DEVELOPMENT OF PLANNING AND EVALUATION MODELS FOR SUPERSTREET

THESIS DEFENSE

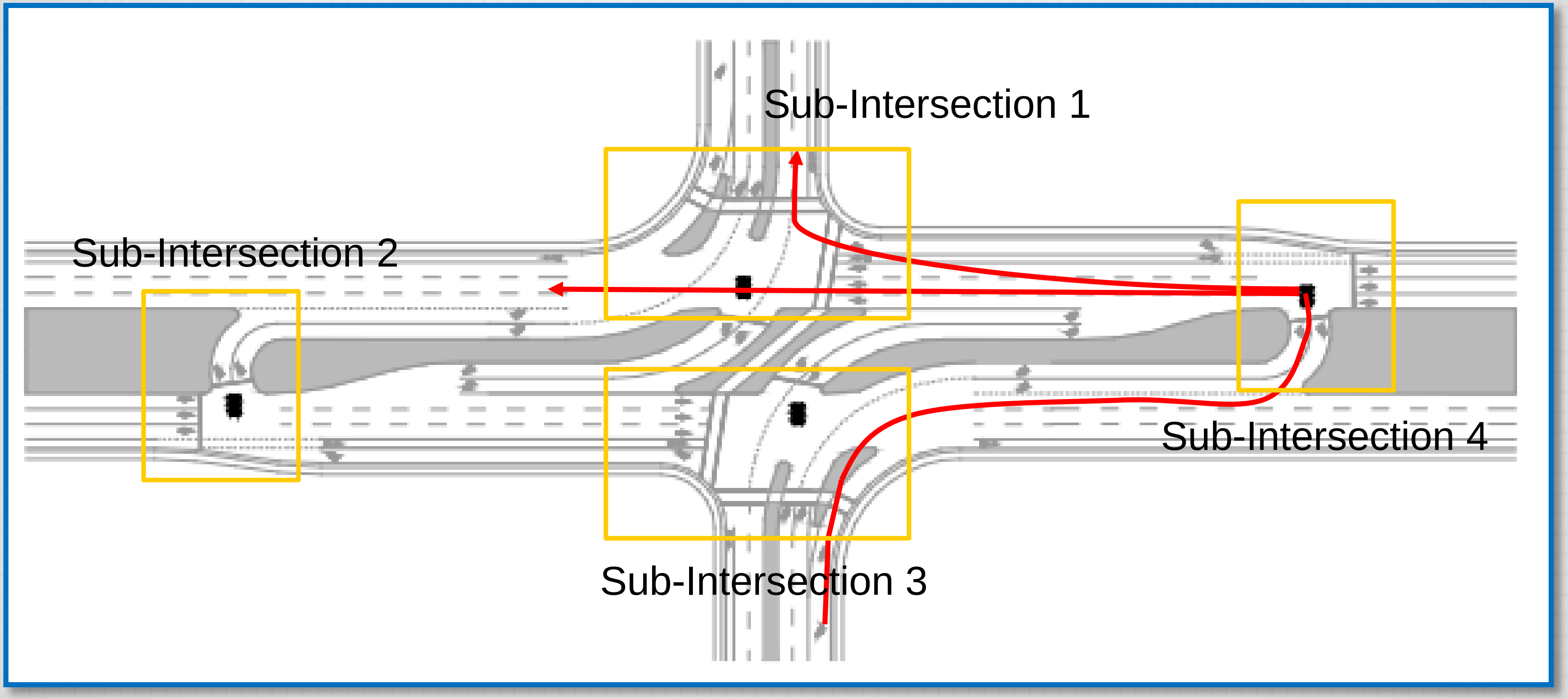
May 3rd 2016

LIU XU

CONTENTS

01 RESEARCH BACKGROUND

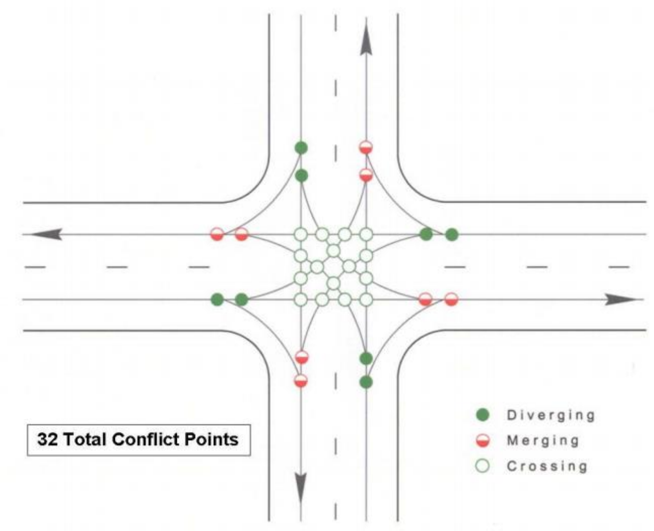
Superstreet



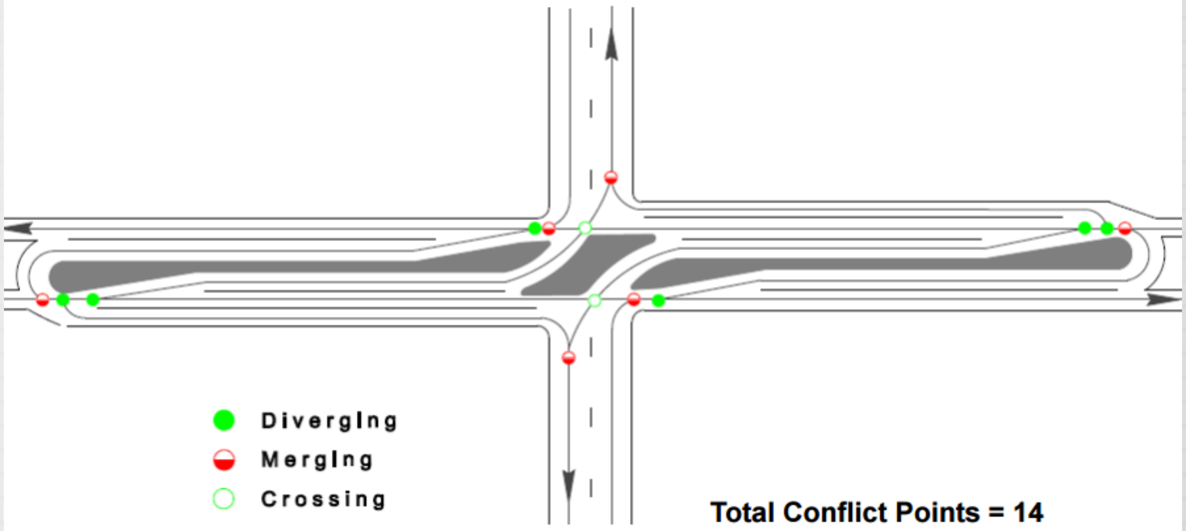
Research Background

❑ BENEFITS: ➡ *Increased Popularity of Superstreets.*

- Economical Benefits: *Less expensive than an interchange;*
- Safety Benefits: *Reduction in number & severity of the collisions;*
- Operation Benefits: *Provide signal progressions along the arterial; and*
- Environmental Benefits: *Reduction in pollutions*.



Conventional Intersection



Superstreet

Research Background

❑ Literature Review

- A number of studies in the literature have confirmed its safety benefits. (Hummer, 2001, 2007, 2008, 2009, 2010, 2012; Kim, 2007; Edara, 2007).

- The distance between the main intersection and U-turn crossover is the dominating factor that influence a Superstreet's safety performance (Liu, 2007; Hochestein, 2009; Hugues, 2010; Olarte, 2011).

- In fact, over the past decades, only limited studies (Olarte, 2011) have attempted to address the issues of design and operations of a Superstreet.

- A newly published report (FHWA, 2014) also indicated the lack of sufficient information in the area of designing a Superstreet.

Existing Literature fall short on the subjects of Design and Evaluation of Superstreets.

Critical Issues

❑Limitations of Existing Studies

Model-1

- How to determine the U-turn offset length that dominates the geometric design of a Superstreet?
- What would be the criteria for determining the need of installing signals for a Superstreet?

Model-2

- How to assess whether the bay length among a signalized Superstreet is sufficient to prevent any spillback from happening?

Model-3

- How to design a proper signal timing plan, considering its unique geometric layouts?
- How to minimize the delay experienced by the minor road drivers due to the detour operations in a Superstreet.

02 THESIS FRAMEWORK

THESIS FRAMEWORK

Evolution Of Superstreets' Development

Un-signalized

1 Minimum U-turn Offset Model for Un-signalized Superstreet

a. Critical Components of U-turn offset
b. Key Input/ Output
c. Model Development
d. SSAM Evaluation
e. Extended Applications

Signalized

2 Interval-based Bay Length Evaluation Models for a Signalized Superstreet

a. Operational Analysis
b. Critical Issues
c. Model Development
d. Model Validation

Signalized

3 Two-stage Signal Optimization Model for a Signalized Superstreet

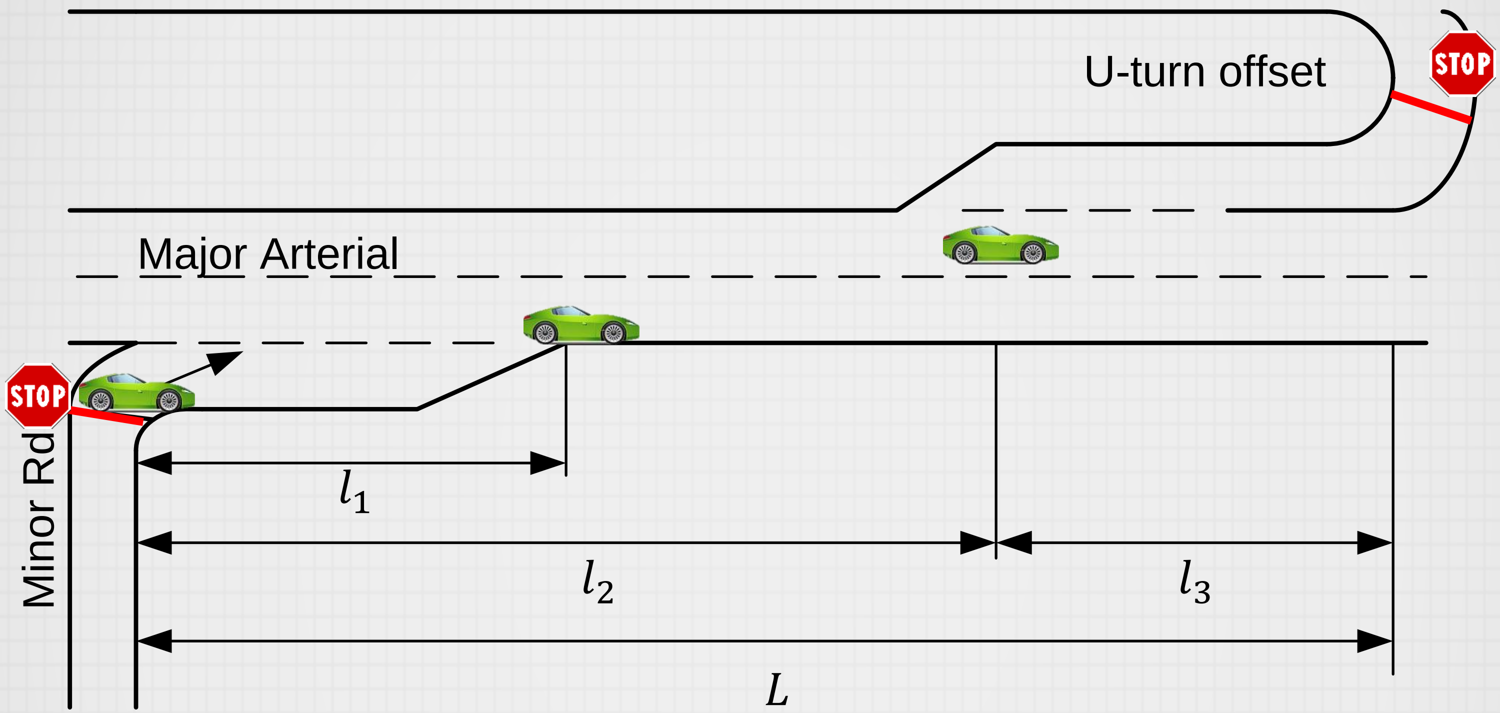
a. General Algorithm
b. Signal Control Algorithm
c. Solution
d. Case Study

03

MINIMUM U-TURN OFFSET MODEL FOR A UN-SIGNALIZED SUPERSTREET

Model-1

Critical Components of U-turn Offset



- l_1: Acceleration and merging length;
- l_2: Lane-changing length;
- l_3: Deceleration and initial queue length;
- L: Minimum U-turn offset.

Key Components

Input :

Acceleration Rate: a_1

Headway Distribution: $(\lambda, \bar{t}, t_m, t_{nr})$

Critical Gap Distribution: (μ, σ)

Headway Distribution: $(\lambda, \bar{t}, t_m, t_{nr})$

Critical Gap Distribution: (μ, σ)

Deceleration Rate: a_2

M/M/1 Queuing System:

$$\rho = \frac{arrival\_rate}{service\_rate}$$

Output

Acceleration & Merging Length: l_1

Lane-Changing Length: l_2

Deceleration & Initial Queue Length: l_3

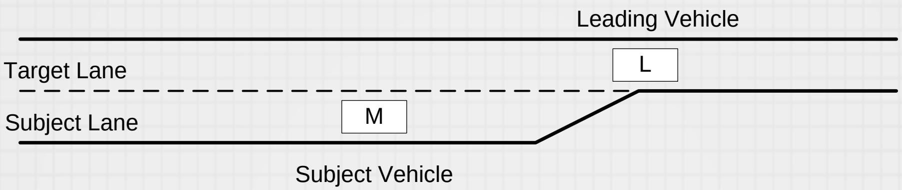
Minimum U-turn Offset Length: L

Notation	Description
a_1	AASHTO recommended acceleration rate
λ	Parameter for major road headway distribution shifted negative exponential distribution
$\bar{t}$	Average gap in second from major traffic
t_m	the minimum headway from major traffic
t_{nr}	the maximum headway from major traffic
μ	Mean of critical gap distribution
σ	Deviation of critical gap distribution
a_2	AASHTO recommended deceleration rate
$\rho = \frac{arrival\_rate}{service\_rate}$	Parameter for M/M/1 system
l_1	Acceleration & merging length
l_2	Kth lane changing length
l_3	Deceleration & initial queue length
L	Minimum U-turn offset length

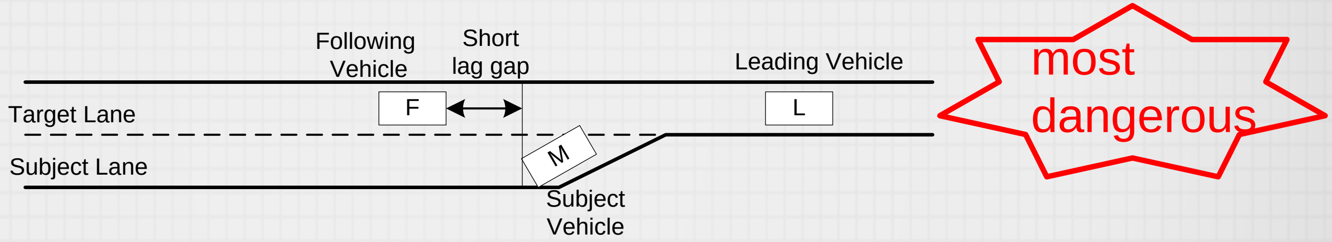
Merging Scenarios

The merging maneuver, based on the relative gaps between the leader and the follower, can be classified into three distinct types:

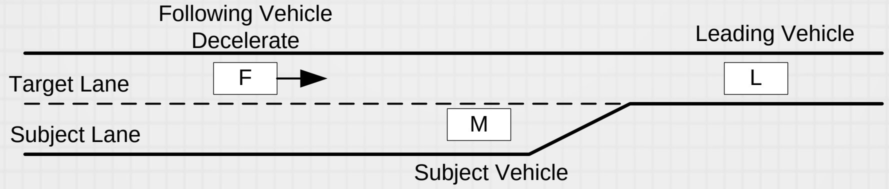
1) **Free merging:**



2) **Forced merging**: the follower was 'forced' to break pedal to maintain safe space headway;

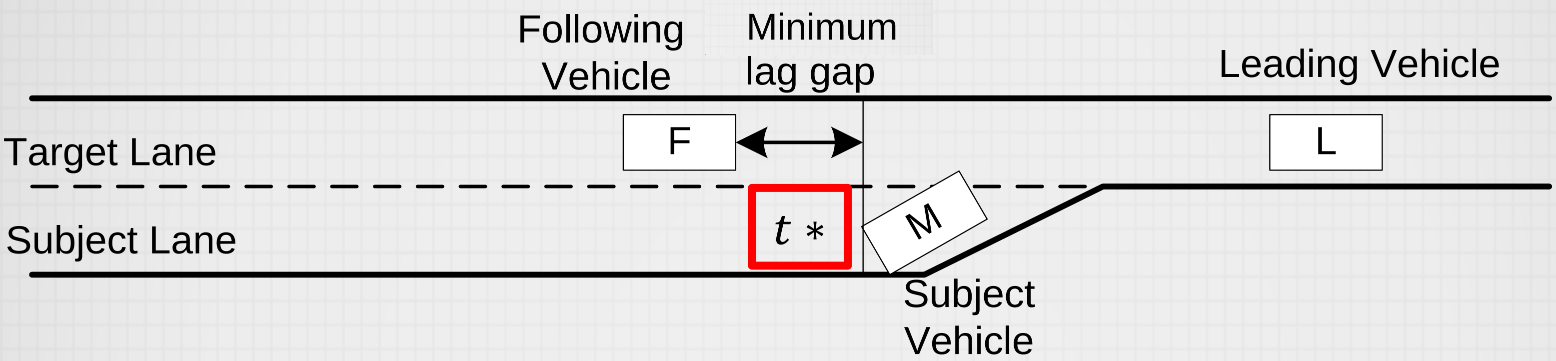


3) **Cooperative merging:**



Resource:Hidas, P. (2005). Modelling vehicle interactions in microscopic simulation of merging and weaving. *Transportation Research Part C: Emerging Technologies*, *13*(1), 37-62.

Merging Scenarios



$$(h-t_r)\cdot v_1 - l_v = \frac{(v_1-v_0)^2}{2a_2}$$

$$t^* \geq \frac{(v_1-v_0)^2}{2a_2 v_1} + \frac{l_v}{v_1} + t_r$$

where v_1 is the speed of mainline traffic;

v_0 is the speed of subject vehicle;

t_r is the average reaction time, 1.0s;

h is a given time headway;

l_v is the AASHTO recommended passenger car length, 20ft;

a_2 is the AASHTO recommended deceleration rate, 11.2ft/s^2.

