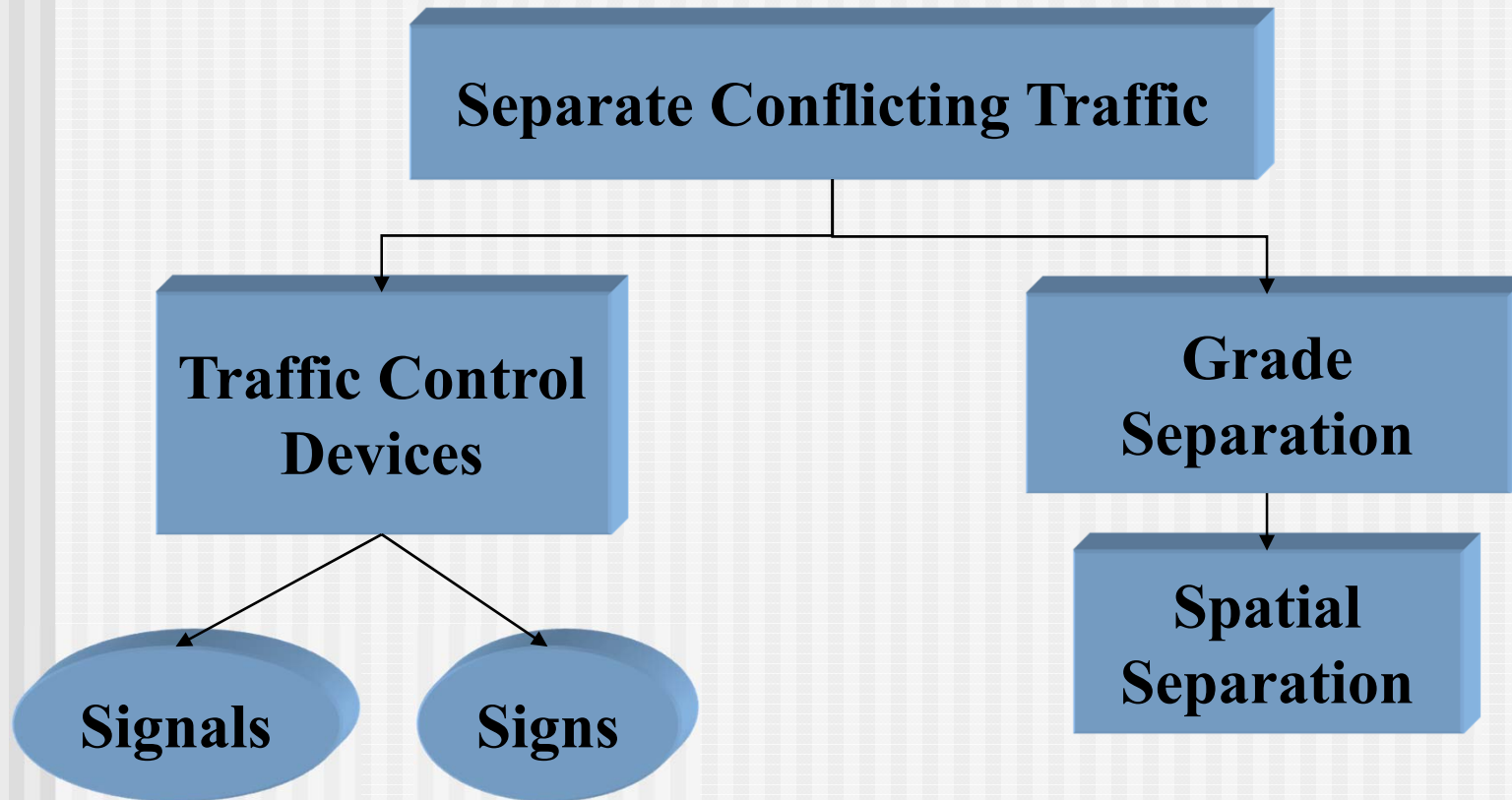


CE 7670: Advanced Traffic Signal Systems

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Winter 2003

Intersections





Middlebelt Road and Five Mile Road Intersection, Livonia, MI

SB Approach



NB Approach



Warren Avenue and Mack Avenue Intersection, Detroit, MI

SB Approach



NB Approach

Intersection of Grand River Avenue, Milford Road and Pontiac Trail, Lyon Township, MI



