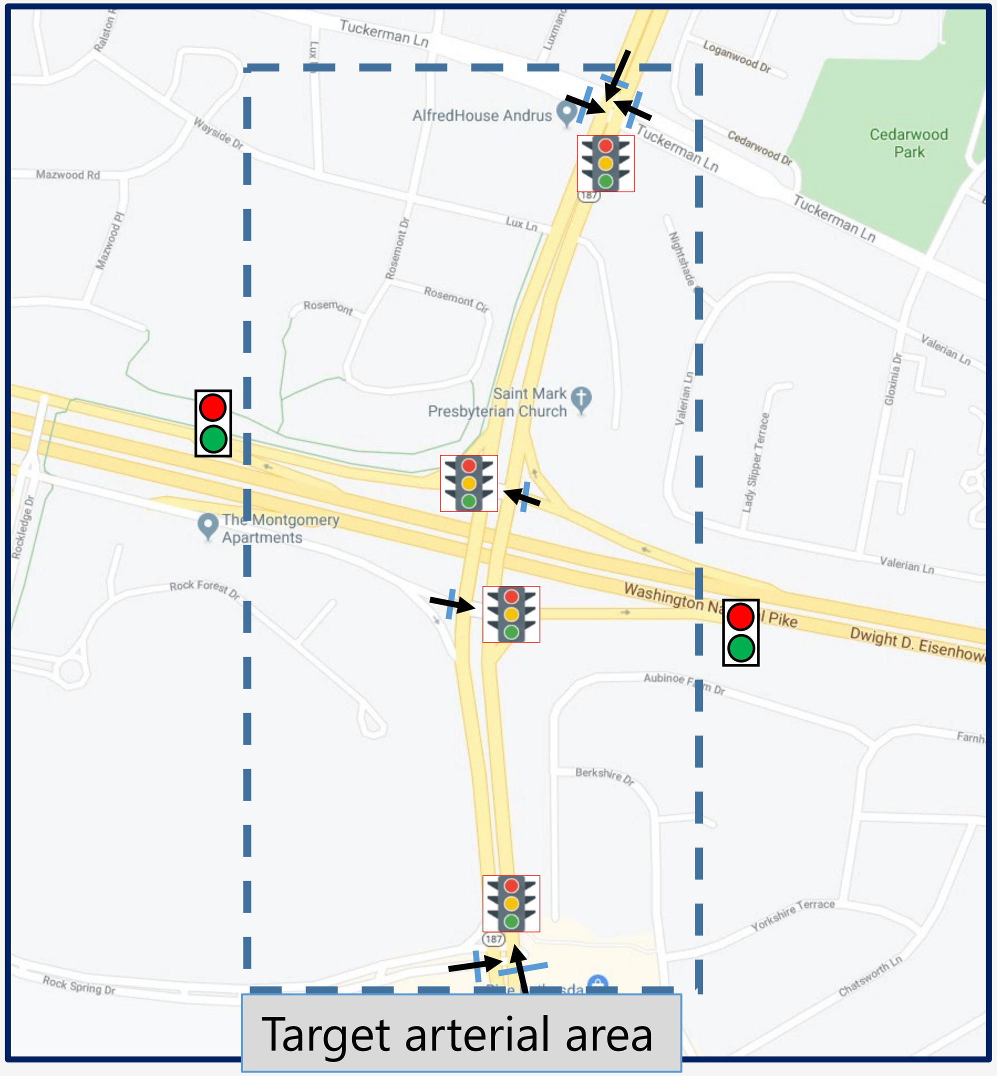
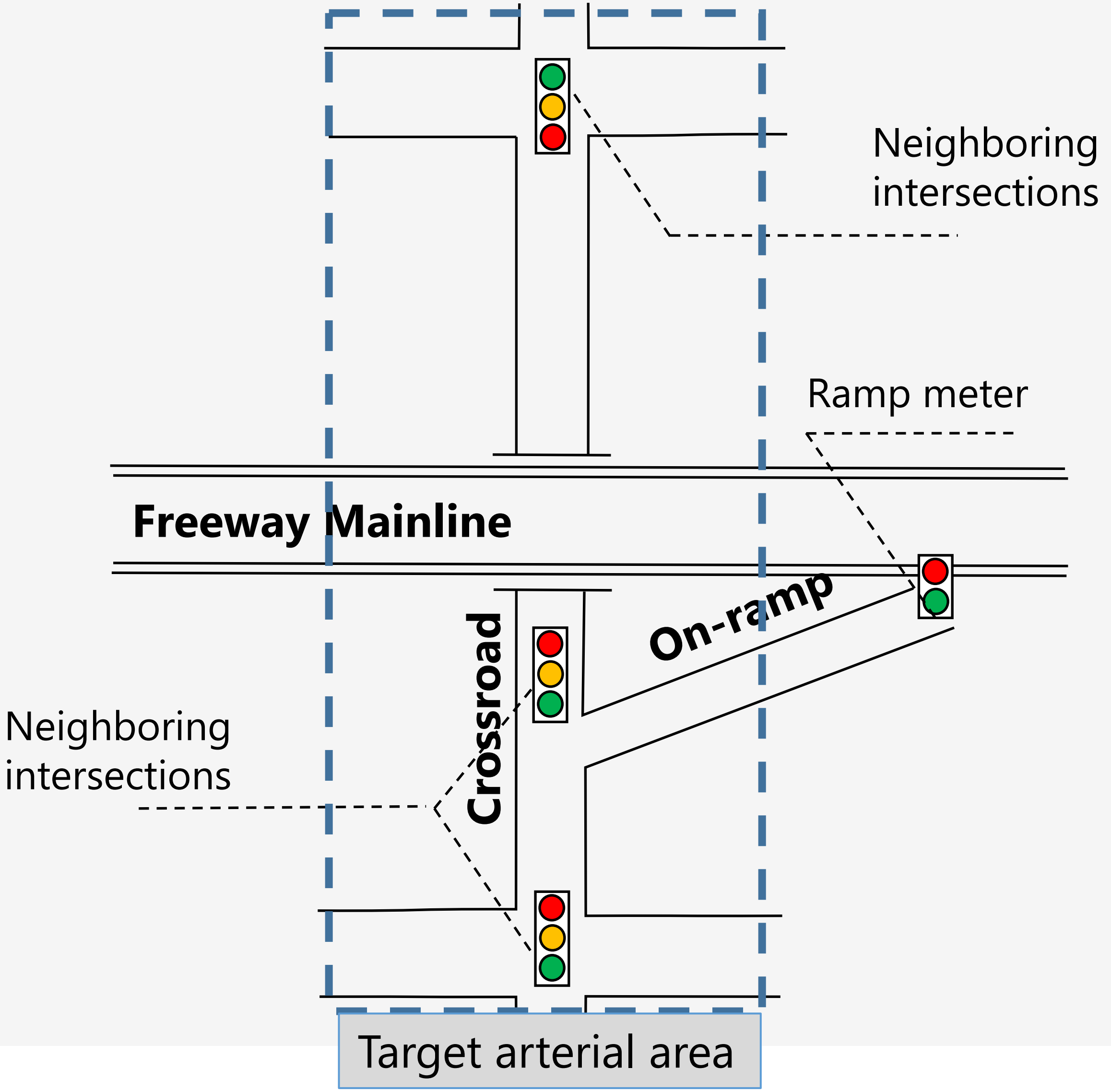
Arterial-friendly Local Ramp Metering Control

