CIVE 440

Traffic Engineering and Simulation – Traffic signals

McGill

Faculty of Engineering
Department of Civil Engineering and Applied Mechanics
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LEVEL III CONTROL

Traffic signals

- Advantages
  - orderly movement of traffic
  - better efficiency at high capacities and increased maximum capacity if well designed and the controller is regularly updated
  - reduce the frequency and severity of certain types of crashes, especially right-angle collisions
  - option of traffic light coordination to reduce idling in the city, and thus reduce delays and greenhouse gas emissions
  - interrupt heavy traffic at intervals to permit other traffic, vehicular or pedestrian, to cross

Traffic signals

- Disadvantages
  - expensive, require regular maintenance and review
  - excessive delay, particularly in low flow conditions
  - disobedience, enforcement
  - may unintentionally reroute unwanted traffic onto side streets
  - other types of collisions increase, particularly:
    - rear-end collisions in the dilemma zone, or
    - reduced caution with false sense of security

JUSTIFICATION

Implementation of a traffic light control requires justification over regular stop-controlled intersections (default). Components:

- Daily traffic flow as well as peak demand
  - Through movement
  - Left turn
  - Pedestrians
- Road safety audits
- Built environment
  - School crossing
  - At-grade rail crossing

Justification design guide:

FHWA (MUTCD Chapter 4)

- Section 4C.01 Studies and Factors for Justifying Traffic Control Signals
  - Warrant 1, Eight-Hour Vehicular Volume
  - Warrant 2, Four-Hour Vehicular Volume
  - Warrant 3, Peak Hour
  - Warrant 4, Pedestrian Volume
  - Warrant 5, School Crossing
  - Warrant 6, Coordinated Signal System
  - Warrant 7, Crash Experience
  - Warrant 8, Roadway Network
  - Warrant 9, Intersection Near a Grade Crossing

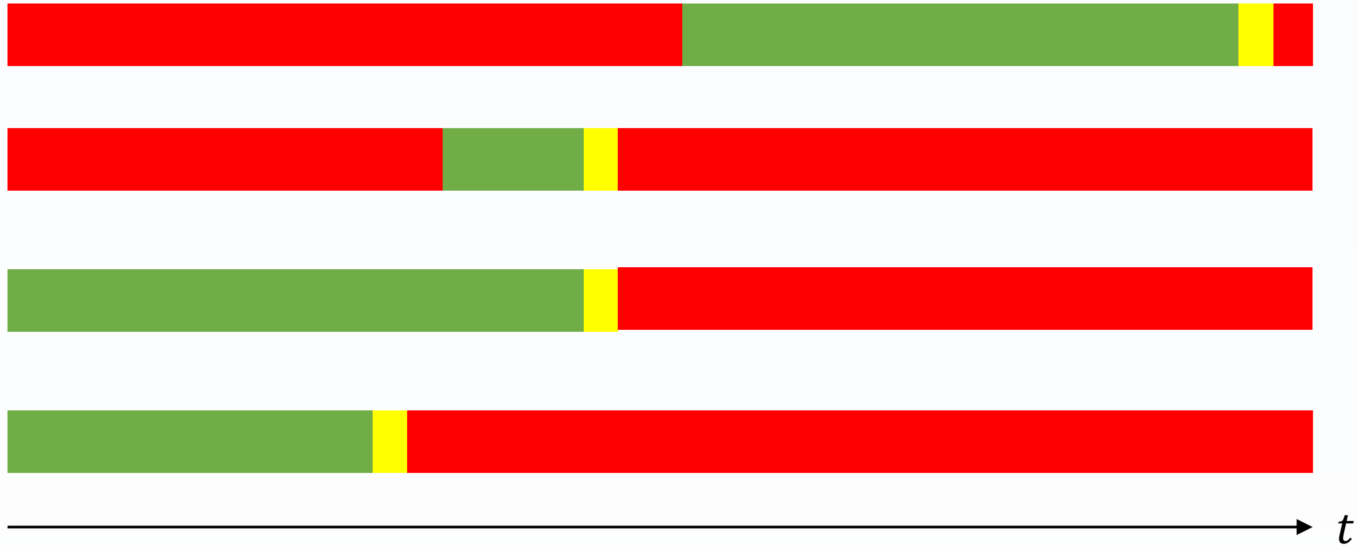
http://mutcd.fhwa.dot.gov/pdfs/2009r1r2/mutcd2009r1r2edition.pdf

Specific to Québec:

MTQ (Tome V, Chapter 8)

- 8.5.1.4 Critères de justification des feux de circulation
  - Critère 1 : Débit minimal de véhicules durant 6 heures
  - Critère 2 : Débit minimal de véhicules durant 4 heures
  - Critère 3 : Débit minimal de véhicules durant une heure
  - Critère 4 : Sécurité
  - Critère 5 : Retard minimal durant une heure
  - Critère 6 : Débit minimal de piétons
  - Critère 7 : Débit minimal d'écoliers

http://www3.publicationsduquebec.gouv.qc.ca/produits/ouvrage_routier/normes/norme6.fr.html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TERMINOLOGY

Cycle:

- A signal cycle is one complete rotation through all of the indications provided. In general, every legal movement receives a "green" indication during each cycle

Cycle length C:

- The time in seconds it takes to complete one cycle.

Phase:

- Set of intervals that allow a designated movement or set of movements to flow safely before conflicting movements begin
- Includes green, yellow and clearance interval

Interval:

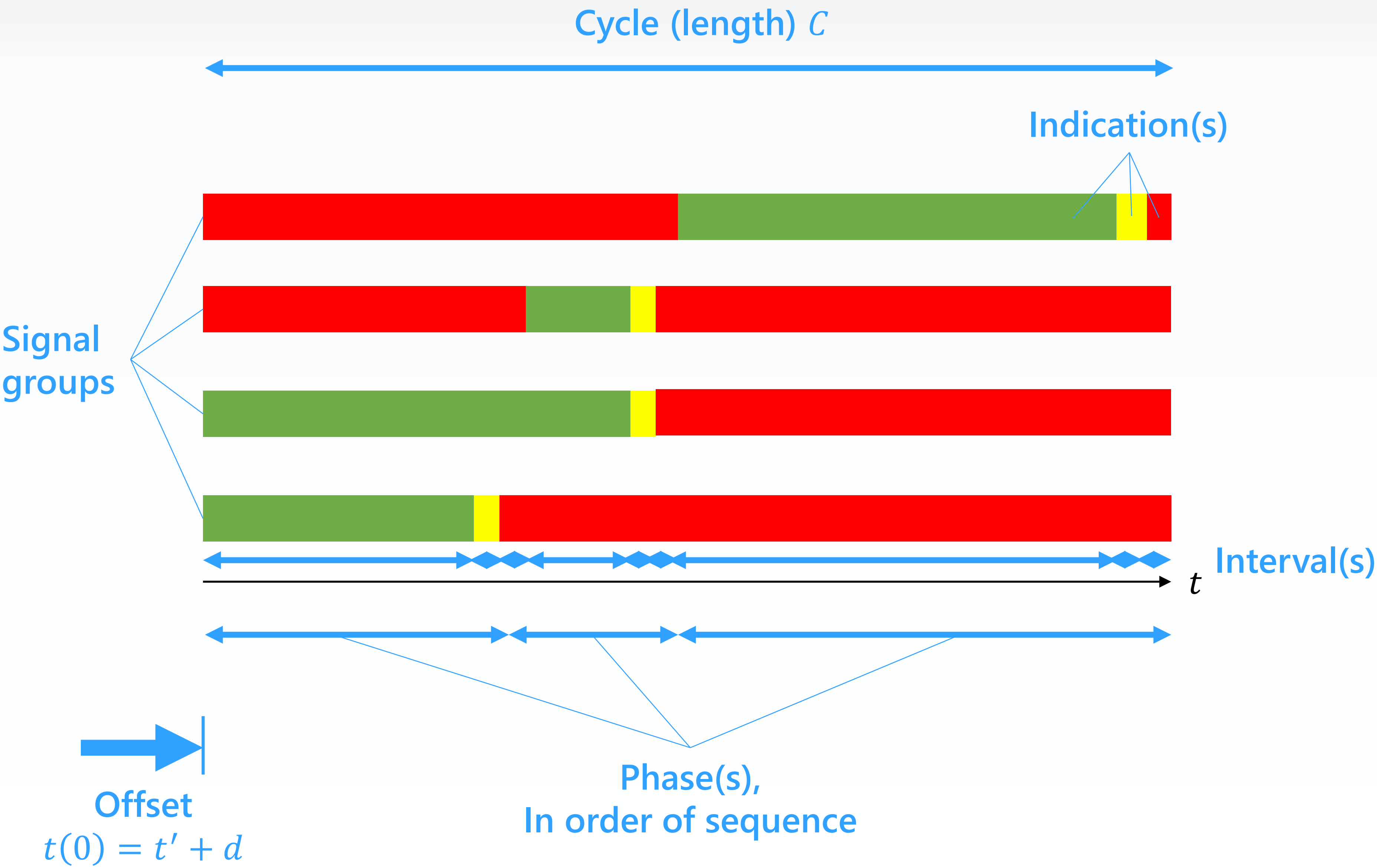
- The period of time during which no signal indication changes. Types of intervals are:
  - Change interval – yellow indication
  - Clearance interval – used to allow vehicles that entered on yellow to leave (all red)
  - Green interval – G_i for the ith flow
  - Red interval – R_i for the ith flow

Signal group:

- Group of lanes/movements which share the same traffic light indication and are thus controlled by the same controller. E.g.: NB/SB, EB/WB.