Milford Road approach



Pontiac Trail approach



Grand River Avenue approach

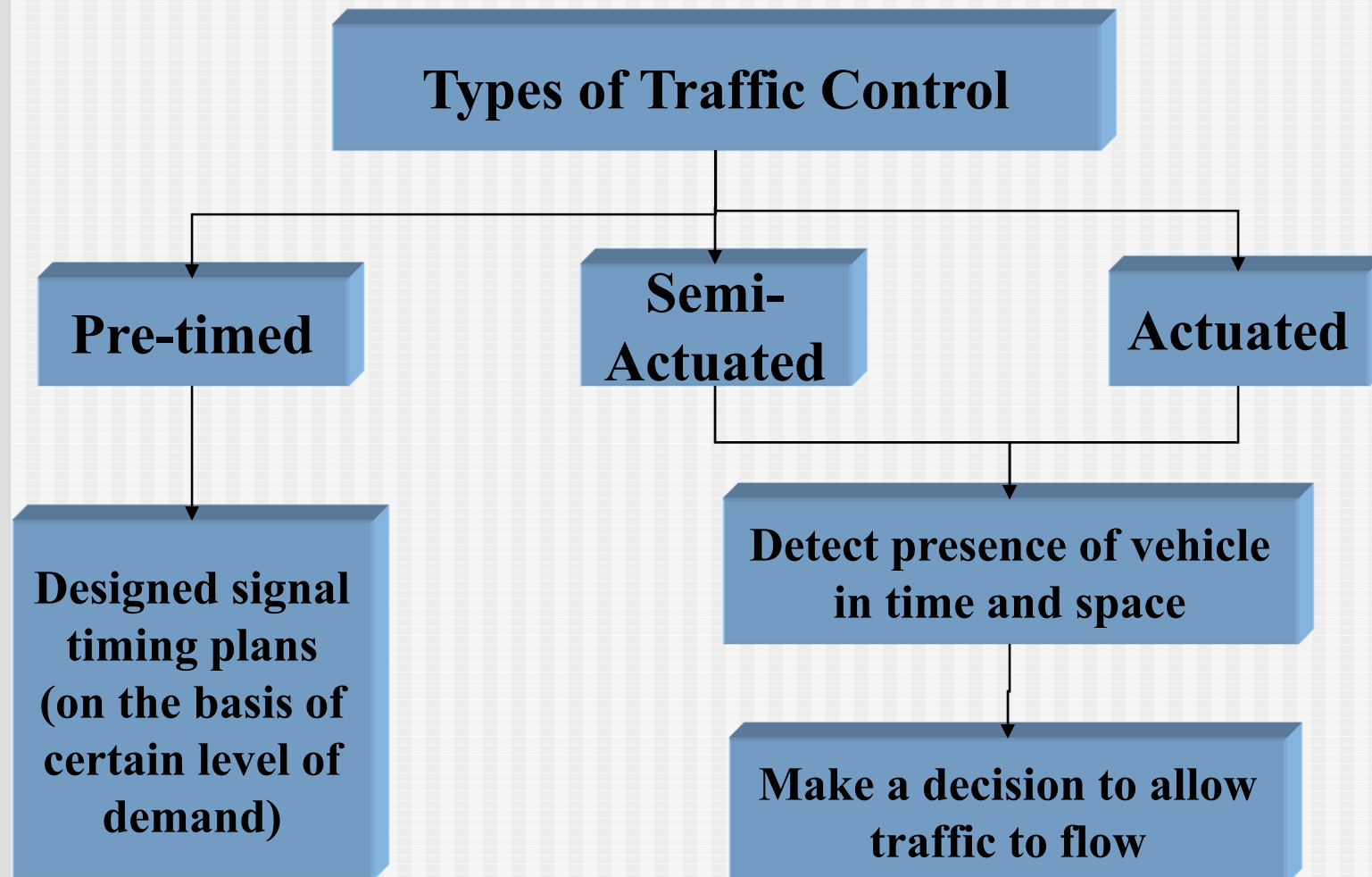
Signal Timing Improvement Practices

NCHRP 172

- Advantages of traffic signals
 - provide for the orderly movement of traffic
 - increase the traffic-handling capacity of the intersection
 - reduce the frequency of certain types of crashes
 - Right-angle
 - Left-turn head-on
 - Rear end

-
- coordinated to provide for continuous movement of traffic at a definite speed along a given route
 - interrupt heavy traffic at intervals to permit other traffic, vehicular or pedestrian, to cross

Signalized Intersections



Types of Traffic Control

■ Pre-timed

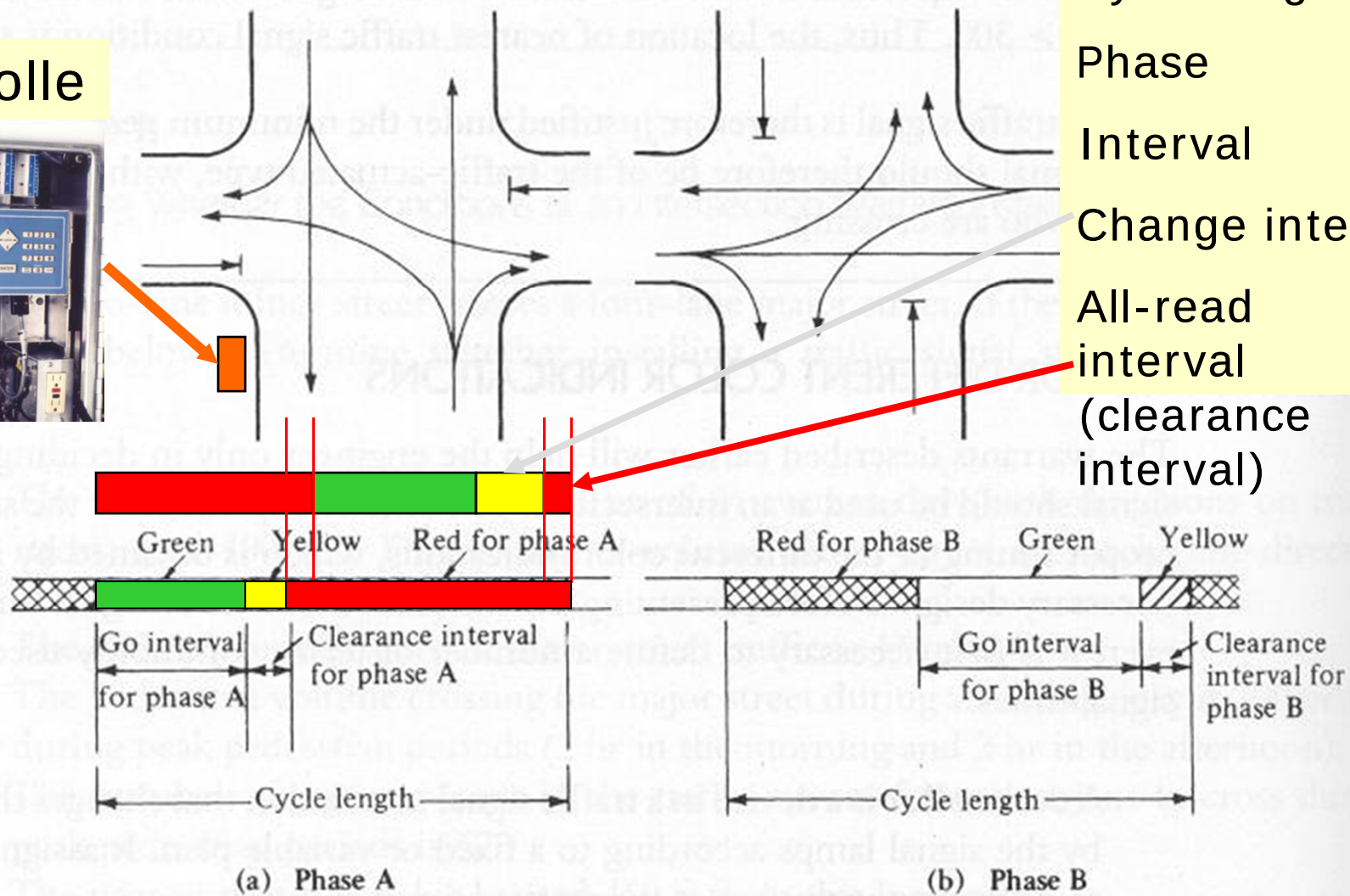
- operates on a clock
- same cycle length and split for the designed period