Project Progress Report

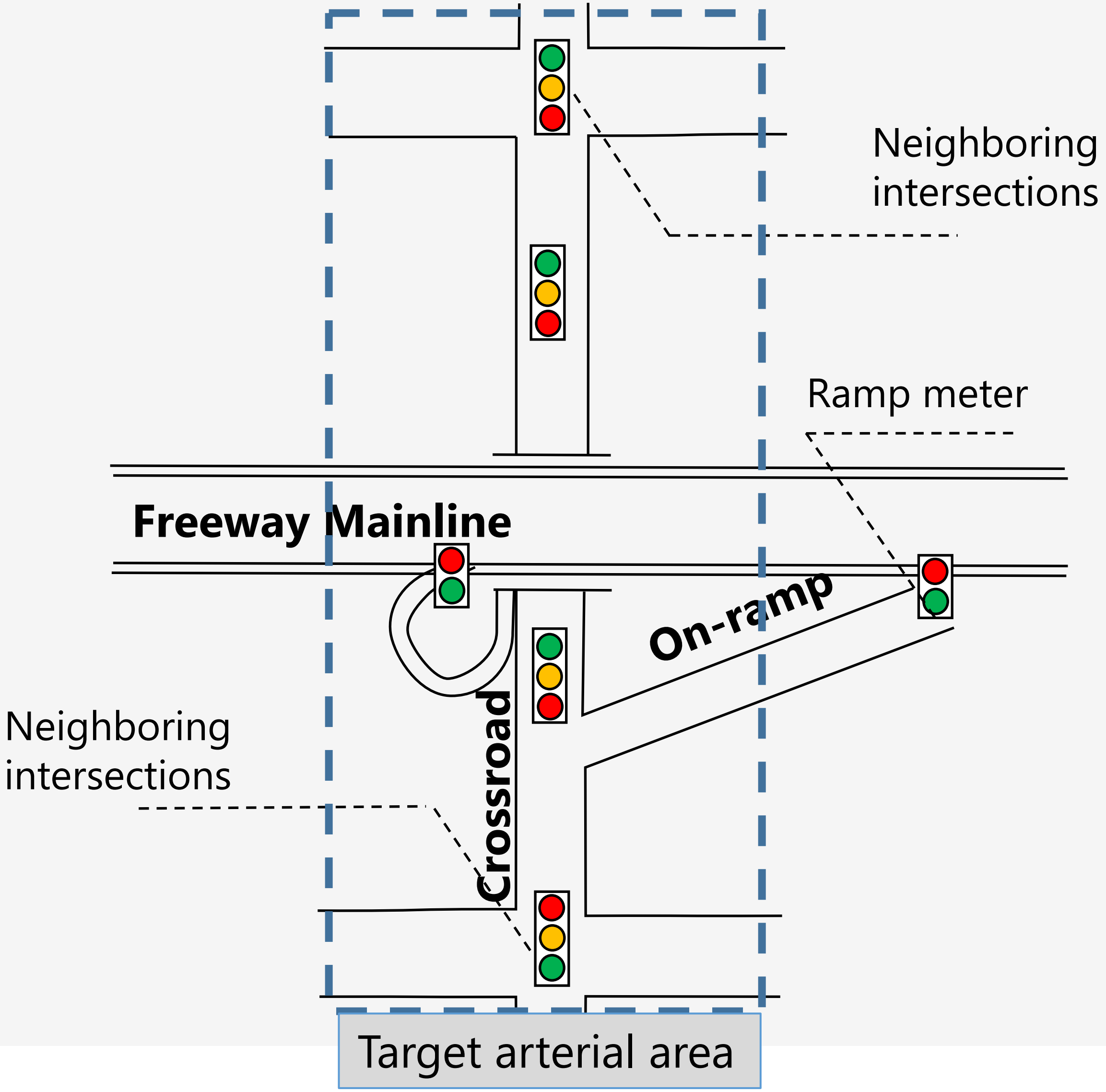
02/26/2020

Type-1 freeway on-ramp

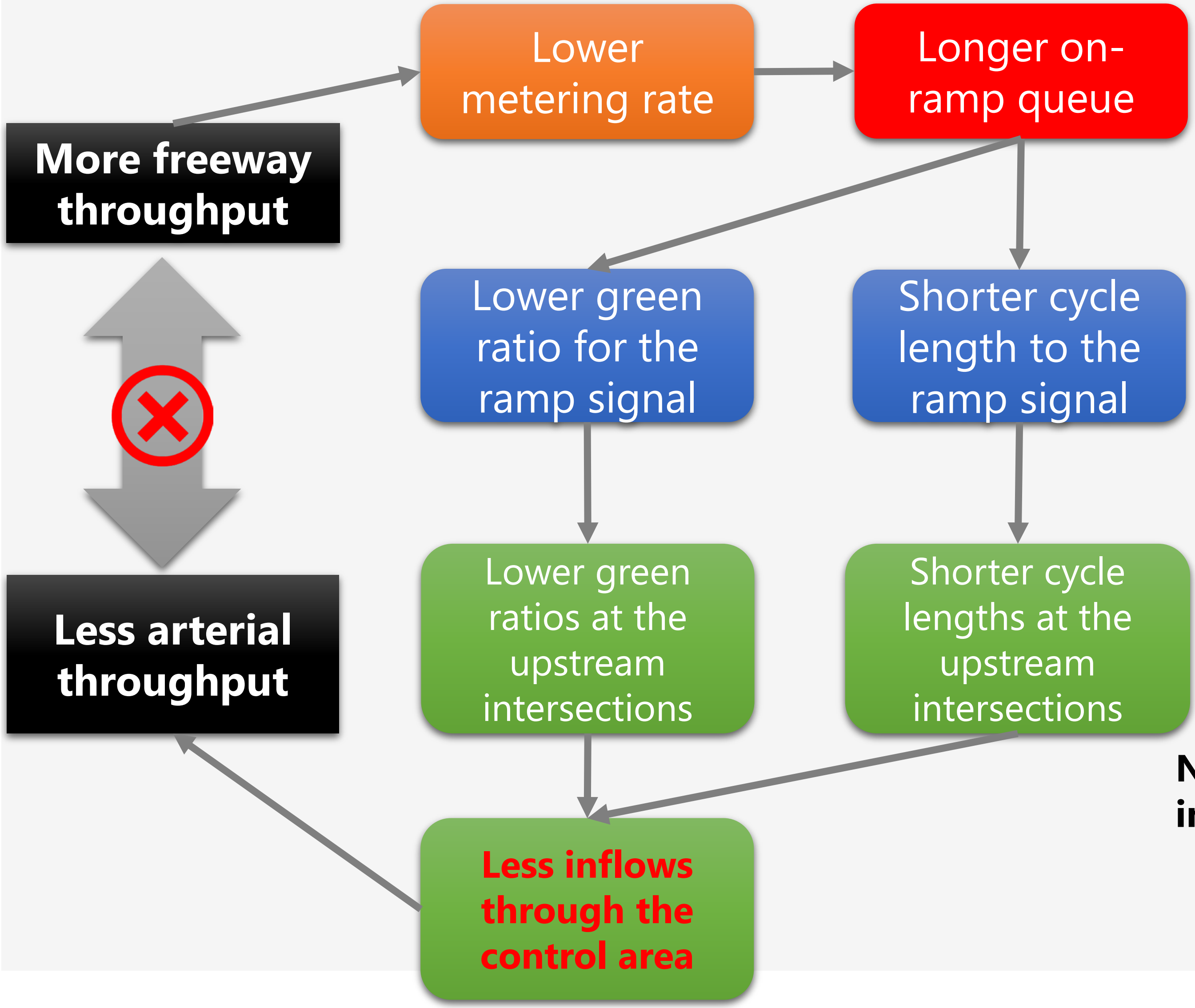


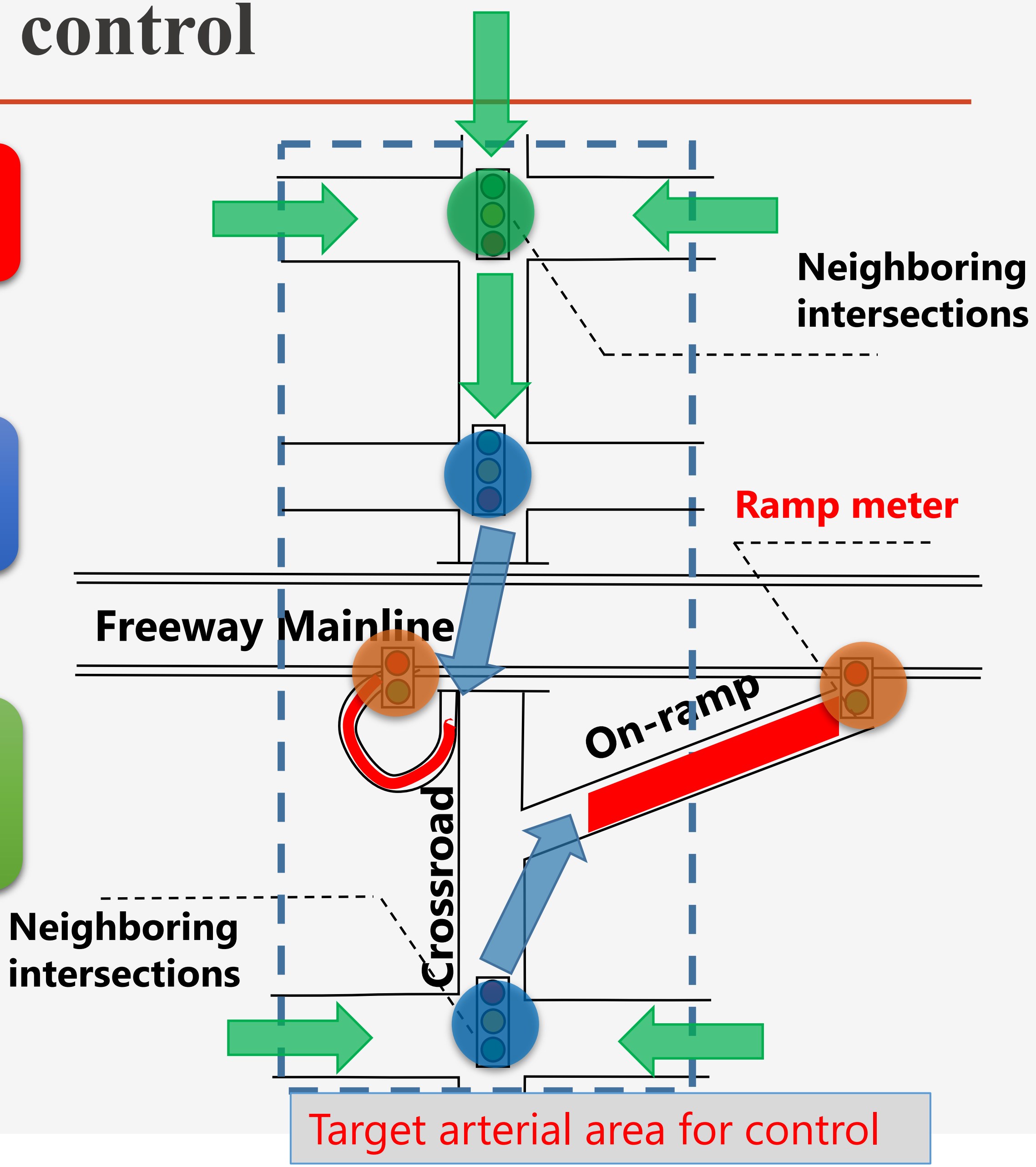


Type-2 freeway on-ramp



Impacts by a local-ramp metering control

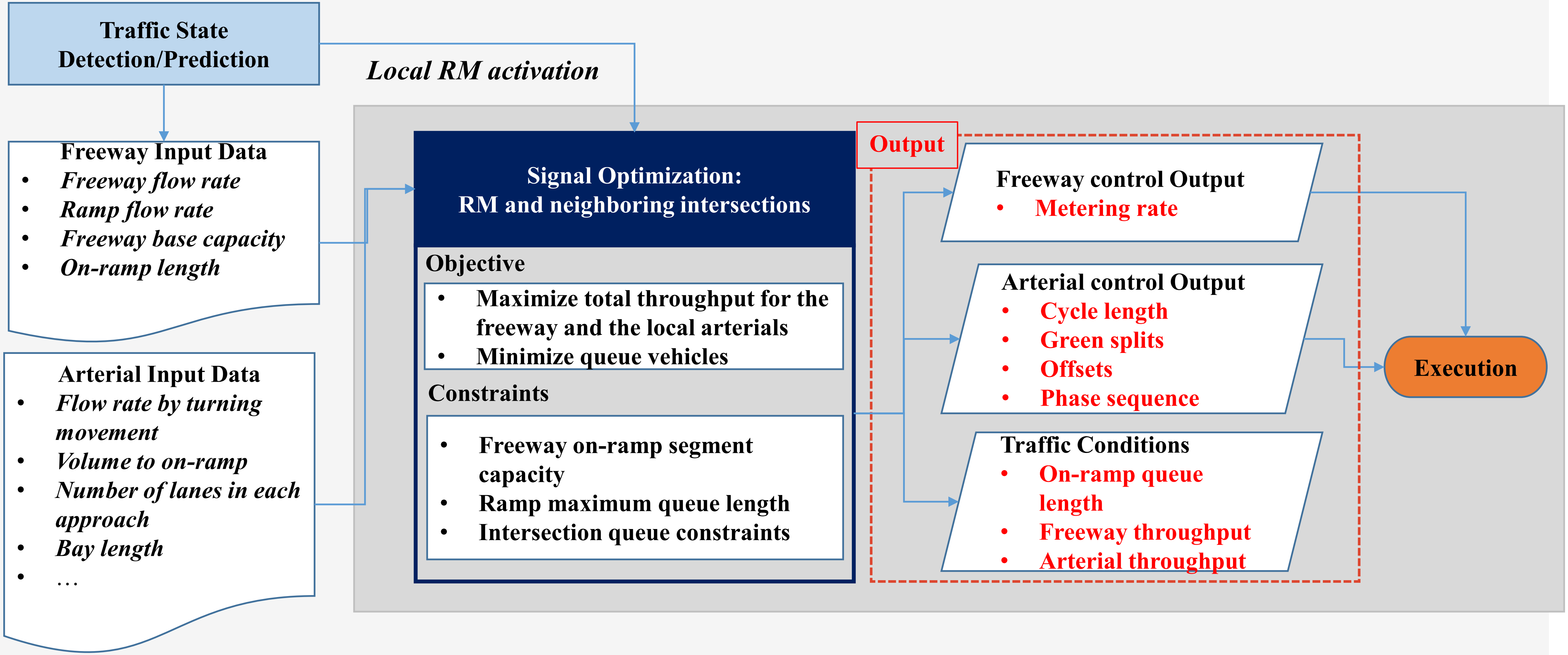




Research issues

- How to balance the benefits between the freeway and its local intersections near the on-ramps?
- How to prevent the on-ramp queue spillback during the peak hours?
- How to avoid queue spillover at the intersection's turning bays?
- How to provide progression for vehicles within the control area, and minimize queues at the boundaries?

System structure



Methodology

Objective

Maximize **Freeway throughput**

+ α x **Arterial throughput**

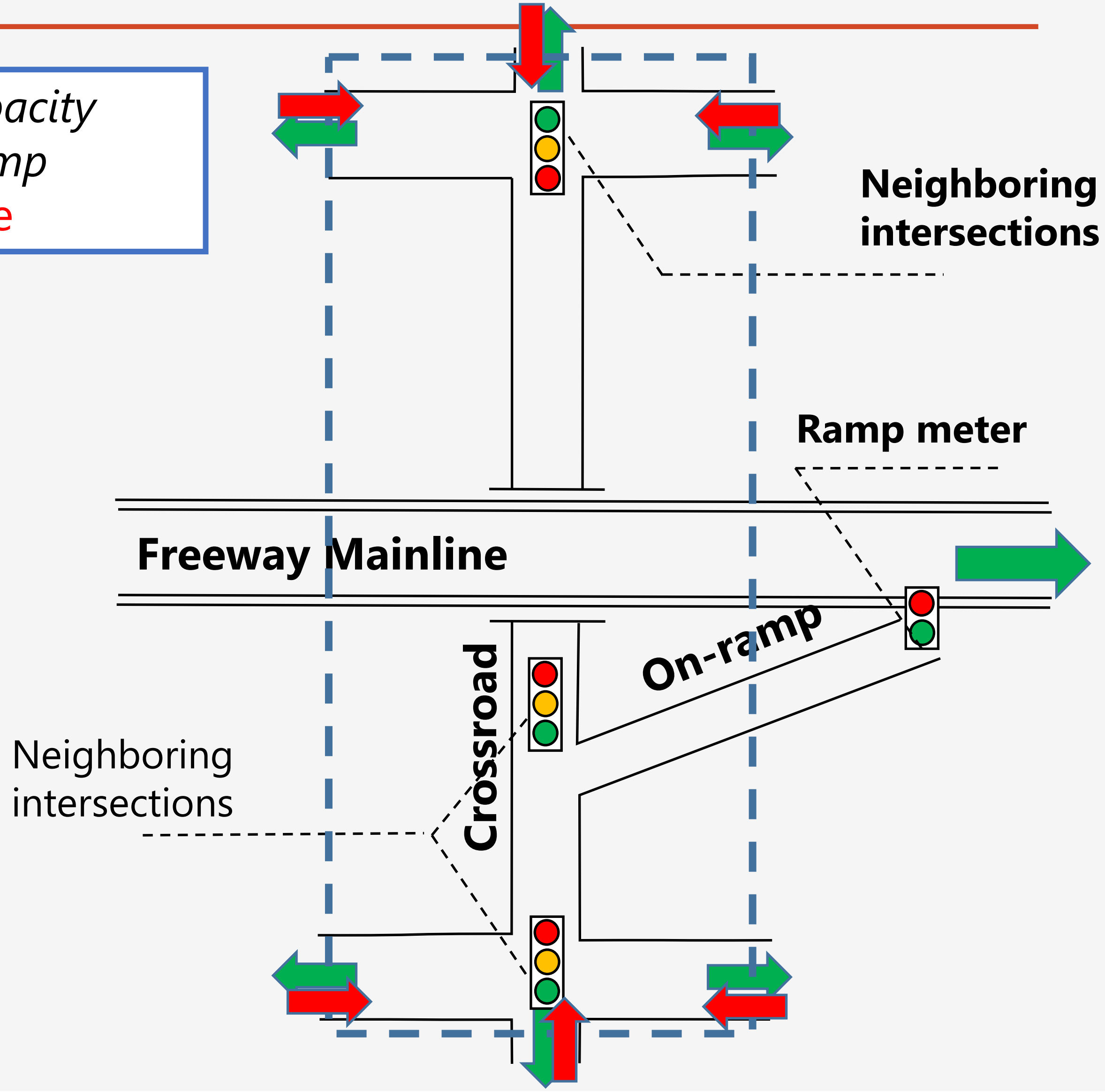
- Β x **total queue vehicles**

