CIVE 440

Traffic Engineering and Simulation – Signal Delay and Design

McGill
Faculty of Engineering
Department of Civil Engineering and Applied Mechanics
Fall 2016

MEASURES OF EFFECTIVENESS

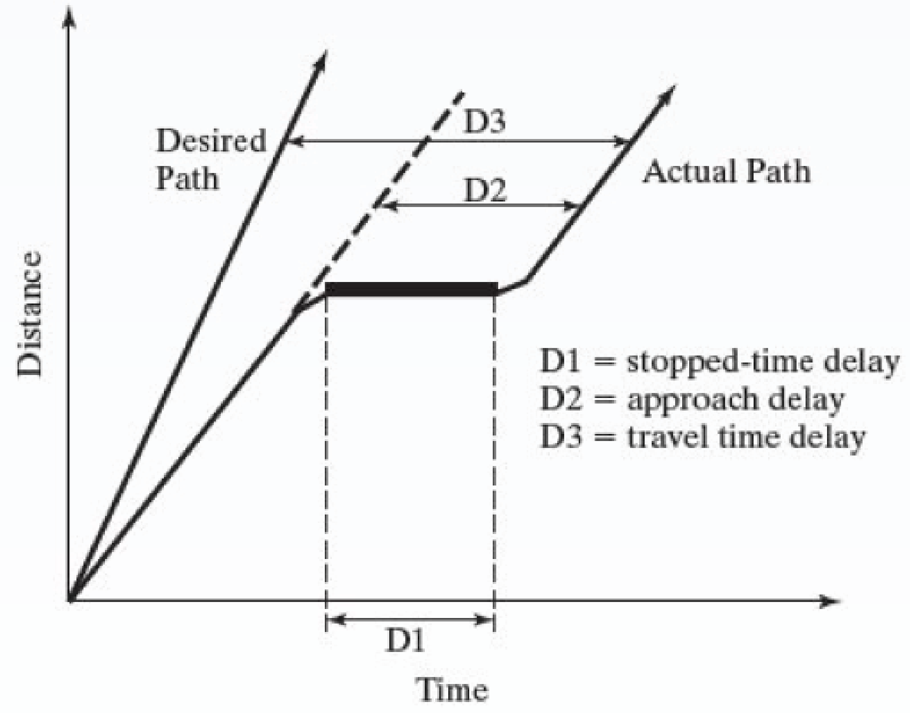
How do we evaluate the performance of a signalized intersection?

- Speed: may be useful for safety considerations
- Capacity: general measure of traffic performance, though dependant on externalities (number of lanes, parking, lane changes)
- Delay: amount of time consumed in traversing the intersection
  - Queue lengths: related to delay and capacity
  - % Stops: also related to delay

DELAY

Types of delay:

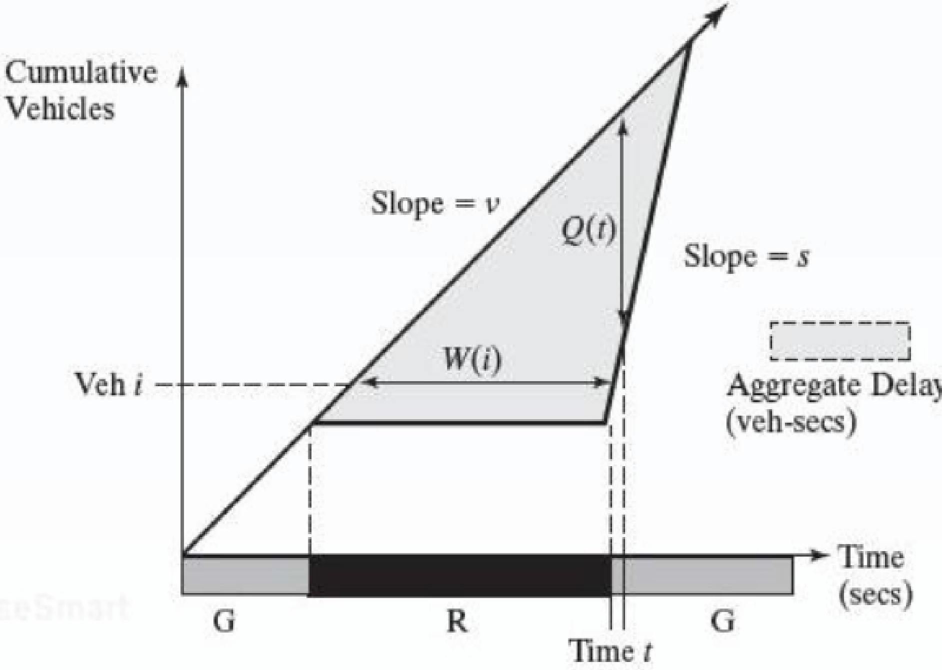
- Stopped-time delay
- Approach delay
- Time-in-queue delay
- Travel time delay
- Control delay
  - time-in-queue
  - acceleration-deceleration



DELAY MODELS

Theory:

- Uniform arrivals λ or v
- Vehicles arriving leave during green
  - Vehicles accumulate when red (queue)
  - Queue dissipates during following green
  - If the queue does not dissipate before the next red, the intersection is **oversaturated,** or "stable"
- Delay calculated for each signal group.



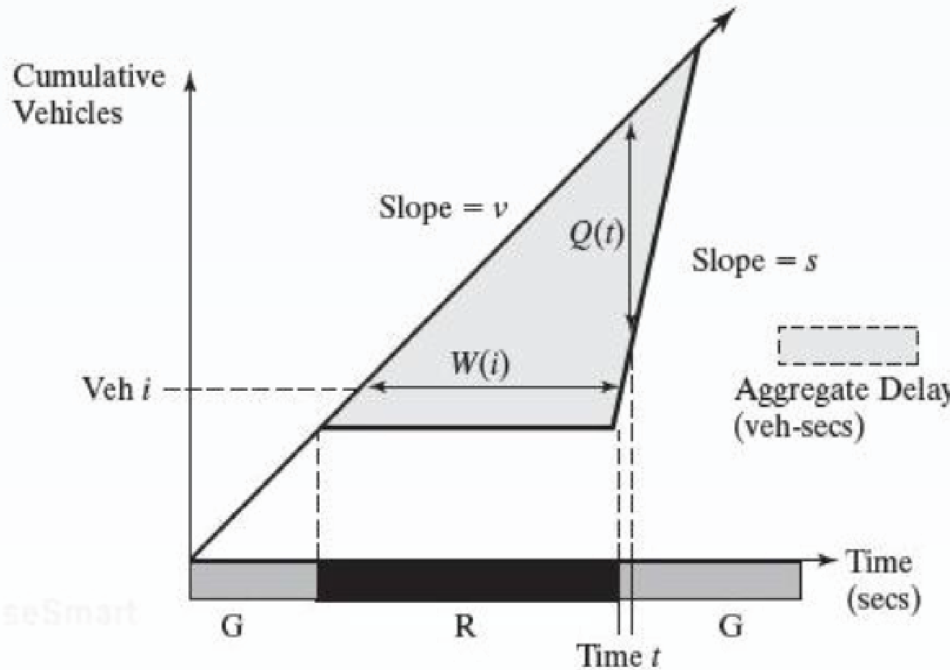
DELAY MODELS

Critical parameters:

- Total time vehicle i spends waiting in queue ($W(i)$)
- Number of vehicles in queue ($Q(i)$)
- Total delay of the queue

$$\sum_i W(i) \text{ or } \sum_i Q(i)$$

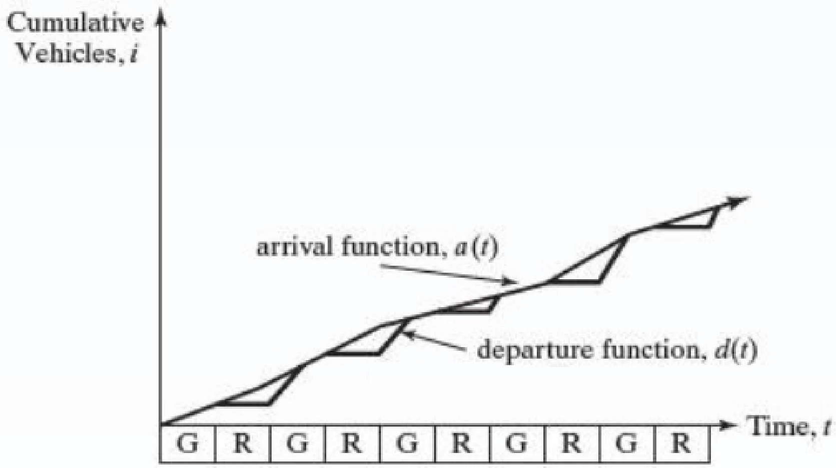
- That is, the area bounded by λ and μ



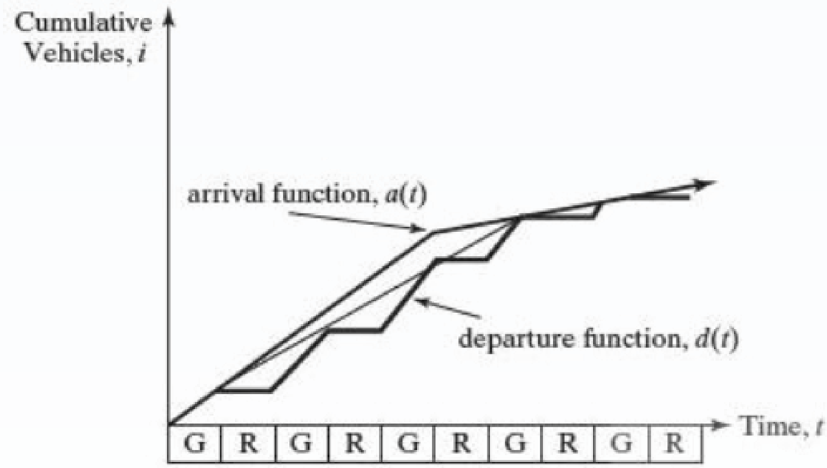
This model makes the following assumptions:

- Uniform arrival rate
  - Arrivals are usually random i.e. inter arrival time is not uniform
- Queue is building at a point rather than in space
  - There is growth in backward queue while vehicles arrive at the end of the queue
- We shall explore means to relaxing some of these assumptions

Variable arrival rate (cycle scale) and over saturation:



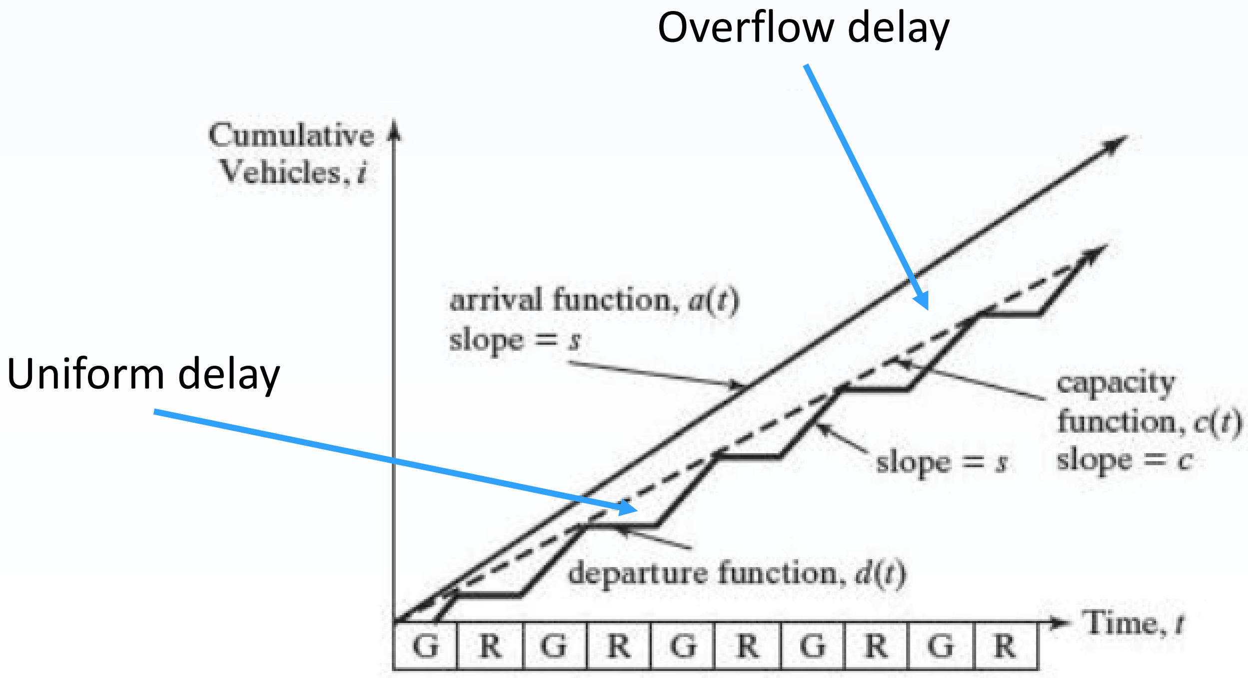
(a) Stable Flow



(b) Individual Cycle Failures Within a Stable Operation

Signal failure when the arrival rate λ is greater than the departure rate μ for a succession of cycles.

- Temporary "failures" over one to two cycles considered acceptable and within normal operating tolerance



(c) Demand Exceeds Capacity for a Significant Period

DELAY

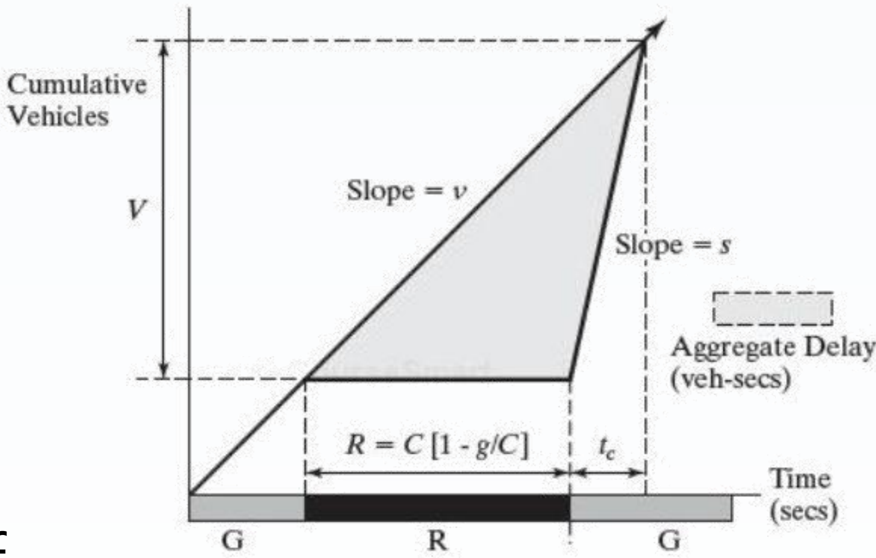
Subtypes of queue delay:

- Uniform delay is the delay based on an assumption of uniform arrivals and stable flow with no individual cycle failures.
- Random delay is the additional delay, above and beyond uniform delay, because flow is randomly distributed rather than uniform at isolated intersections.
- Overflow delay is the additional delay that occurs when the capacity of an individual phase or series of phases is less than demand or arrival flow rate

WEBSTER'S MODEL

Cycle time model to minimise uniform delay (***UD***).
Generalized as:

$$UD = \frac{RQ(i)}{2}$$



By convention, traffic models are not developed in terms of red time, so we convert it into green time:

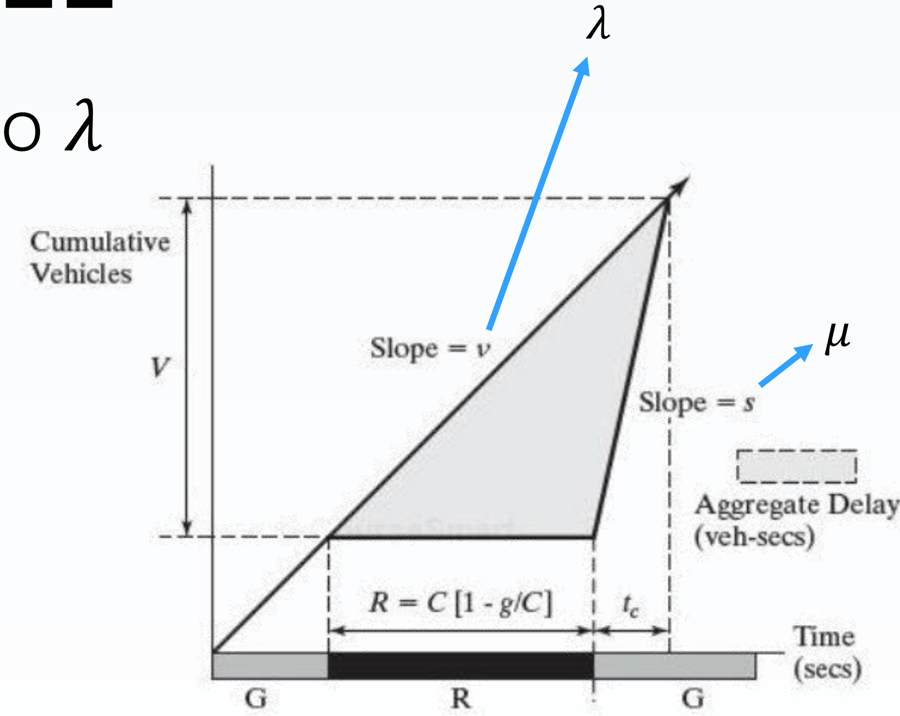
$$R = C\left[1 - \left(\frac{g}{C}\right)\right]$$

WEBSTER'S MODEL

$Q(i)$ is linearly proportional to λ (μ is 0 during red).

- Therefore:

$$Q(i) = \lambda(R + t_c) = \mu t_c$$



Solve for t_c…

$$t_c = \frac{R}{\frac{\mu}{\lambda} - 1}$$

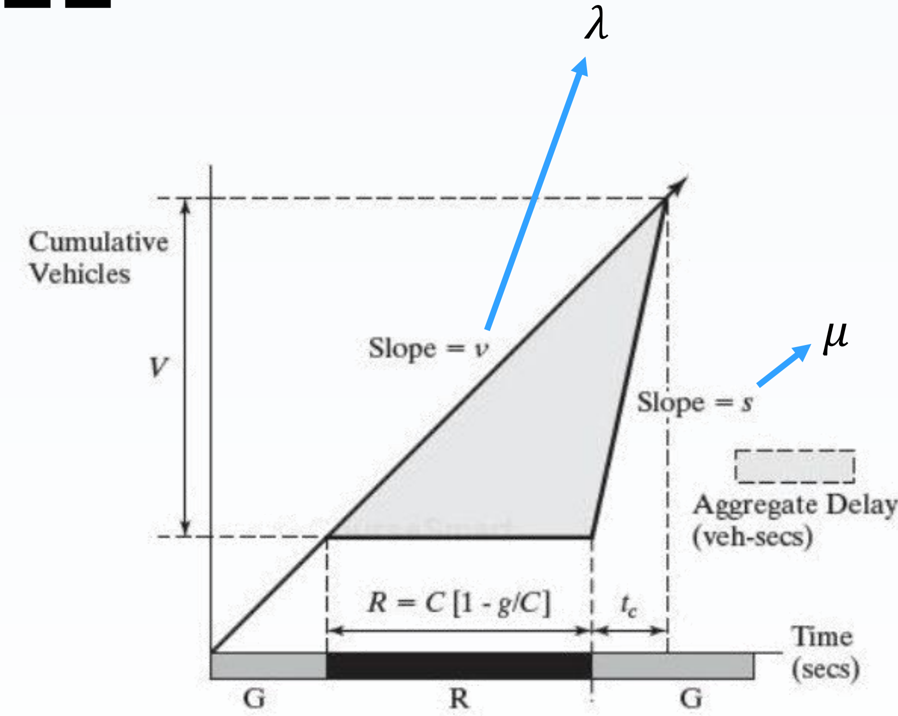
WEBSTER'S MODEL

Substituting back into $Q(i)$...

$$Q(i) = \lambda\left(R + \frac{R}{\frac{\mu}{\lambda} - 1}\right)$$

$$= R\left(\frac{\mu\lambda}{\mu - \lambda}\right)$$

$$= C\left[1 - \left(\frac{g}{C}\right)\right]\left(\frac{\mu\lambda}{\mu - \lambda}\right)$$