■ Actuated

- makes use of detectors (sensors)
 - buried in the road
 - video detection
 - at all approaches
 - at some approaches- semi-actuated

Terms relating to signal timing

Controller



■ Actuated

- give green time only to the approaches with waiting vehicles
- change the signal as soon as they have been served
- used where traffic volumes are not steady



**Burton and Eastern
Grand Rapids, MI**



Burton and Kalamazoo
Grand Rapids, MI





**Ottawa and Michigan
Grand Rapids, MI**

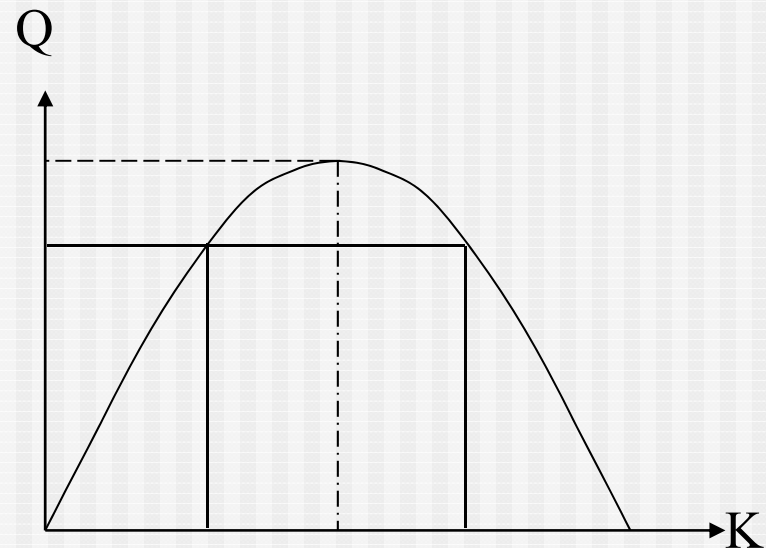
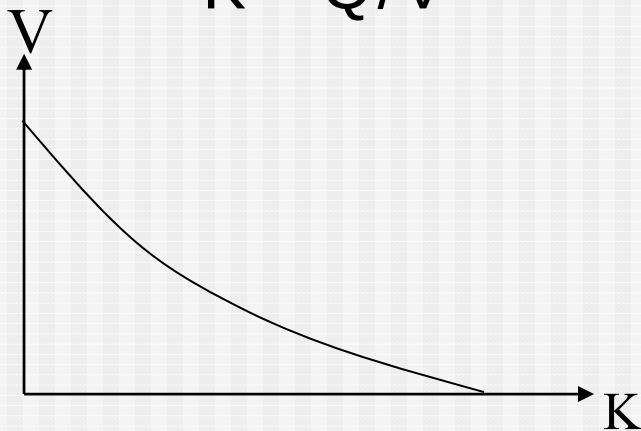


Ottawa and Michigan Grand Rapids, MI



3 Parameters of Traffic Flow

- Macroscopic:
 - Speed (V)
 - Density (K)
 - Flow Rate (Q)
 - $K = Q/V$

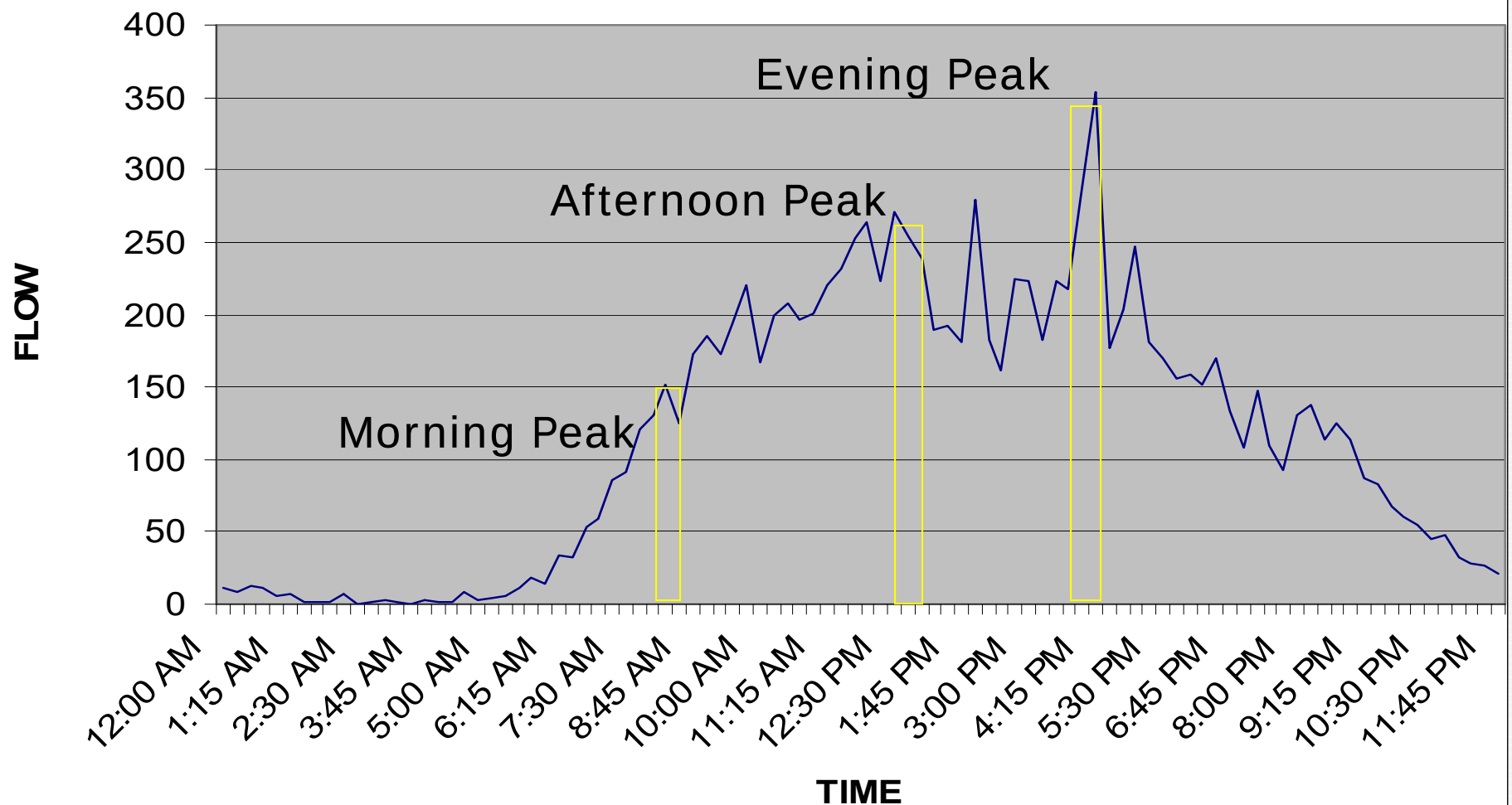


Volume

- AADT- Average Annual Daily Traffic
- ADT – Average Daily Traffic
- Hourly Volume and Use
 - Peak hour volume- vehicles per hour
 - Capacity analyses
 - Safety analyses
 - Operational analyses –traffic signals
 - DHV: design hourly volume, 30th highest hourly volume

