane group;
- $f_{LB}$ = adjustment factor for local buses that stop within intersection area;
- $f_{RB}$ = adjustment factor for area type;
- $f_{LU}$ = adjustment factor for lane utilization;
- $f_{LT}$ = adjustment factor for left turns in lane group;
- $f_{RT}$ = adjustment factor for right turns in lane group;
- $f_{LB}$ = pedestrian adjustment factor for left-turn movements; and
- $f_{RB}$ = pedestrian-bicycle adjustment factor for right-turn movements.

Base saturation flow rate = 1,900 (pc/h/ln)
HCM Saturation Flow Rate Factors

### Lane Group Capacity

#### V/C Ratio

\[ c_1 = \frac{g_i}{C} \]  

(16-6)

where

- \( g_i \) = capacity of lane group i (veh/h),
- \( c_1 \) = saturation flow rate for lane group i (veh/h), and
- \( \frac{g_i}{C} \) = effective green ratio for lane group i.

#### v/c Ratio

The ratio of flow rate to capacity (v/c), often called the volume to capacity ratio, is given the symbol \( X_i \) in intersection analysis. It is typically referred to as degree of saturation. For a given lane group i, \( X_i \) is computed using Equation 16-7.

\[ X_i = \frac{v_i}{c_i} = \frac{v_i}{\frac{g_i}{C}} = \frac{v_i C}{g_i} \]  

(16-7)

where

- \( X_i \) = (v/c)i = ratio for lane group i,
- \( v_i \) = actual or projected demand flow rate for lane group i (veh/h),
- \( c_i \) = saturation flow rate for lane group i (veh/h),
- \( g_i \) = effective green time for lane group i (s), and
- \( C \) = cycle length (s).
Signalized Intersection LOS

LOS

The average control delay per vehicle is estimated for each lane group and aggregated for each approach and for the intersection as a whole. LOS is directly related to the control delay value. The criteria are listed in Exhibit 16-2.

<table>
<thead>
<tr>
<th>LOS</th>
<th>Control Delay per Vehicle (s/veh)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>≤ 10</td>
</tr>
<tr>
<td>B</td>
<td>&gt; 10–20</td>
</tr>
<tr>
<td>C</td>
<td>&gt; 20–35</td>
</tr>
<tr>
<td>D</td>
<td>&gt; 35–55</td>
</tr>
<tr>
<td>E</td>
<td>&gt; 55–80</td>
</tr>
<tr>
<td>F</td>
<td>&gt; 80</td>
</tr>
</tbody>
</table>

The Knack