For any randomly given subject vehicle, the minimum acceptable headway must be no less than t*

Acceleration & Merging Length

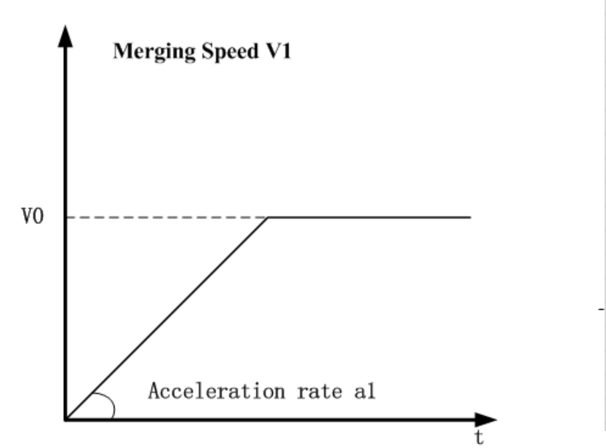
Assumptions

During the merging process, subject vehicle has to accelerate from stop. Assuming

(1) The subject vehicle accelerates from 0_{mph} with a fixed acceleration rate until reaches speed limit v_1, and then stay at the same speed until reach the U-turn location;

(2) Critical headway t_c follows a certain distribution, in this case, assume critical gap for drivers from side street follows normal distribution~ $N(\mu, \sigma^2)$;

(3) The headway follows negative exponential distribution since the car arrival follows Poisson distribution.



Acceleration & Merging Length

Set a link between headway distribution and U-turn offset Length

For a random vehicle, at time point t, the lane-changing probability can be:

$$F(t) = P\{h \geq t_c(t)\}$$

Where t_c denotes the critical gap for a certain driver at time point t.

If assuming that at time point $t + \Delta t$, where $\Delta t \to 0$,

Successful Merging Probability at $t + \Delta t$

$$\boxed{p_1(t+\Delta t)} = p_1(t) + (1 - p_1(t))\Delta t(F(t))$$

Since $\Delta t \to 0$, we can have $t_c(t + \Delta t) = t_c(t)$, then

$$\frac{p_1(t+\Delta t) - p_1(t)}{\Delta t} = [1 - p_1(t)] \cdot F(t)$$

$$-In[1 - p(t)]dP = \int_0^{+\infty} F(t) \cdot dt$$

F(t) is not a constant but a function with respect to time. So we cannot have closed form of P(t)

how to calculate F(t) ?

F(t) stands for the probability for a random driver merging into major road at any time point t. It is a function with respect to both time and human characteristic.

Merging Length

The probability of a driver having a critical gap equals t_c* at time point *t* is

$$f(t_c^*)dt = \left[\frac{1}{\sigma\sqrt{2\pi}} \exp\frac{-(t_c^* - \mu)}{2\sigma^2} \right] dt$$

Meanwhile, the headway distribution follows shifted negative exponential distribution as

$$\Pr(h \geq t_c) = \begin{cases} e^{-\lambda(t_c - t_m)}, \; for & t_c \geq t_m \\ 0, \quad for & t_c < t_m \end{cases}$$

Where $\lambda = 1/(\bar{t} - t_m)$ while $\bar{t}$ is the average gap (s) and t_m is the minimum headway(s).

A random vehicle to conduct a successful merging can be

$$\boxed{\int_{t_c=0}^{\infty} \Pr(h \geq t_c) f(t_c) dt}$$

Merging Length

There exist two thresholds t_m, t_{nr} that stands for the lower bound and upper bound, respectively (Pollatschek, 2002).
Therefore, the overall merging probability can be expressed as:

$$\int_{t_c=0}^{\infty} \Pr(h \geq t_c) f(t_c) dt = \boxed{\int_{t_c=0}^{\max(t_m,t*)} \Pr(h \geq t_c) f(t_c) dt} + \boxed{\int_{t_c=\max(t_m,t*)}^{t_{nr}} \Pr(h \geq t_c) f(t_c) dt} + \boxed{\int_{t_c=t_{nr}}^{\infty} \Pr(h \geq t_c) f(t_c) dt}$$

$$\boxed{\text{since } \Pr(h \geq t_c) = 0, \quad \int_{t_c=0}^{\max(t_m,t*)} 0 * f(t_c) dt = 0}$$

$$\boxed{\text{since } f(t_c) dt \approx 0, \quad \int_{t_c=t_{nr}}^{\infty} \Pr(h \geq t_c) f(t_c) dt \approx 0}$$

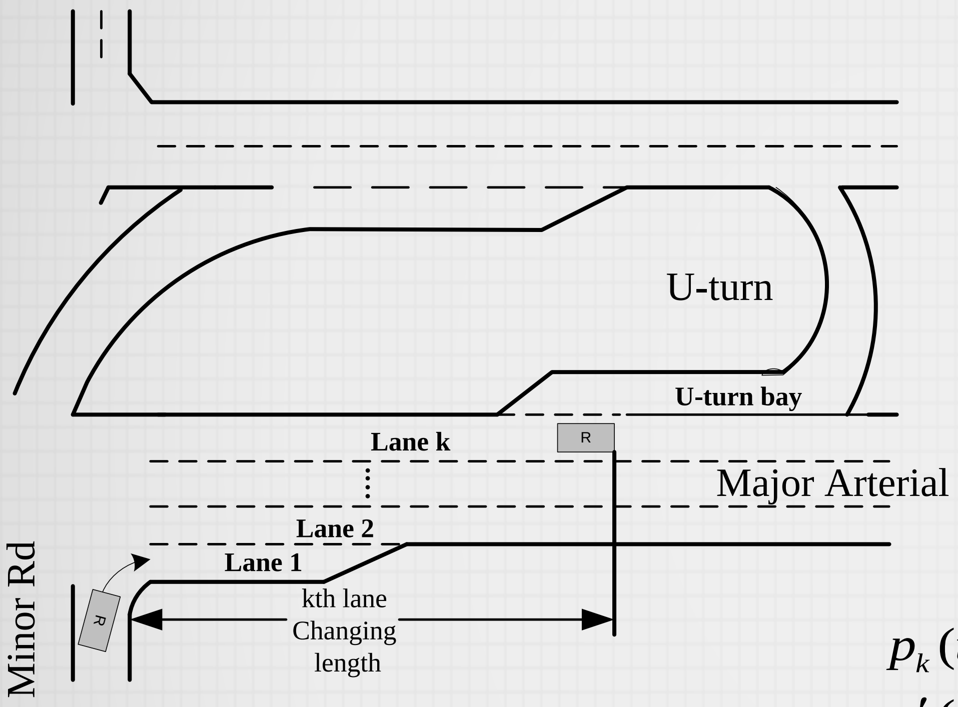
Finally, we can have

$$\int_{t_c=0}^{\infty} \Pr(h \geq t_c) f(t_c) dt = \boxed{\int_{t_c=\max(t_m,t*)}^{t_{nr}} \Pr(h \geq t_c) f(t_c) dt} = e^{\lambda t_m + \frac{-2\mu\lambda\sigma^2 + \lambda^2\sigma^4}{2\sigma^2}} \boxed{\int_{\max(t_m,t*)}^{t_{nr}} \frac{1}{\sigma\sqrt{2\pi}} \exp \frac{-\left[t_c - (\mu - \lambda\sigma^2)\right]^2}{2\sigma^2} dt}$$

$$\boxed{N \sim (\mu - \lambda\sigma^2, \sigma^2)}$$

Resource: Pollatschek, M. A., Polus, A., & Livneh, M. (2002). A decision model for gap acceptance and capacity at intersections. *Transportation Research Part B: Methodological*, *36*(7), 649-663.

Lane-Changing Length



Overlapping with Acceleration & Merging Length l_1

As $p_k(t)$ denotes the probability that the vehicle is in lane *k* at time point *t*.

Remember for 1st successful merging probability, we have:

$$p_1(t+\Delta t) = p_1(t) + (1 - p_1(t))\Delta t F(t)$$

For k_{th} lane change, we can get

$$p_k(t+\Delta t) = p_k(t) + [1 - p_k(t)] \cdot p_{k-1}(t) \cdot \Delta t \cdot F(t)$$

$$p_k'(t) = [1 - P_k(t)] \cdot \boxed{p_{k-1}(t)} \cdot \boxed{F(t)}$$

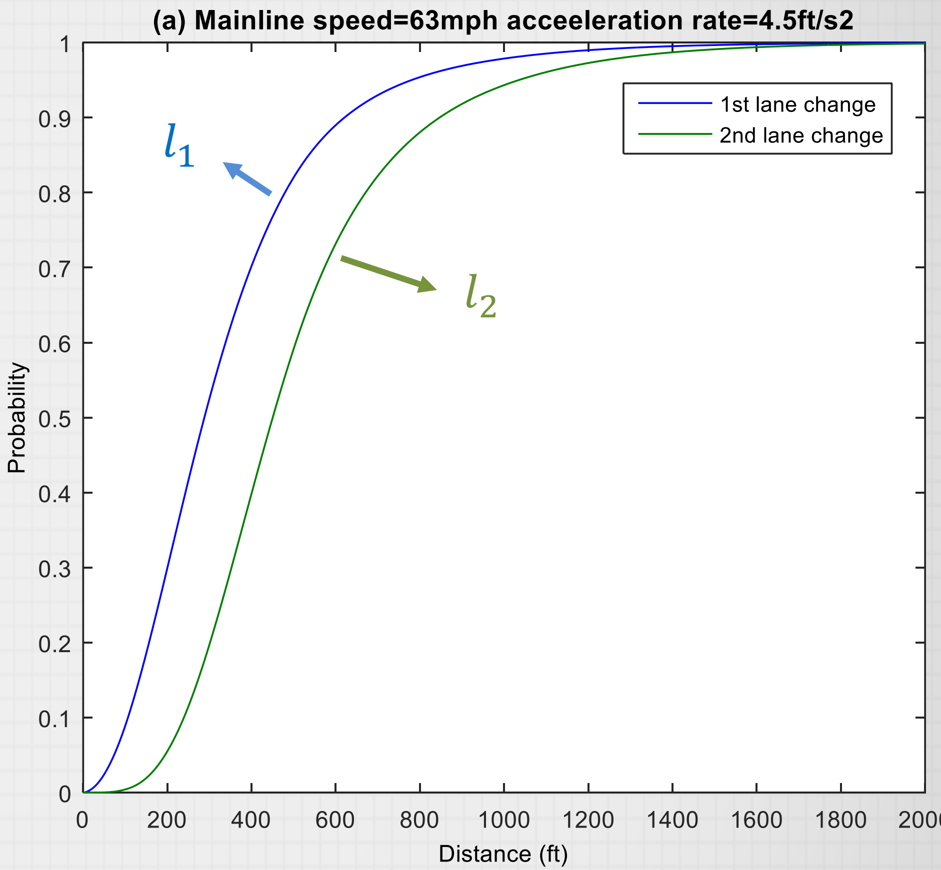
Because both F(t) and p(t) are not a constant but functions with respect to time. So we cannot have closed form of $P_k(t)$

Numerical Example

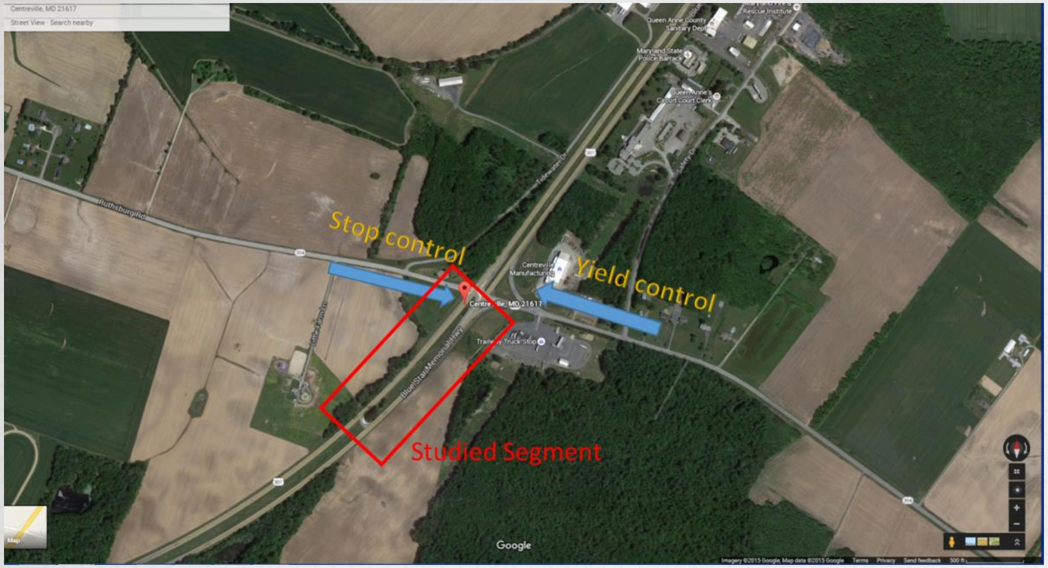
Given the headway distribution of arterial traffic and the predetermined overall successful rate, we can get the relationship between probability of kth lane changes and the required distance.

A numerical example is shown on the right-hand side:

PARAMETERS	VALUES
t_{wr}	11 seconds
t_m / t_{sa}	2 seconds
$\bar{t}$	5.6 seconds
$\bar{\lambda}$	0.28
$\bar{\mu}$	0.67
a_1	4.0~4.5 ft/s2
a_2	11.2 ft/s2
v_1	63~67 mph



SSAM Evaluation



US 301 @ Ruthsburg Rd, MD

- Stop control for EB minor road
- Yield control for WB minor road
- Studies Segment:
South-Bound U-turn Segment

❑Scenario 1: 1500ft southern U-turn offset (*Field implementation*)

❑Scenario 2: 1100ft southern U-turn offset (*Mean of model output*)

❑Scenario 3: 700ft southern U-turn offset (*Shortened U-turn offset*)

▪The only difference between three scenarios is the length of southern U-turn offset. The rest of geometrics are the same for all scenarios and are measured from the field.

SSAM Measurements

- Minimum Time To Collision (TTC)
- Minimum Post-Encroachment Time (PET)

→ Severity of Conflict Events

- Initial Deceleration Rate (DR)
- Maximum Speed (MaxS)
- Maximum relative Speed Difference (DeltaS)
- Maximum Deceleration Rate (MaxD)
- Maximum “post collision” DeltaV (MaxDelatV)

→ Severity of Collisions

MaxDeltaV is the maximum **speed change** of either vehicle in the conflict.

Safety Comparison(Scenario 2 VS. Scenario 1)

- U-turn Segment safety performance Comparison (1100ft VS. 1500ft)

SSAM Measures	Mean (1100ft)	Variance (1100ft)	Mean (1500ft)	Variance (1500ft)	t value	t critical	Sigfinicant	Mean Difference
TTC	0.217	0.184	0.217	0.144	-0.002	1.668	NO	0
PET	0.08	0.026	0.083	0.02	-0.018	1.668	NO	-0.003
MaxS	22.441	8.983	22.97	11.465	-0.868	1.668	NO	-0.529
DeltaS	8.678	9.953	9.942	27.489	-1.263	1.668	NO	-1.265
DR	-1.004	5.08	-1.203	5.582	0.443	1.668	NO	0.2
MaxD	-2.482	9.743	-2.838	10.67	0.587	1.668	NO	0.355
MaxDeltaV	4.485	2.711	5.113	7.253	-1.214	1.668	NO	-0.628

No significant difference in terms of conflict **severity**!

Conflict Types	Mean (1100ft)	Variance (1100ft)	Mean (1500ft)	Variance (1500ft)	t value	t critical	Signficant	Mean Difference
Crossing	0	0	0	0	0	1.86	NO	0
Rear-end	5	22	7.2	21.7	-0.744	1.86	NO	-2.2
Lane changing	1	2	1	0.5	0	1.86	NO	0
Total	6	36	8.2	22.7	-0.642	1.86	NO	-2.2

No significant difference in terms of **No. of conflicts.**

No statistically significant difference between 1500ft and 1100ft in terms of both number of conflicts and all SSAM measurements.

Safety Comparison(Scenario 3 VS. Scenario 2)

❑ U-turn Segment safety performance Comparison (700ft VS. 1100ft)

SSAM Measures	Mean (700ft)	Variance (700ft)	Mean (1100ft)	Variance (1100ft)	t value	t critical	Sigfinicant	Mean Difference
TTC	0.19	0.187	0.217	0.184	-0.136	1.668	NO	-0.026
PET	0.078	0.028	0.08	0.026	-0.01	1.668	NO	-0.002
MaxS	22.952	8.076	22.441	8.983	1.044	1.668	NO	0.511
DeltaS	13.111	37.605	8.678	9.953	3.966	1.67	YES	4.433
DR	-0.57	2.399	-1.004	5.08	0.909	1.677	NO	0.434
MaxD	-2.907	10.679	-2.482	9.743	-0.797	1.668	NO	-0.425
MaxDeltaV	6.791	10.316	4.485	2.711	3.943	1.67	YES	2.306

More Sever Collisions under 700ft scenario.

Conflict Types	Mean (700ft)	Variance (700ft)	Mean (1100ft)	Variance (1100ft)	t value	t critical	Sigfinicant	Mean Difference
Crossing	0	0	0	0	0	1.86	NO	0
Rear-end	5.4	6.3	5	22	0.168	1.86	NO	0.4
Lane changing	2.8	0.7	1	2	2.449	1.86	YES	1.8
Total	8.2	7.2	6	36	0.748	1.86	NO	2.2

Increased possible lane-changing collisions under 700ft when comparing 1100ft

a. Increased possible lane-changing collisions under 700ft than in 1100ft;
b. More sever collisions under 700ft than in 1100ft.