Offset:

- Demarks the offset in time with respect to other traffic lights for the purpose of coordination.



Cycle (length) C
Indication(s)
Signal groups
Interval(s)
t
Offset
$t(0) = t' + d$
Phase(s),
In order of sequence

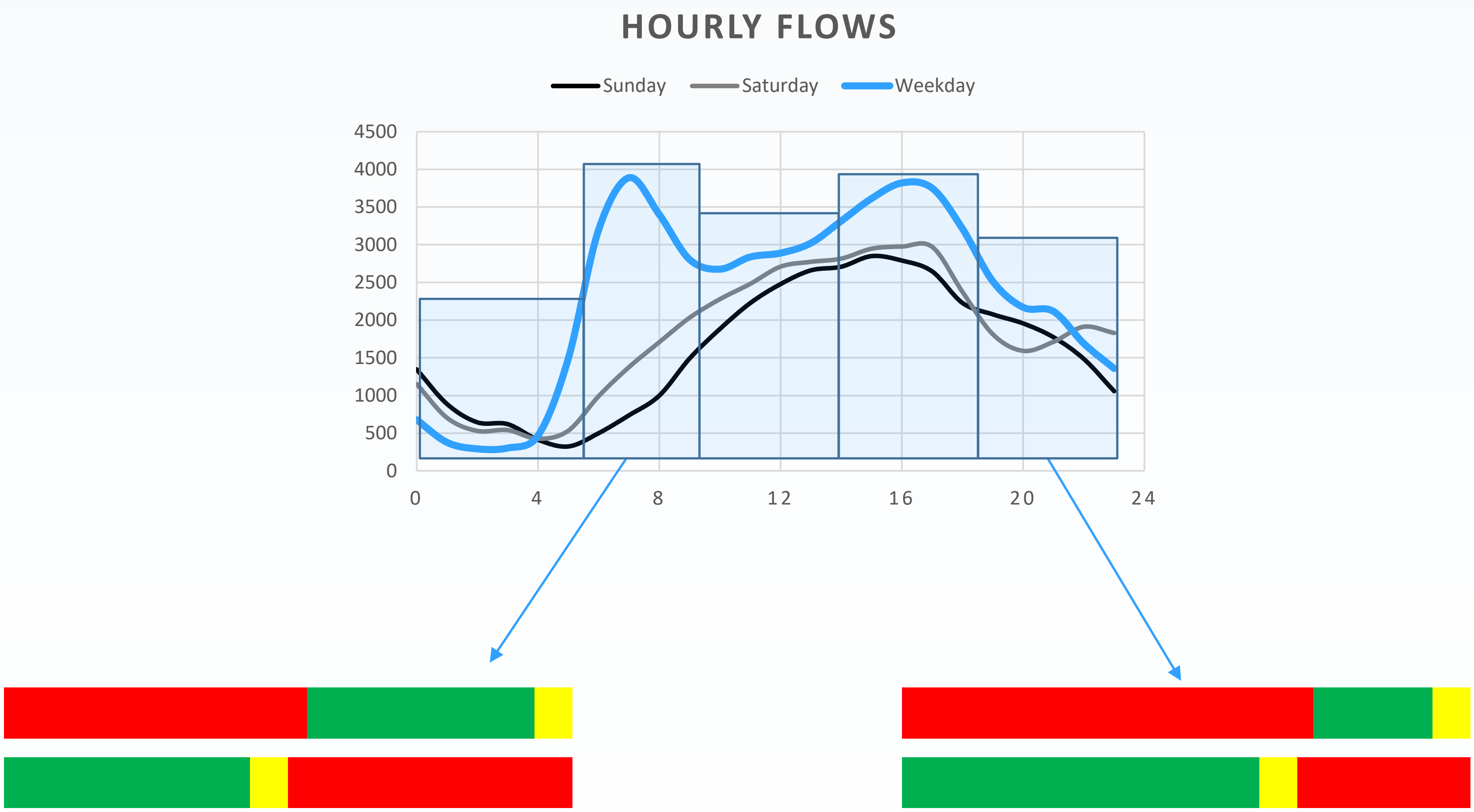
CONTROLLER TYPES

Pre-timed:

- Cycle length, phase sequence, and timing of each interval are constant
- You can set different pre-timed operation for different time periods of the day (TOD)

Semi-actuated:

- Detectors are placed on the minor road. The light remains green for the major road unless there is an actuation on the minor street (until a “failsafe” phase change).
- The switch happens when the minor road’s pre-timed green time expires or when there are no calls to the actuator from the minor road’s detector.


HOURLY FLOWS
Sunday
Saturday
Weekday
4500
4000
3500
3000
2500
2000
1500
1000
500
0
0
4
8
12
16
20
24

Fully actuated:

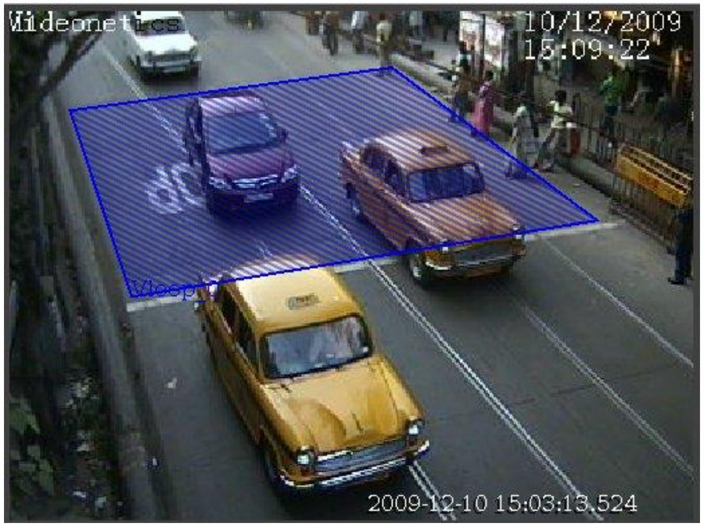
- Detectors placed on every lane of every approach.
- The cycle length, sequence of phases, and green time split may vary from cycle to cycle.

Centralised/real-time control:

- A large number of signals are controlled by a single computer by utilizing data from the detectors throughout the network. Enables:
  - Load balancing
  - Queue counting and dissipation triage
  - Real-time calculation of signal plan
  - Avoid gridlock from resonant cycle times
  - Over watch and monitoring of entire network by a small team of operators.

Actuated and real-time controllers have limited use where traffic light coordination is implemented! Why?