- *Available capacity under the ramp metering rate*

Vehicles moving out of the control area (movements by green arrows), based on

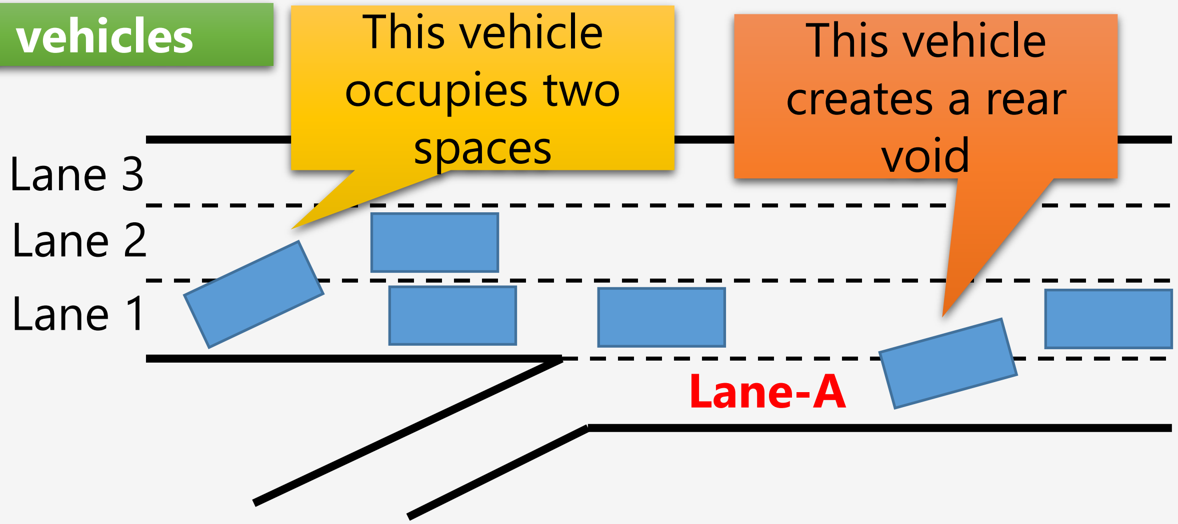
- The demand that can enter the target area
- Green ratios at the boundary intersections

Queuing vehicles not within the control area (see **red arrows**)



Methodology

Freeway capacity reduction due to on-ramp merging vehicles



Rear void

Speed difference between Lane-1 and Lane-A

Deceleration to accommodate merging flows

Create a void due to one lane change

On-ramp vehicles

Capacity reduction due to all created voids

Upstream lane change

Density on lane 2

Density on lane 1

On-ramp merging vehicles

Assumption*: the density ratio between lanes 1 and 2 remains approximately* ***stable*** *before and after the ramp-weavings.*