Safety Comparison(Scenario 3 VS. Scenario 1)

❑ U-turn Segment safety performance Comparison (700ft VS. 1500ft)

SSAM Measures	Mean (700ft)	Variance (700ft)	Mean (1500ft)	Variance (1500ft)	t value	t critical	Sigfinicant	Mean Difference
TTC	0.19	0.187	0.217	0.144	-0.159	1.664	NO	-0.027
PET	0.078	0.028	0.083	0.02	-0.031	1.664	NO	-0.005
MaxS	22.952	8.076	22.97	11.465	-0.036	1.664	NO	-0.018
DeltaS	13.111	37.605	9.942	27.489	3.502	1.664	YES	3.168
DR	-0.57	2.399	-1.203	5.582	1.816	1.664	YES	0.633
MaxD	-2.907	10.679	-2.838	10.67	-0.13	1.664	NO	-0.069
MaxDeltaV	6.791	10.316	5.113	7.253	3.434	1.664	YES	1.678

More sever collisions under 700ft

Conflict Types	Mean (700ft)	Variance (700ft)	Mean (1500ft)	Variance (1500ft)	t value	t critical	Significant	Mean Difference
Crossing	0	0	0	0	0	1.86	NO	0
Rear-end	5.4	6.3	7.2	21.7	-0.761	1.86	NO	-1.8
Lane changing	2.8	0.7	1	0.5	3.674	1.86	YES	1.8
Total	8.2	7.2	8.2	22.7	0	1.86	NO	0

Increased possible lane changing collisions for 700ft when comparing 1500ft

a. **Increased possible lane-changing collisions under 700ft than in 1500ft;**
b. **More sever conflicts under 700ft than in 1500ft.**

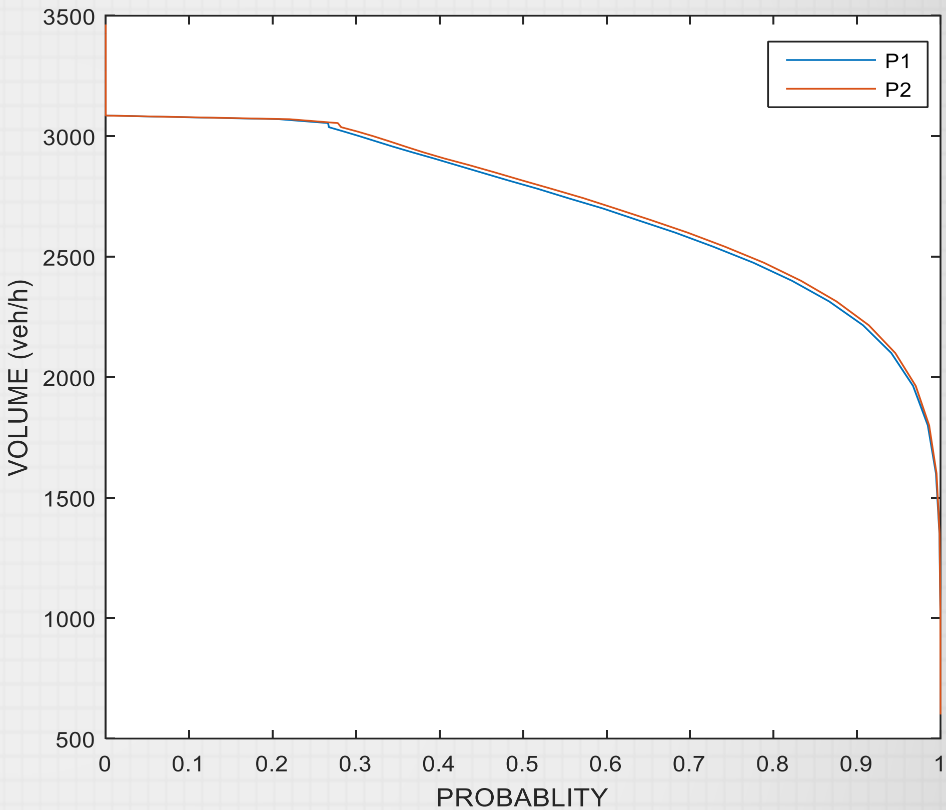
Extended Application

Set the criteria for installing signal to accommodate the increased traffic

The overall merging successful probability decreases with growing volume level.

A numerical example is shown on the right-hand side:

PARAMETERS	VALUES
t_{nr}	11 seconds
t_m / t_{sa}	2 seconds
$\overline{t}$	5.6 seconds
$\overline{\lambda}$	0.28
$\overline{\mu}$	0.67
a_1	4.0~4.5 ft/s2
a_2	11.2 ft/s2
v_1	63~67 mph



The Relationship between Traffic Demand and the Probability of Conducting Twice Lane-Changes

04

INTERVAL-BASED BAY LENGTH EVALUATION MODELS FOR A SIGNALIZED SUPERSTREET

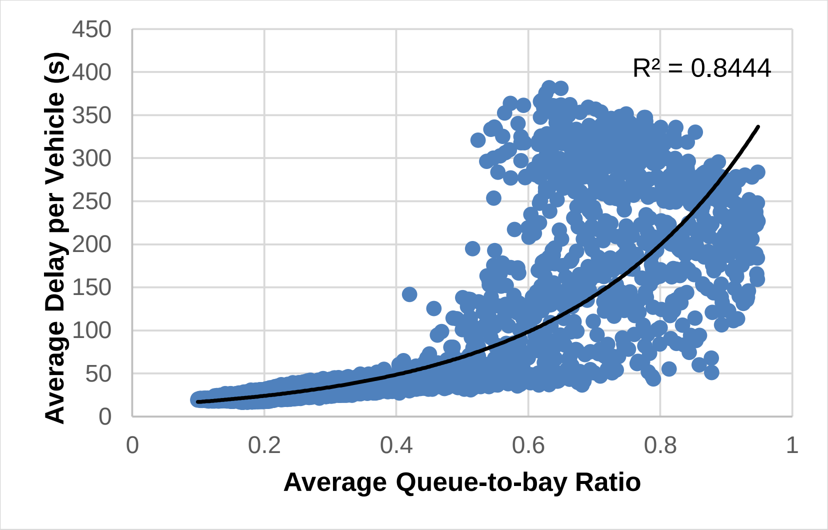
Model-2

Operation Analysis

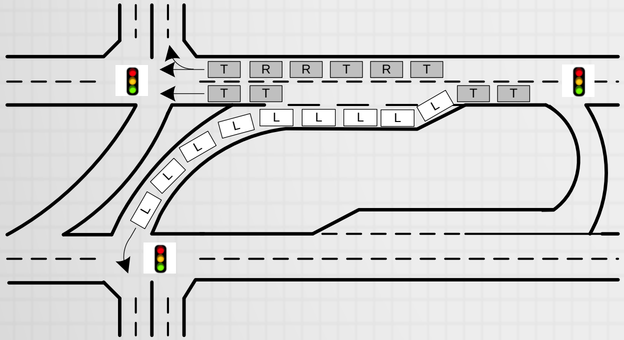
Field Survey and VISSIM Calibration

- This study has conducted a field survey at a signalized Superstreet Intersection **(MD3 & Waugh Chapel Rd)** to calibrate key parameters in VISSIM;
- The collected data include queue lengths, signal plan and traffic flow rates.
- Extensive simulation results reveal that the exponentially increased delay when Q/L ratio approaches to 1.

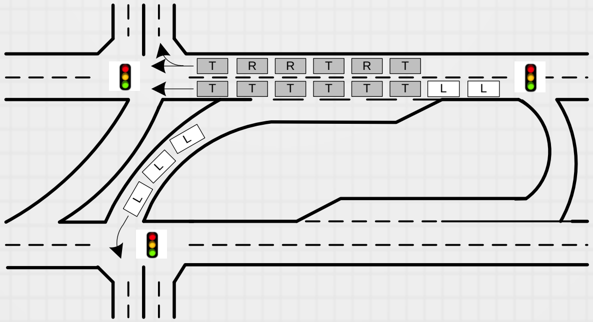
Figure. Scatter plot of average delay v.s. average QL ratio



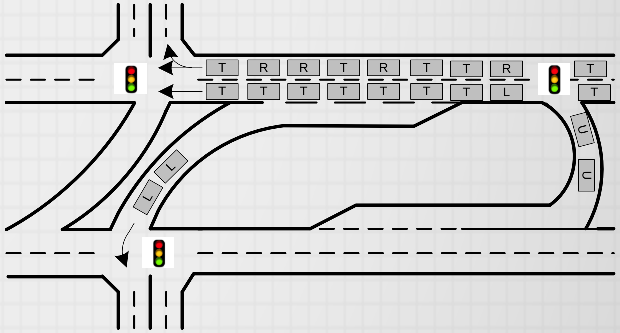
Possible blockages among a Superstreet are shown below:



(A) Left-turn lane group partially blocks the right-through lane group



(B) Right-through lane group completely blocks the left-turn lane group



(C) Through lane group completely blocks the upstream lane groups

Critical Issues

Interval-based queue estimation models

- Traffic flow and signal design can both contribute to the formation of queues in a superstreet

➢ *Incoming traffic fluctuates over time*

➢ *Signal coordination plan is another key factor to determine queue length*

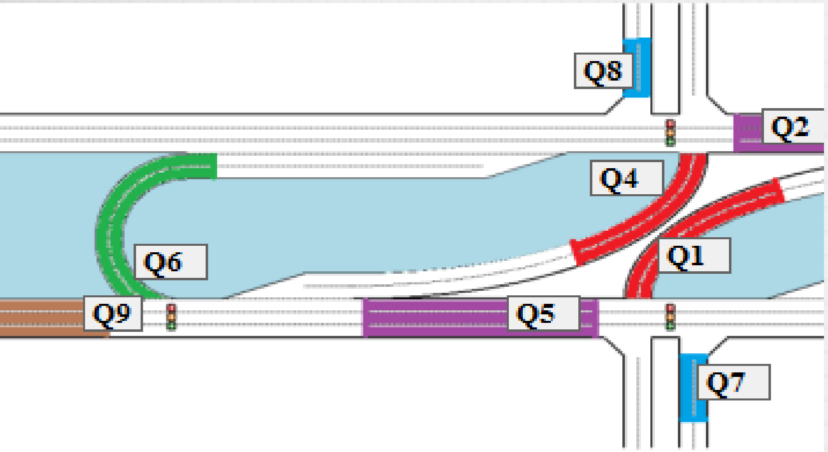
➢ *Develop interval-based queue estimation models to take into account of the both uncertainties.*

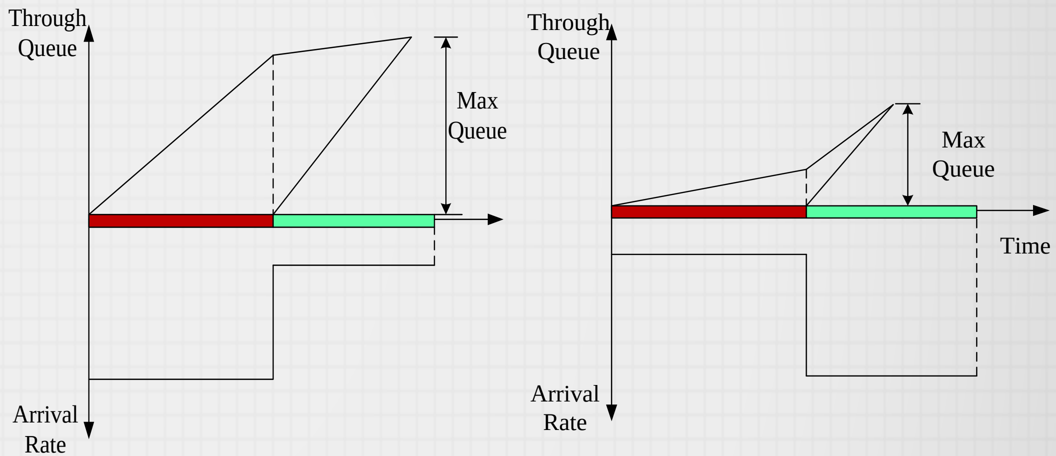
- Two types of queues:

1) **External Queues:** only influenced by flow fluctuation
2) **Internal Queues:** influenced by both flow fluctuation and signal coordination

Queue lengths under different signal coordination plan

- For main intersection through-Q: Q5, departures from Q6 and Q9 are two sources for it’s incoming flow.
- 1) through and right-turn movements from Q9;
- 2) departures from Q6



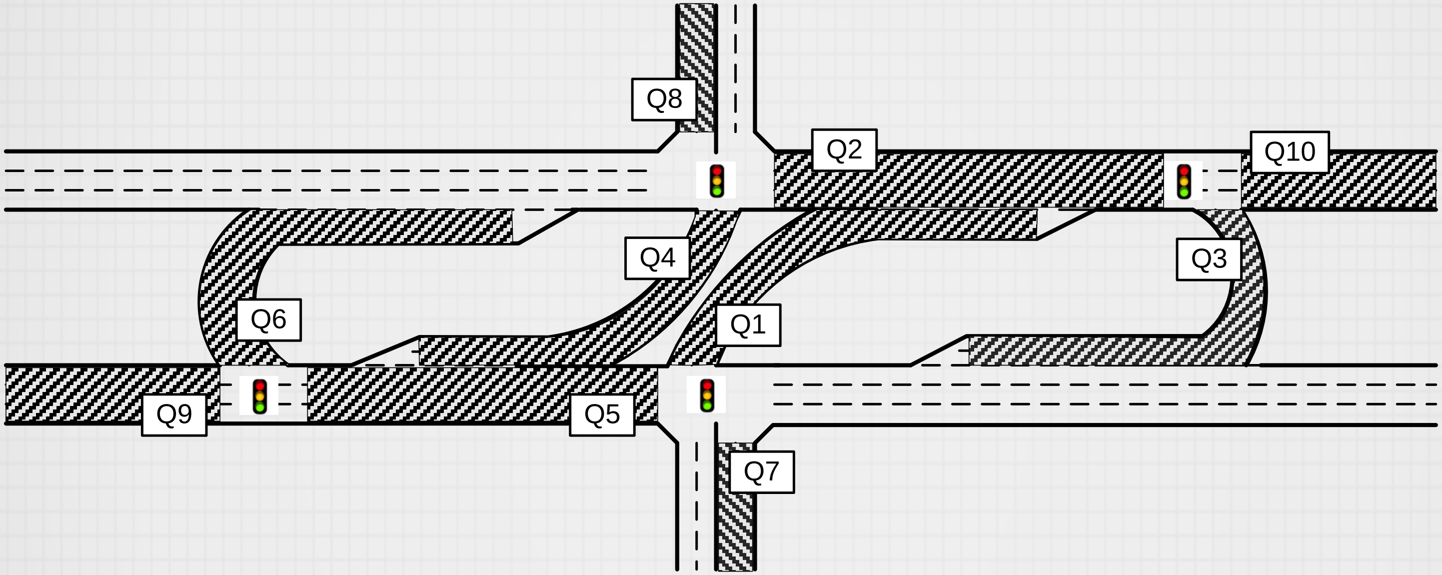


Worst Case=
Largest arrival rate+
worst signal coordination

Best Case=
Smallest arrival rate+
Best signal coordination

Model Development

- Spatial distributions of all potential queues among a Signalized Superstreet

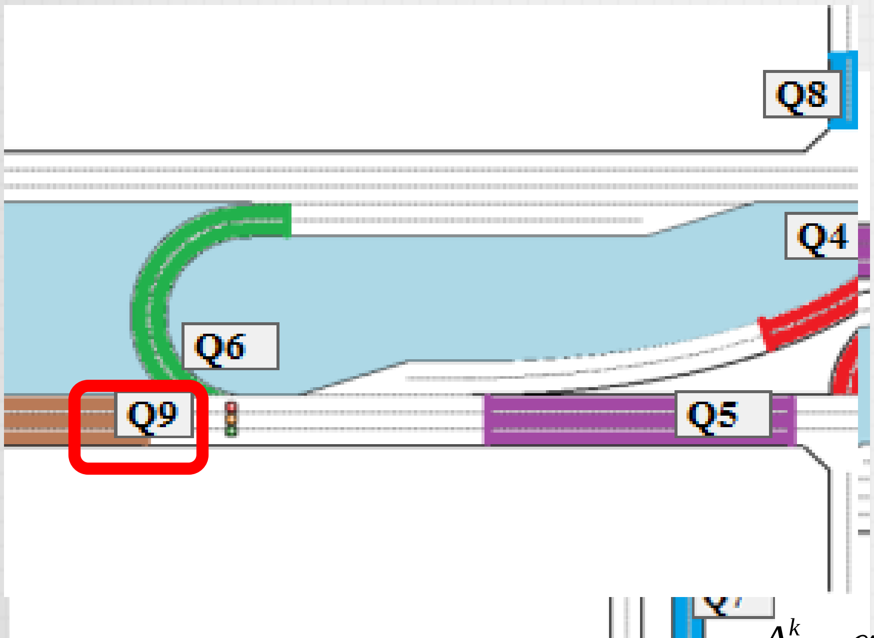


- Type-1 (Q7, Q8, Q9,Q10): Through queues at major & minor road
- Type-2 (Q3, Q6): U-turn queues at the crossover intersection
- Type-3 (Q1, Q4): Left-turn queues at main intersection
- Type-4(Q2, Q5): Through queues at main intersection

Interval-based Queue Model

- **Q5:** **Through queues at the main intersection**

  - Departures from Q6
  - Through and Right-turn departures from Q9



$$D_{9TR}^{k}=\begin{cases}0 & \text{During Red Time}\\ \min(s_9, A_{9TR}^{k}+q_{9TR}^{k}) & \text{During Green Time}\end{cases}$$

where : s_9 is the saturation flow rate for link 9;

β_{9TR} is the through and right-turning ratio for Q9;

A_{9TR}^{k} is the arrived vehicle for through and right-turn movements in Q9 at time k;

q_{9TR}^{k} is the queued through and right-turning vehicles in Q9 at time k.

For Q6, all the departures from it should merging into Q5, so at any time point k, the departures from Q6 to Q5 can be expressed as:

$$D_{6}^{k}=\begin{cases}0 & \text{During Red Time}\\ \min(s_6, A_{6}^{k}+q_{6}^{k}) & \text{During Green Time}\end{cases}$$

where : s_6 is the saturation flow rate for link 6;

A_6^k is the arrived vehicle in Q6 at time point k;

q_6^k is the vehicles in Q6 at time point k.

Arrivals at Q5:

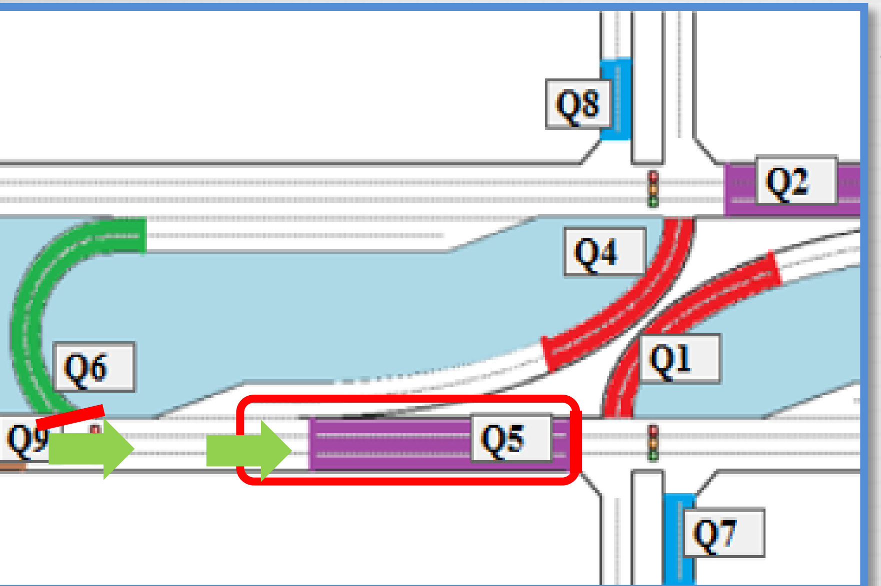
$$A_5^k=\alpha D_{9TR}^{k-\sigma}+(1-\alpha)D_6^{k-\tau}, \alpha=0,1$$

$where: \sigma\ is$ the travel time from Q9 to Q5

τ is the travel time from Q6 toQ5;

while α is a binary variable.