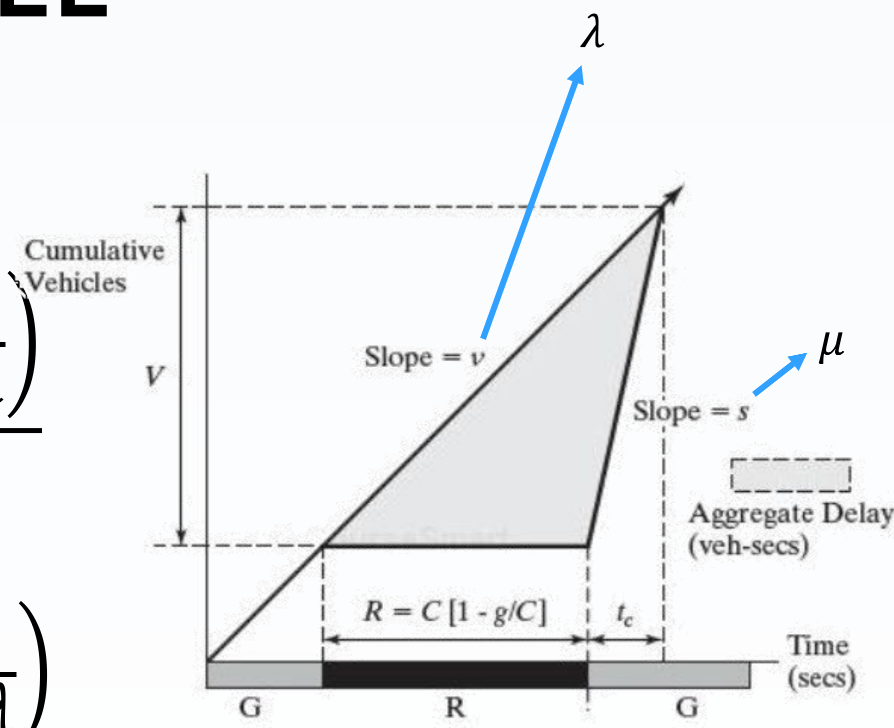


WEBSTER'S MODEL

Finally...

$$UD = \frac{RC\left[1-\left(\frac{g}{C}\right)\right]\left(\frac{\mu\lambda}{\mu-\lambda}\right)}{2}$$

$$UD = \frac{C^2\left[1-\left(\frac{g}{C}\right)\right]^2\left(\frac{\mu\lambda}{\mu-\lambda}\right)}{2}$$



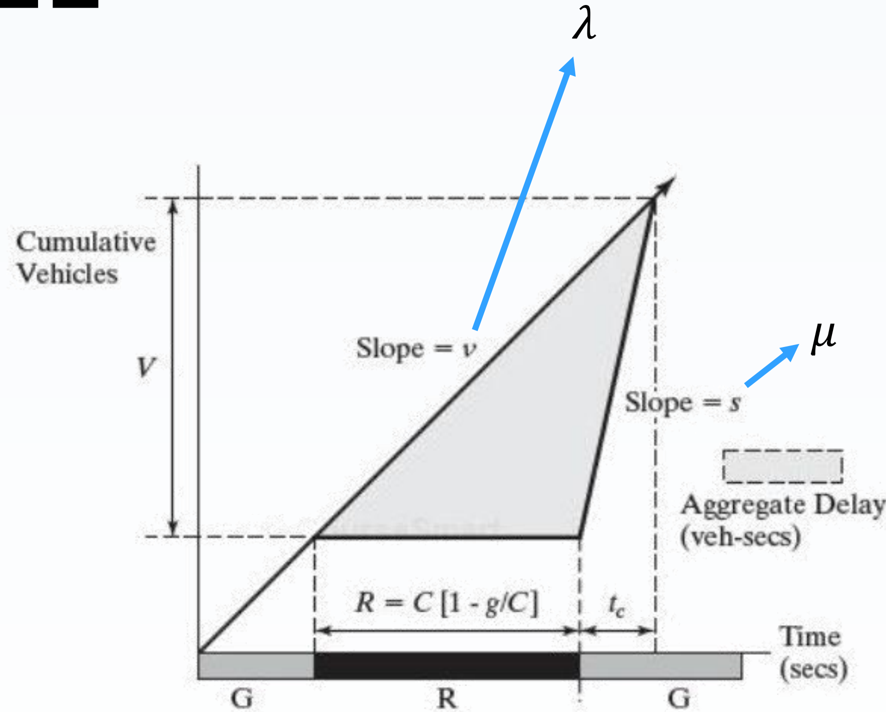
WEBSTER'S MODEL

The average, per-vehicle UD, $\overline{UD}$, is given by dividing UD by the number of arriving vehicles per cycle, i.e. λC:

$$\overline{UD} = \frac{C\left[1-\left(\frac{g}{C}\right)\right]^2\left(\frac{\mu}{\mu-\lambda}\right)}{2}$$

where λ =arrivals (demand)

μ = saturation flow (s)

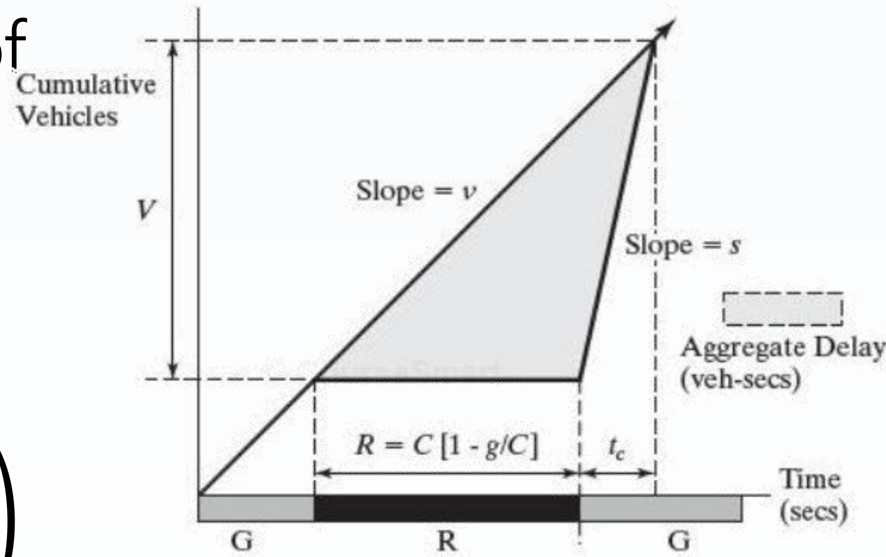


WEBSTER'S MODEL

$\overline{UD}$ can be expressed as a function of capacity instead of saturation flow using:

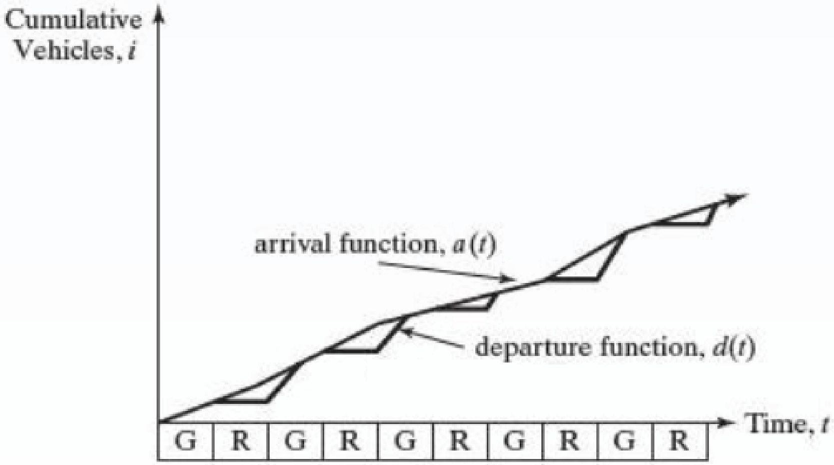
$$s = \frac{cC}{g}$$

$$\overline{UD} = \frac{\left[1 - \left(\frac{g}{C}\right)\right]^2 \left(\frac{C}{C - \frac{vg}{c}}\right)}{2}$$

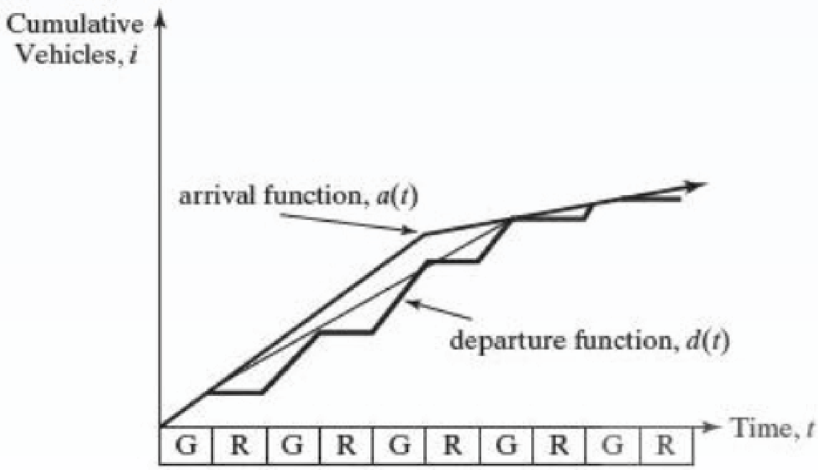


RANDOM DELAY

Flows are never perfectly uniform in the short term, the arrival rate varies from cycle to cycle.



(a) Stable Flow



(b) Individual Cycle Failures Within a Stable Operation

RANDOM DELAY

How do we account for random delay ($\boldsymbol{RD}$)?