SUBARU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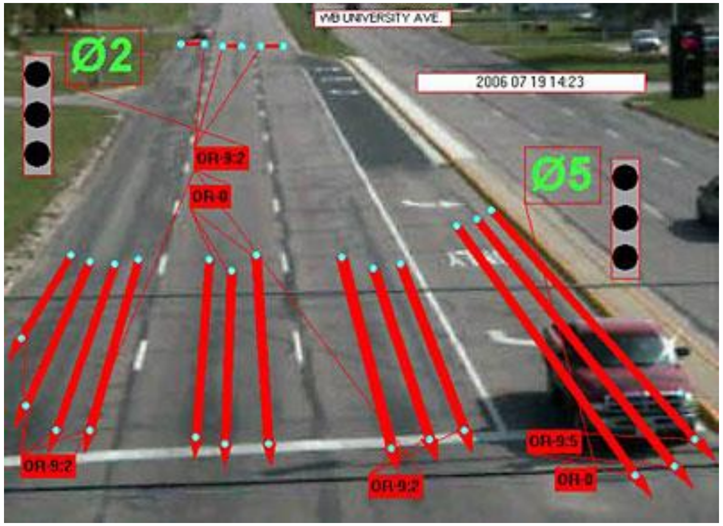
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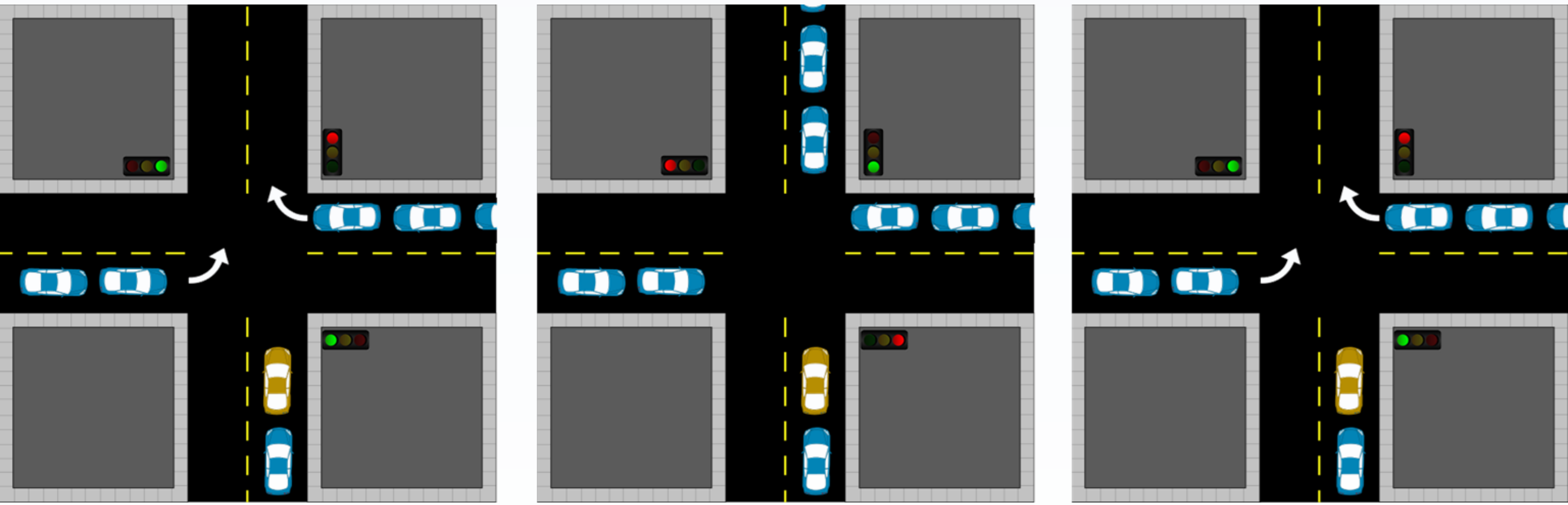
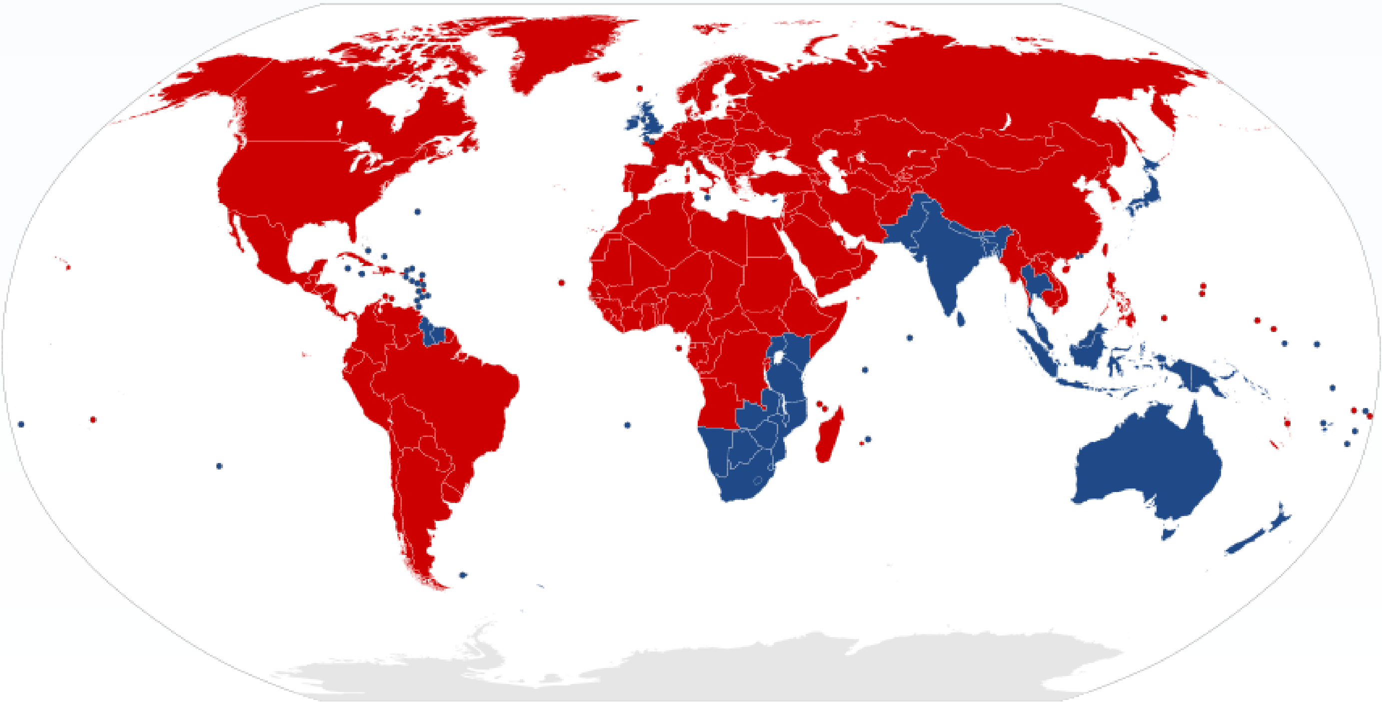