Interval-based Queue Model



When Q5's red and Q9's green is concurrent, we could find the queue as:

$$\bar{Q}_5 = \begin{cases} \int_{t_0}^{t_0+R_5} D_{9TR}^{t-\sigma} dt + \int_{t_1}^{t_1+t*} D_{9TR}^{t-\sigma} dt, if R^5 + t* \le g^9 \\ \int_{t_0}^{t_0+R_5} D_{9TR}^{t-\sigma} dt + \int_{t_1}^{t_1+t*} A_5^t dt, if R^5 + t* > g^9 > R^5 \\ \int_{t_0}^{t_0+g_9} D_{9TR}^{t-\sigma} dt + \int_{t_0+g_9}^{t_0+R_5} D_6^{t-\tau} dt + \int_{t_1}^{t_1+t*} D_6^{t-\tau} dt, if R^5 > g^9 \end{cases}$$

where t_1 is the intial time of green phase of Q5

$t*$ is time to dissipate initial queue

s is the saturation flow rate.

- The queue dissipate time $t*$ can be derived using:

$$\int_{t_0}^{t_0+R_5} A_5^k dt = \int_{t_1}^{t_1+t*} (s_5 - [\alpha D_{9TR}^{k-\sigma} + (1-\alpha) D_6^{k-\tau}]) dt$$

$\alpha = 0$ or 1;

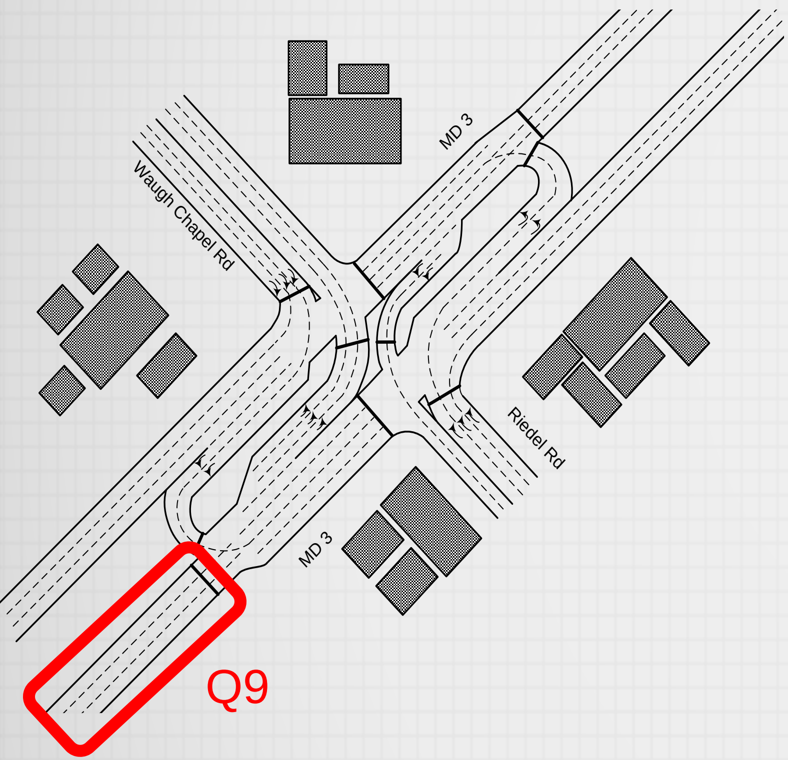
By taking into consideration of incoming traffic fluctuation, we can have the maximum queue interval as:

$$Q_5^{\max} = \bar{Q}(A_5^{\max})$$

$$Q_5^{\min} = \underline{Q}(A_5^{\min})$$

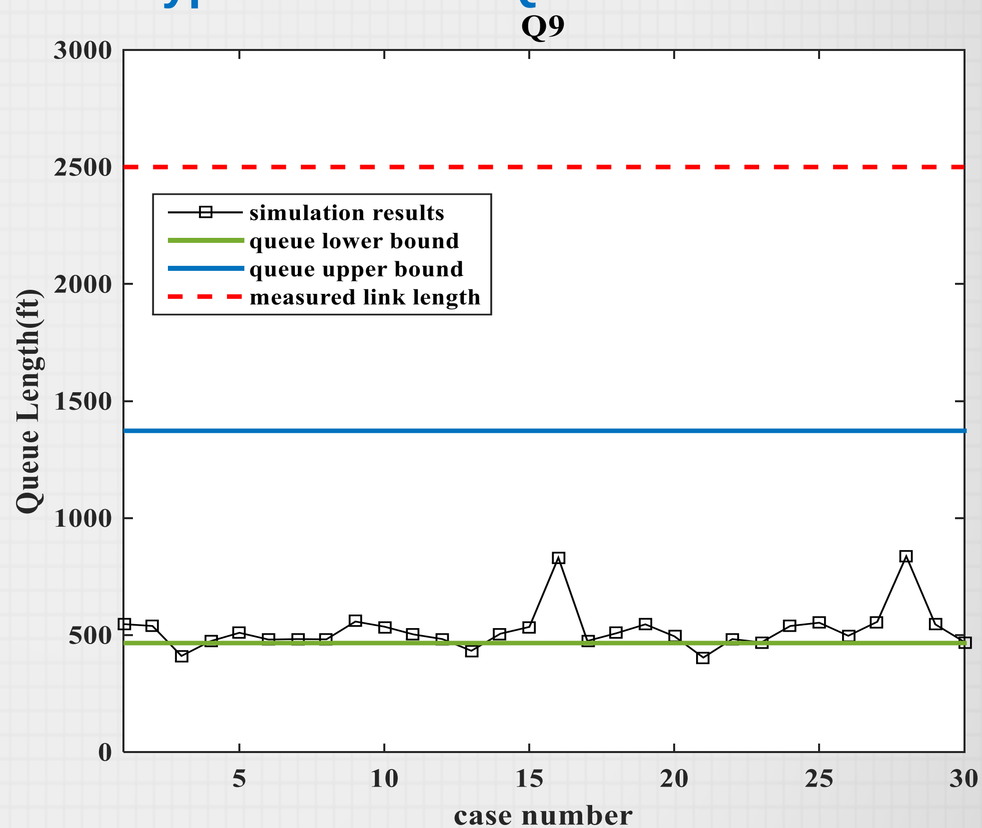
Model Validation

- Field Collected peak hour traffic data are used for the case study
- ➢ Most of the simulated maximum queues fall within the estimated intervals.



MD 3 @ Waugh Chapel Rd

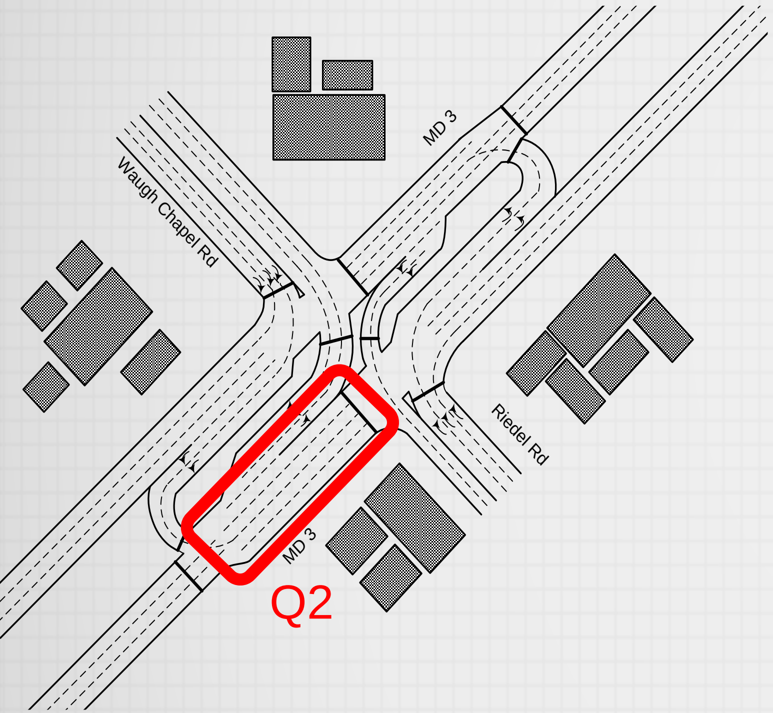
Type 1:External Q



The distribution of simulated maximal queue length (ft)

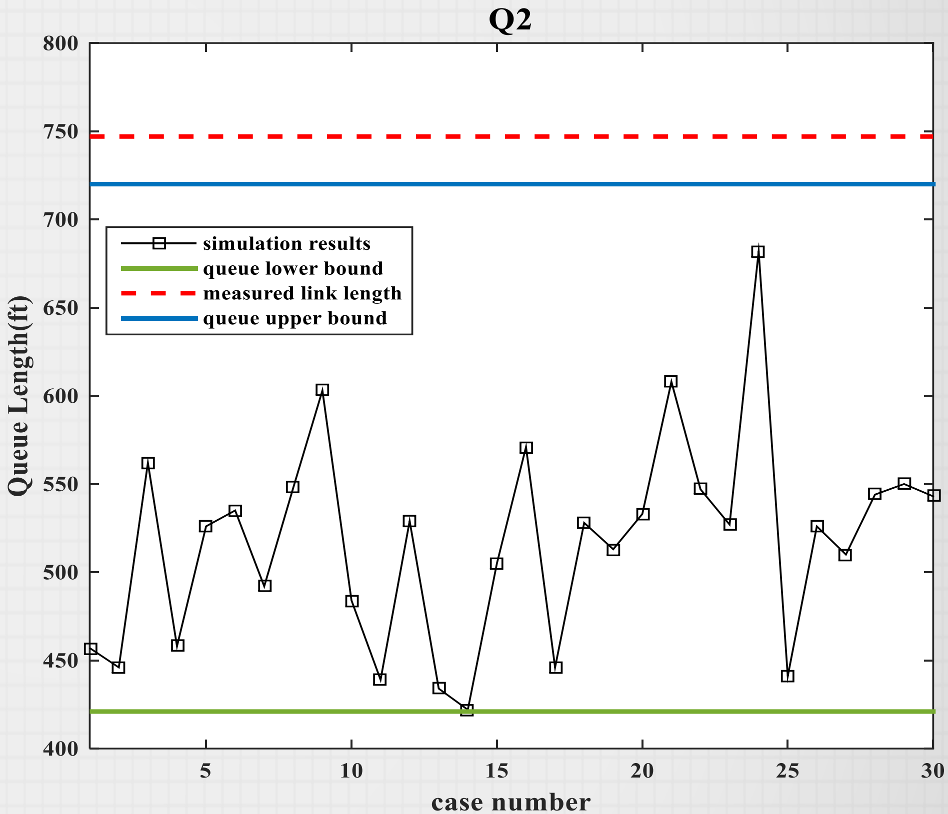
Model Validation

- Field Collected peak hour traffic data are used for the case study
- ➢ Most of the simulated maximum queues fall within the estimated intervals.



MD 3 @ Waugh Chapel Rd

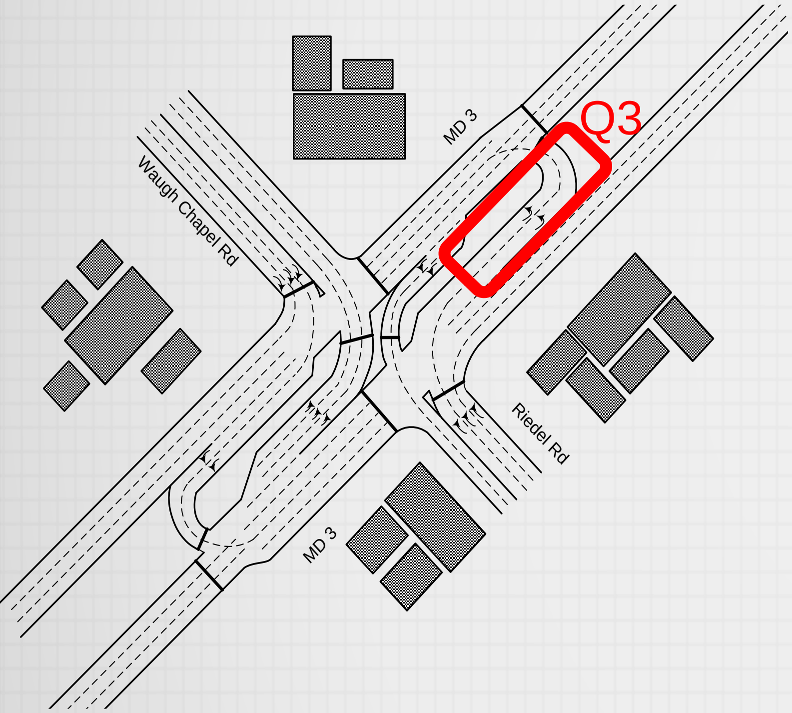
❑ **Type-4(Q2): Main through queue**



The distribution of simulated maximal queue length (ft)

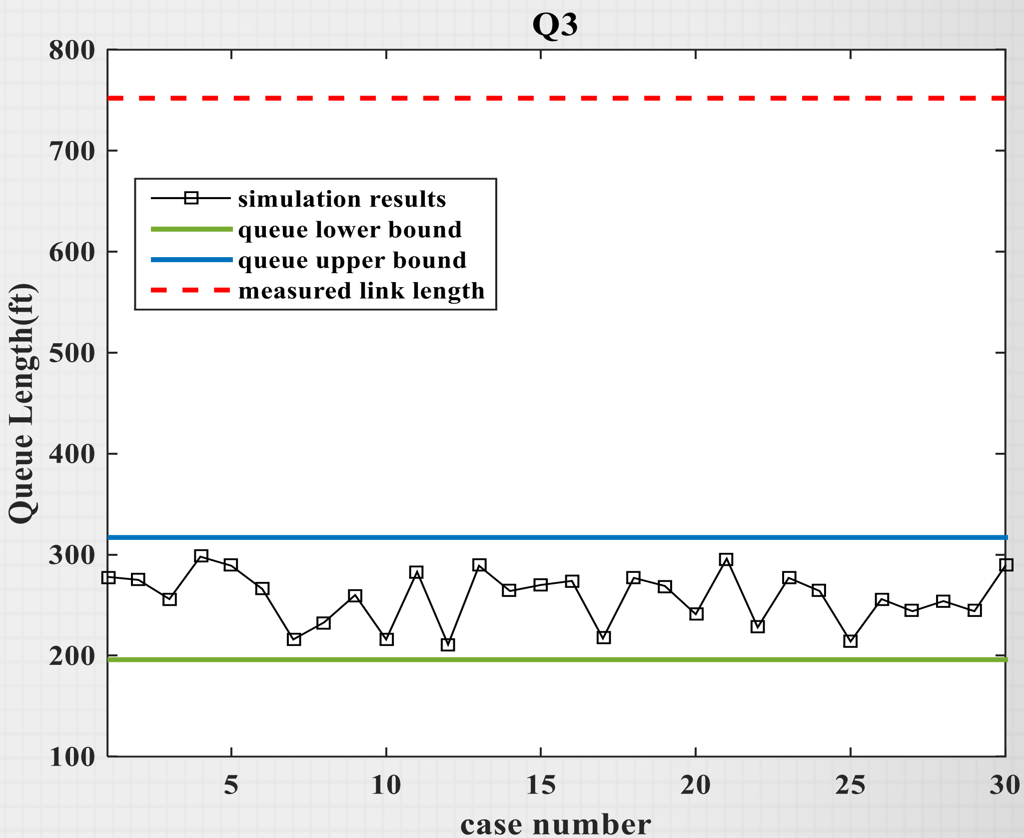
Model Validation

- Field Collected peak hour traffic data are used for the case study
  - ➢ Most of the simulated maximum queues fall within the estimated intervals.



MD 3 @ Waugh Chapel Rd

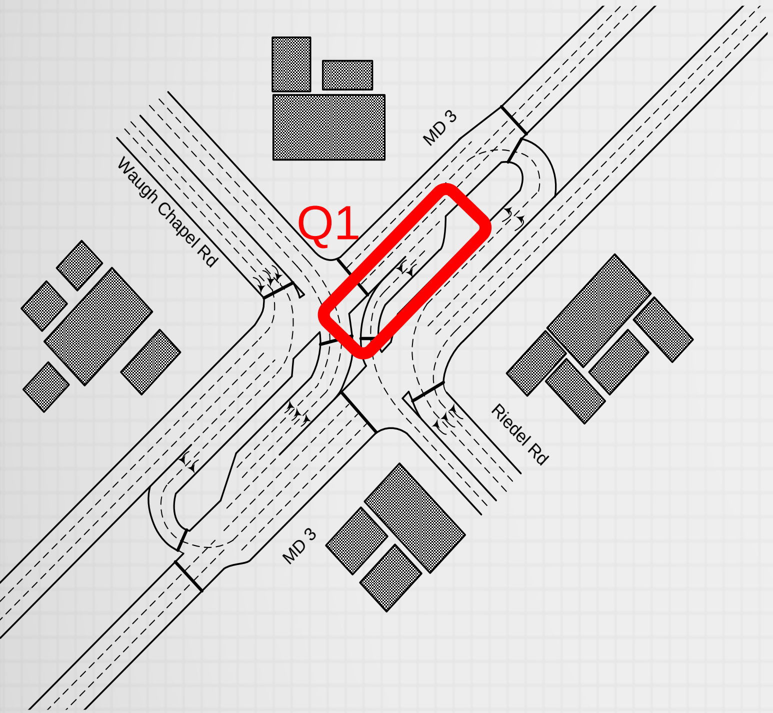
❑ **Type-2(Q3): U-turn queue**



The distribution of simulated maximal queue length (ft)

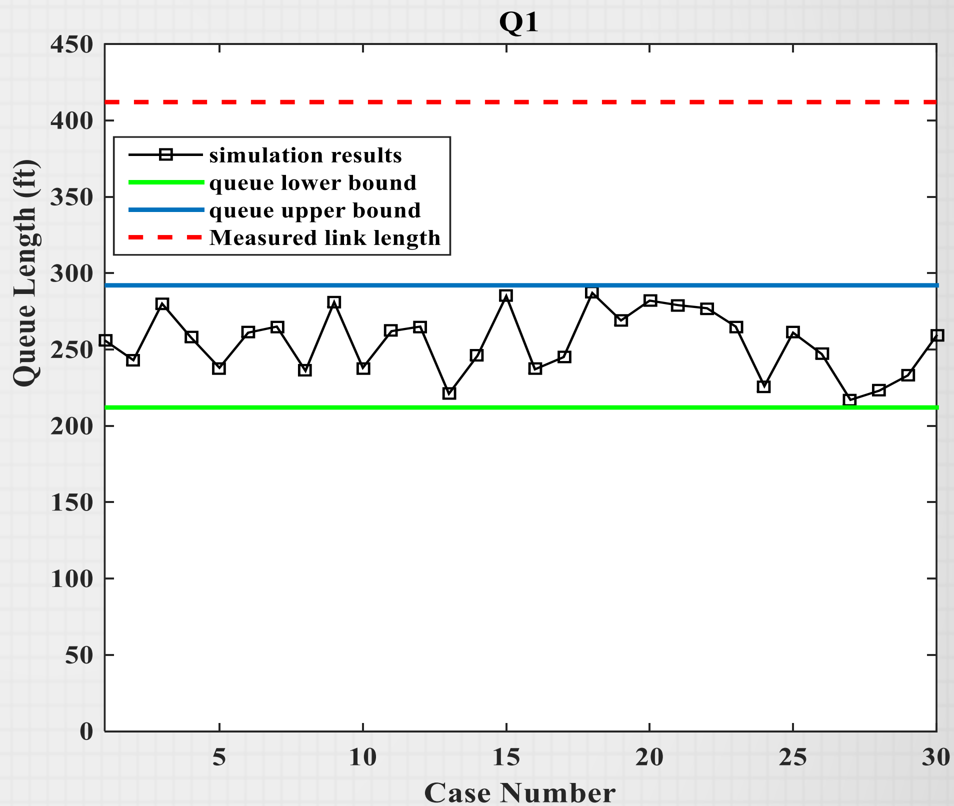
Model Validation

- Field Collected peak hour traffic data are used for the case study
- ➢ Most of the simulated maximum queues fall within the estimated intervals.