- Many academic approaches
- Another Webster model:

$$RD = \frac{\left(\frac{v}{c}\right)^2}{\left[2v\left(1-\left(\frac{v}{c}\right)\right)\right]}$$

In reality, this is a high estimate. Empirical adjustment for total delay:

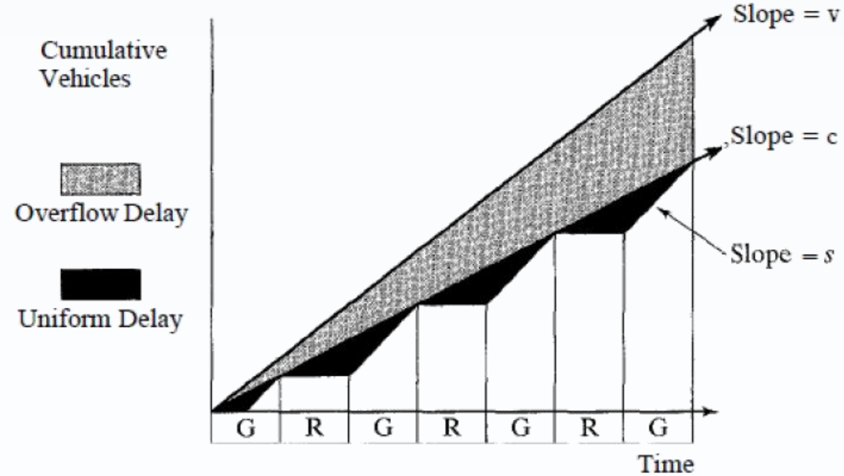
$$TD = 0.9[UD + RD]$$

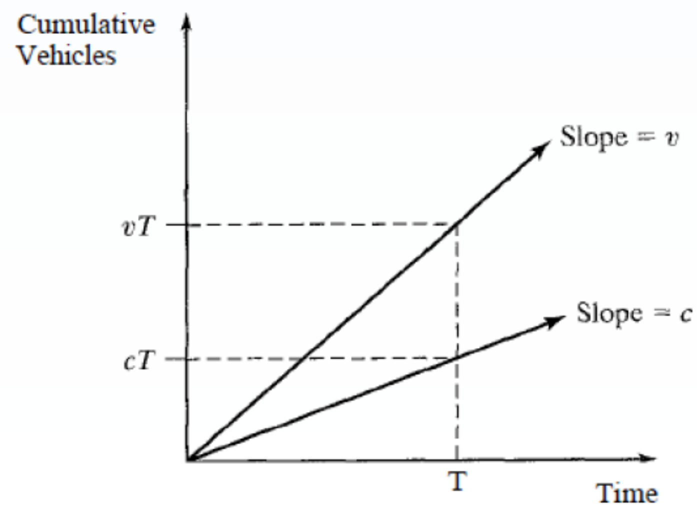
OVERFLOW DELAY MODEL

Overflow (saturation) when $v/c > 1$

Then, assuming uniform arrival, simplified OD calculation:

$$OD = \frac{c\left[1-\left(\frac{g}{c}\right)\right]}{2}$$

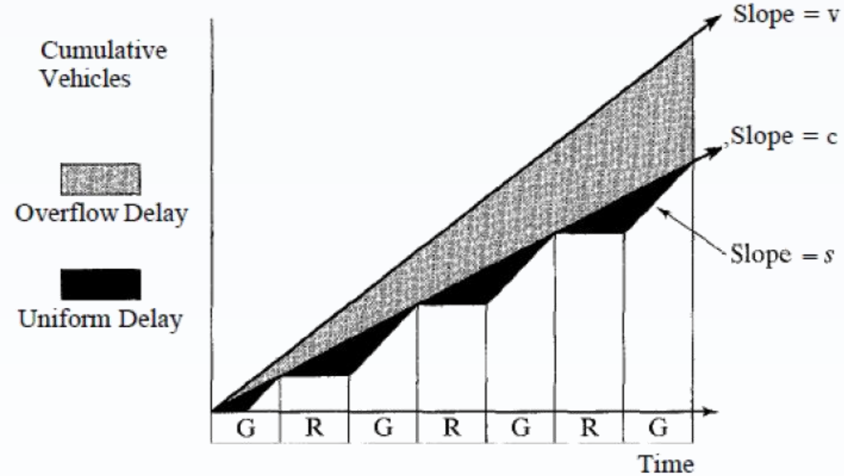




OVERFLOW DELAY MODEL

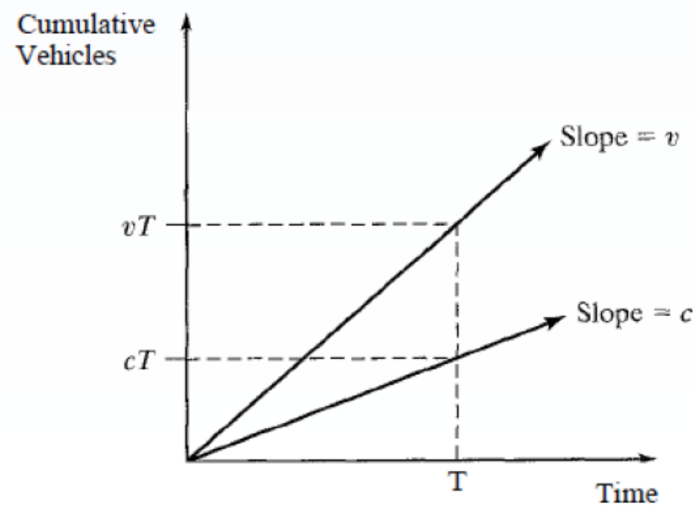
Overflow delay ($\boldsymbol{OD}$) for given time T given by:

$$OD = \frac{T(vT - CT)}{2} = \frac{T^2(v - c)}{2}$$



Average overflow delay:

$$\overline{OD} = \frac{OD}{CT} = \frac{T\left(\frac{v}{c} - 1\right)}{2}$$

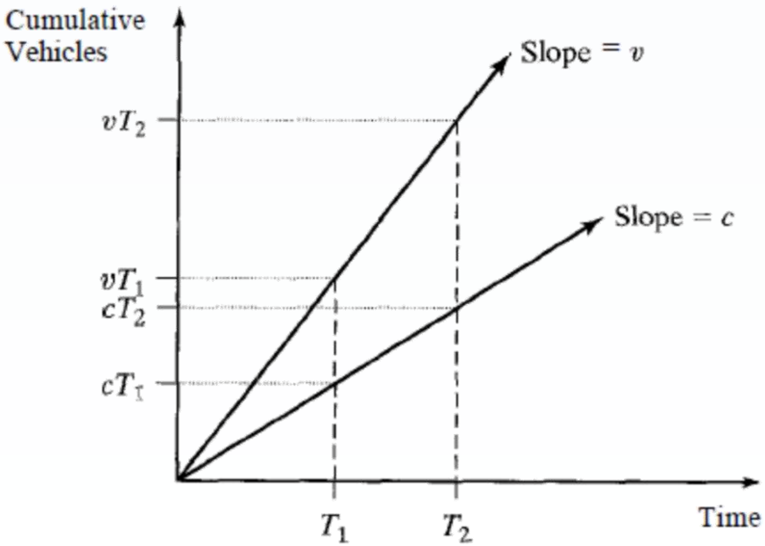


REMARKS ON WEBSTER'S MODEL

OD increases with T (varies over time).

- Vehicles that arrive early during the overflow period have smaller overflow delay
- We can develop an average delay between two intervals T1 and T2 and use it in the formula to find delay over a specific interval:

$$OD = \frac{(T1+T2)\left[\left(\frac{v}{c}\right)-1\right]}{4}$$



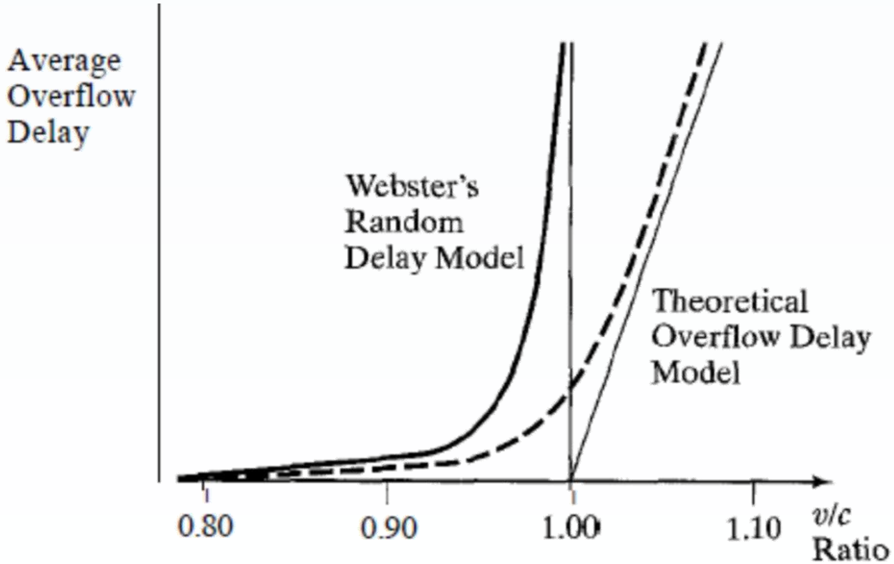
ISSUES WITH WEBSTER'S MODEL

Random delay increases dramatically as v/c approaches 1

- $\lim_{\frac{v}{c} \to 1.00} OD = \infty$

Overflow delay is 0 when $v/c = 1$ and increases linearly with v/c.

- Both are not accurate reflections of reality at saturation.



EXAMPLE

Determine the average delay for the following approach with a flow rate of 1000 veh/h, a saturation flow rate of 2800 veh/h, an allocation of 55% green time, and an intersection cycle of 90 seconds.

EXAMPLE

Determine the average delay over a period of two hours for the following approach with a flow rate of 1900 veh/h, a saturation flow rate of 2800 veh/h, an allocation of 55% green time and an intersection cycle of 90 seconds.

How does the delay during the first 15 minutes compare with the delay during the last 15 minutes?

AKCELIK'S MODEL

Akcelik's model from the Australian Road Research Board:

$$OD = \frac{cT}{4}\left[(X - 1) + \sqrt{(X-1)^2 + \left(\frac{12(X - X_o)}{cT}\right)}\right]$$

$$X_o = 0.67 + \left(\frac{sg}{600}\right)$$

$$OD = 0.0 \text{ s/veh for } X \leq X_o \qquad (17\text{-}26)$$

where: T = analysis period, h
X = v/c ratio
c = capacity, veh/h
s = saturation flow rate, veh/sg, (vehs per second of green)
g = effective green time, s

HCM MODEL

Classic delay (d) model used by the Highway capacity manual (Chapter 16) for level of service evaluation:

$$d = d_1(PF) + d_2 + d_3$$

- d = control delay per vehicle (s/veh)
- d_1 = uniform control delay (s/veh)
- PF = uniform delay progression adjustment factor
- d_2 = incremental of signal control (s/veh)
- d_3 = initial queue delay (s/veh)

PROGRESSION ADJUSTMENT FACTOR

The uniform delay progression adjustment factor accounts for effects of signal progression (coordination):

$$PF = \frac{(1-P)f_{PA}}{1-\left(\frac{g}{C}\right)}$$

For non-coordinated intersections, use $AT = 3$. Unbuilt, use $AT = 4$. Else, see 16-11.