Number of lane-changing vehicles due to ramp flows

Capacity reduction due to vehicle weavings

Methodology

Constraints

On the on-ramp

Queues caused by excessive volumes

+

Queues caused by vehicles per cycle from the upstream intersections

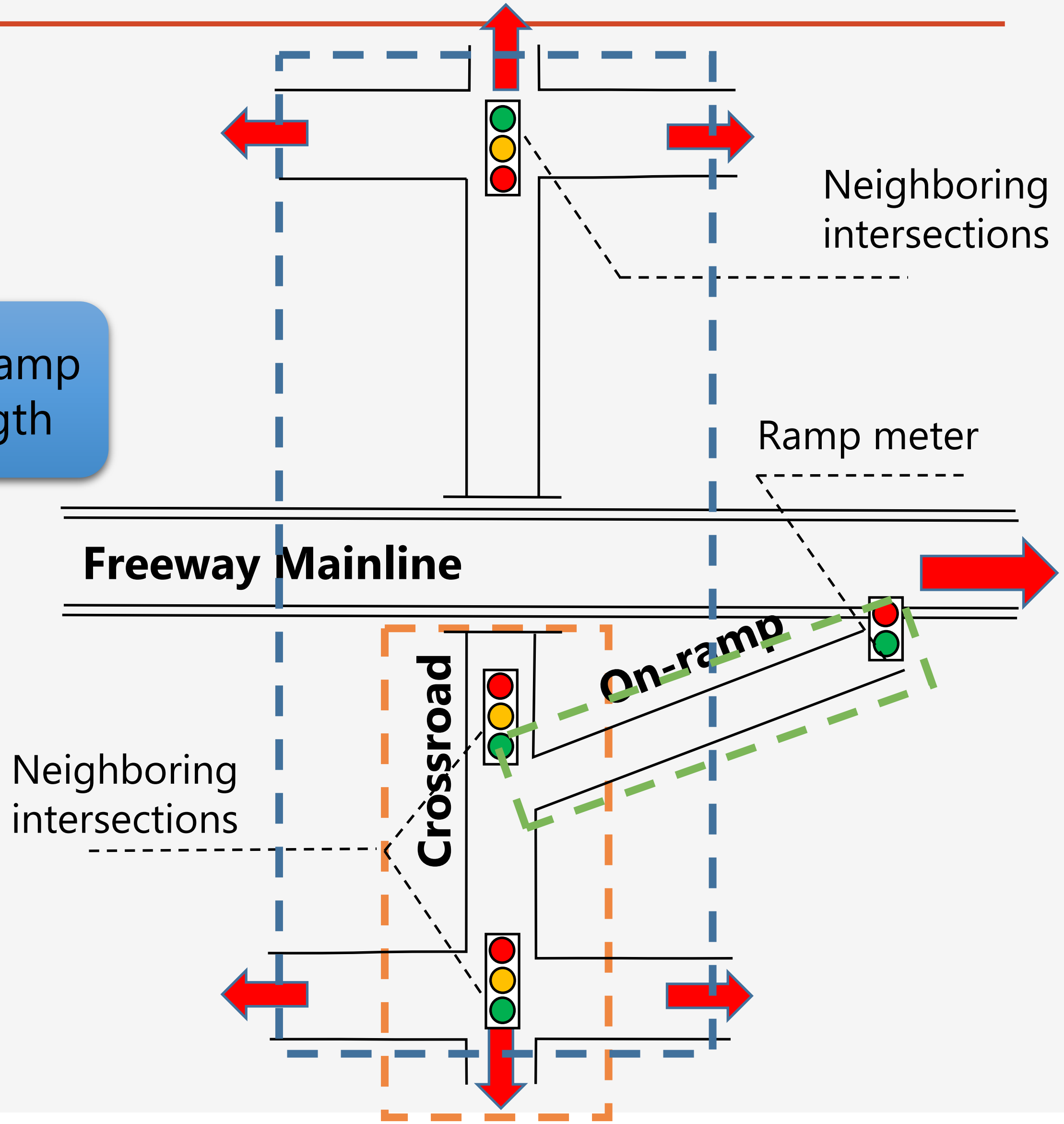
→ On-ramp queue ≤ On-ramp length

On the arterial links

Vehicle queues on an arterial link ≤ Bay length/ link length

Vehicles entering a link ≤ Discharging capacity

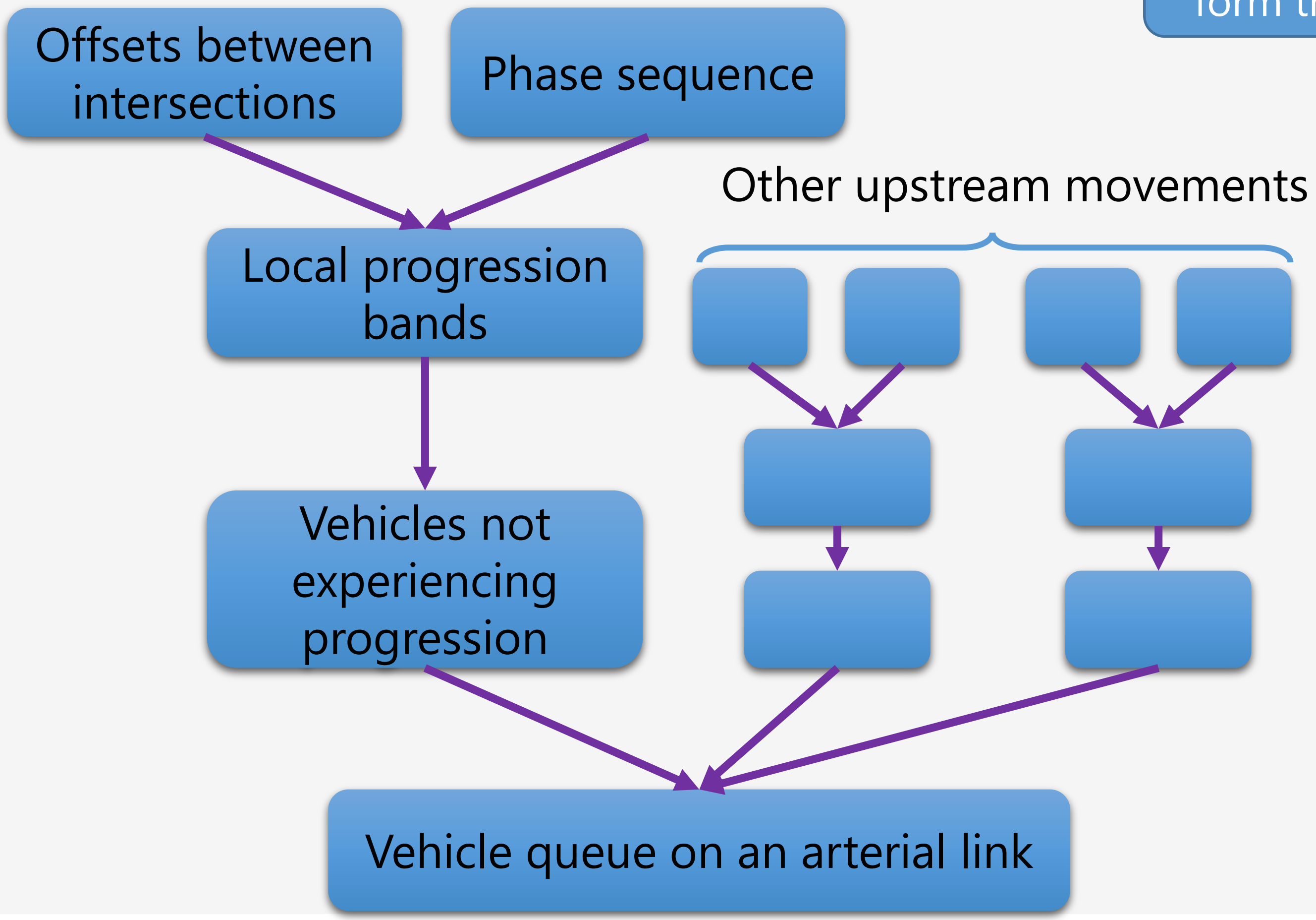
Vehicles not experiencing local progression = Queueing vehicles