MD 3 @ Waugh Chapel Rd

❑ **Type-3(Q1):Main left-turn queue**



The distribution of simulated maximal queue length (ft)

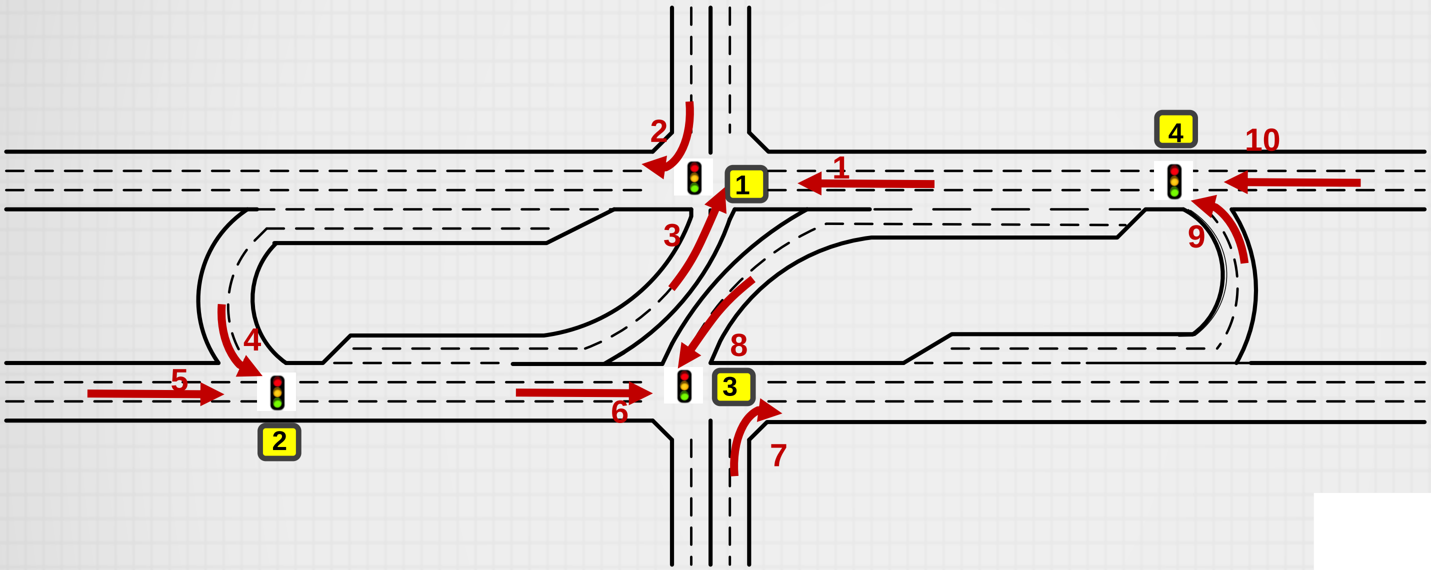
05

TWO-STAGE SIGNAL OPTIMIZATION MODEL FOR A SIGNALIZED SUPERSTREET

Model-3

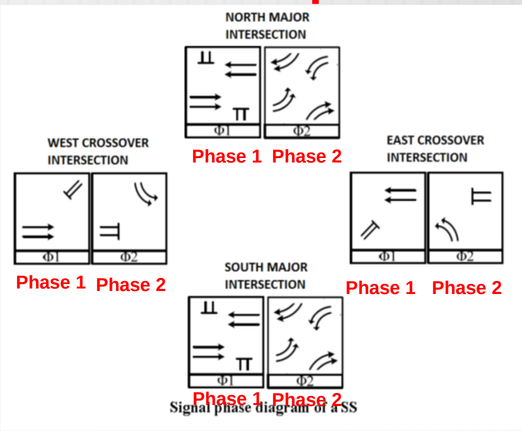
Index for Movements

Index	Movements
1	WB through at sub 1
2	Right-turn at sub 1
3	Left-turn at sub 1
4	U-turn at sub 2
5	EB through at sub 2
6	EB through at sub 3
7	Right-turn at sub 3
8	Left-turn at sub 3
9	U-turn at sub 4
10	WB through at sub 4



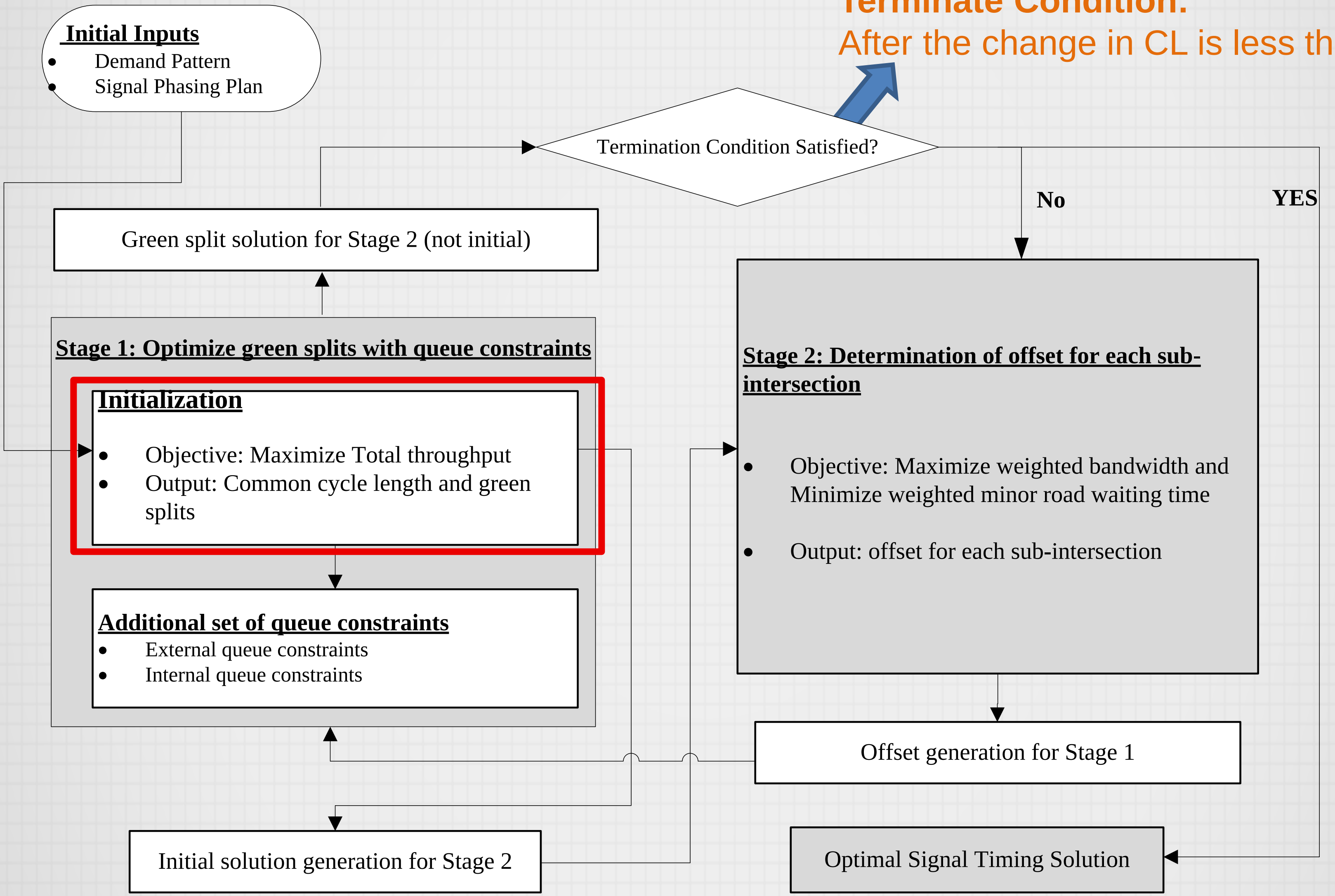
i : index for sub-intersections. 1 stands for northern sub; 2 for western sub; 3 for southern sub and 4 stands for eastern sub.

Phase plan:



General Algorithm Two-stage MILP

Terminate Condition:
After the change in CL is less than 1s?



Stage 1-Initial (Wong, 2003)

- Control Objective: **Capacity Maximization**

$$\text{Objetive Function: } Max(\sum_{i \in I} \mu_i)$$

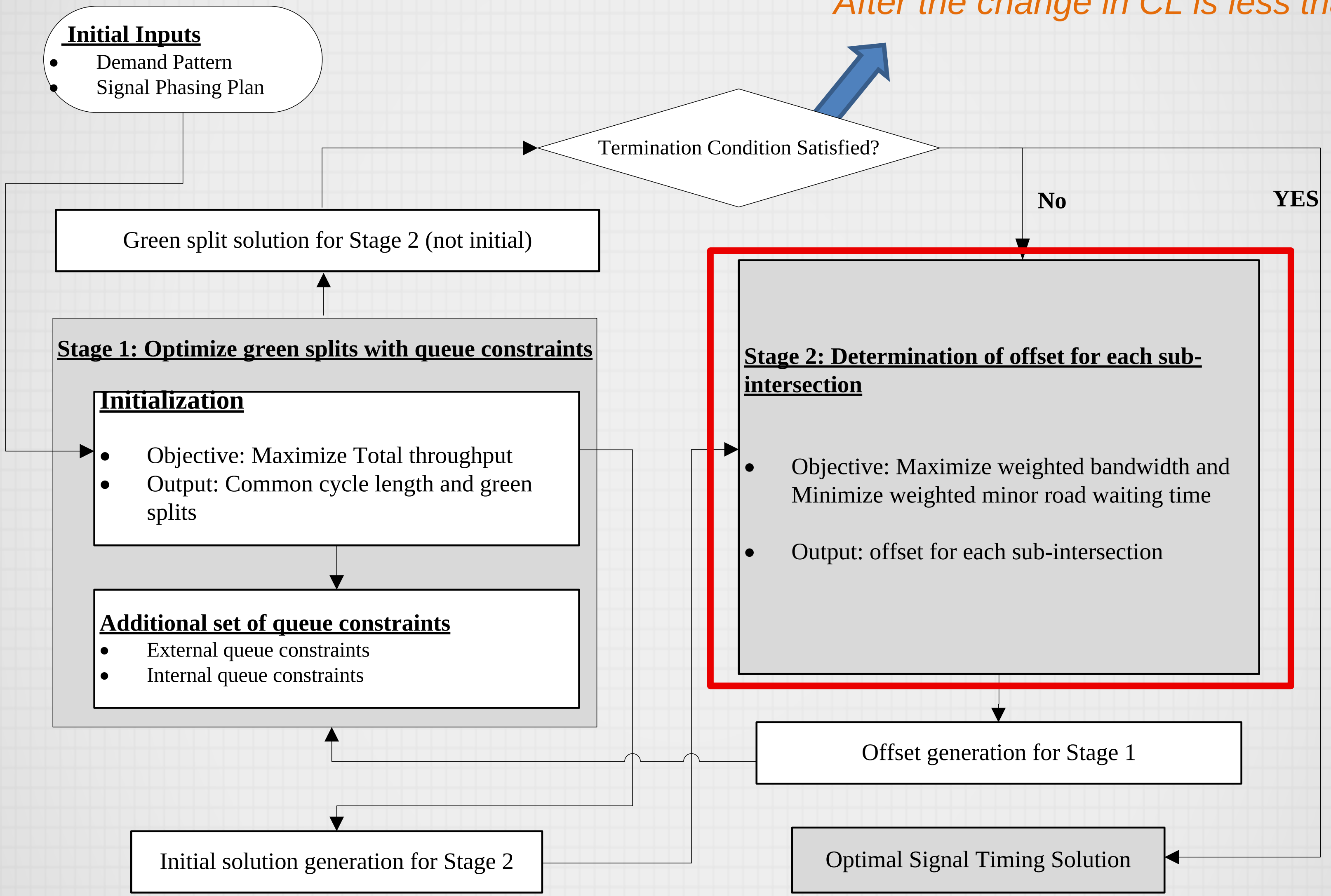
- *Subject to:*
- Traffic rate will not exceed the saturation flow rate: $\mu_i \alpha_{ij} q_{ij} \le s(\phi_{ij} - \xi \times t_l)$
- Cycle length constraints: $\frac{1}{C_{\max}} \le \xi \le \frac{1}{C_{\min}}$
- Green ratio constraints: $g_{\min} \times \xi \le \phi_{ij} \le g_{\max} \times \xi$
- Sum of green time cannot exceed cycle length: $\phi_{ij_1} + \phi_{ij_2} = 1;\ j_1, j_2 \in J$ and $j_1 \ne j_2$

Initial solution for stage 2: *Cycle length and Green splits*

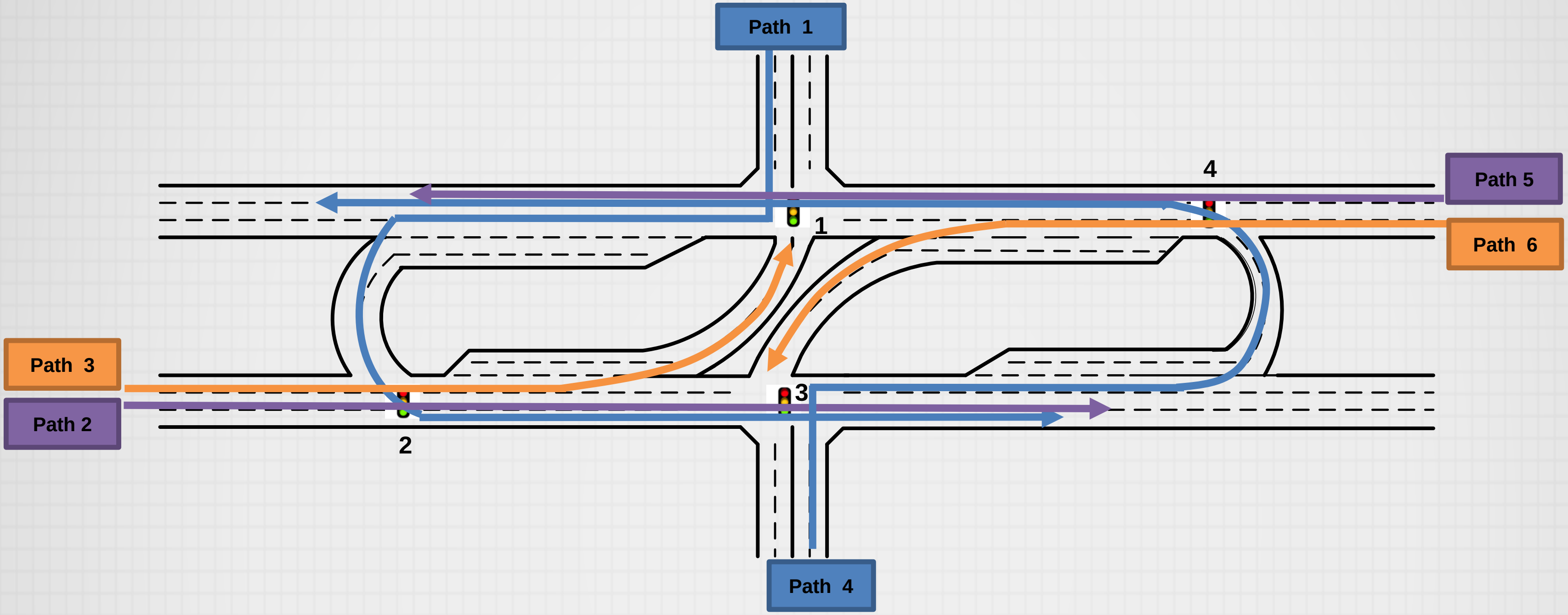
Resource: Wong CK, Wong SC. Lane-based optimization of signal timings for isolated junctions. Transportation Research Part B: Methodological. 2003 Jan 31;37(1):63-84.

General Algorithm

Terminate Condition:
After the change in CL is less than 1s



Critical Paths



- **Path 1, 4:** Through and left-turn movements from the minor road , including 3 signals which are 1-2-3 or 3-4-1;
- **Path 2, 5:** Through and right-turn movements on arterial, including 2 signals which are 2-3 or 4-1;
- **Path 3, 6:** Left-turn movements on arterial, including 2 signals which are 2-1 or 4-3.