EXHIBIT 16-12. PROGRESSION ADJUSTMENT FACTOR FOR UNIFORM DELAY CALCULATION

Green Ratio (g/C)	Arrival Type (AT)					
	AT 1	AT 2	AT 3	AT 4	AT 5	AT 6
0.20	1.167	1.007	1.000	1.000	0.833	0.750
0.30	1.286	1.063	1.000	0.986	0.714	0.571
0.40	1.445	1.136	1.000	0.895	0.555	0.333
0.50	1.667	1.240	1.000	0.767	0.333	0.000
0.60	2.001	1.395	1.000	0.576	0.000	0.000
0.70	2.556	1.653	1.000	0.256	0.000	0.000
f_{PA}	1.00	0.93	1.00	1.15	1.00	1.00
Default, R_p	0.333	0.667	1.000	1.333	1.667	2.000

Notes:
$PF = (1 - P)f_{PA}/(1 - g/C)$.
Tabulation is based on default values of f_{PA} and R_p.
$P = R_p * g/C$ (may not exceed 1.0).
PF may not exceed 1.0 for AT 3 through AT 6.

HCM UNIFROM DELAY

Essentially based off of Webster's model.

$$d_1 = \frac{0.5C\left(1 - \frac{g}{C}\right)^2}{1 - \left(\min(1, x)\frac{g}{C}\right)}$$

Where $x = \left(\frac{v}{c}\right)_{\mathrm{ci}}$ for the phase involving the signal group of g

HCM INCREMENTAL DELAY

Accounts for effect of random and oversaturation queues (RD), adjusted for duration of analysis period and type of signal control.

- This delay component assumes that there is no initial queue for lane group at start of analysis period.

$$d_2 = 900T\left((x-1) + \sqrt{(x-1)^2 + \frac{8kIx}{cT}}\right)$$

where T = duration of analysis period (h)

k = incremental delay factor that is dependent on controller settings

I = upstream filtering/metering adjustment factor (I = 1.0 for isolated intersections)

c = lane group capacity (veh/h)

$x = \left(\frac{v}{c}\right)_{\mathrm{ci}}$ for the phase involving the signal group of g

EXHIBIT 16-13. k-VALUES TO ACCOUNT FOR CONTROLLER TYPE

Unit Extension (s)	Degree of Saturation (X)					
	≤ 0.50	0.60	0.70	0.80	0.90	≥ 1.0
≤ 2.0	0.04	0.13	0.22	0.32	0.41	0.50
2.5	0.08	0.16	0.25	0.33	0.42	0.50
3.0	0.11	0.19	0.27	0.34	0.42	0.50
3.5	0.13	0.20	0.28	0.35	0.43	0.50
4.0	0.15	0.22	0.29	0.36	0.43	0.50
4.5	0.19	0.25	0.31	0.38	0.44	0.50
5.0[a]	0.23	0.28	0.34	0.39	0.45	0.50
Pretimed or nonactuated movement	0.50	0.50	0.50	0.50	0.50	0.50

Note:
For a given unit extension and its k_{min} value at X = 0.5: $k = (1 - 2k_{min})(X - 0.5) + k_{min}$, $k \geq k_{min}$, and $k \leq 0.5$.
a. For unit extension > 5.0, extrapolate to find k, keeping $k \leq 0.5$.

HCM INITIAL QUEUE DELAY

When a residual queue from a previous time period causes an initial queue (OD) to occur at the start of the analysis period (T), additional delay is experienced by vehicles arriving in the period since the initial queue must first clear the intersection.

- A procedure for determining this initial queue delay is described in detail in Chapter 16, Appendix F of HCM.
- If this is not the case, a value of zero is used for d_3.

AGGREGATED DELAY

Delays are calculated for each signal group (as they share red and green time).

- An intersection-wide delay is measured as the weighted average delay across signal groups by signal group flows.

$$d_{intersection} = \frac{\sum d_i q_i}{\sum q_i}$$

LEVEL-OF-SERVICE (LOS) FOR INTERSECTIONS

The HCM level of service for intersections or individual signal groups is based off of delay.

- The LOS grades the individual delay according to the following table:

EXHIBIT 16-2. LOS CRITERIA FOR SIGNALIZED INTERSECTIONS

LOS	Control Delay per Vehicle (s/veh)
A	≤ 10
B	> 10–20
C	> 20–35
D	> 35–55
E	> 55–80
F	> 80

SIGNAL TIMING DESIGN

Key elements of signal design:

- Development of a safe and effective phase plan and sequence
- Determination of vehicular signal needs
- Determination of pedestrian signal needs

SIGNAL PHASE PLAN

Development of a safe and effective phase plan and sequence:

- Requires engineering judgement
- Treatment of left turns is an important feature that drives the development of the phase plan
- Basic phase plan is a two phase signal plan with all left turns being permitted
- Additional phases may be added to provide protection for some or all left turns or pedestrians
  - But, additional phases add to the lost time, so we need to weigh the inefficiency and lost time to the cycle against improved efficiency in operation of left-turning and other vehicles affected

Two criteria are used to determine the need for a protected or partially protected phase for left-turn:

- $Q_{LT} \geq 200\ veh/hr$

  —and—

- $xprod = Q_{LT}\frac{Q_0}{N_0} \geq 50{,}000$

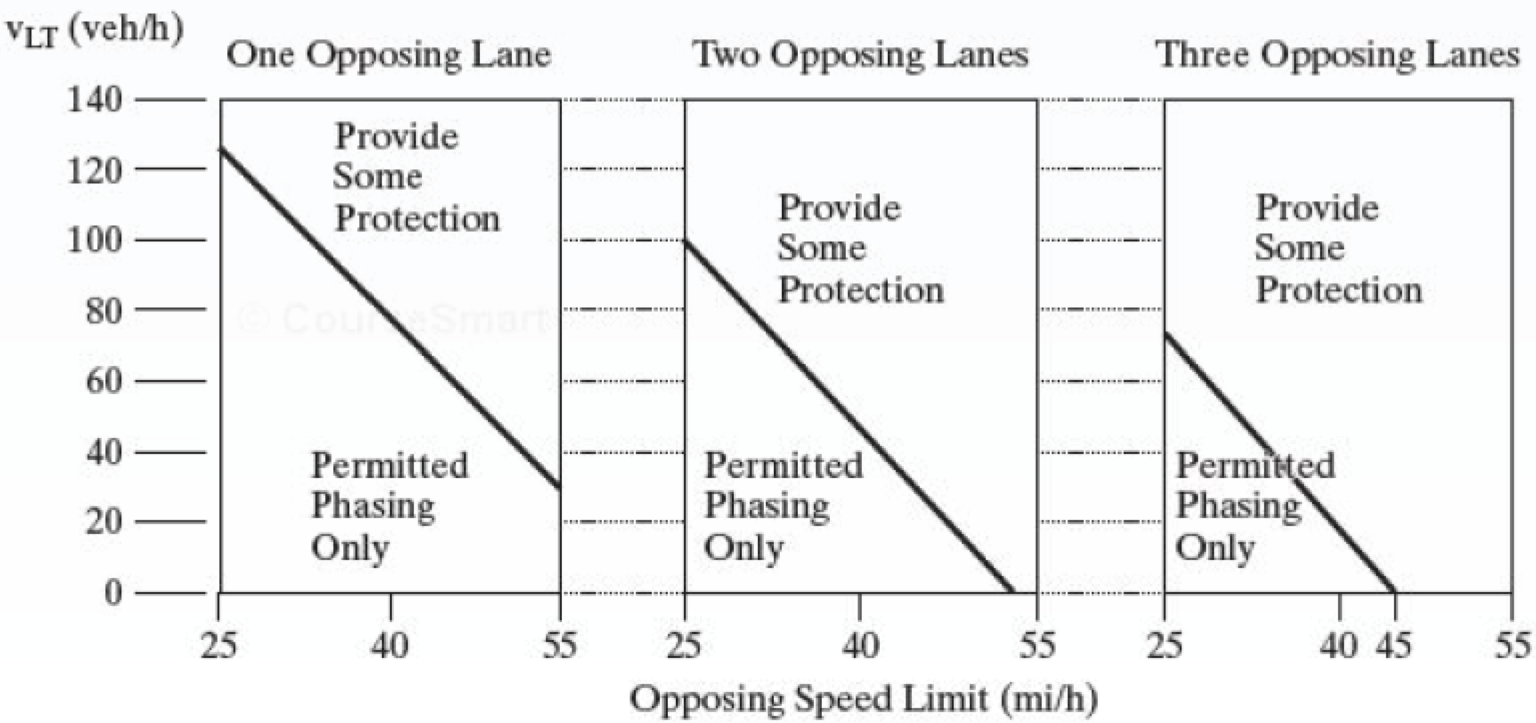
where Q_{LT} = left-turn flow rate, veh/hr

Q_0 = opposing through movement flow rate

N_0 = no. of lanes for opposing through movements

Additional guidelines:

- Sight distance
- Number of left turn accidents > 8
- Opposing speed limit is greater than or equal to $70\ km/h$



Phasing is employed to minimize conflicts.