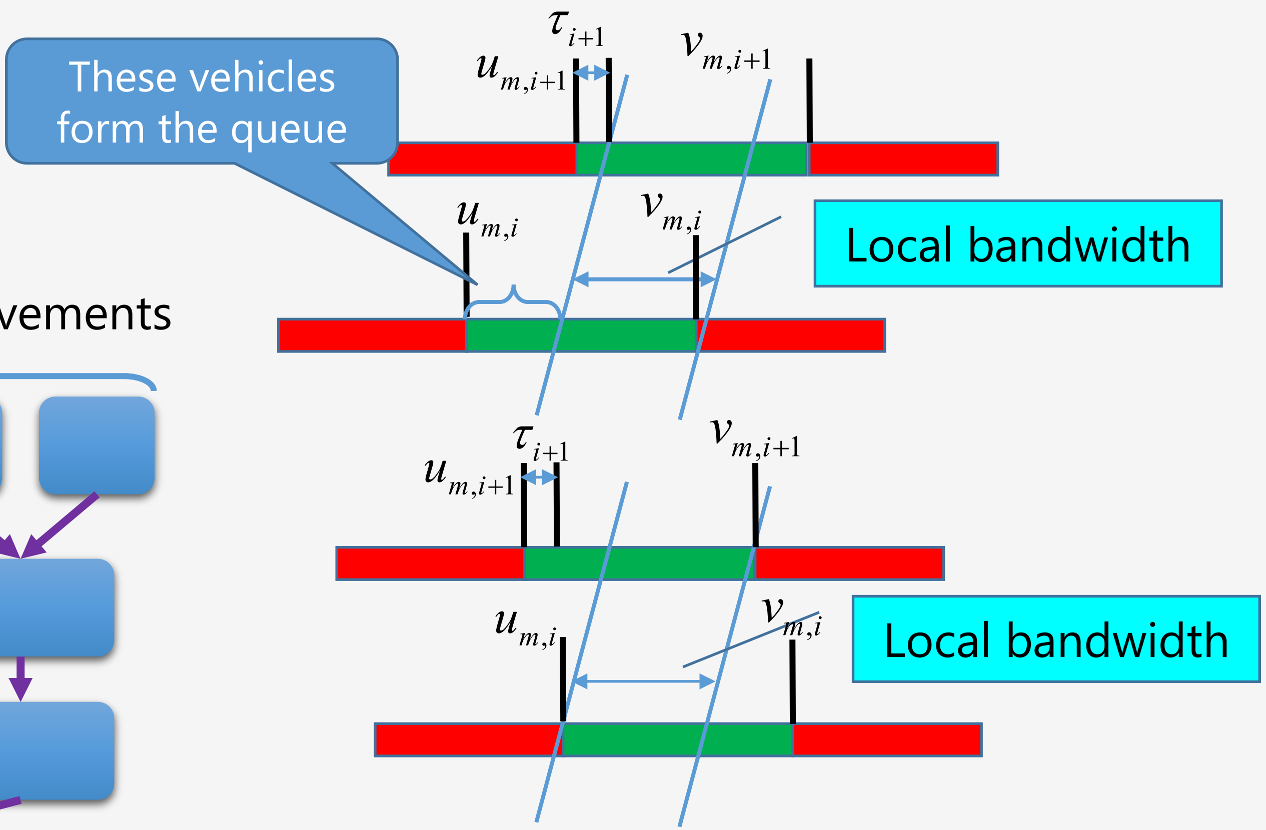


Key control logic –providing progression for vehicles of all movements within the control area

Constraints

Local progression bands





u, v: start, and end of a green phase

For Type-2 ramps

Additional Constraints

Dual on-ramp Control

Freeway Flow rate$_{upstream}$ - (off-ramp flow rate) + total on-ramp flow rate <= Freeway Capacity$_{downstream}$

On-ramp demand = actual volume on link x turning ratio to the ramp

Not directly controlled by signal

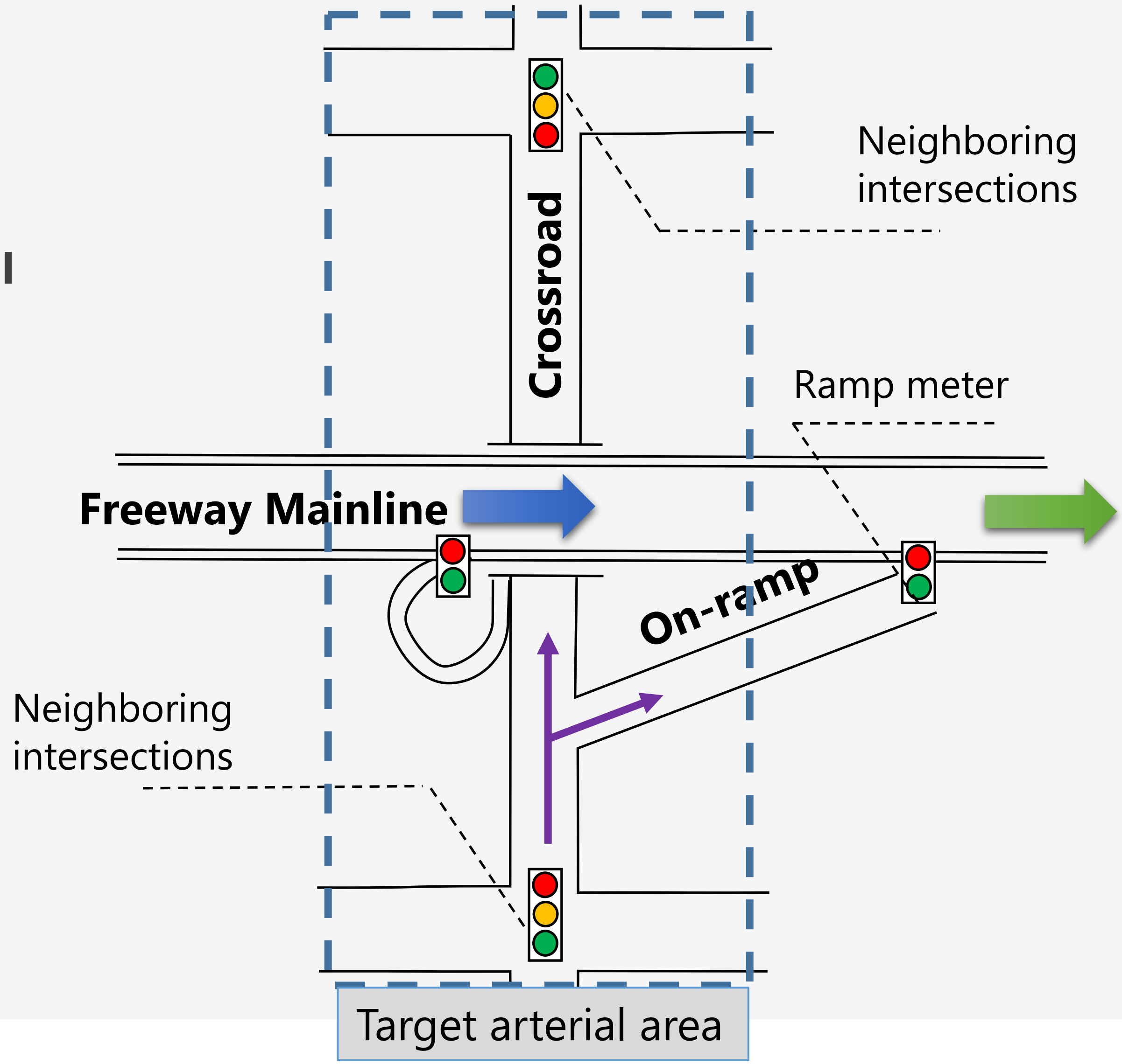
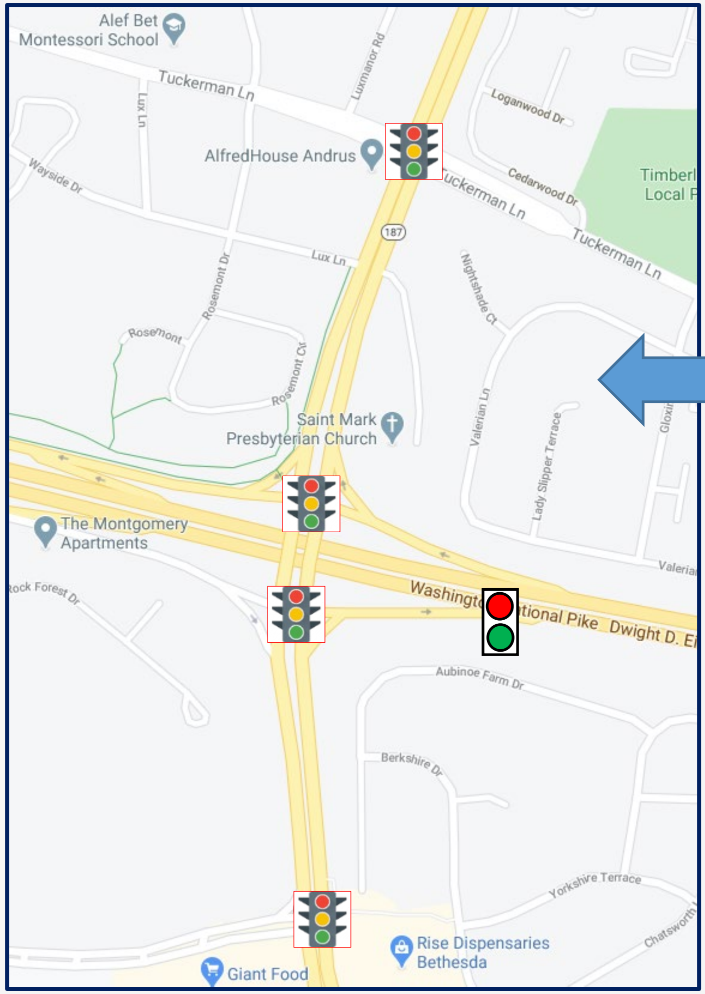


Illustration of model application



Freeway mainline:
- ***2 lanes (excluding HOV lane), 3000 vph***

Metered on-ramp:
- ***792 ft, one lane***
- ***975 ft and 1882 ft, one lan***

Volume input and turning ratios obtained from ITMS.