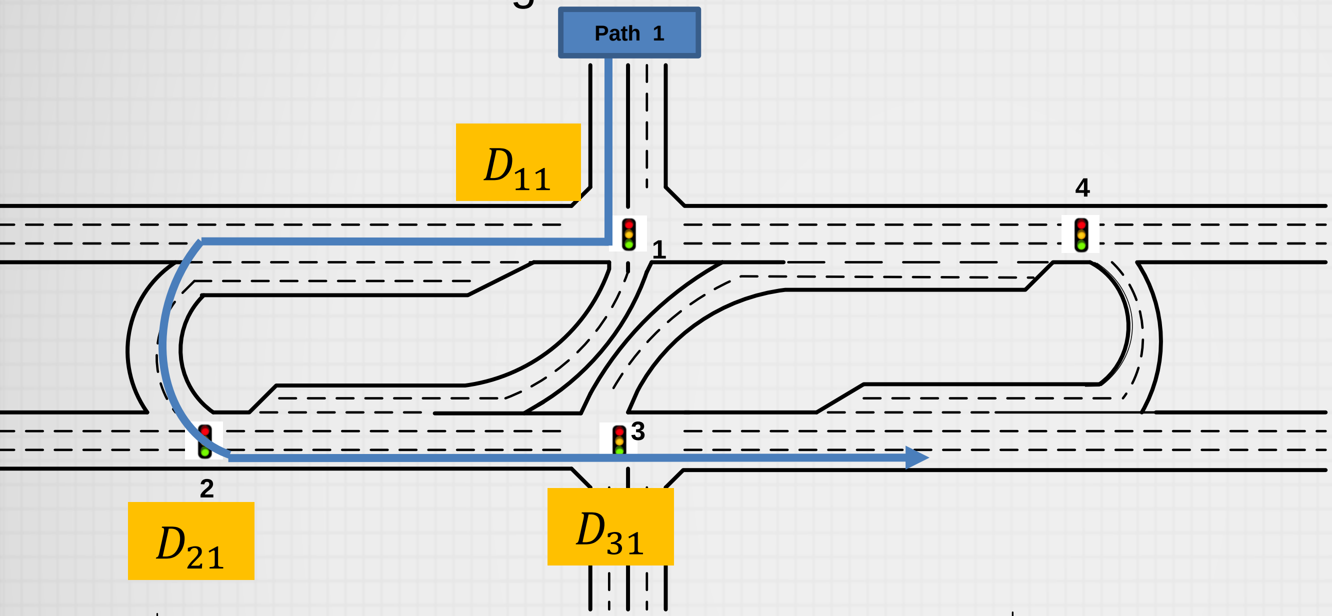
Stage-2 Control Objective:

Green Band Maximization & Minor Road Waiting Time Ratio Minimization

Objective Function: $Max(\sum_{p \in P} \eta_p b_p - f_k \sum_{k \in K} D_{ik})$

Stage 2

- Minor Road Waiting time constraints:



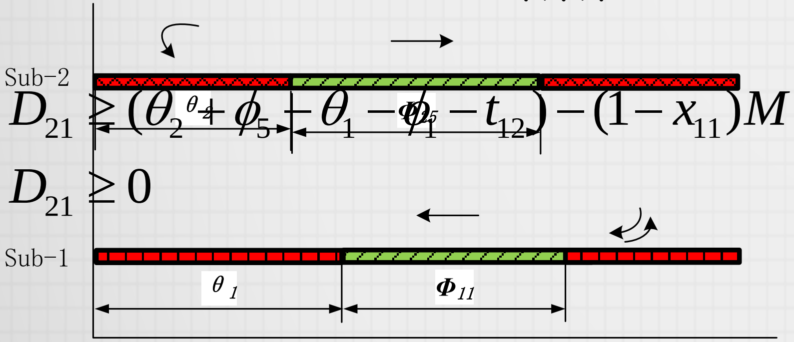
$$x_{11} \geq \frac{\theta_2 + \phi_5 - \theta_1 - \phi_1 - t_{12}}{M}$$

$$x_{11} \leq \frac{\theta_2 + \phi_5 - \theta_1 - \phi_1 - t_{12}}{M} + 1$$

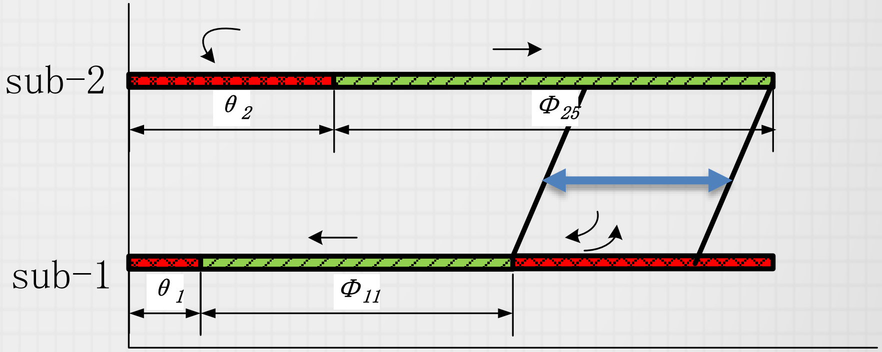
$x_{11} = 0,1$ are binary variables

$$D_{21} \geq (\theta_2 + \phi_5 - \theta_1 - \phi_1 - t_{12}) - (1 - x_{11})M$$

$$D_{21} \geq 0$$



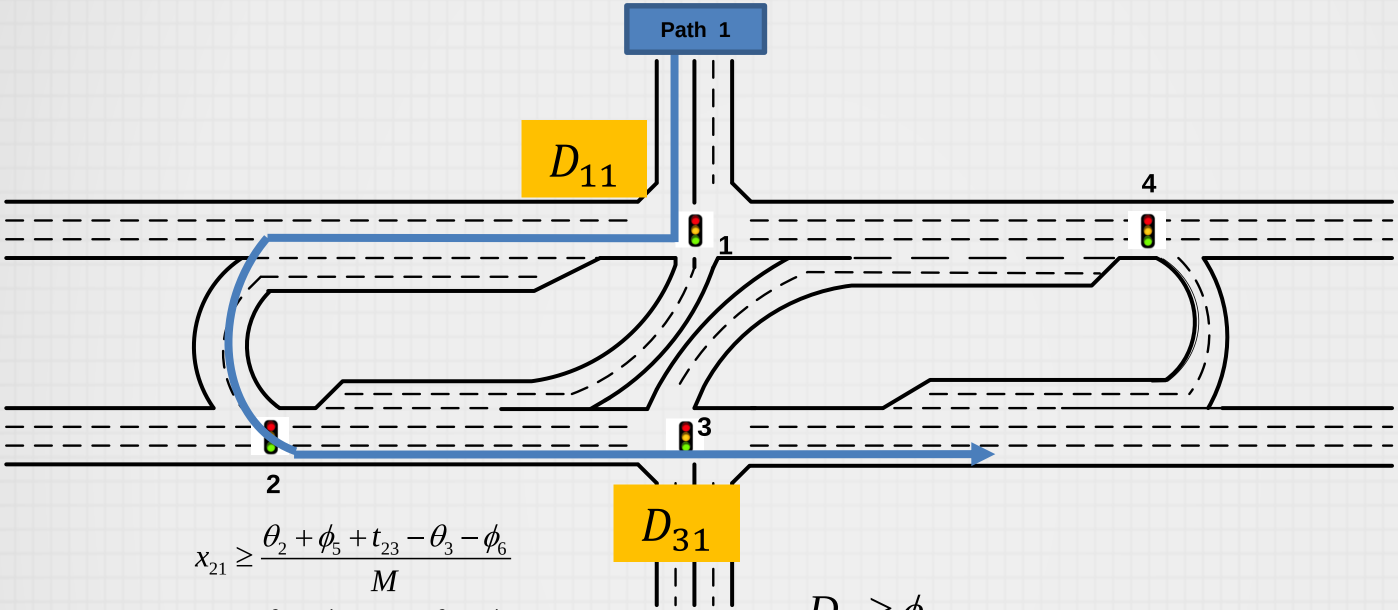
$x_{11} = 0$



$x_{11} = 1$

Stage 2

- Minor Road Waiting time constraints:



$$x_{21} \geq \frac{\theta_2 + \phi_5 + t_{23} - \theta_3 - \phi_6}{M}$$

$$x_{21} \leq \frac{\theta_2 + \phi_5 + t_{23} - \theta_3 - \phi_6}{M} + 1$$

$$x_{31} \geq \frac{\theta_1 + \phi_1 + t_{12} + t_{23} - \theta_3 - \phi_6}{M}$$

$$x_{31} \leq \frac{\theta_1 + \phi_1 + t_{12} + t_{23} - \theta_3 - \phi_6}{M} + 1$$

$x_{21}, x_{31} = 0,1$ are binary variables

$$D_{11} \geq \phi_1$$

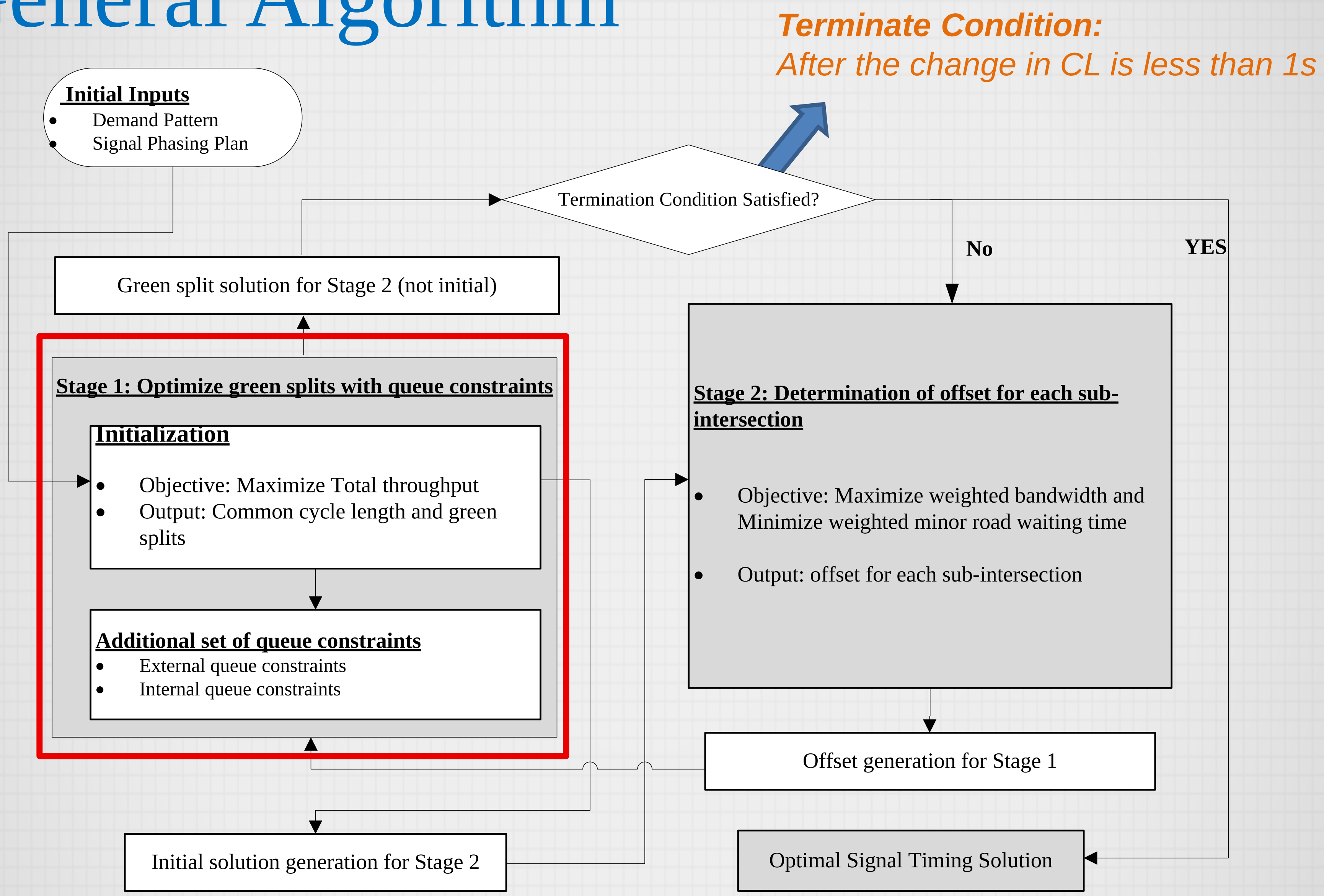
$$D_{31} \geq (1 - \theta_2 - \phi_5 - t_{23} + \theta_3) - (1 - x_{11})M - (1 - x_{21})M$$

$$D_{31} \geq (1 - \theta_1 - \phi_1 - t_{12} - t_{23} + \theta_3) - x_{11}M - (1 - x_{21})M$$

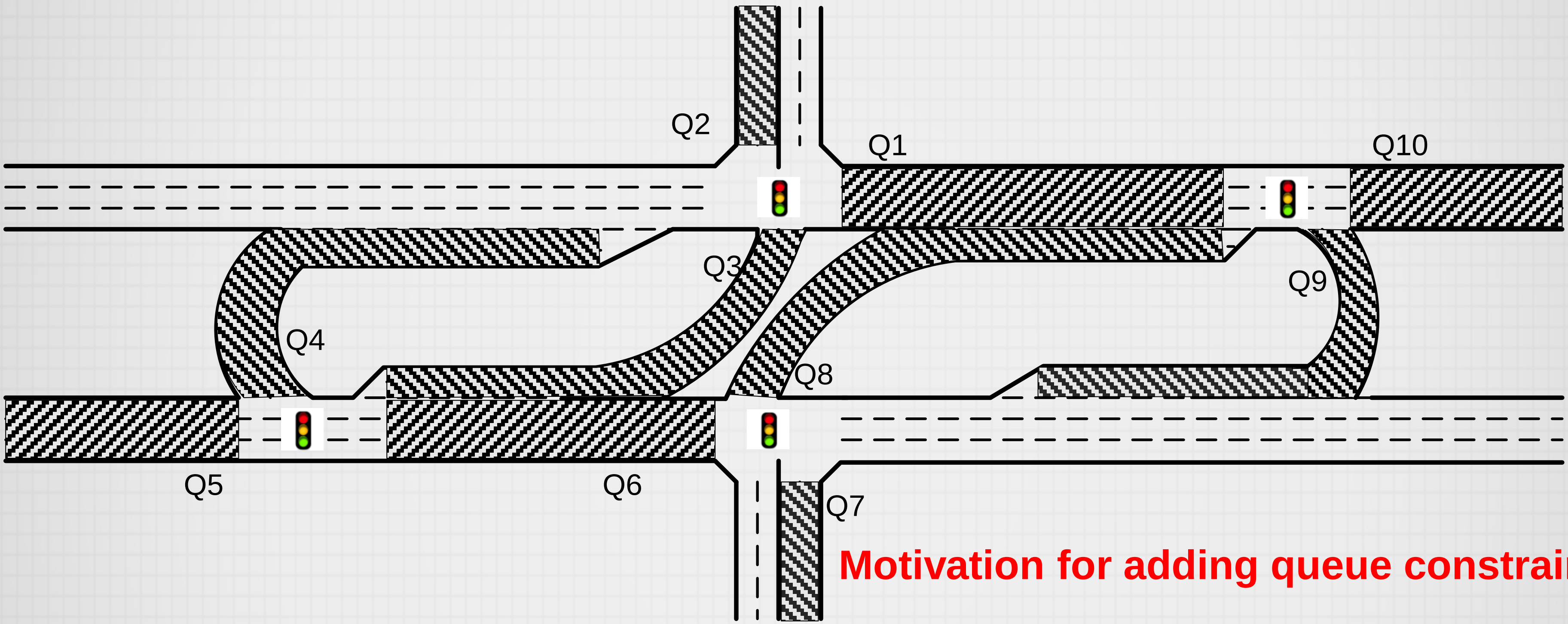
$$D_{11}, D_{31} \geq 0$$

$$\boxed{D_{11} + D_{21} + D_{31} \leq \lambda \xi * (t_{12} + t_{23})}$$

General Algorithm



Spatial Distribution of Potential Queues



Motivation for adding queue constraints:

Cycle Length is the key contributing factor to the queue formations!

- ❑ External Queues: Q2,Q5,Q7,Q10;
- ❑ Internal Queues: Q1,Q6,Q3,Q8,Q4,Q9.

Stage 1-Queue Constraints

- While keeping the previous *Objective function* & *All Constraints* in Initial;

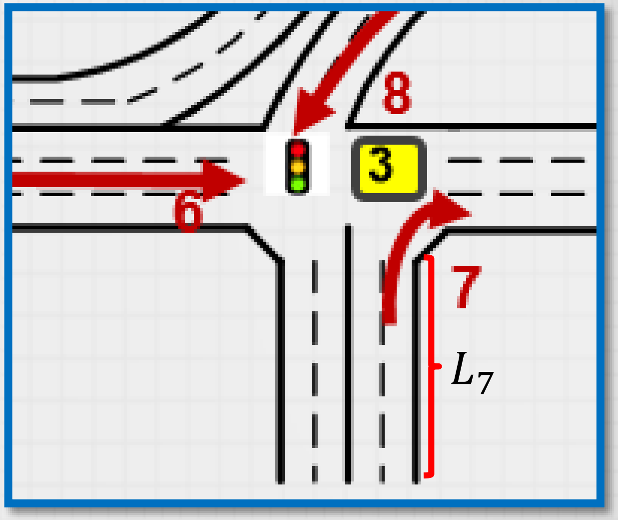
- Maximum Queue won't exceed the link length:

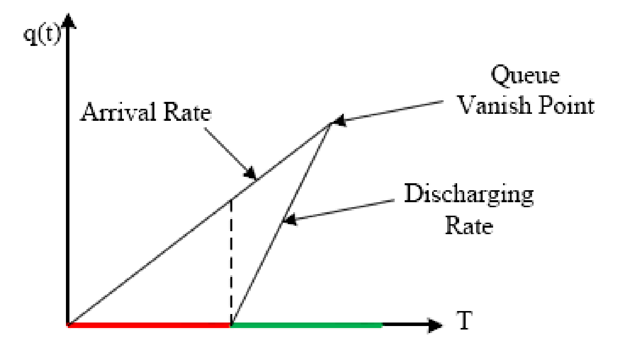
--For **External Queues:**

❑ For Q7:

$$L_7 \le \frac{(1-\phi_{37}+t_l*\xi)*\alpha_7 q_7 * s}{(s-\alpha_7 q_7)\xi}$$

$$(1-\phi_{37}+t_l*\xi)\alpha_{37}q_{37}s \le L_7(s-\alpha_{37}q_{37})\xi$$





Queue formation process

Stage 1-Queue Constraints

- **Internal Queues:**
  - For U-turn Queue (Q4):

Define binary variables:

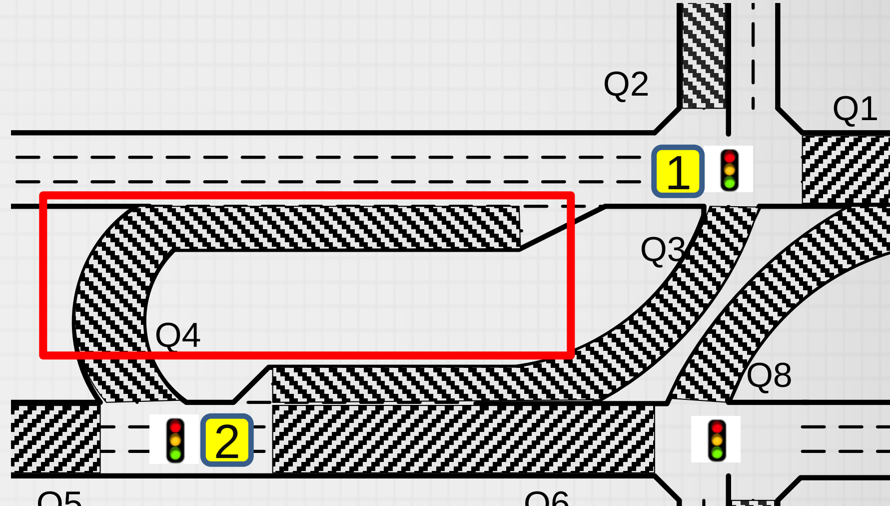
$$y_4 \geq \frac{\theta_2 + \phi_{25} - t_{12} - \theta_1 - \phi_{11}}{M}$$

$$y_4 \leq \frac{\theta_2 + \phi_{25} - t_{12} - \theta_1 - \phi_{11}}{M} + 1$$

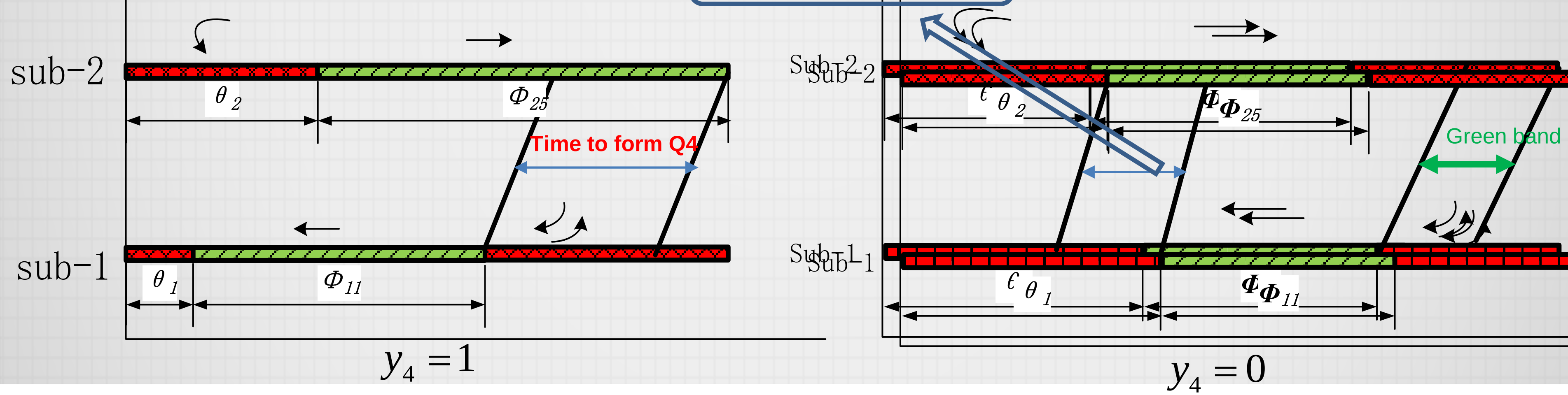
$$Q_4' \geq (\theta_2 + \phi_{25} - \theta_1 - \phi_{11} - t_{12})\alpha_{24} q_2^{LT} - (1 - y_4)M$$

$$Q_4' \geq 0;$$

f_1, *binary* parameter, $=1$ if $\theta_1 > \theta_2 - t_{12}, o.w = 0$

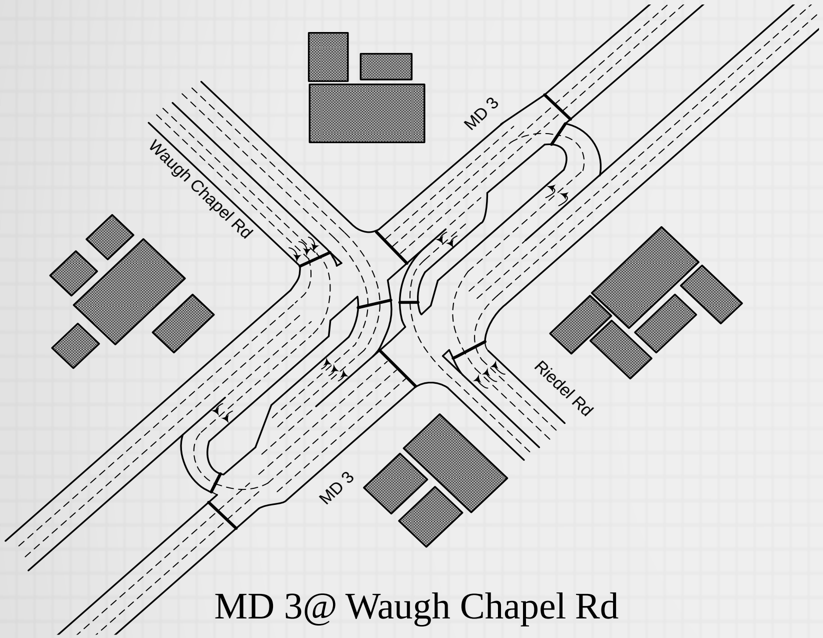


Q4 cannot exceed link length 4: $\boxed{\alpha_4 q_2^{LT} f_1(\theta_1 - \theta_2 + t_{12})} + Q_4' \leq \xi L_4$

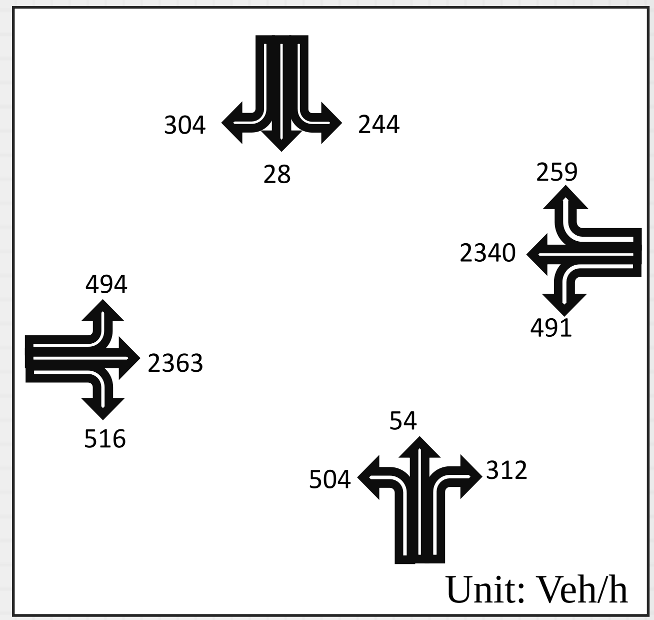


Case Study

❑ Input demand:



MD 3@ Waugh Chapel Rd

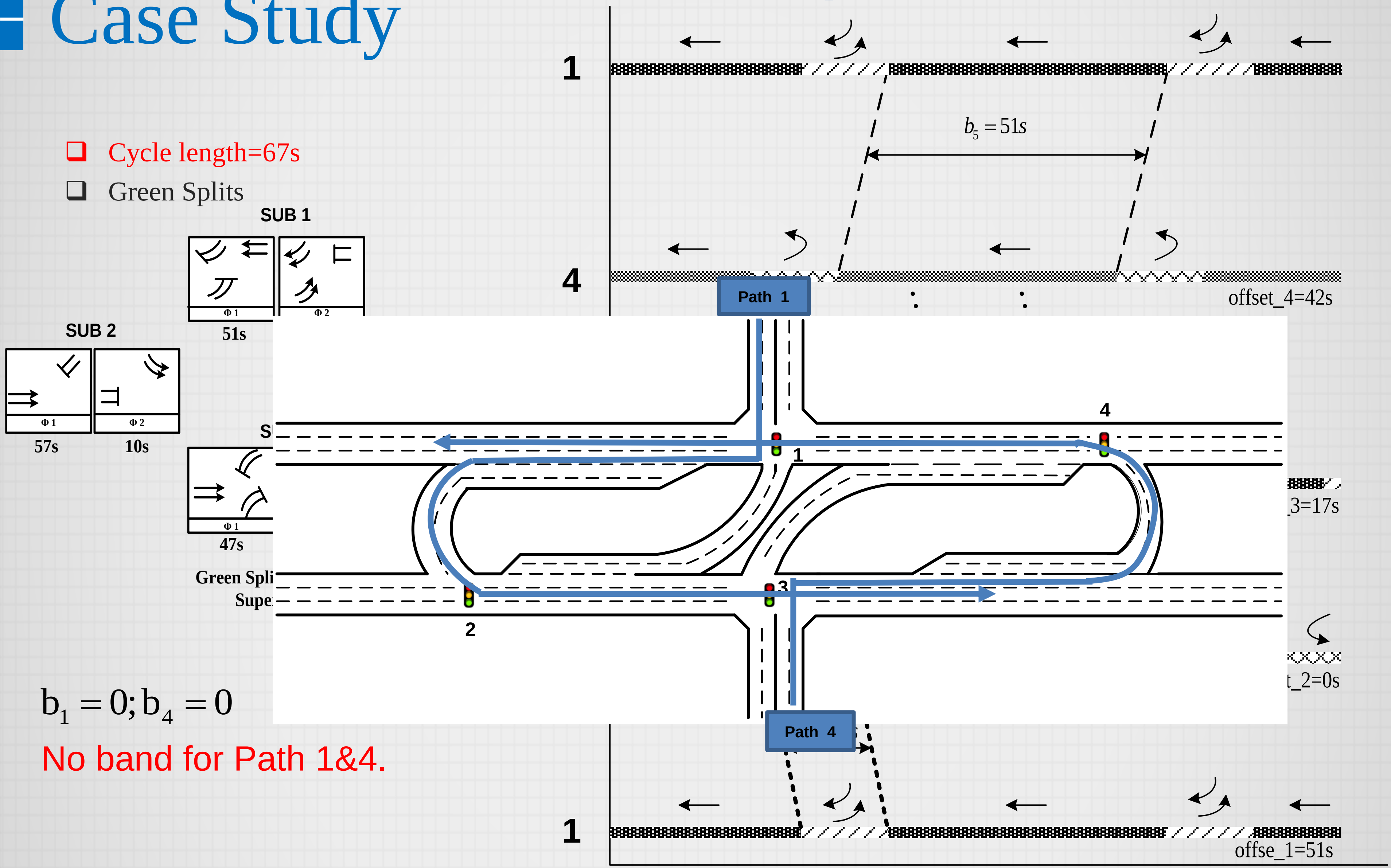


Using the MD 3@ Waugh Chapel Rd field collected traffic data, the model has ran 4 times to get the optimized signal plan.

The maximum waiting time for minor Rd drivers are not exceeding the upper bound.

Case Study

- Cycle length=67s
- Green Splits

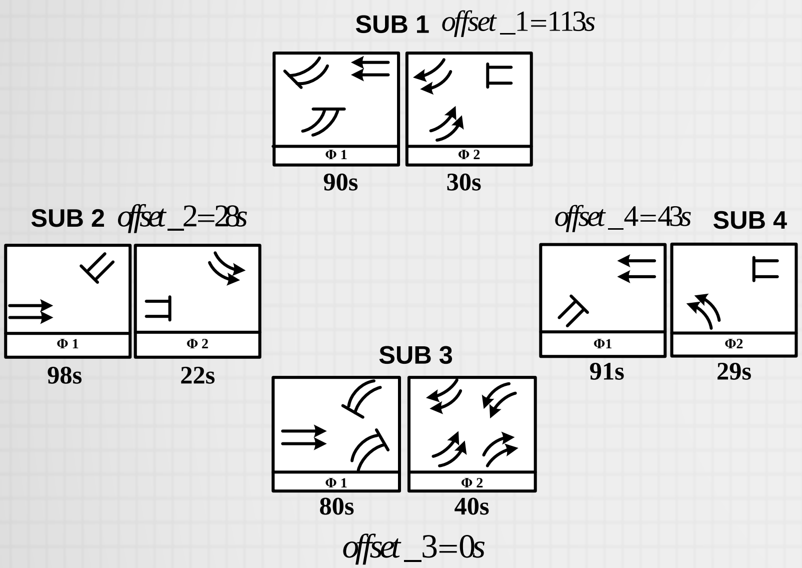


$b_1 = 0; b_4 = 0$

No band for Path 1&4.

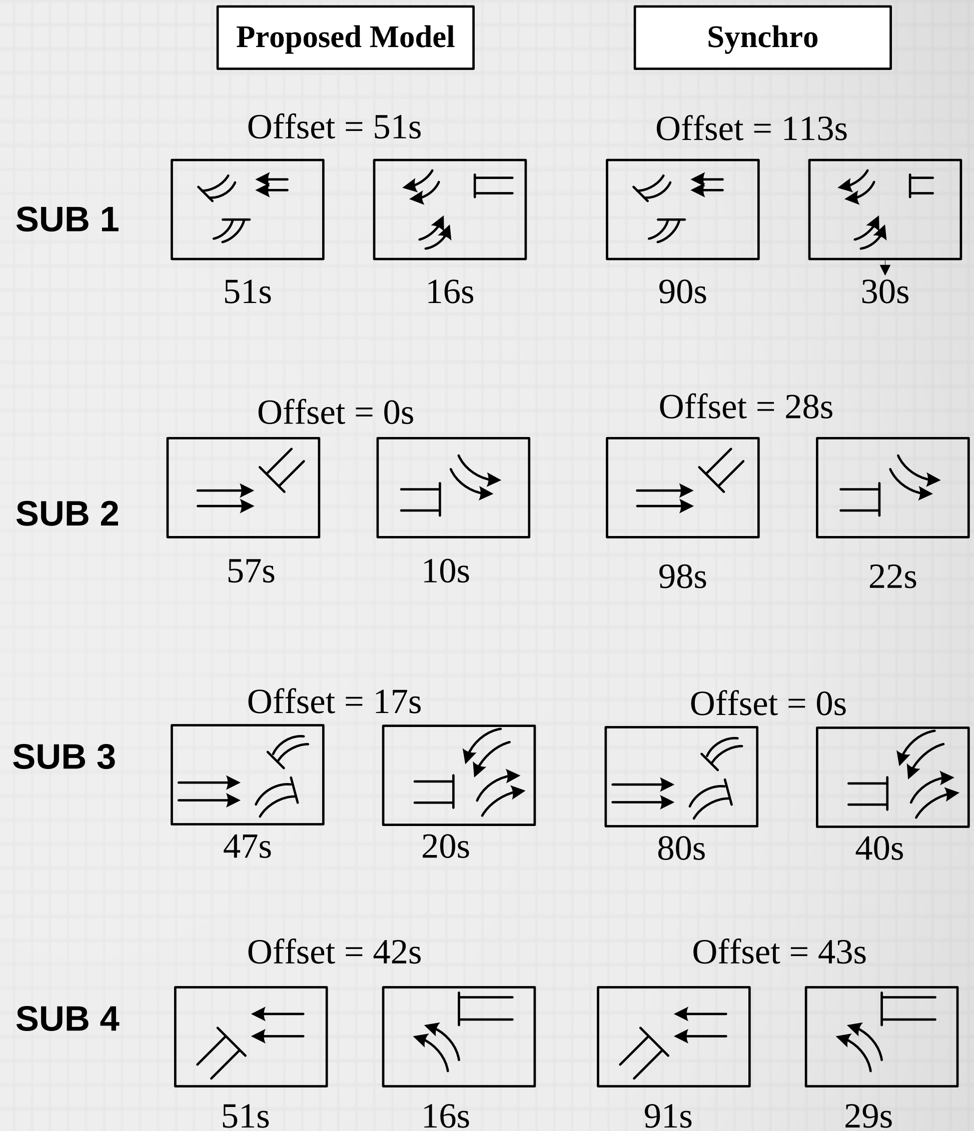
Case Study

- **Comparison Solution (Synchro)**
- Cycle Length: 120s
- Green Splits:



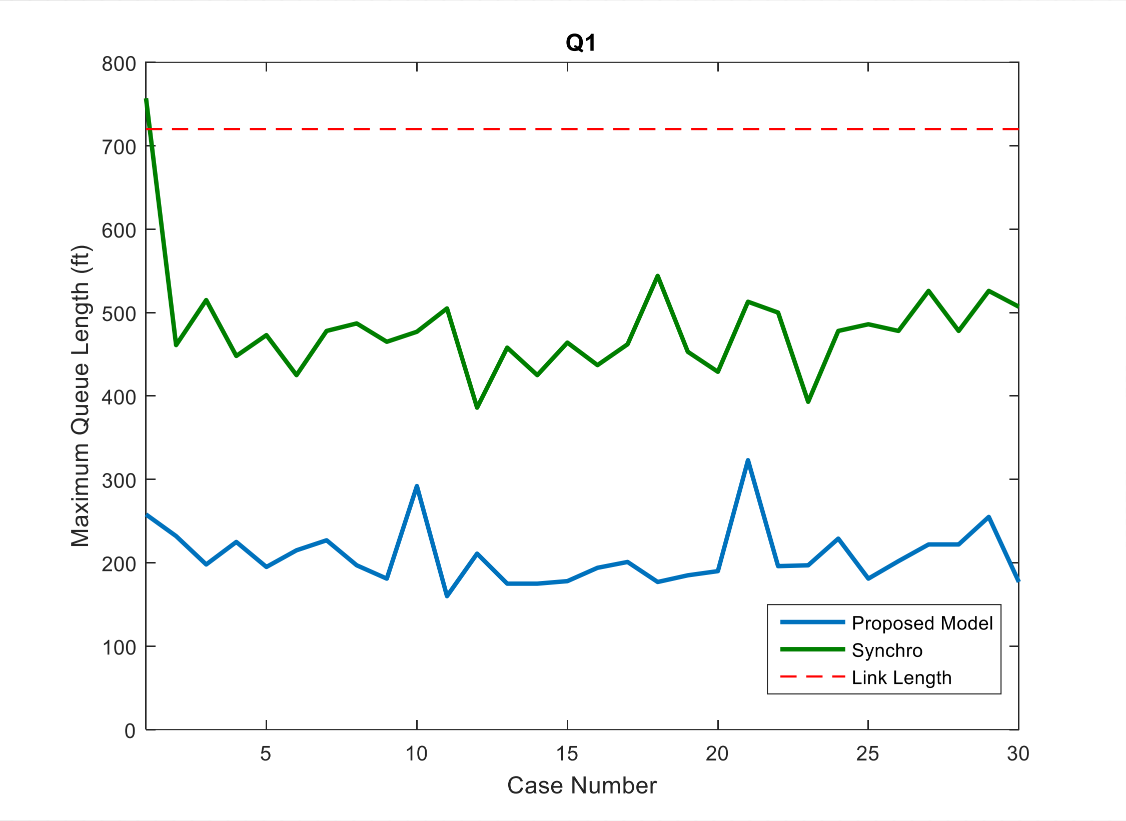
Comparison Case: Synchro Optimization Solution