In addition to managing conflicts, additional phasing tends to:

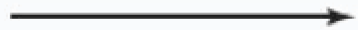
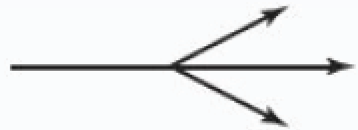
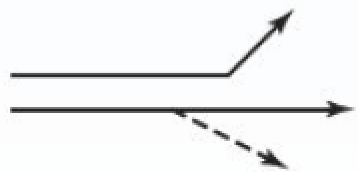
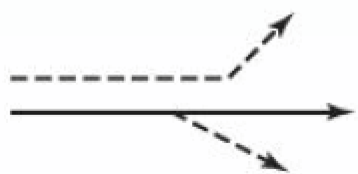
- increase lost time per cycle, reducing capacity
- but in some circumstances might increase the efficiency of left-turn flow
  - This is especially true when left turns are significant and left turn lanes are shared with through movements or overflow onto through movement lanes

Phase plans must be consistent with intersection geometry, lane-use assignments, volumes and speeds and pedestrian crossing requirements. **Right-angle and head-on conflicts should never be permitted.**

PHASE DIAGRAMS

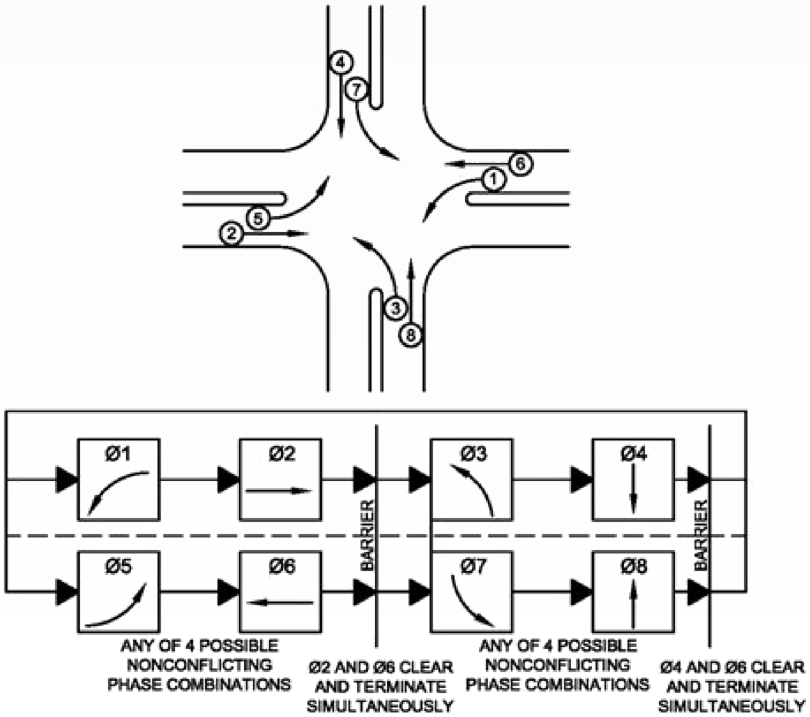
Conventions:

- A solid arrow denotes a movement without opposition.
  - All through movements are unopposed by default.
  - An unopposed left turn has no opposing through vehicular flow.
- Opposed left- and/or right-turn movements are shown as a dashed line.
- Turning movements made from a shared lane(s) are shown as arrows connected to the through movement with which they share the lane(s).
- Turning movements from an exclusive lane(s) are shown as separate arrows, not connected to any through movement.

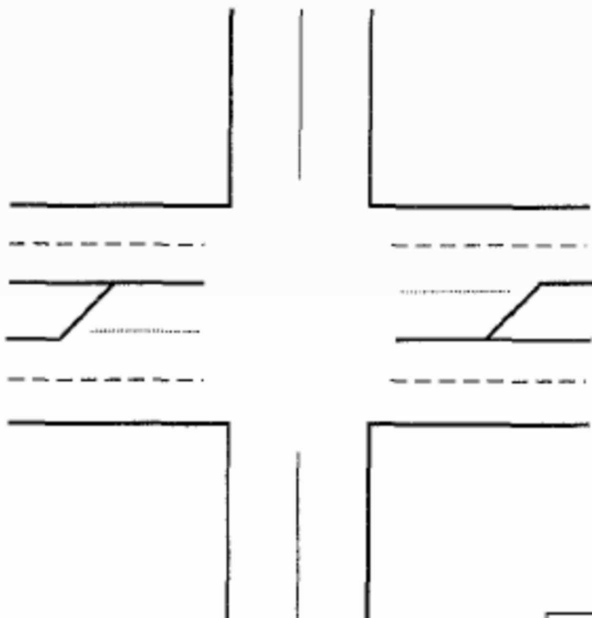
Through movement without turning movement.	
Through movement with protected right and left turns from shared lanes.	
Through movement with permitted right and left turns from shared lanes.	
Through movement with protected left turn from exclusive lane and permitted right turn from shared lane.	
Through movement with permitted left turn from exclusive lane and permitted right turn from shared lane.	

"RING BARRIER" DIAGRAM

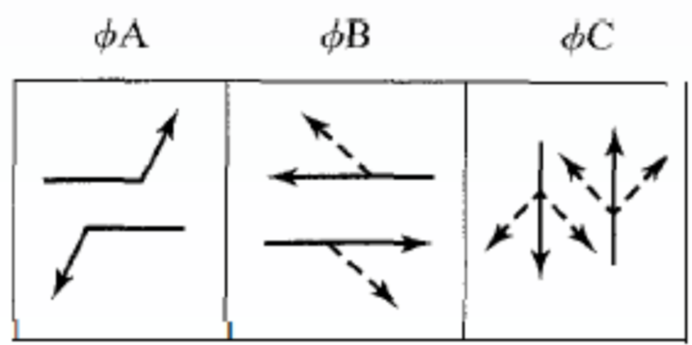
A ring diagram shows which movements are controlled by which "ring" on a signal controller. A "ring" of a controller generally controls one set of signal faces.



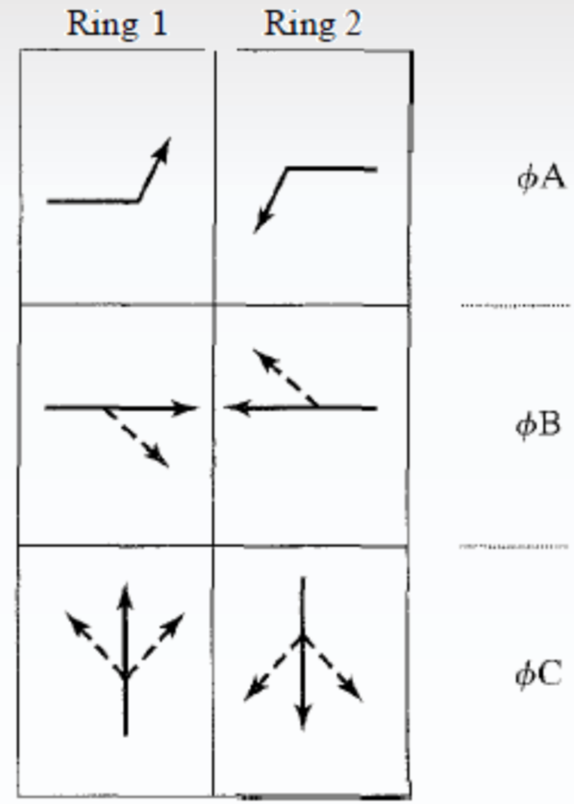
EXCLUSIVE LEFT-TURN PHASING



(a) Intersection Layout



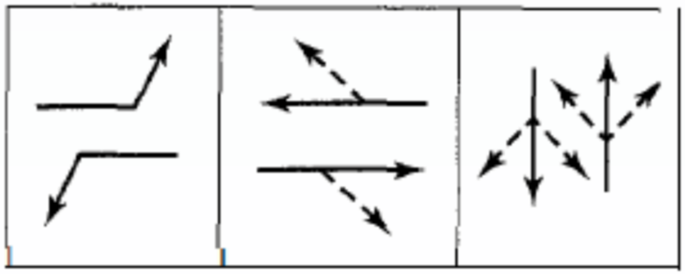
(b) Phase Diagram



(c) Ring Diagram

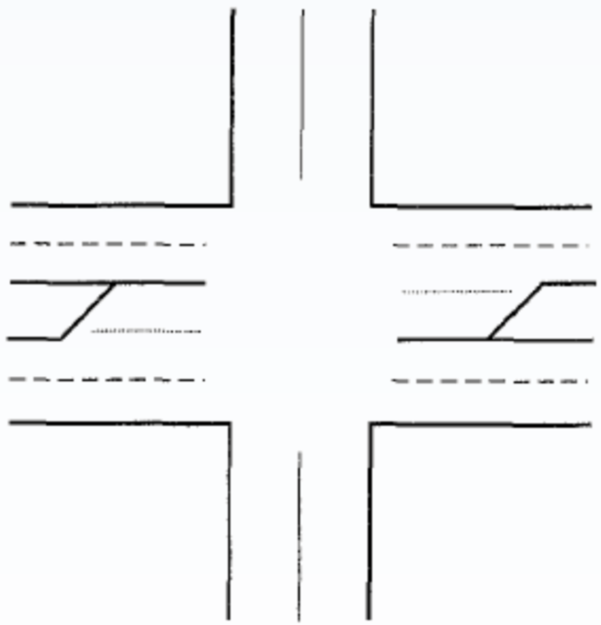
How does the order on a protected left turn matter? For:

- Pedestrians
- Cyclists
- Through traffic
- Accident rates

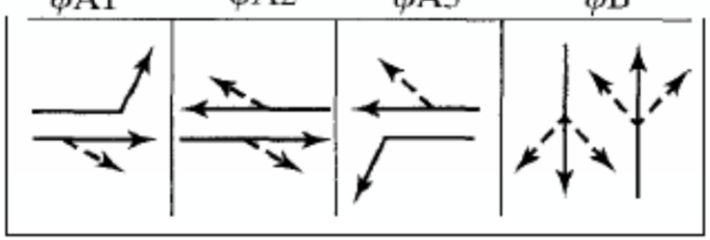


(b) Phase Diagram

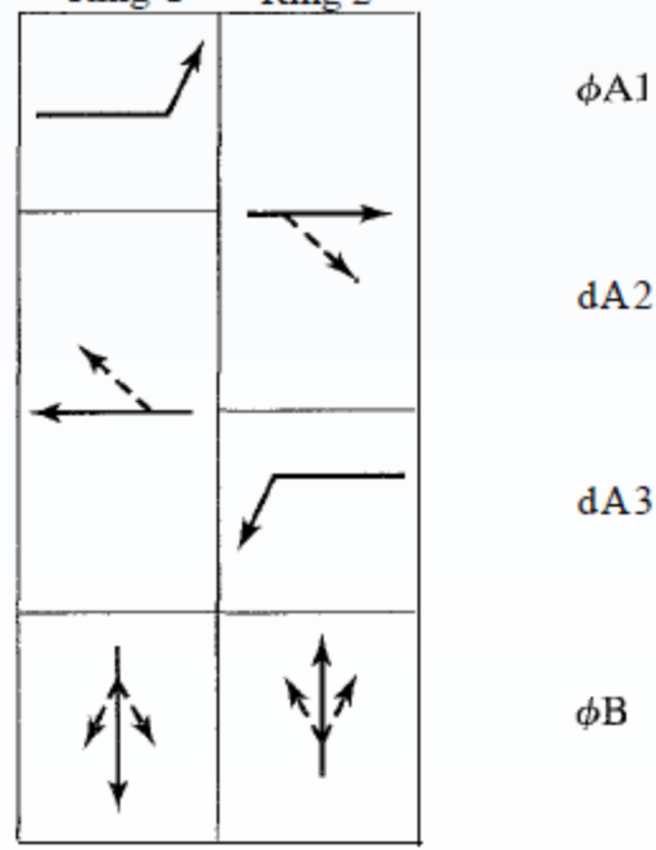
LEADING AND LAGGING GREEN PHASES



(a) Intersection Layout

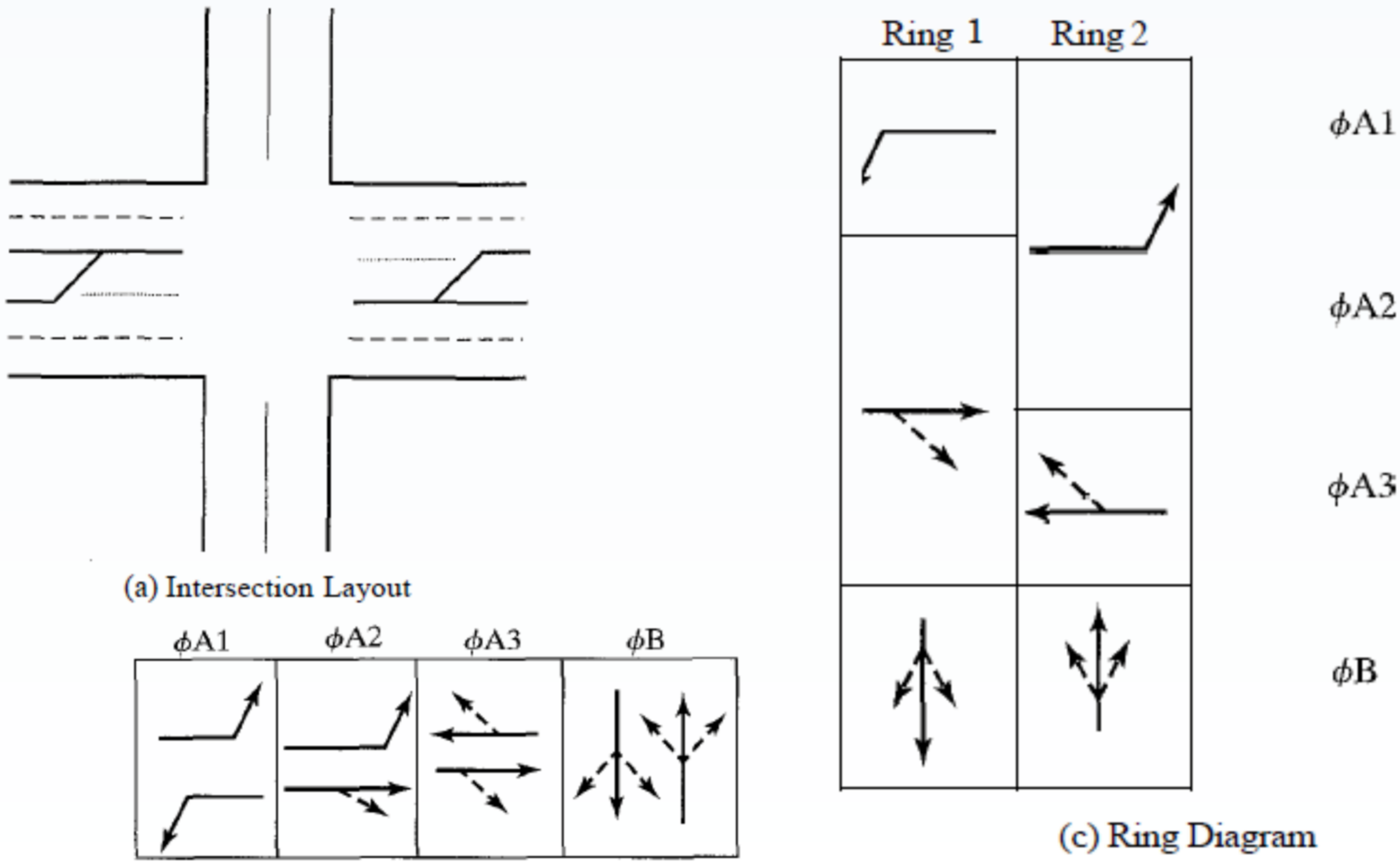


(b) Phase Diagram



(c) Ring Diagram

EXCLUSIVE LEFT-TURN PHASE WITH LEADING GREEN

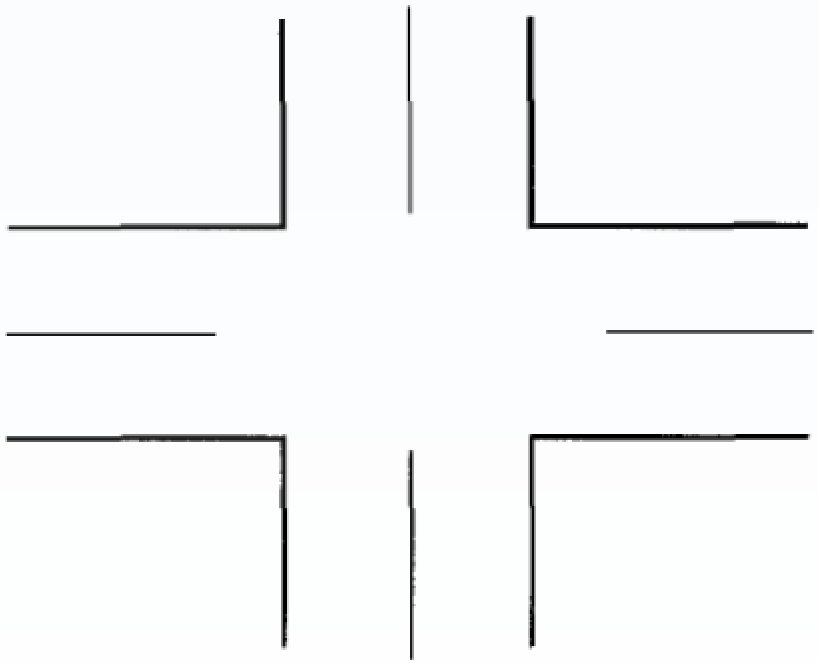


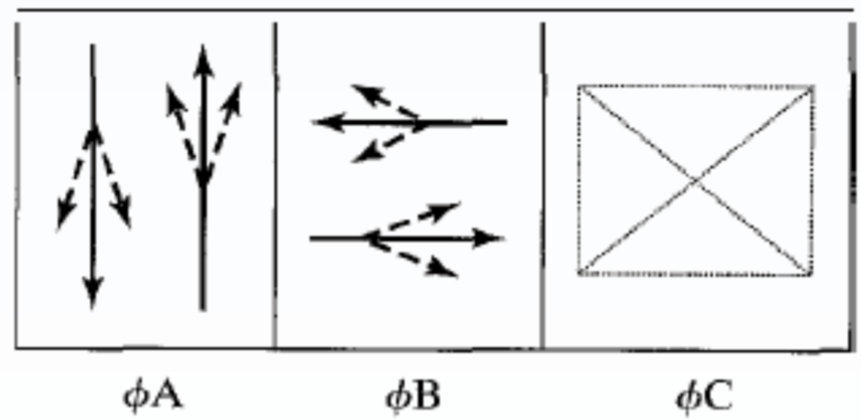
(a) Intersection Layout

(b) Phase Diagram

(c) Ring Diagram

EXCLUSIVE PEDESTRIAN PHASE





VEHICULAR REQUIREMENTS

Determination of vehicular signal needs

- Timing of yellow and all-red (clearance) intervals for each signal phase (security)
- Determination of the sum of critical lane volumes (Q_c)
- Determination of lost times per phase (t_L) and per cycle (L)
- Determination of an appropriate cycle length (C)
- Allocation of effective green time to the various phases defined in the phase plan
    - often referred to as “splitting” the green

YELLOW TIME

The yellow time Y is an interval that ensures that a vehicle that is upstream of the STOP line when the green indication is withdrawn to continue at the approach speed and enter the intersection on yellow.

- Breaking suddenly to avoid a red or yellow light is extremely dangerous and a common cause of increases in accidents at traffic lights.
- Drivers must judge their ability to clear the intersection safely.

"Entering the intersection" is interpreted as the front wheels crossing over the STOP line.

- This is the technical, engineering definition

Yellow time must be carefully selected to avoid the dilemma zone. The dilemma zone occurs when a driver:

- cannot decelerate safely to a complete stop before the STOP line

  —and—

- simultaneously cannot cross the STOP line before the light turns red operating the vehicle at a reasonable speed.

Dilemma zones occur frequently at high-speed intersections. What else influences deceleration?

A few different models exist. Traffic Engineering Handbook adapted metric model:

$$Y = t_r + \frac{\left(\frac{5}{18}\right) v_{85}}{2a + 2gG}$$

Where g = gravitational acceleration (m/s²)

t_r = driver perception/reaction time, usually 1.0 seconds

v_{85} = 85th percentile speed (km/h)

a = deceleration of vehicle (m/s²)

G = grade ($\pm$ $[0.0 - 1.0]$)

D-50-1

Le panneau D-50-1 doit être installé en amont d'une intersection lorsque les deux conditions suivantes sont présentes :

– la vitesse affichée est de 80 km/h ou moins;
– la distance permettant de voir les feux de circulation est inférieure à la distance indiquée au tableau 3.9–1.

Ce panneau doit également être installé pendant un mois suivant l'installation de nouveaux feux de circulation.

Tableau 3.9–1
Distance de visibilité des têtes de feux

Vitesse affichée (km/h)	Distance mesurée depuis la ligne d'arrêt (m)
30	50
50	100
60	150
70	200
80	250
90	300
100	400

3.10 Préparez-vous à arrêter

Les panneaux « Préparez-vous à arrêter » (D-60-1 à D-60-4), ainsi que les feux de circulation associés dans le cas du panneau D-60-1, doivent être munis d'un système de relève afin de s'assurer que le fonctionnement des clignotements ou des phases programmées se fait normalement.