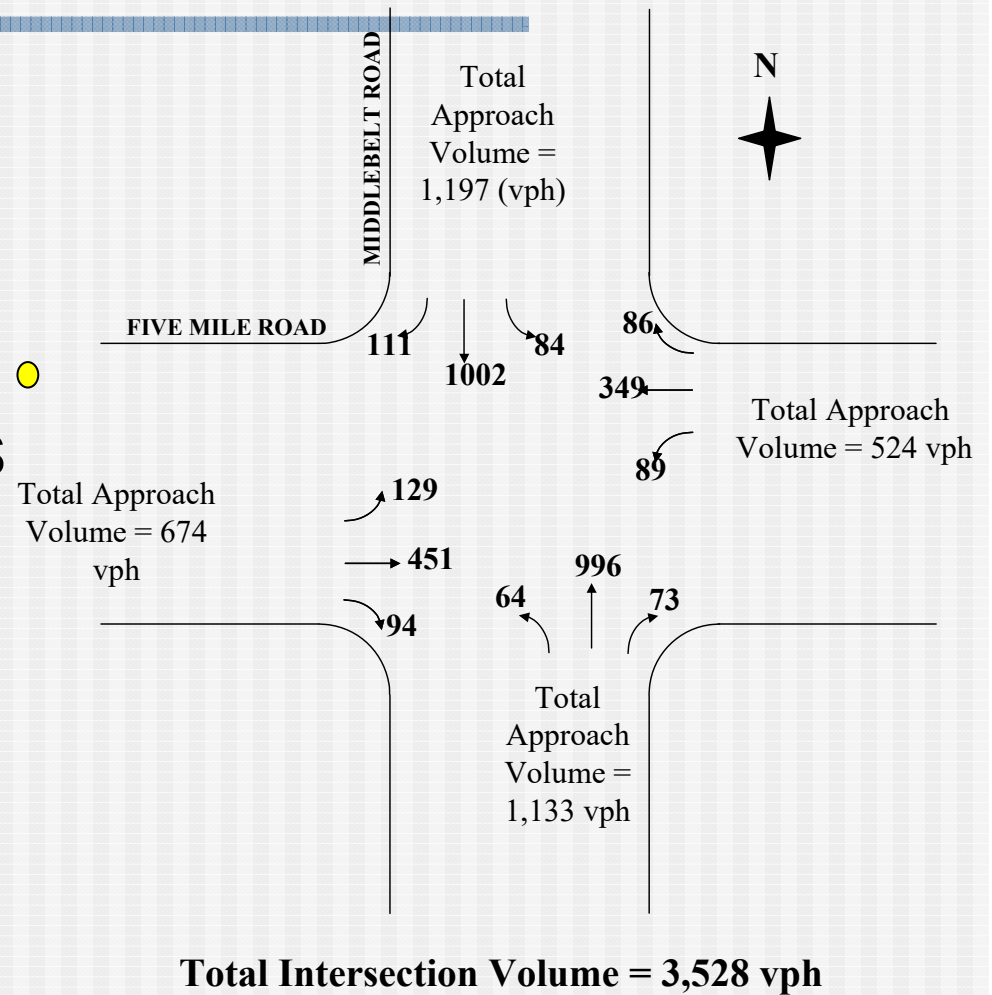
Traffic Volumes on Kercheval Road, Grosse Pointe Farms

ADT = 10,889 vpd



■ Traffic Volume

- Approach volume
- Turning counts
- Classification counts
 - Trucks
 - Buses



- Flow rates
- Peak Hour Factor

$$PHF_{\text{intersections}} = \frac{\text{Peak hour volume}}{(\text{Peak 15 minute volume}) * 4}$$

$$PHF_{\text{freeways}} = \frac{\text{Peak hour volume}}{(\text{Peak 5 minute volume}) * 12}$$

Transportation and Traffic Engineering Handbook

- The objective of signal timing
 - alternate the right of way between traffic streams
 - Minimize average delay to all vehicles and pedestrians,
 - Minimize total delay to any single group of vehicles and pedestrians
 - Minimize possibility of crash-producing **conflicts**

Traffic Conflict

- An evasive action taken by a driver to avoid an impending collision

Signal Timing Design for Isolated Intersections

- Cycle length
 - shortest cycle that will accommodate the demand present and produce the lowest average delay
 - Typical range = 60 seconds to 120 seconds

■ Minimize intersection delay

■ Delay = actual time – expected time

- Function of each individual vehicles, driver behavior, etc

■ Types of delay

- Travel time delay- hard to measure at an intersection
- Stopped time delay- physical counting and analysis



Rochester and Wattles Intersection,
Troy, MI

Webster's Delay Model

■ Average delay/vehicle (d)

$$d = \frac{C (1 - \lambda)^2}{2 (1 - \lambda x)} + \frac{x^2}{2q (1 - x)} - \underbrace{0.65 \left(\frac{c}{q^2} \right)^{1/3} * x^{(2+5x)}}_{\text{This term accounts for 10\% of the delay}}$$

Where:

c = cycle length

x = degree of saturation = $q/(\lambda s)$

q = flow rate

λ = g/c ratio

s = saturation flow

g = green time

-
- Thus, the equation can be re-written as:

$$d = \frac{9}{10} \left[\frac{C (1 - \lambda)^2}{2 (1 - \lambda x)} + \frac{x^2}{2q (1 - x)} \right]$$

Example

- Given:

- $Q = 600 \text{ vph}$
 - $q = 600\text{vph}/3600 \text{ sec per hour}$
 - $= 1/6 \text{ vehicles per sec}$
- $G = \text{green time} = 28 \text{ seconds}$
- $Y = \text{yellow interval} = 4 \text{ seconds}$
- $C = \text{cycle length} = 60 \text{ seconds}$
- 15 vehicles are discharged in fully loaded green
- Assume, saturation flow = 1,800 vph

$$\text{Effective green} = \underset{\substack{\text{green} \\ \text{time}}}{28 \text{ sec}} + \underset{\substack{\text{yellow} \\ \text{time}}}{4 \text{ sec}} - \underset{\substack{\text{starting} \\ \text{delay}}}{2 \text{ sec}} = 30 \text{ sec}$$

$$\lambda = g/c = 30/60 = \frac{1}{2}$$

$$x = q/(\lambda s) = \frac{600}{0.5 * 1800} = \frac{2}{3}$$

$$d = \frac{9}{10} \left[\frac{60 \left(1 - \frac{1}{2}\right)^2}{2 \left(1 - \frac{1}{2} * \frac{2}{3}\right)} + \frac{\left(\frac{2}{3}\right)^2}{2 * \frac{1}{6} \left(1 - \frac{2}{3}\right)} \right]$$

$$d = 13.725 \text{ sec/vehicle}$$

Webster's Equation for Cycle Length

- Based on computer simulation and field observations
- To minimize delay

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

Where:

C_{opt} = optimum cycle length

L = lost time = starting delay, usually 1-2 sec per phase

Y = $f(\text{no. phases, ratio of approach vol./saturation vol.})$

$$Y = \sum y_i \quad y_1 = q_i/s_i = \text{flow/saturation flow}$$

-
- Very sensitive to small changes in lost time and saturation flow
 - For moderate traffic volumes the equation tends to yield very short cycle length
 - For heavy traffic volumes, where Y approaches 1.0, the equation will produce very long cycle lengths
 - The Webster calculation should be used as a “pointer” for selection within a range of predetermined acceptable cycle lengths

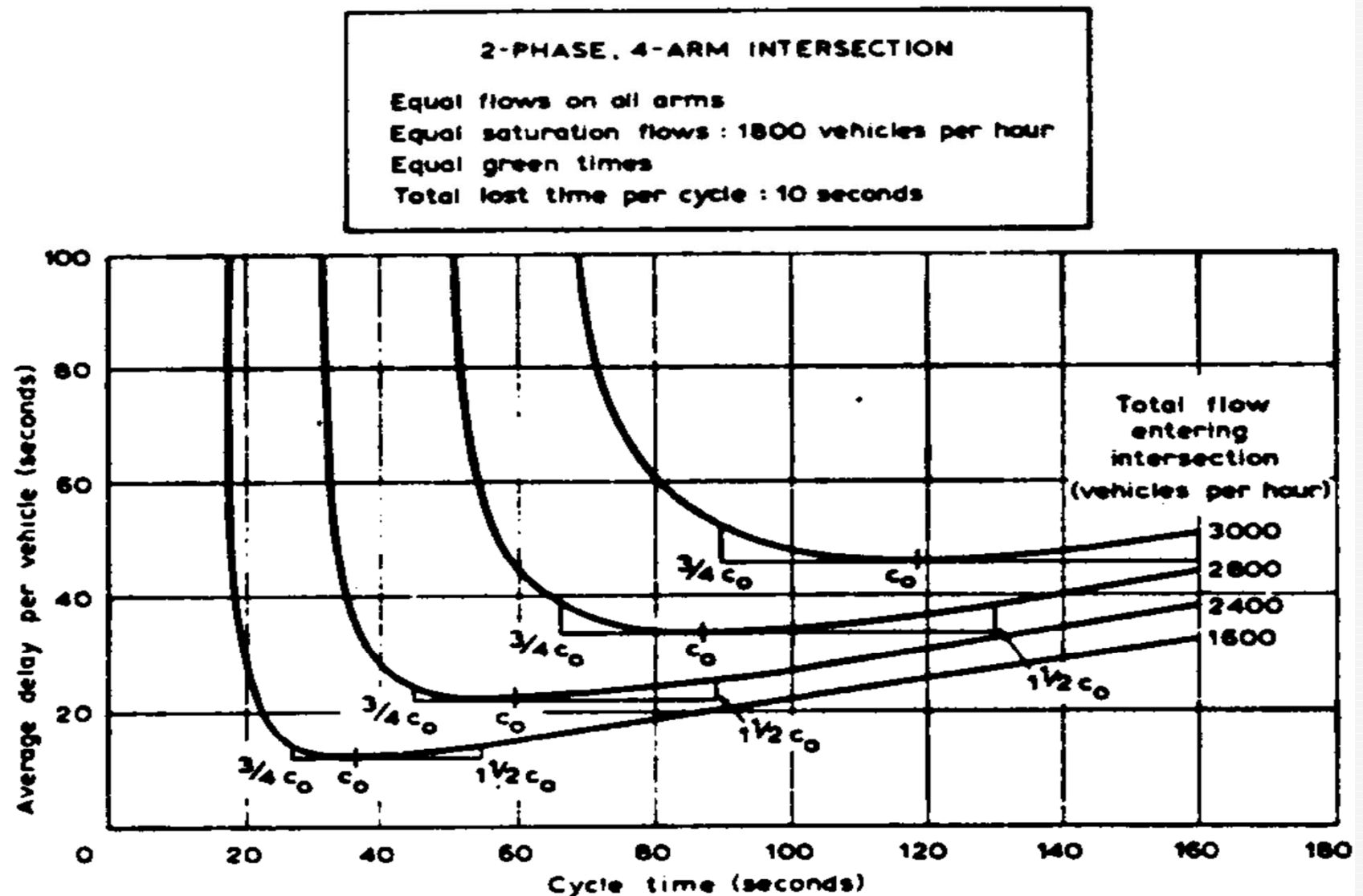


FIGURE 1 Effect on delay of variation in cycle length.