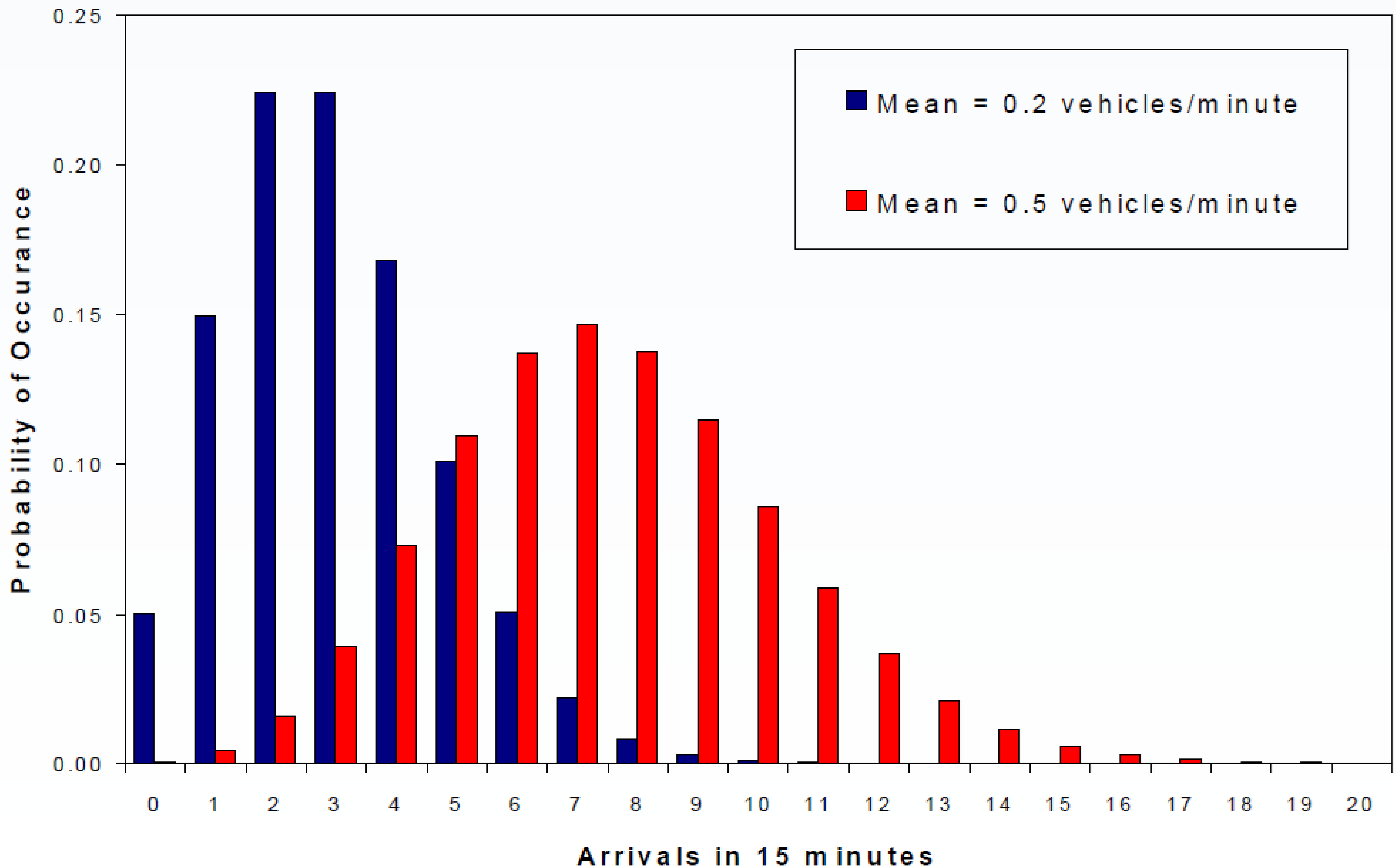
LEFT-TURNS

Left-turns (in drive-on-the-right countries) complicate traffic flow and reduce capacity as they are typically in conflict with oncoming traffic.



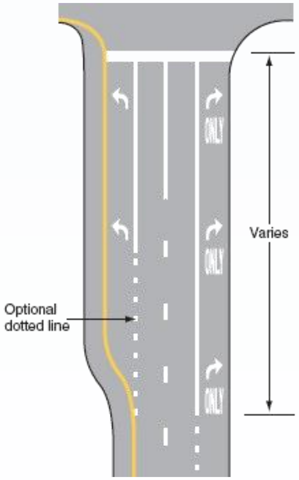
The same principle applies for right-turns for heavily congested pedestrian and cyclist facilities.

Capacity reductions from left-turns are most important when opposing flows are high and uniform.



Counter-measures to ease capacity reductions from left- turns:

- Reduce opposing traffic or convert uniform arrivals into platoons
- Convert to one-way streets (e.g. St-Laurent, Maisonneuve, Rue University)
- Implement dedicated left-turn lanes
- Ban left-turns



Each of these countermeasures has pros and cons.

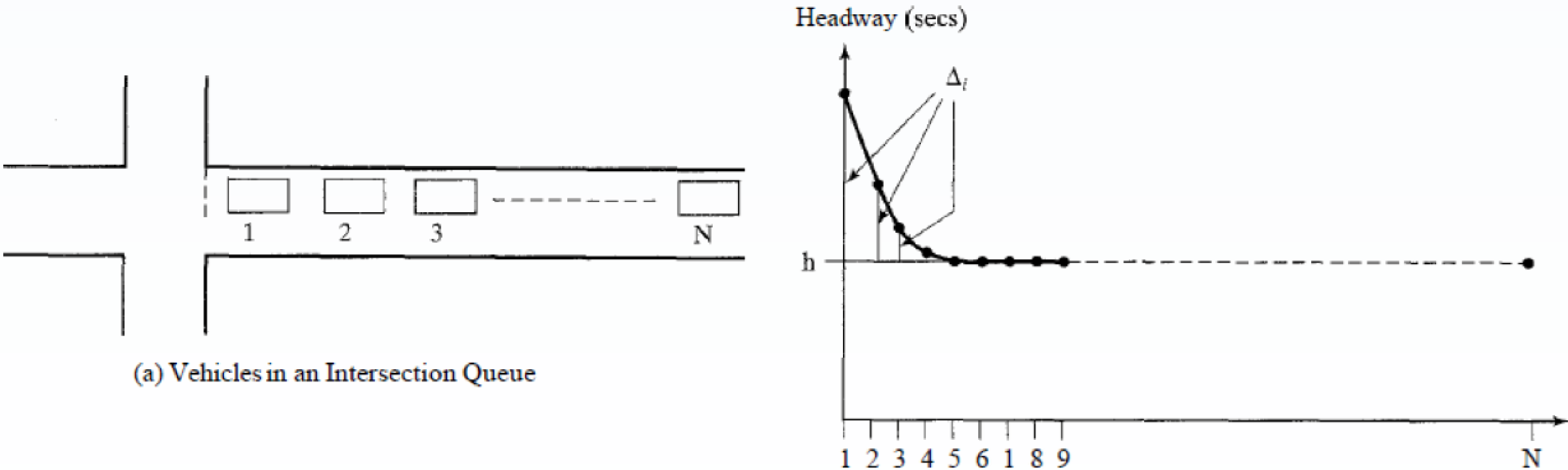
Types of movements:

- Permitted: Movement is permitted, but drivers will be faced with opposing traffic and are responsible for finding a gap in the opposing traffic. Typical of left turns.
- Protected: Movement is permitted and protected by stopping the movement of opposing vehicles. Typically communicated through a flashing green light. Typical of left turns.
  - **Compound left turns**: Mix of permitted and protected phases for the same signal group in any order.
- Un-opposed: Movement is permitted and protected due to the nature of the intersection. Default state for through traffic and typical for rural/suburban right-turns and for various signal groups at T-branch intersections.

TRAFFIC FLOW AT INTERSECTIONS

Traffic flows as a gas, not a liquid: flows are compressible.

- A few seconds after a traffic light turns green:



(a) Vehicles in an Intersection Queue

(b) Average Headways Departing Signal

SATURATION FLOW

Saturation flow $\boldsymbol{s}$ is the maximum theoretical flow rate per hour per lane that flows through a section of road.

- $s \neq c$ (capacity) with the exception of controlled-access highways
- s is the uncontrolled flow rate , i.e. without traffic signals
- additionally, s' is the saturation flow under strict ideal conditions, including:
  - no traffic control
  - no congestion
  - no road obstacles including the road itself
- $s > c$ for un-controlled-access roads

s can be estimated from headway:

$$s = \frac{3600}{h}$$

- where h is the saturation headway in seconds
- s' typically lies within $[1900{,}2200]$ veh/h/ln for human drivers
- s' for automated cars, it is likely orders of magnitude higher, limited primarily by size of vehicle and traveling speed.

s can also be obtained from s' according to geometric and traffic adjustment factors as spelled out in the HCM.

Headways at the start of the intersection are always larger:

- The first 1-2 drivers must react to the green light
- All other drivers react successively to the driver infront of them.
- This effect is mitigated by drivers anticipating the start of leading vehicles:
  - Vehicles further in line incur less delay from startup
- As human drivers increase their speed, they also increase their following distance (headway-spacing relationship is not-linear when gap < car length) in proportion to their speed to maintain safety.

This effect is most pronounced with the first 3-5 vehicles and usually dissipated by either

- the tenth car
- for cars arriving at the end of the queue just as it regains full speed (dissipates)

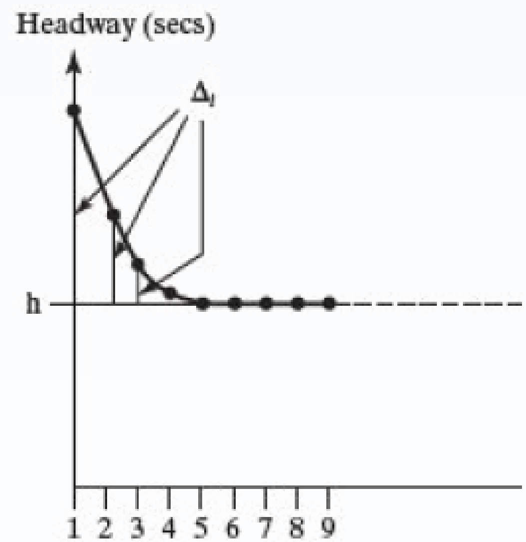
Similar effects can be observed in pedestrians and cyclists, though less pronounced because

- following distances vary much less for pedestrians as pedestrian velocity is less of a safety concern and acceleration is quasi-instantaneous.
- cyclist acceleration varies greatly

The time above h that each vehicle j requires is represented as Δ_j.