Arterial signals
- ***90s<cycle<180s***
- ***Lost time: 2 s, Minimum phase duration: 9 s***
- ***Right turn on red***

Study period: 1 hr

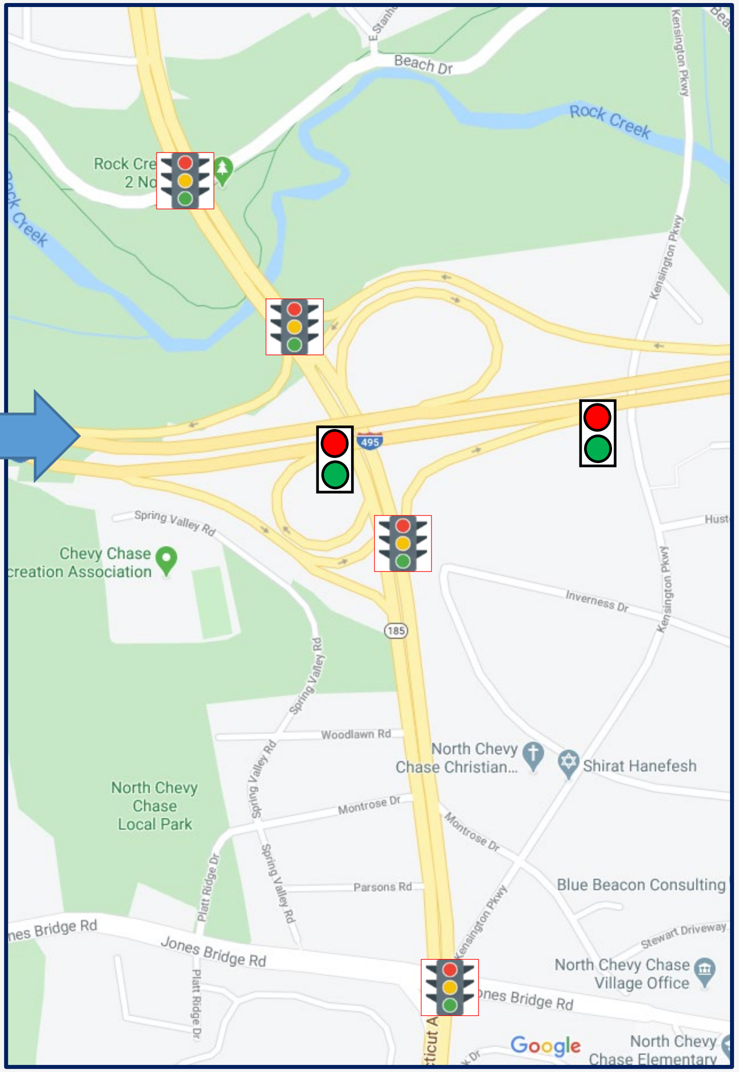
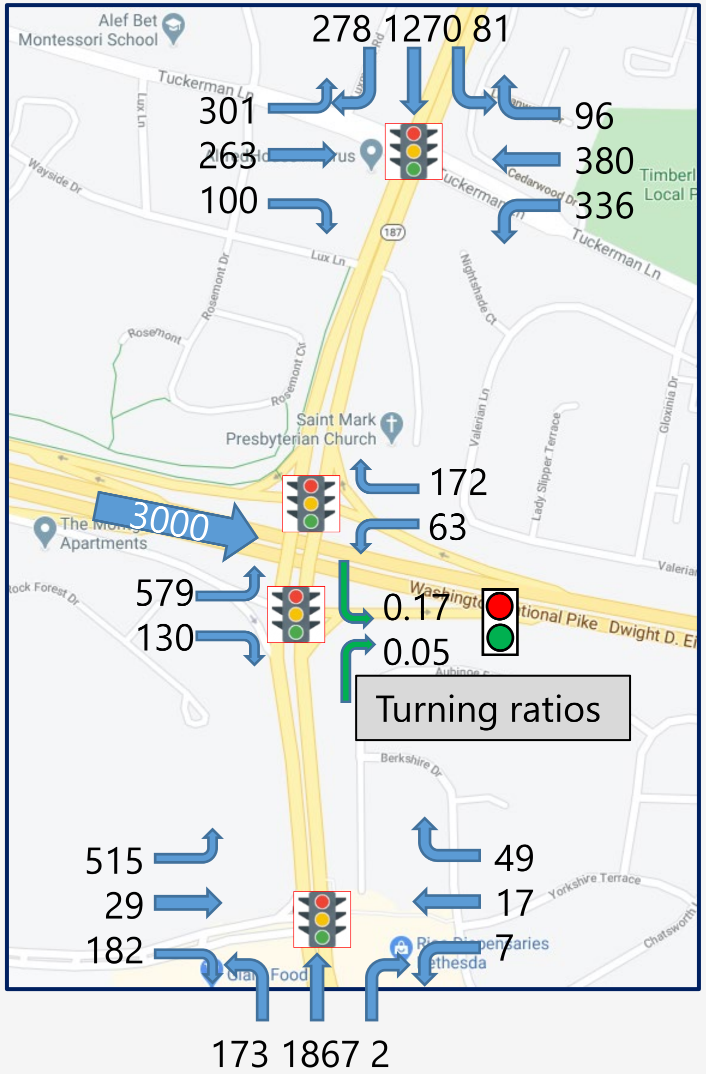
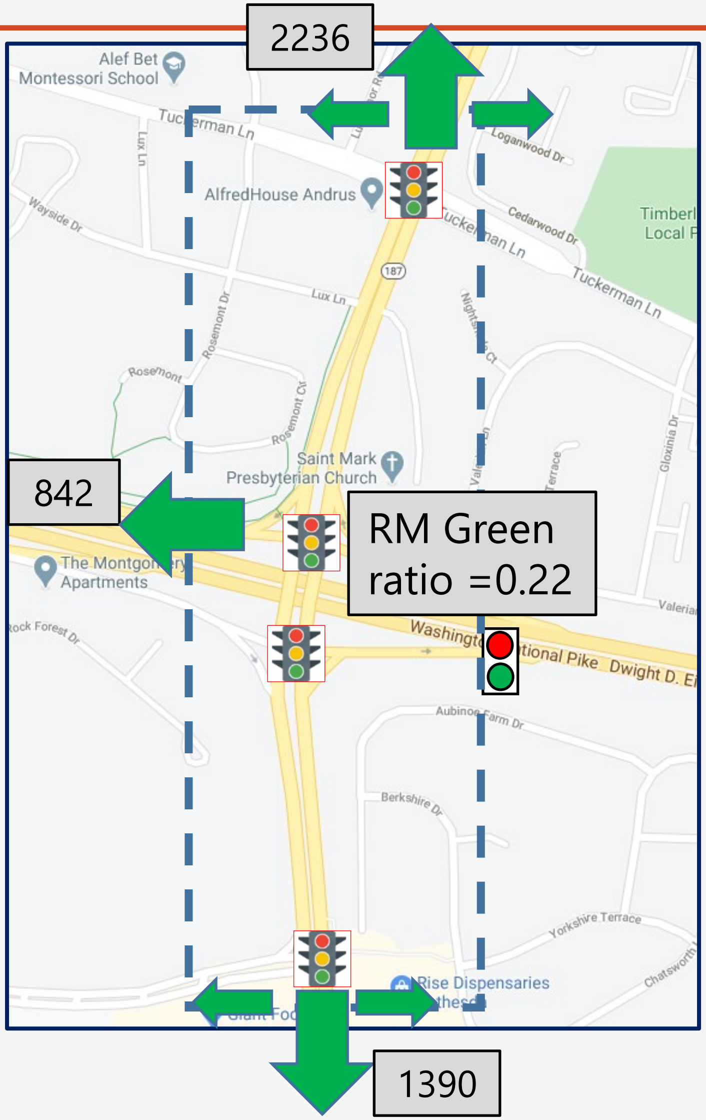


Illustration of model application



Volume input (total=6890 veh)



Optimization results

Cycle length =95s

Arterial throughput = 6537 veh/hr

Freeway throughput = 3222 veh/hr

Type A on-ramp queue = 587 ft(27 veh)

Type B on-ramp queue = 205 ft(9 veh)