Saturation Flow

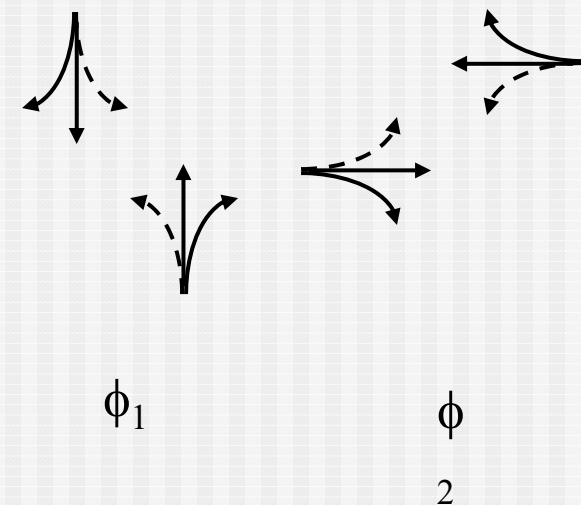
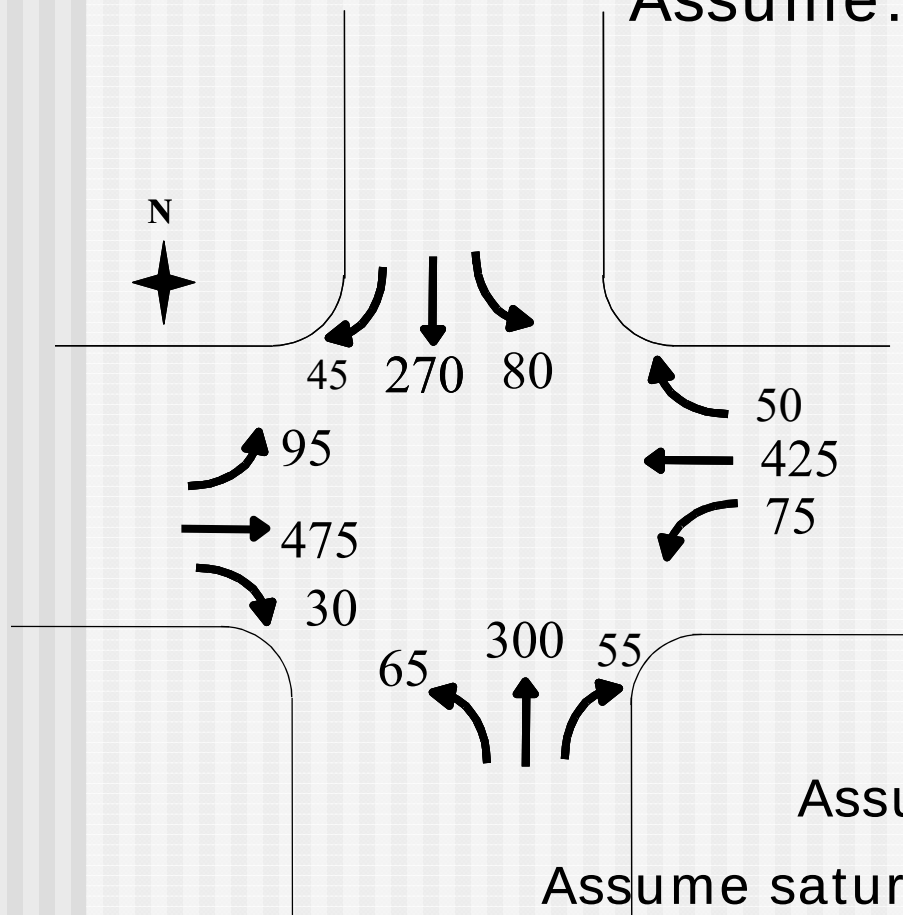
- Observed in the field during peak hours
 - Maximum number of vehicles per unit of time per green cycle observed as:
 - at least one car waiting to be served at the beginning of the green, and
 - a full stream of vehicles passing through the green, and
 - At least one car waiting at the end of the green cycle
 - Discard the flow altogether if there are gaps in the traffic stream

Saturation Flow

- Count number of vehicles for 10 signal cycles
 - At one intersection
 - At one approach
 - Excluding left-turn vehicles
- Result is vehicles per hour of green (vphg)

Example: Webster's Model

Assume: 3 x 3 lane road
Simple Two-Phase Design



Assume saturation flow = 1,800 vphg
Assume saturation flow (left turn) = 1,000 vphg

$$y_{N-S} = \max[(300+55)/1800, (270+45)/1800, 65/1000, 80/1000] = 0.197$$

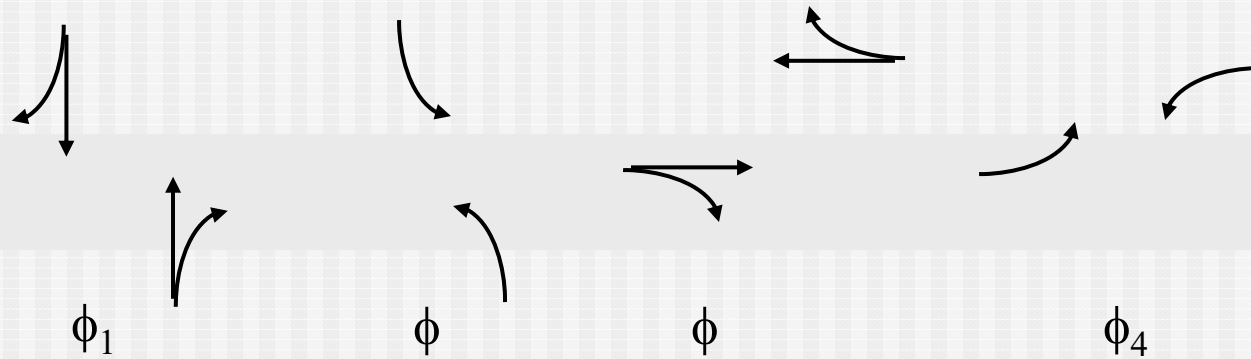
$$y_{E-W} = \max[(475+30)/1800, (425+50)/1800, 95/1000, 75/1000] = 0.281$$

$$Y = \sum y_i = 0.197 + 0.281 = 0.478$$

$$L = 2 \text{ seconds} * 2 \text{ phases} = 4 \text{ seconds}$$

$$C_{opt} = \frac{1.5 (4) + 5}{1 - 0.478} = \frac{11}{0.522} = 21.07 \text{ sec}$$

Assume: a four phase signal design with exclusive left turn phases (also assuming warrants for left- turns are met)



Assume: left-turn² saturation flow rate of 1,000 vphg³

$$y_{N-S\ LT} = \max(65/1000, 80/1000) = 0.08$$

$$y_{E-W\ LT} = \max(95/1000, 75/1000) = 0.095$$

$$Y = \sum y_i = 0.197 + 0.281 + \mathbf{0.08} + \mathbf{0.095} = \mathbf{0.653}$$

$$L = 2 \text{ seconds} * 4 \text{ phases} = 8 \text{ seconds}$$

$$C_{opt} = \frac{1.5 (8) + 5}{1 - 0.653} = \frac{17}{0.347} = 48.99 \text{ sec}$$