Start-up lost time l_1 is given by:

$$(l_1) = \sum_j \Delta_j$$



The total green time required to move n vehicles is given by:

$$G(n) = l_1 + nh$$

There is also a lost time l_2 associated with stopping the queue at the end of the cycle

- Difficult to observe because queue needs to be large enough to consume all the green time
- Defined as the time the last vehicle's wheels cross the stop line and the initiation of the *next* green

We denote the lost time T_{Li} for signal group i:

$$T_{Li} = l_1 + l_2$$

Therefore, the effective green time (g_i) for signal group i is given by:

$$g_i = G_i + Y_i - T_{Li}$$

where: g_i = effective green time for movement(s) i, s
G_i = actual green time for movement(s) i, s
Y_i = sum of yellow and all red intervals for movement(s) i, ($Y_i = y_i + ar_i$)
y_i = yellow interval for movement(s) i, s
ar_i = all-red interval for movement(s) i, s
t_{Li} = total lost time for movement(s) i, s

INTERSECTION CAPACITY

Capacity c_i of an intersection lane or signal group i can be computed as:

$$c_i = s_i \left(\frac{g_i}{C}\right)$$

where: s_i = saturation flow rate for lane or signal group

g_i = effective green time for lane or signal group

EXAMPLE

Calculate the capacity for the signal group with the following parameters:

- Cycle length $C = 60\ s$
- Green time $G = 27\ s$
- Yellow + all-red time $Y + AR = 3.0\ s$
- Saturation headway $h = 2.4\ s/veh$
- Start-up lost time $l_1 = 2.0\ s$
- Clearance lost time $l_2 = 1.0\ s$

CRITICAL LANE

Involves identifying the lane movement shared by a single phase that will control the timing of the green signal of that phase. Consider the following example:

- Two phases
  - One of NB-SB signal groups in one phase
  - One of EB-WB signal group in the following phase
- Traffic intensity differs by lane
- The signal must be timed to accommodate the approach or lane with the highest demand intensity.

The lane with this need is considered the critical lane.

Remember: a phase may be shared by multiple lanes and even multiple signal groups!

The critical lane is not determined from demand. Instead, it is determined from the level of saturation, or the $\boldsymbol{v/c}$ ratio of demand versus the saturation flow (lane capacity).

- The adjusted saturation flow accounts for losses in capacity due to additional factors besides traffic volume such as heavy turning movements and heavy lane changes.

Also, there are multiple phases per cycle. For each phase, there is one critical lane.

LOST TIME

We first determine the primary source of inefficiency introduced by the controller: the total lost time per cycle $\boldsymbol{L}$.

$$L = \sum_i T_{Li}$$

This is static, given a particular choice of phases. Similarly, the total lost time per hour $\boldsymbol{L_H}$ is given by:

$$L_H = L \frac{3600}{C}$$

INTERSECTION CAPACITY

Effective green time in an hour g_H:

$$g_H = 3600 - L_H$$

Theoretical total critical lane capacity (all critical lanes combined):

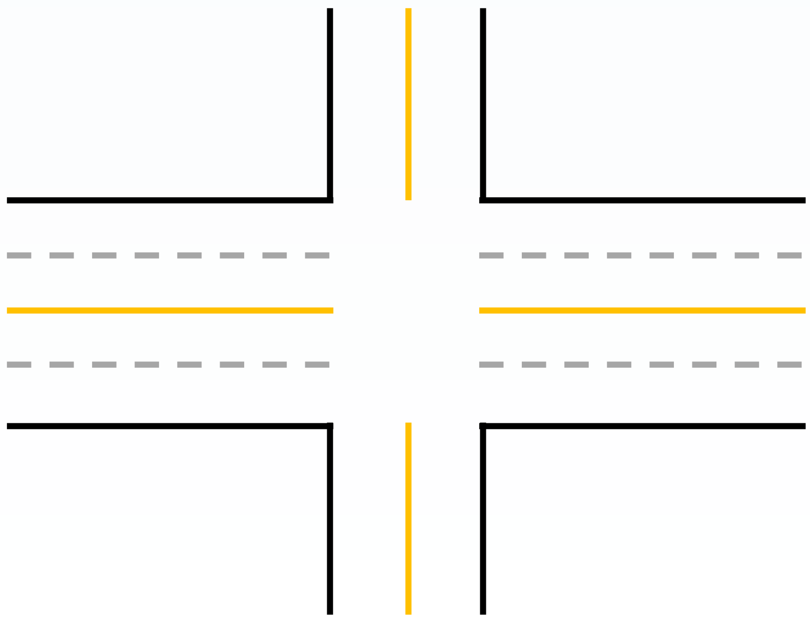
$$Q_c = \frac{g_H}{h}$$

Where: h = saturation headway

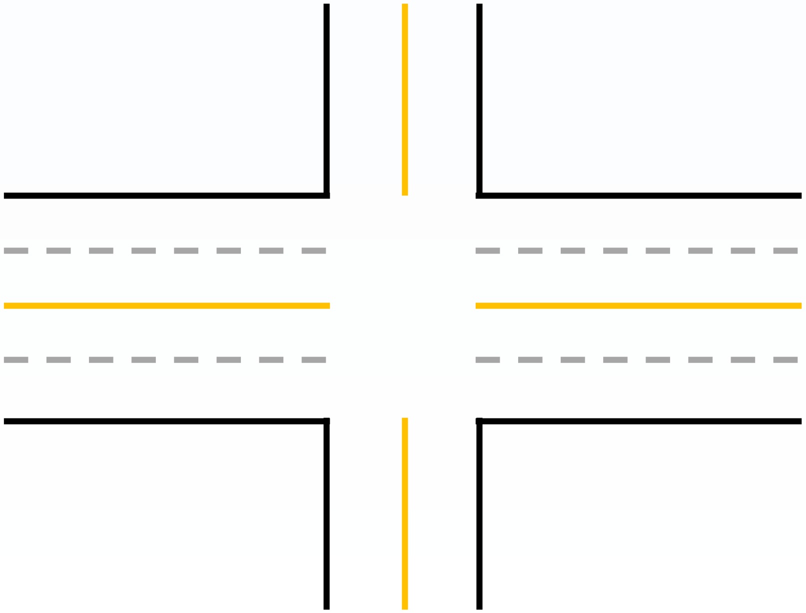
$$Q_c = \frac{3600 - L_H}{h} = \frac{3600\left(1 - \frac{\sum_i T_{Li}}{C}\right)}{h}$$

EXAMPLE

With a two phases 50/50 split, a saturation headway of 2 seconds, lost times of 3 and 4 seconds, and a desired cycle length of 60 seconds, compute the capacity of the following intersection:

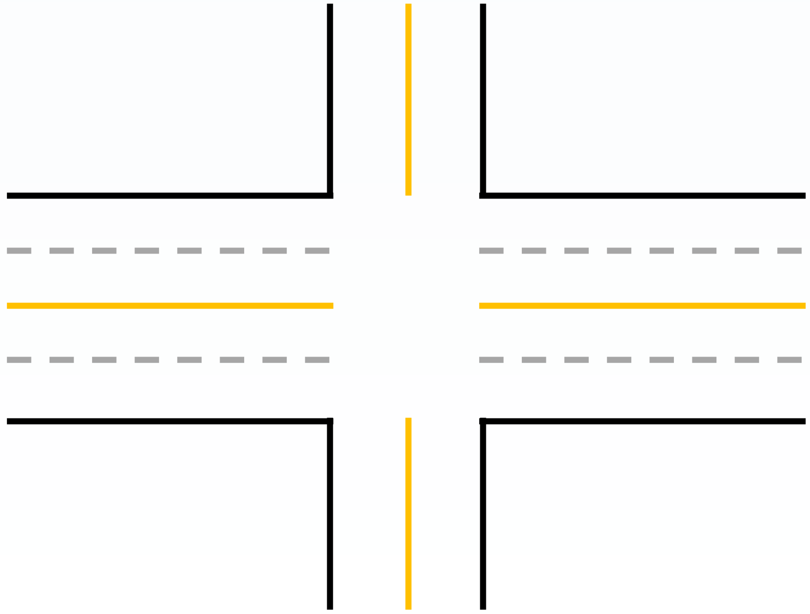


How does the previous example compare with a similar scenario with a saturation headway of 3 seconds?



This calculation is theoretical and simplistic. What process could drastically warp the capacity?

Hint: recall, this is a **two phase** intersection.