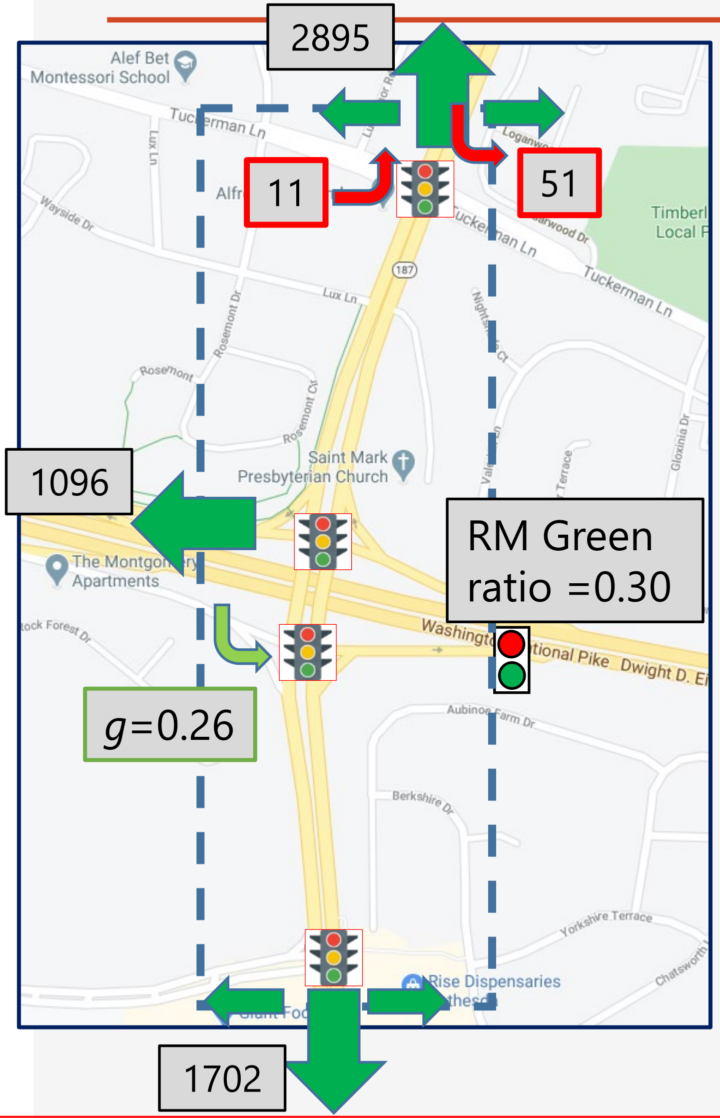
On-ramp inflow = 353 veh

Total restrained vehicle = 0 veh

Suggested RM cycle length = 95 s

Obj: maximize Freeway throughput + Arterial throughput - total queue vehicles

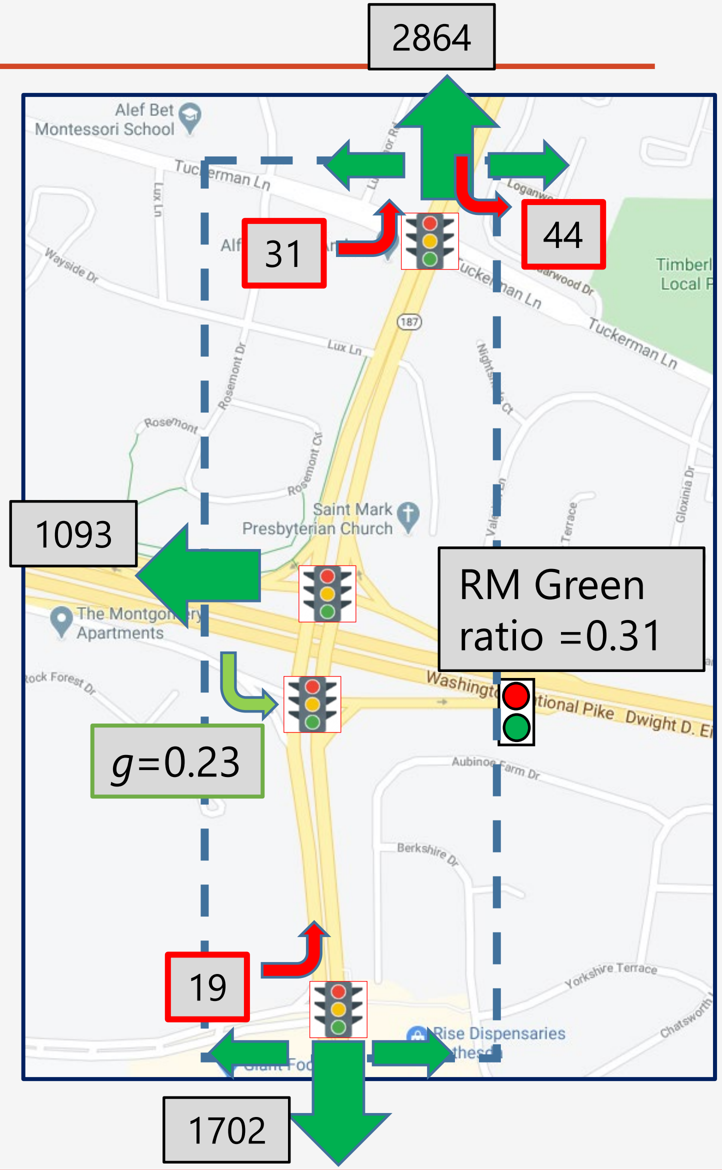
Illustration of model Application



1.3 * volume, ramp length = 972 ft

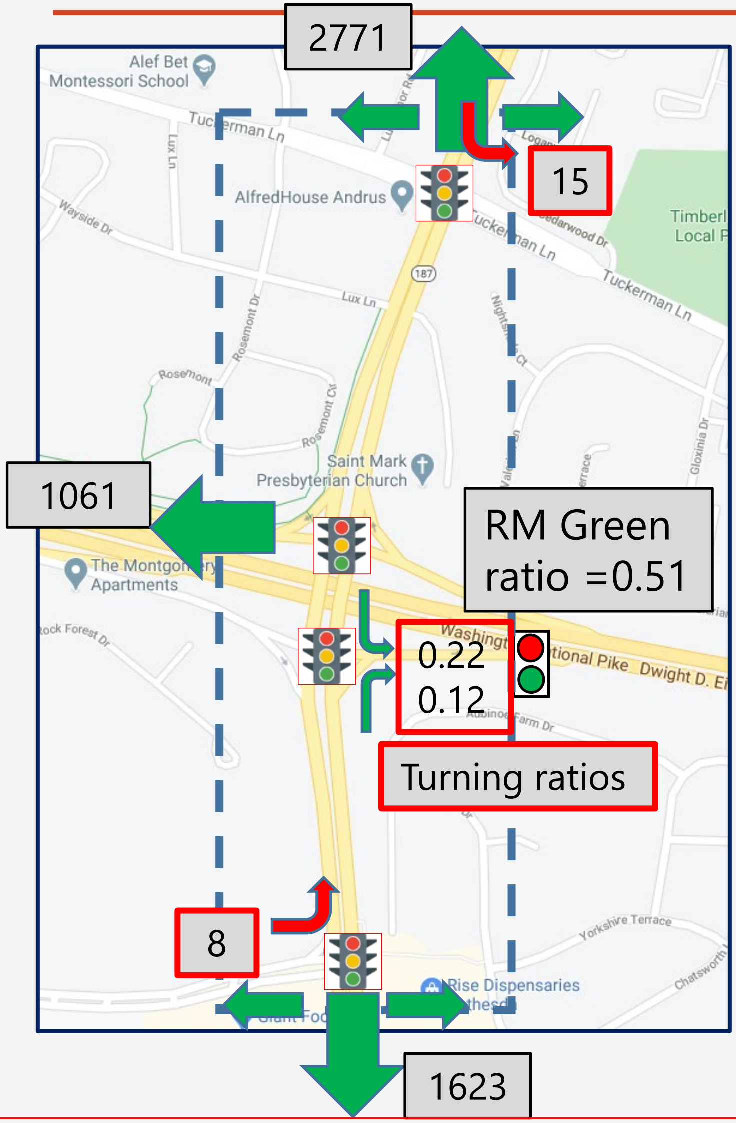
180	Cycle length (s)	160
8436	Arterial throughput (veh/hr)	8405
3083	Freeway throughput (veh/hr)	3069
13	Type A on-ramp queue (veh)	0
23	Type B on-ramp queue (veh)	20
458	On-ramp inflow (veh)	458
62	Total restrained vehicle (veh)	94
90	Suggested RM cycle length (s)	80

- When the on-ramp is shorter, intersection cycle length would decrease, and RM green ratio will increase to make sure the on-ramp queue would not spillback.
- The model would further adjust upstream green ratios to avoid gridlock in the target area, which may increase outside queueing vehicles.



1.3 * volume, ramp length = 450 ft

Illustration of model Application



Cycle length =165s

Arterial throughput = 8167 veh/hr

Freeway throughput = 2712 veh/hr

Type A on-ramp queue = 18 ft(1 veh)

Type B on-ramp queue = 774 ft(35 veh)

On-ramp inflow = 767 veh

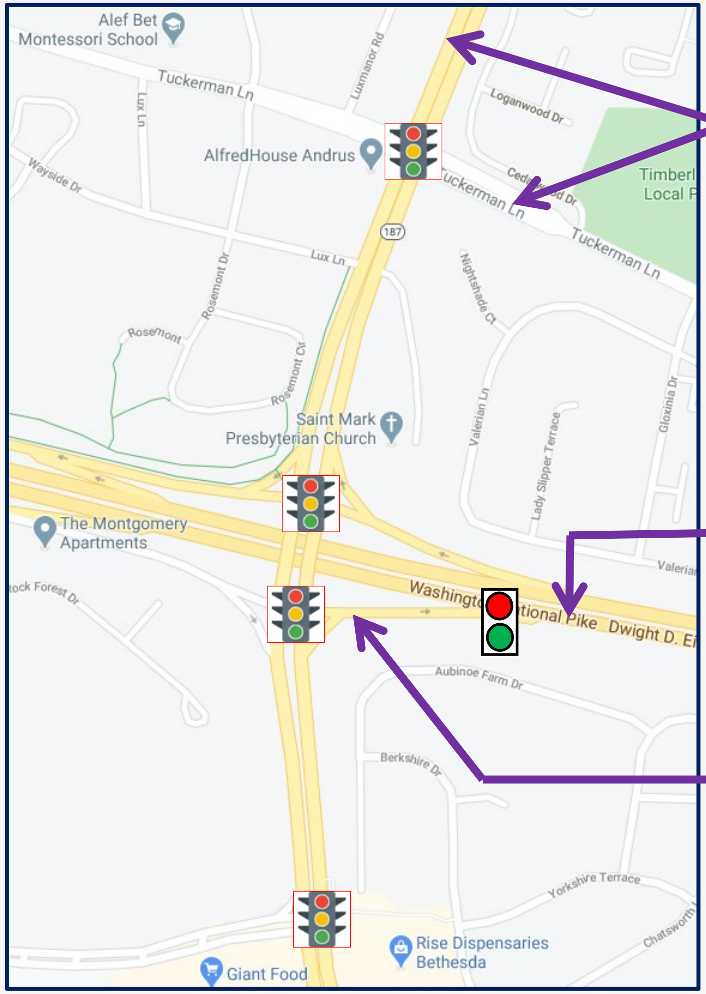
Total restrained vehicle = 23 veh

Suggested RM cycle length = 165 s

- The RM green ratio increases with the turning ratios to the on-ramp.
- Since the on-ramp volume is large, Type-B queue increases, forcing Type-A queue to be shorter.
- The freeway throughput is significantly impacted by the high on-ramp volume.
- Compared to the scenario with lower turning ratios, the intersection cycle length is reduced to decrease the on-ramp queue.