-
- Since 49 seconds is too low, use a 60 second cycle length
 - Evaluate capacity using Highway Capacity Software (HCS) to check the level of service (LOS)
 - May have to revise the timing plan for better LOS
 - HCS provides an idea of how the signal design will work in the field

Splits

- Determined by finding the green time needed to serve the demand, and adding these times with the yellow and all-red times

Clearance Interval

Clearance Interval = Yellow + All Red

$$\text{Yellow} = Y = t + \frac{v}{2(a \pm Gg)}$$

$$\text{All Red} = AR = \frac{w + L}{v} \quad \text{or} \quad \frac{P}{v} \quad \text{or} \quad \frac{P + L}{v}$$

Where:

Y = yellow interval (seconds)

t = driver perception-reaction time for stopping,
taken as 1 sec

v = approach speed (ft/sec) taken as the 85th
percentile speed or the speed limit

a = deceleration rate for stopping taken as 10 ft/sec^2

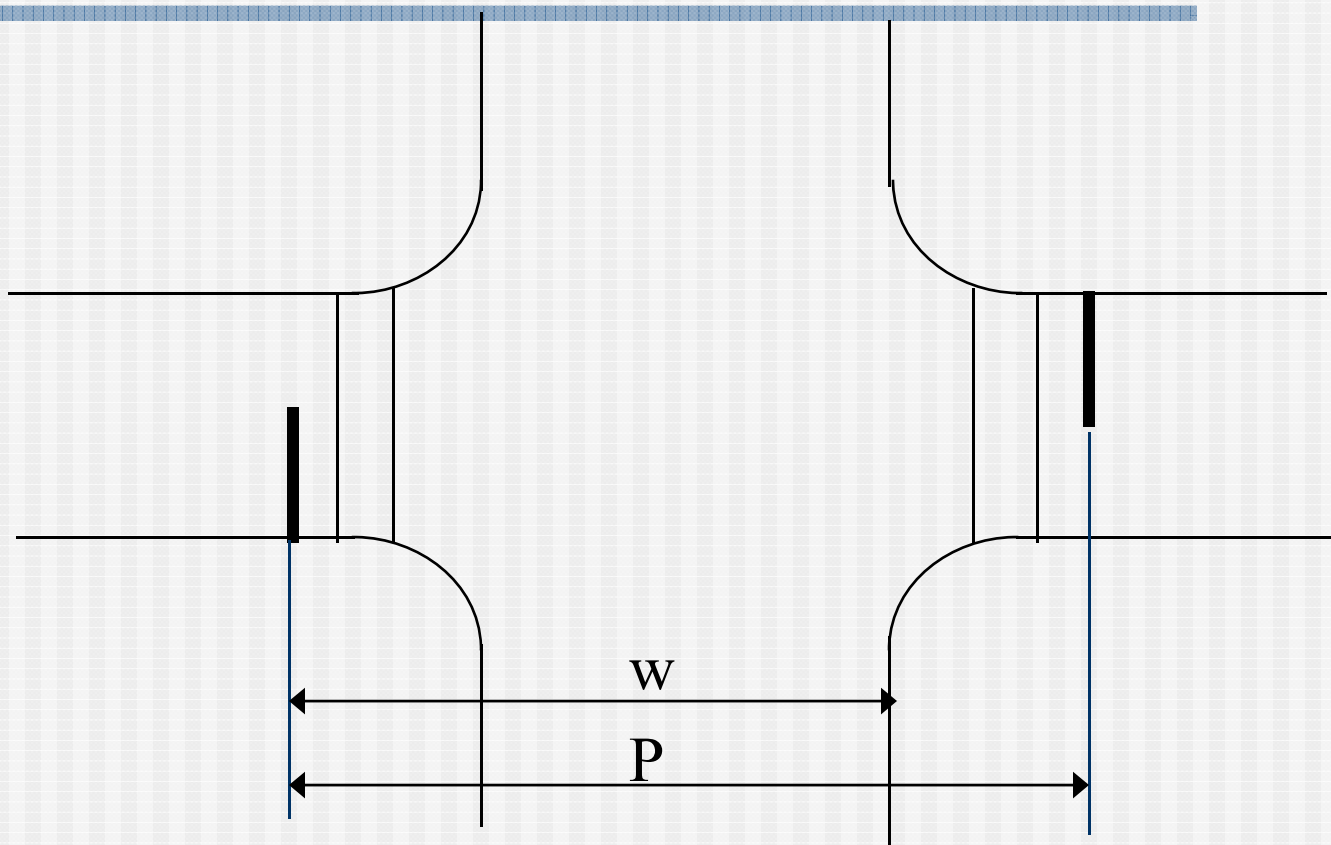
G = percent of grade divided by 100 (positive for
upgrade, negative for downgrade)

L = length of the clearing vehicle,
normally 20 feet

W = width of the intersection in feet, measured
from the upstream stop bar to the
downstream extended edge of pavement

P = width of the intersection (feet) measured
from the near-side stop line to the far side
of the farthest conflicting pedestrian
crosswalk along an actual vehicle path

Clearance Interval



Example Clearance Interval

$$v = 30 \text{ mph} = 44 \text{ feet/sec}$$

$$t = 1 \text{ second}$$

$$a = 10 \text{ feet/sec}^2$$

$$\text{Yellow} = Y = t + \frac{v}{2(a \pm Gg)}$$

$$Y = 1 + 44/20 = 3.2 \text{ seconds}$$

- As the approach speed increases, the amber time increases

- All-red interval is based on approach speed and roadway geometry

$$\text{All Red} = \text{AR} = \frac{w + L}{v} \quad \text{or} \quad \frac{P + L}{v}$$