Influence of:

- Cycle length
- Number of phases

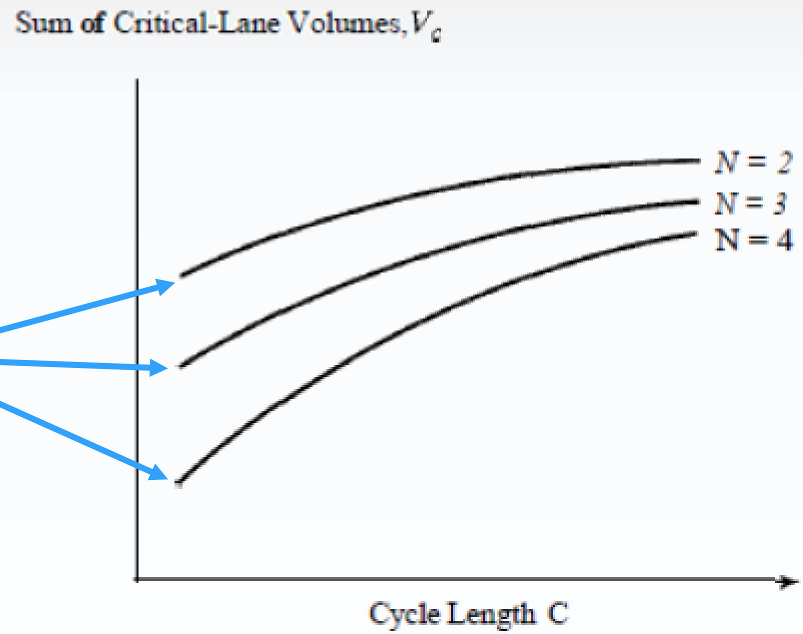


Figure 17.3: Maximum Sum of Critical-Lane Volumes Plotted

Based on this, what is your recommendation for signal design?

SIMPLE OPTIMAL CYCLE

From the previous equation, for a desired capacity, a v/c ratio, and a peak hour factor (PHF), we can calculate a cycle time:

$$C = \frac{\sum_i T_{Li}}{1 - \left(\frac{Q_c h}{3600PHF \times \frac{v}{c}}\right)}$$

- The PHF adjusts for flow intensity in the one hour design period
- $v/c < 1$ ensures that the intersection remains unsaturated

As demand increases:

- desirable cycle length increases exponentially

As v/c increases:

- cycle length increases (for the same capacity)

Aim for a v/c under saturation

- **i.e. $v/c \leq 0.8$**
- **$v/c > 0.9$ tends to fail**

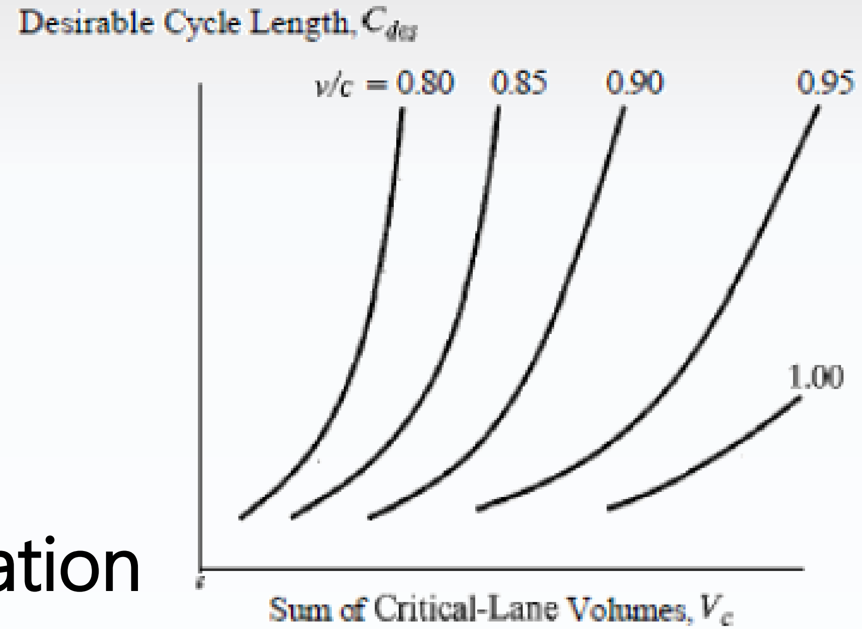


Figure 17.4: Desirable Cycle Length vs. Sum of Critical-Lane Volumes

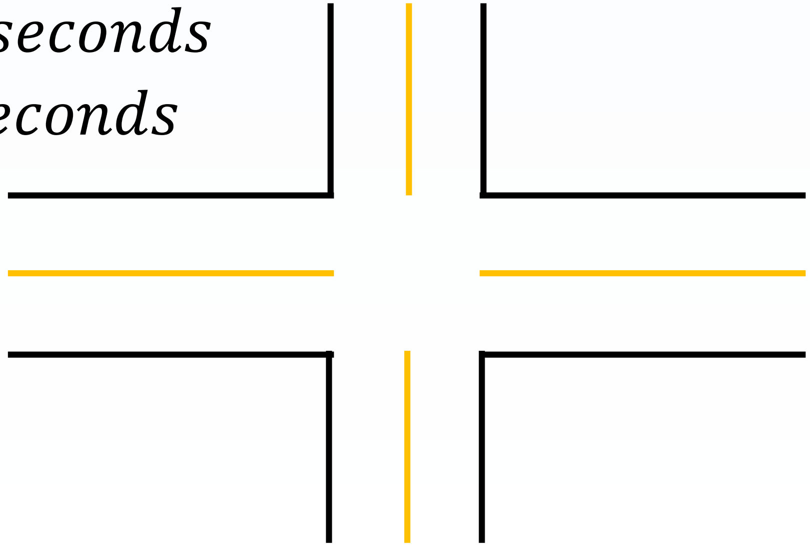
Lower v/c are intrinsically preferable:

- less congestion and less likely that queues extending over multiple cycles will form
- however, a v/c that is too low is an intersection that is overdesigned and wasting money!

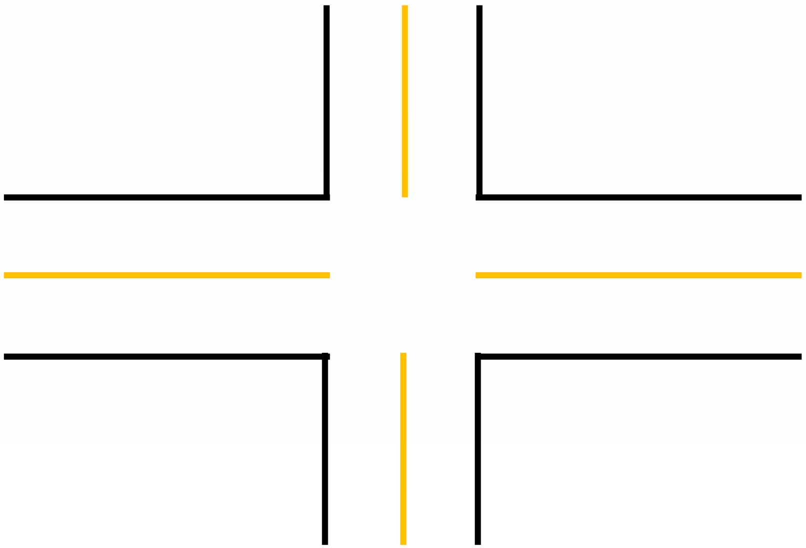
EXAMPLE

Determine an adequate cycle length for the two-phase intersection with the following parameters:

- $Q_c \;=\; 1000\, veh/hr/ln$
- $PHF \;=\; 0.95$
- $v/c \;=\; 0.9$
- $H \;=\; 2.5\, seconds$
- $T_{Li} \;= 4\, seconds$



Design for a $Q_c = 1200$ instead. How does the cycle length compare? Is it reasonable?



There is a hard limit on cycle length imposed by the HCM 2000 design guide:

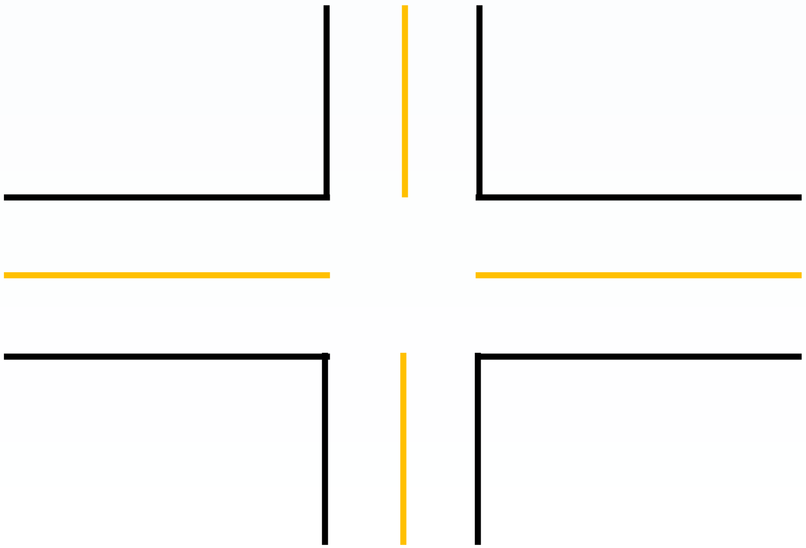
$$\mathbf{40}\boldsymbol{s} \le \boldsymbol{C} \le \mathbf{120}\boldsymbol{s}\text{, using } \boldsymbol{C} = \mathbf{70}\boldsymbol{s} \text{ by default}$$

An exceedingly long traffic light has high recidivism rate and causes excessive uniform delays.