Le panneau « Préparez-vous à arrêter » (D-60-1) indique, à l'avance, la proximité d'une intersection comportant des feux de circulation et, par le clignotement des feux jaunes, que ces feux passeront au rouge.

D-60-1

Le clignotement alterné des deux feux du panneau D-60-1 doit débuter avant la fin de la phase verte des feux de circulation et se prolonger pendant toute la durée de la phase rouge de l'approche signalée.

Le début du clignotement doit être calibré de façon à permettre à l'usager de la route arrivant à la hauteur du panneau, juste avant que les feux clignotent, d'atteindre le carrefour, en respectant la vitesse affichée, avant le début de la période de dégagement des feux de circulation.

Le panneau « Préparez-vous à arrêter » (D-60-2) indique, à l'avance, la proximité d'un passage à niveau et, par le clignotement des feux jaunes, que les feux lumineux, au passage à niveau, clignotent.

D-60-2

ALL-RED TIME

Assuming that a vehicle has just entered the intersection on yellow, the all-red time AR must provide sufficient time for the vehicle to cross the intersection and clear its back bumper past the far curb line (or crosswalk line) before conflicting vehicles are given the green.

Not all jurisdictions use the all-red time, assuming clearance is either not an issue or will occur during reaction time of drivers on the following green phase.

- Safety implications

No-pedestrian model:

- $AR = \frac{W+L}{\frac{5}{18}v_{15}}$

Moderate pedestrian model:

- $AR = Max\left[\frac{W+L}{\frac{5}{18}v_{15}}, \frac{P}{\frac{5}{18}v_{15}}\right]$

High pedestrian model:

- $AR = \frac{P+L}{\frac{5}{18}v_{15}}$

If only average speed (v) is known:

- $v_{15} = v - 8$
- $v_{85} = v + 8$

Where:

W = crossing width of intersection

P = crossing width including pedestrian crossings

L = length of a standard vehicle (5.4-6m

v_{15} = 15th percentile speed of approaching traffic

EXAMPLE

Determine yellow and red times for the following intersection:

- Approach speed = 55 km/h
- Downgrade of 2.5%
- W = 14.4 m
- P = 18 m, moderate pedestrian activity
- Deceleration of 3 m/s

LIMITATIONS

There is a hard limit on combined yellow and red time of:

$$3\,seconds\ <\ Y + AR\ <\ 5\,seconds$$

Why?

CRITICAL LANE VOLUMES

The lane with the highest intensity of demand within each phase is defined as the critical lane. Simple volumes cannot be simply compared.

- Trucks require more time than passenger cars,
- Left and right turns require more time than through vehicles
- Vehicles on a downgrade approach require less time than vehicles on a level or upgrade approach.

Thus intensity of demand is not measured accurately by simple volume.

Where phase plans involve overlapping elements, the ring diagram must be carefully examined to determine which flows constitute critical lane volumes.

Ideally, demand volumes would be converted to equivalents based on all of the traffic and roadway factors that might affect intensity.

Table 18.1: Through Vehicle Equivalents for Left-Turning Vehicles, E_{LT}

Opposing Flow V_o (veh/h)	**Number of Opposing Lanes, N_o**		
	1	**2**	**3**
0	1.1	1.1	1.1
200	2.5	2.0	1.8
400	5.0	3.0	2.5
600	10.0*	5.0	4.0
800	13.0*	8.0	6.0
1,000	15.0*	13.0*	10.0*
≥1,200	15.0*	15.0*	15.0*

E_{LT} for all *protected* left turns = 1.05

Table 18.2: Through Vehicle Equivalent for Right-Turning Vehicles, E_{RT}

Pedestrian Volume In Conflicting Crosswalk (peds/h)	**Equivalent**
None (0)	1.18
Low (50)	1.21
Moderate (200)	1.32
High (400)	1.52
Extreme (800)	2.14

Notes on the tables:

- Opposing volume, V_0, includes only ***the through volume*** on the opposing approach, in veh/h.
- Interpolation in Table 18.1 for opposing volume is appropriate, but values should be rounded to the nearest tenth.
- For right turns, the “conflicting crosswalk” is the cross-walk through which right-turning vehicles must pass.
- Pedestrian volumes indicated in Table 18.2 represent typical situations in moderate-sized communities.
  - Pedestrian volumes in large cities, like Montreal, New York, Chicago, or Boston, may be much higher, and the relative terms used and behaviours (low, moderate, high, extreme) are not well correlated to such situations.
- Interpolation in Table 18.2 is not recommended.

SPLITTING THE GREENS

Once we know the cycle length we need to allocate green time to different phases.

- For this we determine the total effective green and allocate the green in ratio of the critical volumes (for each phase or sub-phase)

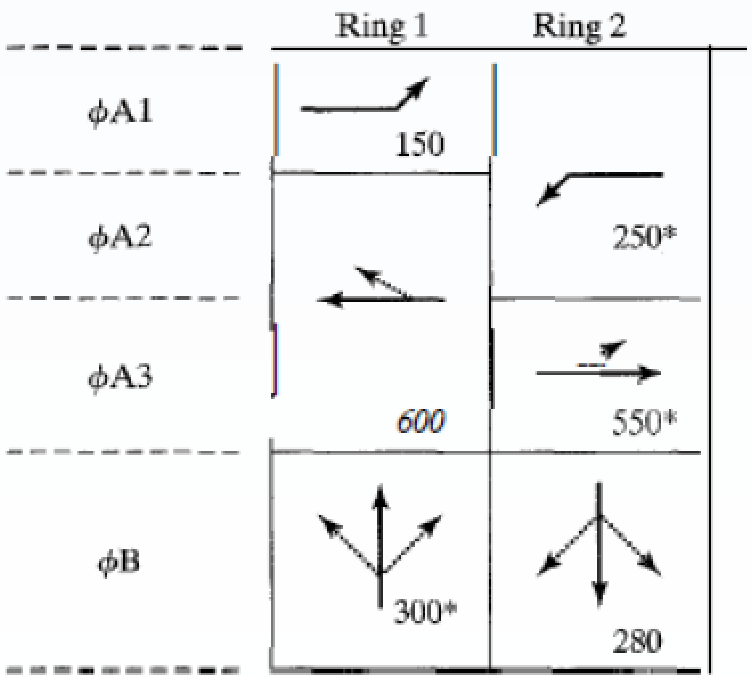
$$g_{TOTAL} = C - L$$

where g_{TOTAL} = total effective green

$$g_i = g_{TOTAL}\left(\frac{V_{ci}}{V_c}\right)$$

EXAMPLE

Split the green time for the following intersection diagram and critical volumes given that $C = 70s$ and $L = 12s$:



PEDESTRIAN REQUIREMENTS

Determination of pedestrian signal needs:

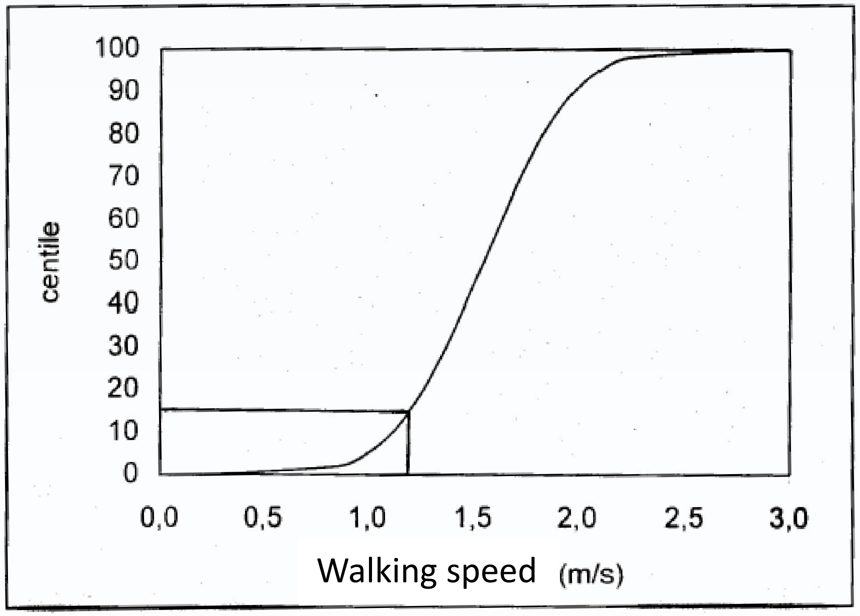
- Determine minimum pedestrian crossing times
- Check to see if vehicular greens meet minimum pedestrian needs (baseline $g_{\min}$ on green split and cycle length)
- If pedestrian needs are unmet by the vehicular signal timing, adjust timing or add pedestrian actuators to ensure pedestrian safety

Consider an intersection of major artery and minor road:

- Vehicle flow on the major artery is more intense than on the small road
  - hence green split for vehicles is more for major road and less for minor road
- This is exactly opposite to what pedestrians would need because the pedestrians get a smaller green to cross the larger width of the road!
- Lets see how to accommodate this!

Walking speed distribution

- 15th centile: $v = 1.2\, m/sec$
- Possible to use a lower centile in special cases (i.e. near retirement home or school zone)



$$T_{crossing} \leq G + Y + AR$$

$$G_{min} = T_{crossing} - Y - AR$$

Several models. HCM 2000:

$$G_{min} = 3.2 + \frac{L}{v} + \left(0.81 \frac{N_p}{W_e}\right) \quad if\ W_e > 3\ m$$

$$G_{min} = 3.2 + \frac{L}{v} + 0.27 N_p \quad if\ W_e \leq 3\ m$$

where L = length of the crosswalk (m)
W_e = width of the crosswalk (m)
N_p = Pedestrian volume **per cycle**
3.2 = Pedestrian reaction yime + safety factor

EXAMPLE

Verify the pedestrian requirements for the following signal plan:

Phase	Green Time G (s)	Yellow + All-Red Y (s)	Lost Time t_L (s)	Pedestrian Requirement G_p (s)
A	40.0	5.0	4.0	20.0
B	15.0	5.0	4.0	30.0

SIGNAL DESIGN CHECKLIST

Signal Design

- Phase plan
  - Based on left and right volumes
- Vehicle requirements
  - Yellow and all red
  - Lost times
  - Equivalent factors
  - Critical volumes
  - Cycle length
  - Splitting the green
- Pedestrian requirements
  - Check if they are met – if not update

That's all for today!