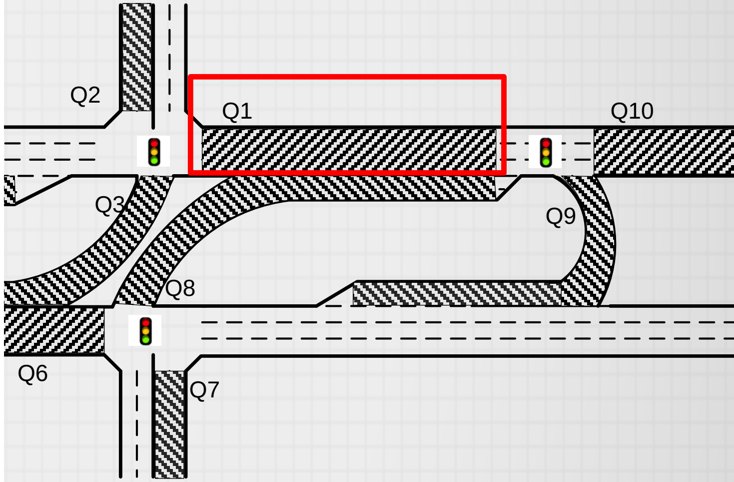
Proposed Model *VS.* Synchro



Case Study

- **Simulation Results Comparison** *(30 cases, 2hr duration per case)*
  - Maximum Queue Length Comparison for Q1—(Main Through Q)

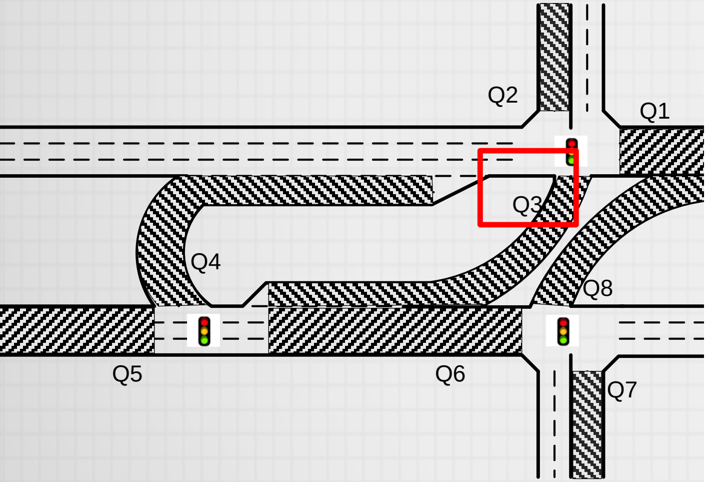


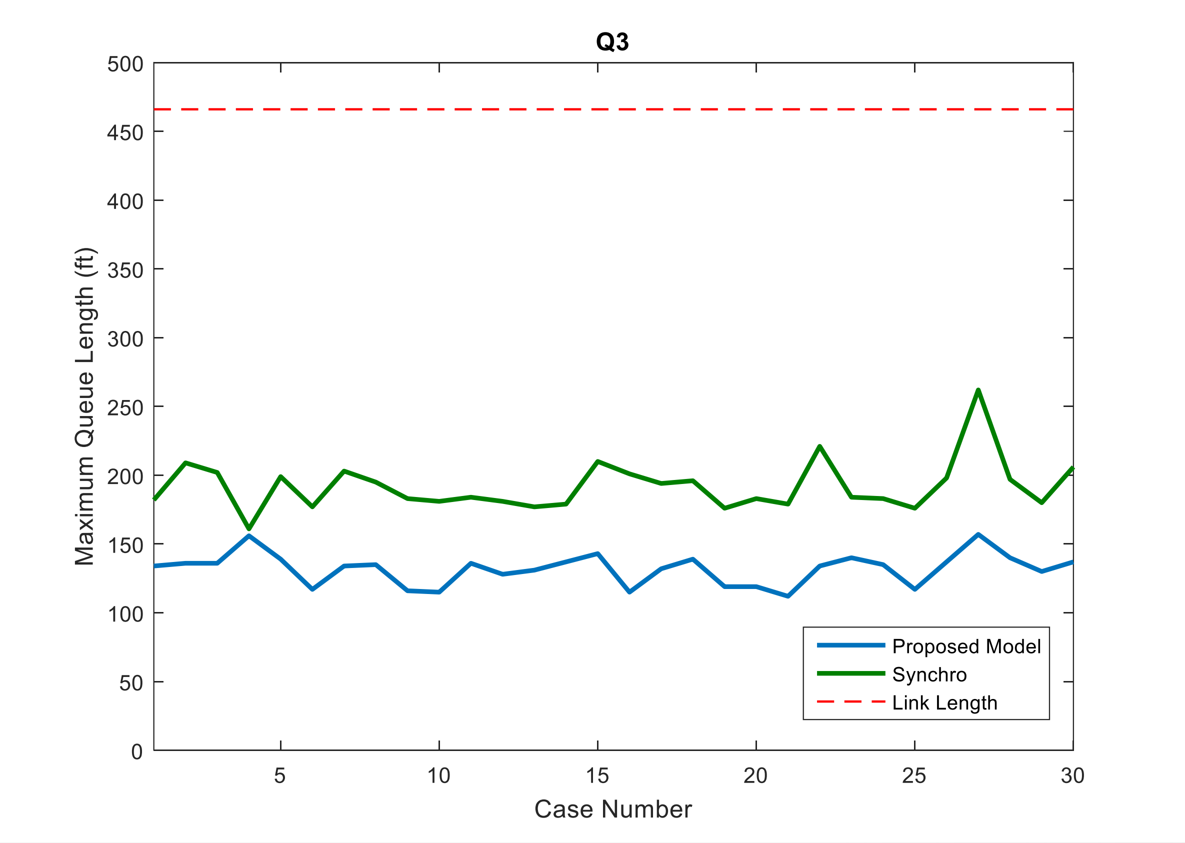


Case Study

❑ **Simulation Result Comparisons** *(30 cases, 2hr duration per case)*

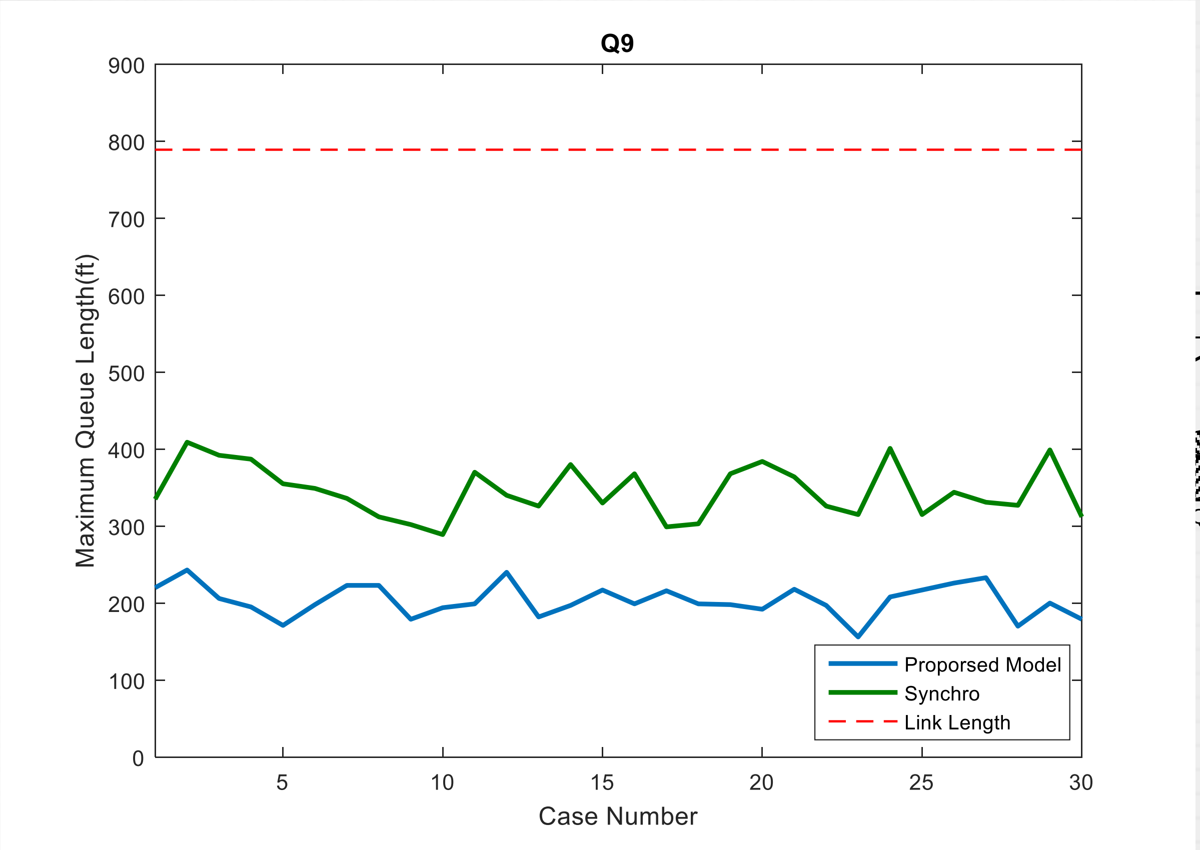
- Maximum Queue Length Comparison for Q3--(Main left-turn Q)

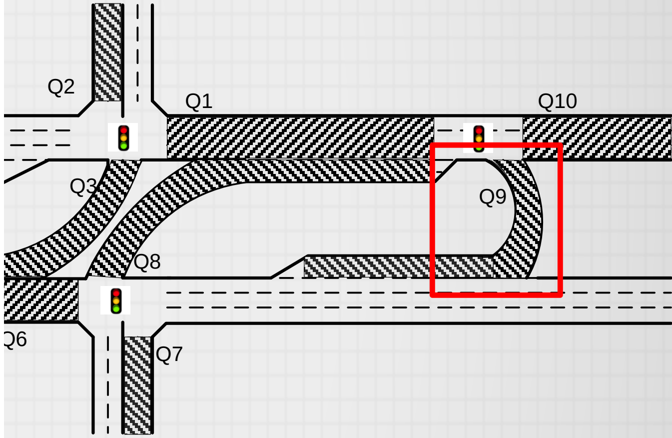




Case Study

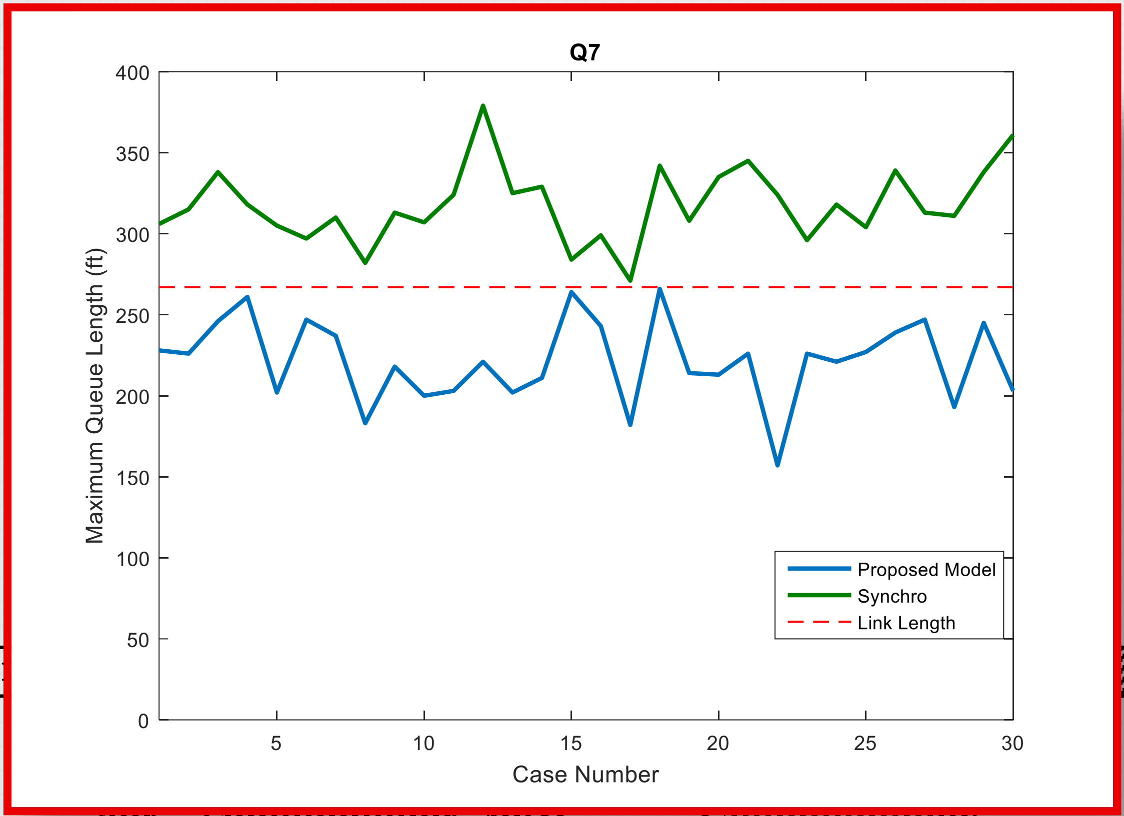
- **Simulation Result Comparisons** *(30 cases, 2hr duration per case)*
  - Maximum Queue Length Comparison for Q9—(U-turn Q)

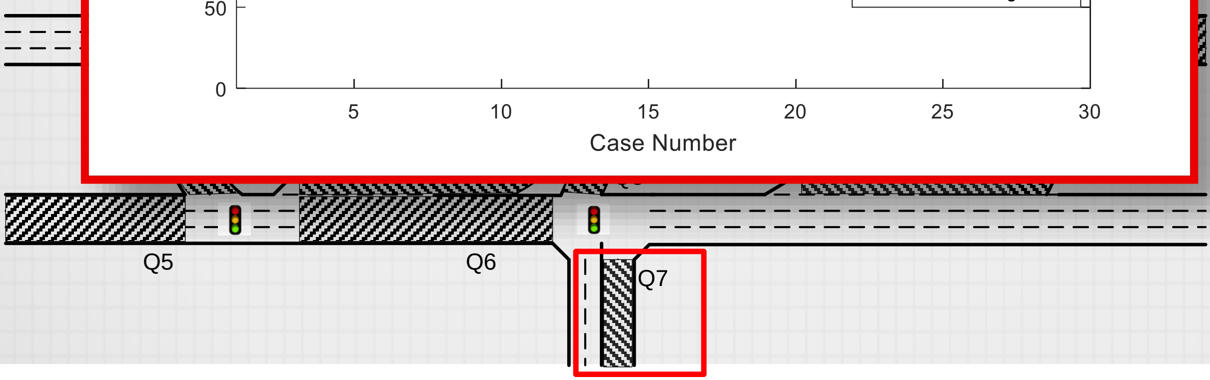




Case Study

- **Simulation Result Comparisons** *(30 cases, 2hr duration per case)*
  - Maximum Queue Length Comparison for External Queues

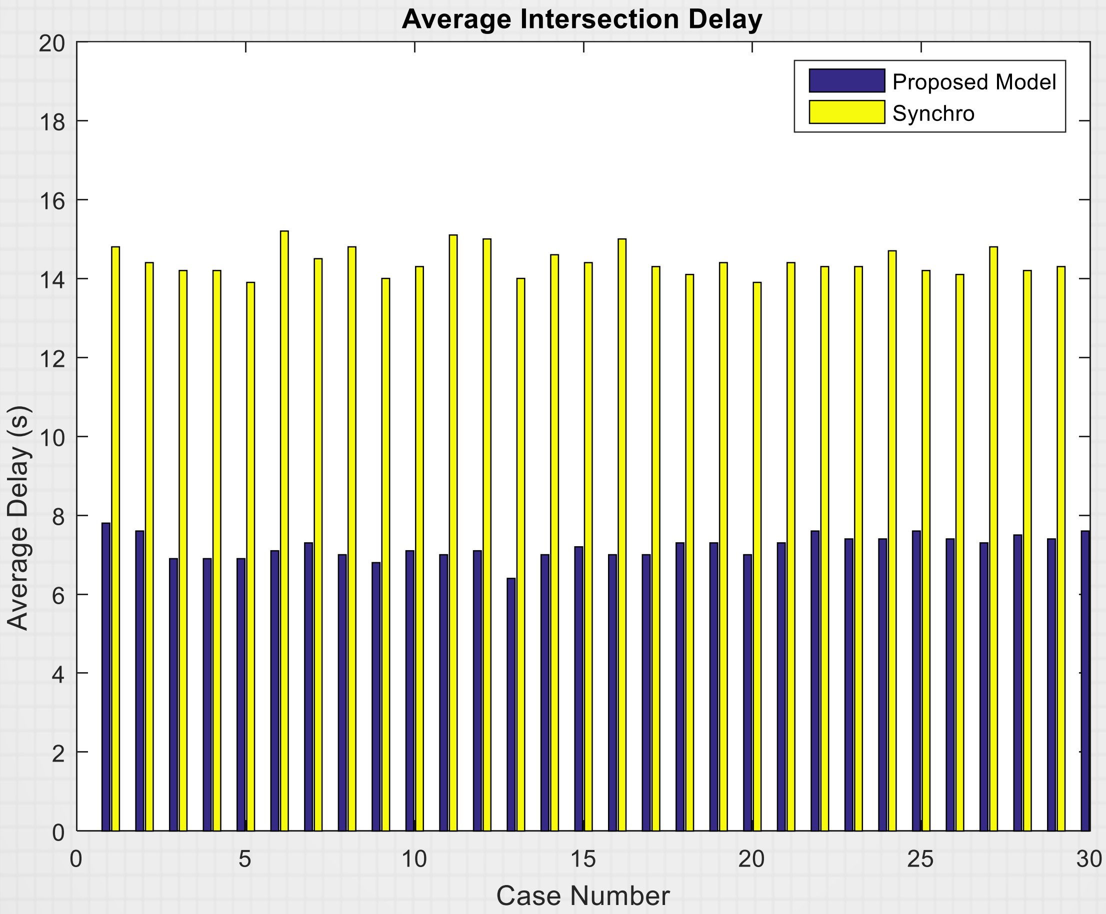




Case Study

❑ Simulation Result Comparisons

- Average Intersection Delay Comparison



06 CONCLUSIONS

Contributions

A. Proposed the procedures and formulations to compute the *minimum required U-turn offset length* for an un-signalized Superstreet;

B. Developed the *interval-based models* for evaluating *the bay length* design in a signalized Superstreet under the given demand variation;

C. Presented an efficient *two-stage signal optimization* model to prevent queue spillback on intersection links and to *minimize the delays* experienced by minor road drivers.

This research offers reliable tools to assist traffic professionals in the design of Superstreets with and without signal control.

Future Work

- ***Field calibration and evaluation on the minimum U-turn offset length model for an Un-signalized Superstreet.***
- ***Evaluation of the impacts of a Superstreet on its neighboring intersections*.**
- ***Coordination of a signal plan for a signalized Superstreet with its neighboring intersections on the same corridor.***

THANKS FOR YOUR TIME