A short traffic light is inefficient (compare L_H!) and should be avoided in general.

- Security risks for vehicles with short cycle times are minimal so this rule is less strictly observed, but pedestrians still need adequate crossing time where applicable!

Design for a $PHF = 0.85$ and $v/c = 0.8$ instead. How does the cycle length compare? Is it reasonable?



ADJUSTED CAPACITY

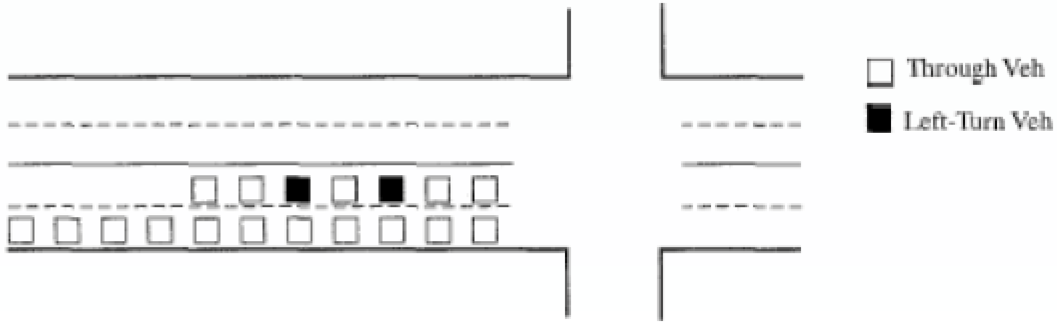
Recall the prior discussion on limitations of theoretical capacity calculations.

- In reality multiple factors influence (i.e. reduce) saturation flow rate (which bottlenecks capacity)
  - Left-turns of all kinds
  - Right turns with heavy pedestrians or cyclist facilities
  - Heavy vehicles
  - Bus stops and busses
  - On-street parking
  - Excessive lane-changing

LEFT-TURN EQUIVALENCY

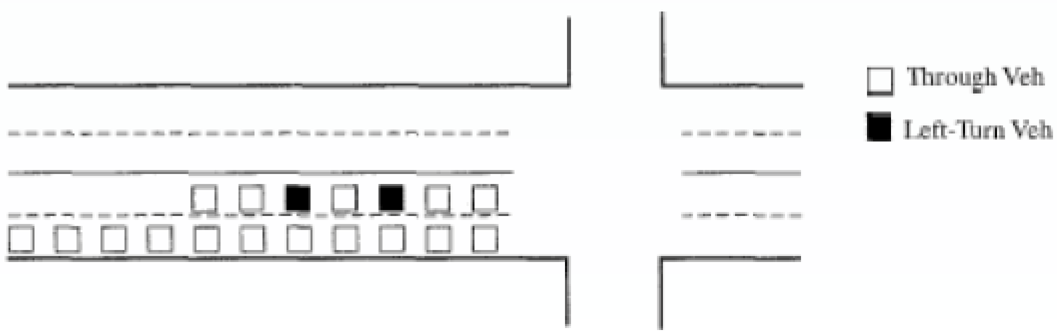
A simplified, empirical adjustment uses the Left-turn Equivalency (LTE) method.

- This is a crude adjustment to saturation flow to account for lost time spent waiting for left turn gaps (including the ensuing lineup of impatient drivers waiting for an opportunity to pass on the right!)



- Left turns may be made from a lane shared with through vehicles (shared-lane operation) or from a lane dedicated to left turning vehicles (exclusive-lane operation)
- LTE only needed for permitted left turn phases. Exclusive protected and unopposed left turn phases are not subject to diminishes in saturation flow rate since movement is unencumbered.

In the following example, lane 1 can serve 11 vehicles while, lane 2 can only serve 2 left-turning vehicles and 5 other cars in the same period of time.

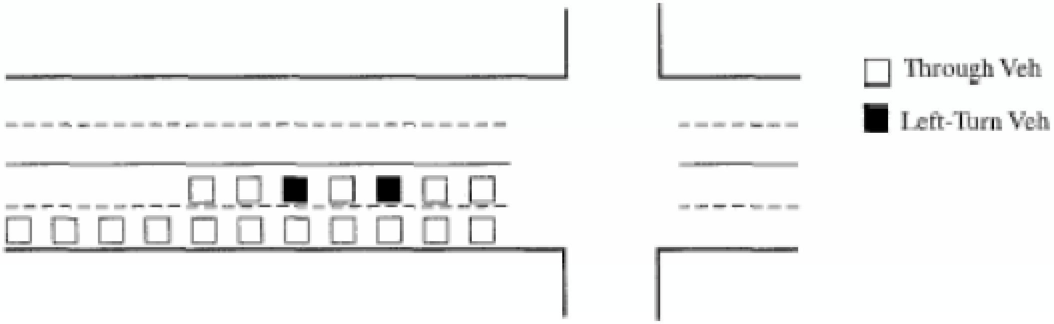


In this example, t represents a unit of time required to process a through vehicle.

$$11\,t = 5t + 2E_{LT}\,t$$

E_{LT} is an equivalency conversion factor between the time taken for through movements and left-turn movements: each left turn takes E_{LT} times the time it takes for a through movement. Rearranging:

$$E_{LT} = \frac{11t - 5t}{2t} = 3.0$$



This equivalency essentially depends on the arrival rate of vehicles in the opposing direction.

- This is modeled through a Poisson distribution and depends on:
  - the average flow rate
  - the number of lanes in the opposing direction
  - location and presence of any upstream traffic lights influencing arrival platoons
- Similar idea for right-turn conflicts with pedestrians and cyclists
- Other equivalents can be devised in a similar manner (e.g. buses and trucks)