$$\text{AR} = (85 + 20) / 44 \quad \text{OR} \quad \text{AR} = (110 + 20) / 44$$

$$= 2.38 \text{ sec} \quad \quad \quad = 2.95 \text{ sec}$$

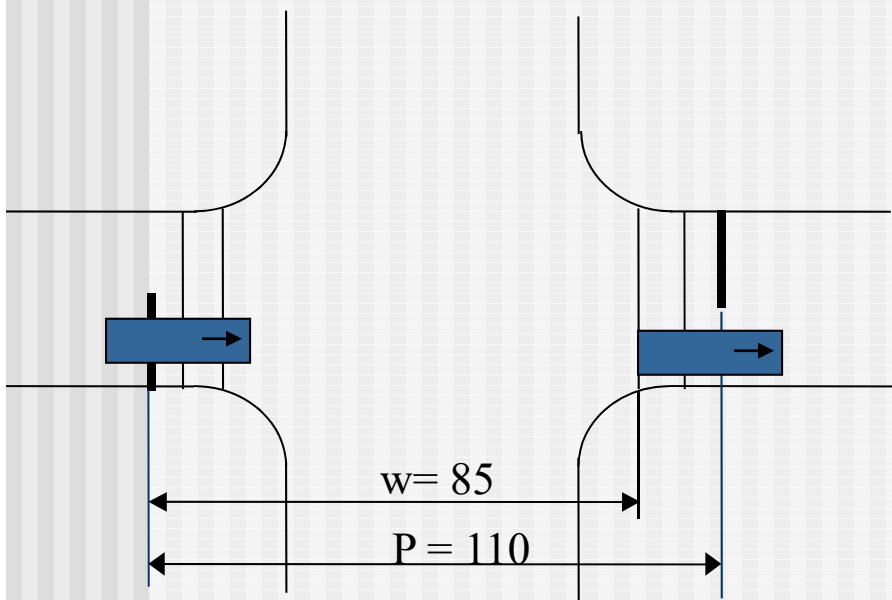
$$\therefore Y = 3.2 \text{ sec}$$

$$\text{AR} = 2.4 \text{ sec}$$

$$\text{CI} = 5.6 \text{ sec}$$

$$\text{Use } Y = 4.0 \text{ sec}$$

$$\text{CI} = 6.4 \text{ sec}$$



Splits

- Assume

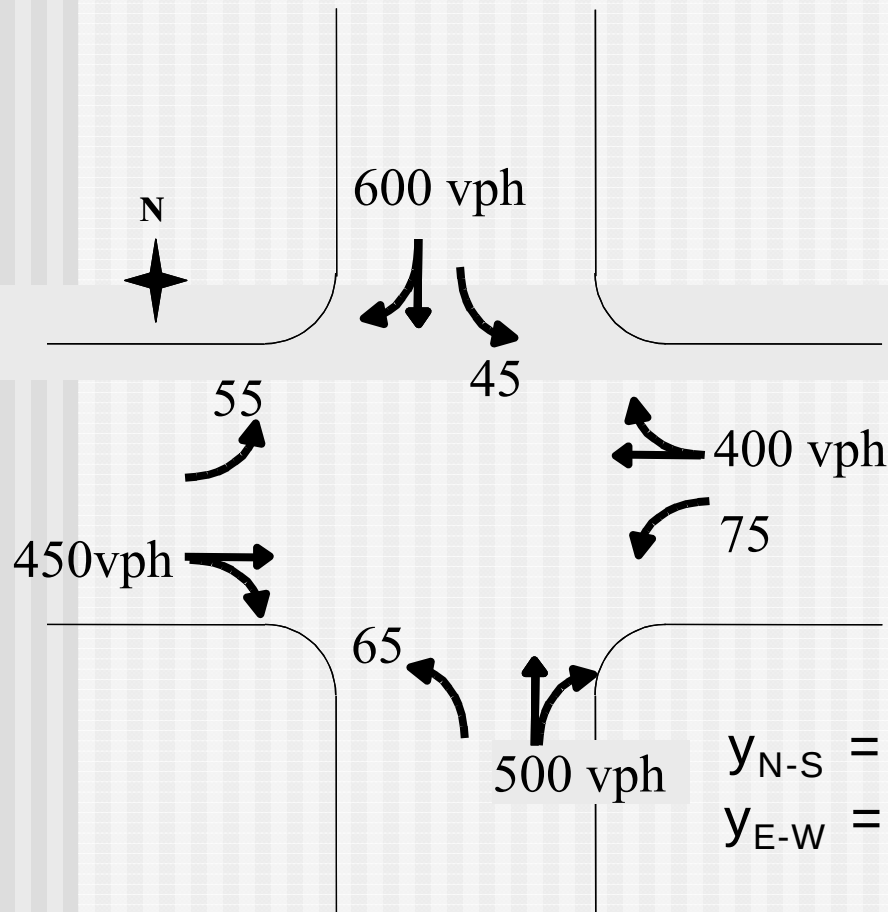
- Two-phase signal design
- 3 x 3 lane intersection
- 60-second cycle length

$$G_{N-S} = \frac{NG * \frac{V_{N-S}}{S_{N-S}}}{\frac{V_{N-S}}{S_{N-S}} + \frac{V_{E-W}}{S_{E-W}}}$$

- Net green time = NG

$$= 60 \text{ sec} - (Y + AR)_{\text{for all phases}}$$

$$= 60 - 2 * 6.4 = 60 - 12.8 = 47.2 \text{ seconds}$$



Assume: saturation flow

•through movements:

$$S_{EB} = 993$$

$$S_{WB} = 1240$$

$$S_{NB} = 1150$$

$$S_{SB} = 960$$

•left turns: $S_{NB,SB,EB,WB} = 1000$

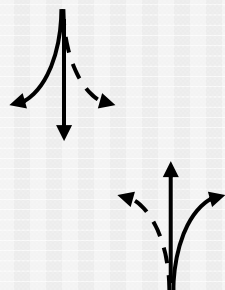
$$y_{N-S} = \max (500/1150, 600/960) = 0.625$$

$$y_{E-W} = \max (400/1240, 450/993) = 0.453$$

$$G_{N-S} = \frac{47.2 (.625)}{0.625 + 0.453} = 21.4$$

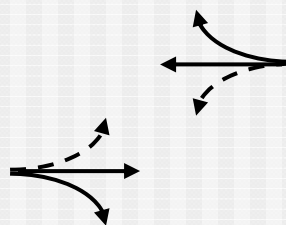
$$G_{E-W} = \frac{47.2 (.453)}{0.625 + 0.453} = 19.8$$

Resulting Signal Timing Plan



ϕ_1
North-South

G = 21.4 sec
Y = 4.0 sec
AR = 2.4 sec



ϕ_2
East-West

G = 19.8 sec
Y = 4.0 sec
AR = 2.4 sec