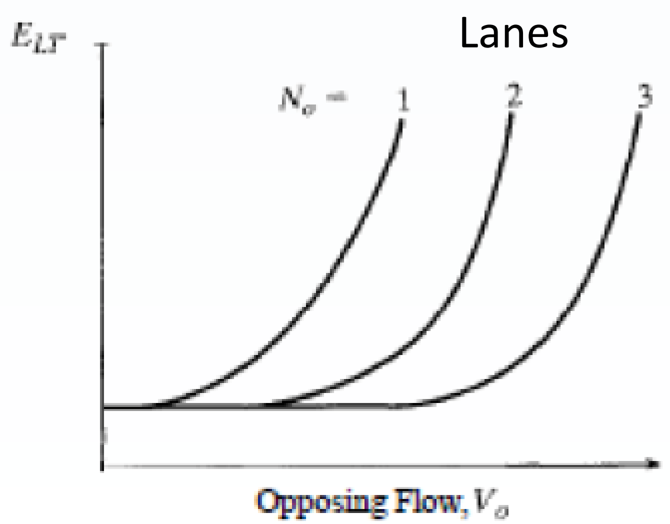


EXHIBIT 23-9. PASSENGER-CAR EQUIVALENTS FOR TRUCKS AND BUSES ON UPGRADES

Upgrade (%)	Length (km)	E_T								
		Percentage of Trucks and Buses								
		2	4	5	6	8	10	15	20	25
< 2	All	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
≥ 2–3	0.0–0.4	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	> 0.4–0.8	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	> 0.8–1.2	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	> 1.2–1.6	2.0	2.0	2.0	2.0	1.5	1.5	1.5	1.5	1.5
	> 1.6–2.4	2.5	2.5	2.5	2.5	2.0	2.0	2.0	2.0	2.0
	> 2.4	3.0	3.0	2.5	2.5	2.0	2.0	2.0	2.0	2.0
> 3–4	0.0–0.4	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	> 0.4–0.8	2.0	2.0	2.0	2.0	2.0	2.0	1.5	1.5	1.5
	> 0.8–1.2	2.5	2.5	2.0	2.0	2.0	2.0	2.0	2.0	2.0
	> 1.2–1.6	3.0	3.0	2.5	2.5	2.5	2.5	2.0	2.0	2.0
	> 1.6–2.4	3.5	3.5	3.0	3.0	3.0	3.0	2.5	2.5	2.5
	> 2.4	4.0	3.5	3.0	3.0	3.0	3.0	2.5	2.5	2.5
> 4–5	0.0–0.4	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	> 0.4–0.8	3.0	2.5	2.5	2.5	2.0	2.0	2.0	2.0	2.0
	> 0.8–1.2	3.5	3.0	3.0	3.0	2.5	2.5	2.5	2.5	2.5
	> 1.2–1.6	4.0	3.5	3.5	3.5	3.0	3.0	3.0	3.0	3.0
	> 1.6	5.0	4.0	4.0	4.0	3.5	3.5	3.0	3.0	3.0
> 5–6	0.0–0.4	2.0	2.0	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	> 0.4–0.5	4.0	3.0	2.5	2.5	2.0	2.0	2.0	2.0	2.0
	> 0.5–0.8	4.5	4.0	3.5	3.0	2.5	2.5	2.5	2.5	2.5
	> 0.8–1.2	5.0	4.5	4.0	3.5	3.0	3.0	3.0	3.0	3.0
	> 1.2–1.6	5.5	5.0	4.5	4.0	3.0	3.0	3.0	3.0	3.0
	> 1.6	6.0	5.0	5.0	4.5	3.5	3.5	3.5	3.5	3.5
> 6	0.0–0.4	4.0	3.0	2.5	2.5	2.5	2.5	2.0	2.0	2.0
	> 0.4–0.5	4.5	4.0	3.5	3.5	3.5	3.0	2.5	2.5	2.5
	> 0.5–0.8	5.0	4.5	4.0	4.0	3.5	3.0	2.5	2.5	2.5
	> 0.8–1.2	5.5	5.0	4.5	4.5	4.0	3.5	3.0	3.0	3.0
	> 1.2–1.6	6.0	5.5	5.0	5.0	4.5	4.0	3.5	3.5	3.5
	> 1.6	7.0	6.0	5.5	5.5	5.0	4.5	4.0	4.0	4.0

EXHIBIT 23-10. PASSENGER-CAR EQUIVALENTS FOR RVs ON UPGRADES

Upgrade (%)	Length (km)	E_R								
		Percentage of RVs								
		2	4	5	6	8	10	15	20	25
≤ 2	All	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
> 2–3	0.0–0.8	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
	> 0.8	3.0	1.5	1.5	1.5	1.5	1.5	1.2	1.2	1.2
> 3–4	0.0–0.4	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
	> 0.4–0.8	2.5	2.5	2.0	2.0	2.0	2.0	1.5	1.5	1.5
	> 0.8	3.0	2.5	2.5	2.5	2.0	2.0	2.0	1.5	1.5
> 4–5	0.0–0.4	2.5	2.0	2.0	2.0	1.5	1.5	1.5	1.5	1.5
	> 0.4–0.8	4.0	3.0	3.0	3.0	2.5	2.5	2.0	2.0	2.0
	> 0.8	4.5	3.5	3.0	3.0	3.0	2.5	2.5	2.0	2.0
> 5	0.0–0.4	4.0	3.0	2.5	2.5	2.5	2.0	2.0	2.0	1.5
	> 0.4–0.8	6.0	4.0	4.0	3.5	3.0	3.0	2.5	2.5	2.0
	> 0.8	6.0	4.5	4.0	4.5	3.5	3.0	3.0	2.5	2.0

EXHIBIT 23-11. PASSENGER-CAR EQUIVALENTS FOR TRUCKS AND BUSES ON DOWNGRADES

Downgrade (%)	Length (km)	E_T Percentage of Trucks			
		5	10	15	20
< 4	All	1.5	1.5	1.5	1.5
4–5	≤ 6.4	1.5	1.5	1.5	1.5
4–5	> 6.4	2.0	2.0	2.0	1.5
> 5–6	≤ 6.4	1.5	1.5	1.5	1.5
> 5–6	> 6.4	5.5	4.0	4.0	3.0
> 6	≤ 6.4	1.5	1.5	1.5	1.5
> 6	> 6.4	7.5	6.0	5.5	4.5

SATURATION FLOW MODEL

A more comprehensive prediction model has been prepared by the HCM as part of chapter 16.

- Starts with an ideal saturation flow rate s'
- A series of eight adjustment factors f_j to take into account various geometric and traffic characteristics of lane i

$$s_i = s' \times \prod_{j=1}^{8} f_j$$

Capacity given by:

$$c_i = \frac{s_i g_i}{C}$$

Cycle design approach from saturation flow:

- For each lane, determine v/s ratio. For each phase, the lane with the highest v/s ratio among the lanes being processed by that phase is the critical lane.
- For a desired v/c ratio, C can be determined from the relationship:

$$\frac{v}{c} = \sum \left(\frac{v}{s}\right)_{ci} \left(\frac{C}{C-L}\right)$$

where ci = a critical lane group

PEAK HOUR FACTOR

The peak hour factor (PHF) is a factor that converts any hourly flow into an equivalent 15-minute intensity (flow per hour) and is very simple to calculate:

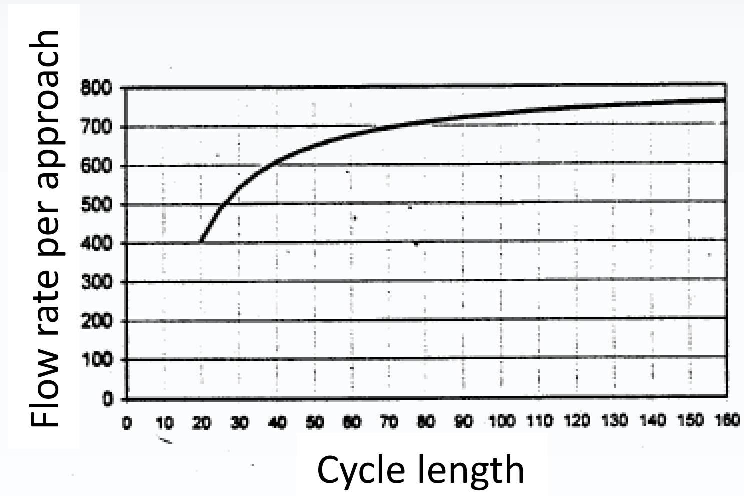
$$PHF = \frac{Q}{4Q_{15}}$$

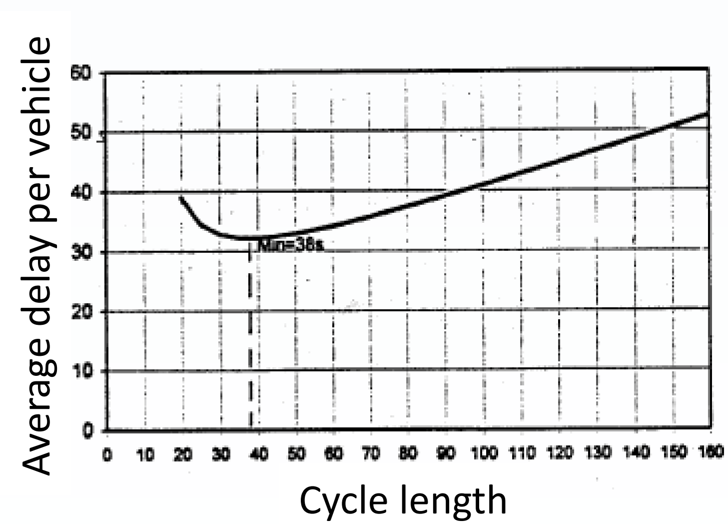
where Q_{15} = the maximum 15 min volume in the hour per 15 minutes

EFFECT OF CYCLE ON DELAY

Long cycles increase average delay.

- The longer a red light, the longer drivers will be made to wait on average under unsaturated conditions.
- If the flow is saturated and drivers are waiting multiple cycles because of congestion, this delay has no effect.
  - **Thus, use long cycles (within hard limits) for high-flow, congested intersection and short cycles for low-flow, unsaturated intersections.**
- Some empirical optimisation equations have been devised…


Flow rate per approach
800
700
600
500
400
300
200
100
0
0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160
Cycle length


Average delay per vehicle
60
50
40
30
20
10
0
Min=38s
0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160
Cycle length

That's all for today!