1.3 * volume, ramp length = 792 ft, increased turning ratio

Upgrading the system to real-time operation



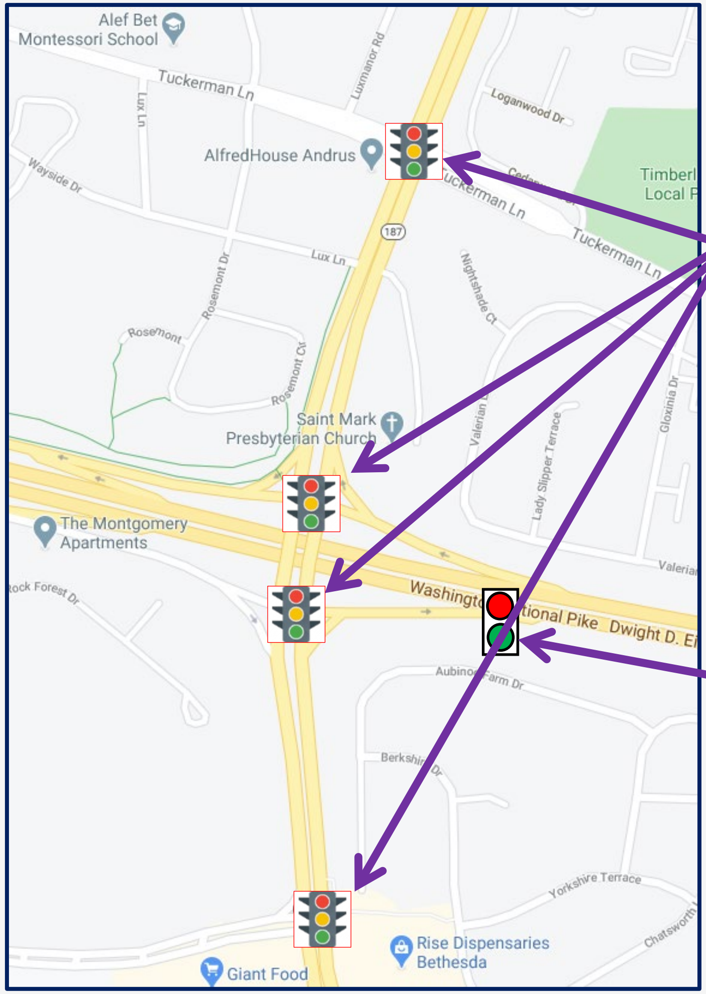
How can real-time detection improve effectiveness of the proposed model?

Volume detection on the arterial
To implement adaptive network signal control, the detected arterial volume, if significantly different from historical data, should be applied to the proposed model to generate a new signal plan.

Volume detection at downstream of the on-ramp
The volume detected at the downstream can be used to calibrate the throughput estimation function in a real-time manner.

Queue detection on the on-ramp
When the detected queue is longer than the estimated Type-A queue, a larger RM green ratio is needed to avoid ramp queue spillback.

Upgrading the system to real-time operation



How can the proposed model benefit real-time operation?

Real-time operation of arterial signals

- The results from the proposed model serves as a base signal plan to minimizes the magnitude of real-time adjustment.
- The developed constraints ensuring that the vehicle queues are shorter than the link length/bay length should still be applied when adjusting signal timing in real-time operation.

Reactive ramp metering control

- Compared to real-time reactive ramp metering strategies, implementing the pre-timed ramp metering before the detection of freeway breakdown can delay or avoid the occurrence breakdown.
- The metering rate of real-time operation, although calculated based on detected data, can be kept close to the optimized value without causing on-ramp queue spillback.
- While reactive ramp metering control strategies may cause fluctuating metering rate due to varying traffic condition, the constant metering rate would minimize the impact of fluctuating on-ramp volume to freeway mainline.

Appendix: Formulations

Objective

$$\max \ T_f + k \times T_a$$

$$T_f = \min\left(V_f + V_o^a, C_w\right)$$

$$k = \frac{\sum_{i \in \Phi} V_i / sN_\phi}{\left(V_f + V_o\right) / C_b N_d}$$

The ratio between arterial and freeway saturation levels

s_o : Saturation flow rate at the on-ramp metering signal (veh/h)

T_f : Freeway throughput (veh/h)

T_a : Local arterial throughput (veh/h)

k : Weighting factor

$V_f\left(V_o\right)$: Freeway mainline(on-ramp) demand (veh/h)

Φ : Boundary intersections

N_d : Number of lanes at the downstream of the on-ramp

V_o^a : Actual flow onto the on-ramp

Appendix: Formulations

Constraints --- on-ramp

These constraints assume stable volume in the study period and can be easily updated to constraints handling piece-wise demand

$$l_p + l_c \le L_o$$

$$l_p = \max\left(V_o^a - s_o r_o, 0\right) \times T$$ On-ramp queue accumulates if inflow is higher than outflow on the on-ramp

$$l_c = V_o^a / 3600\xi$$ Number of vehicles arriving at the on-ramp per cycle

$$V_o^a = \sum_{m \in \Omega} V_m^a$$ Sum of the volume from the movements entering the on-ramp

l_p : Queues caused by excessive demand (veh)

V_o^a : Actual flow onto the on-ramp

l_c : Queues caused by arrivals from upstream intersection in every cycle (veh)

s_o : Saturation flow rate at the on-ramp metering signal (veh/h)

ξ : Reciprocal of cycle length (1/C)

Ω : Set of movements entering the on-ramp

r_o : Metering rate

g : Green ratio

T : Time horizon of study (hr)

s_m : Saturation flow rate for movement m (veh/h)

Appendix: Formulations

Constraints --- local progression

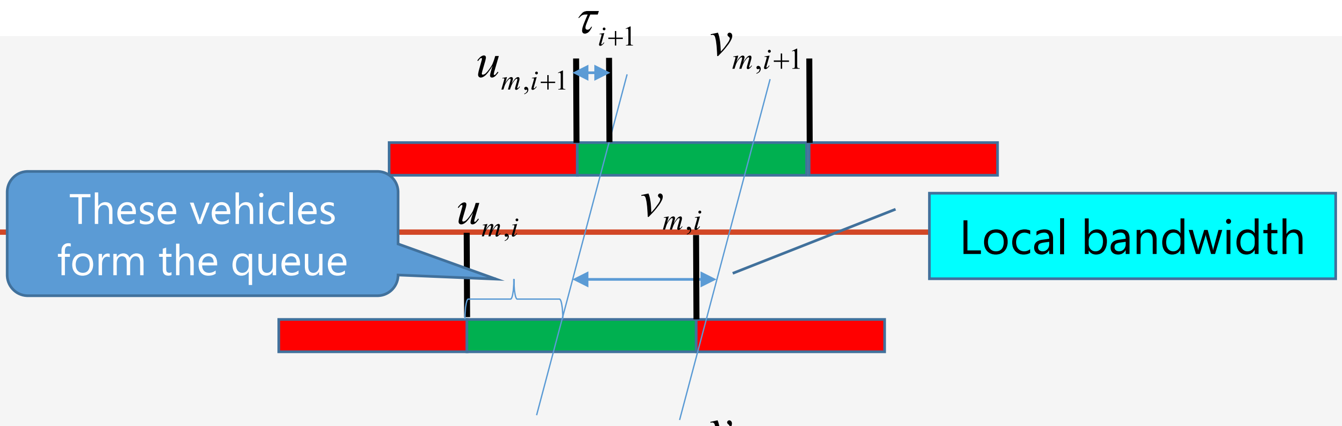
Local bandwidth:

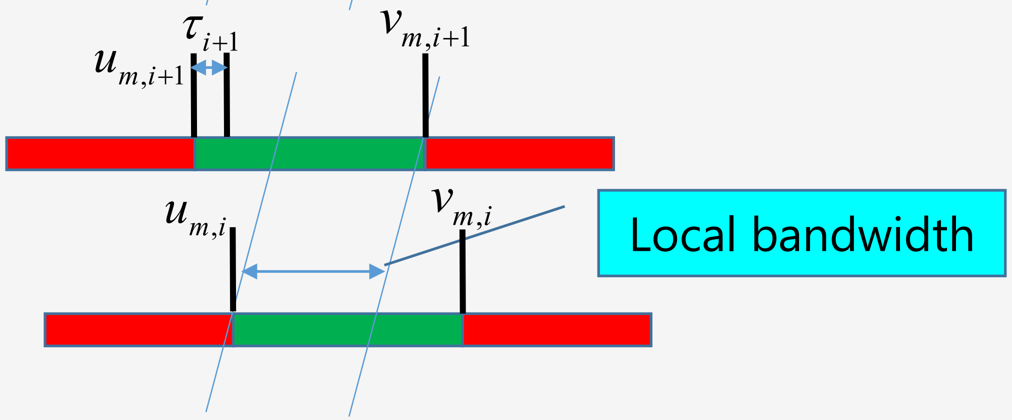
$$b_{m,i} = \min\left(v_{m,i}, v_{m,i+1}\right) - \max\left(u_{m,i}, u_{m,i+1} + \tau_{i+1}\right)$$

Vehicles arriving out of the local band will form the queue:

$$l_{a,i} = \sum_{m} V^{a}_{m',i-1} r_{m,i-1}\left(g_{m,i-1} - b_{m,i-1}\right)\frac{C}{3600}$$

- Applied to all pairs of adjacent intersections





u, v: start, and end of a green phase

Link queue smaller than the bay length/link length:

$$l_{a,i} \le L_{b,i}, l_{a,i} \le L_{l,i}$$

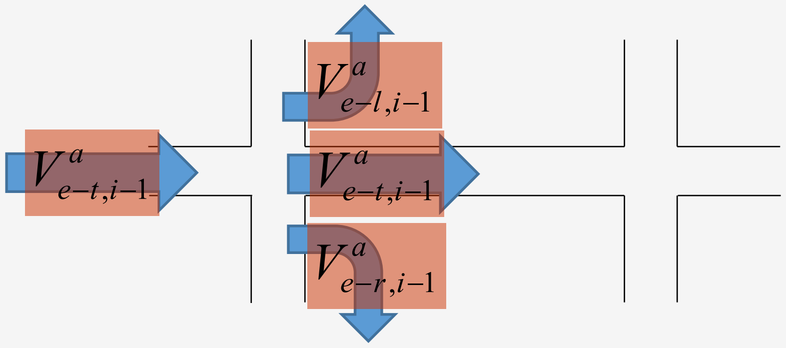
$b_{m,i}$: local bandwidth of movement m
$l_{a,i}$: sum of vehicles not enjoying progression from all movements m
$L_{b,i}$($L_{l,i}$): bay length (link length) (ft)
m(m'): movements entering/leaving an intersection

Appendix: Formulations

Constraints --- arterial link

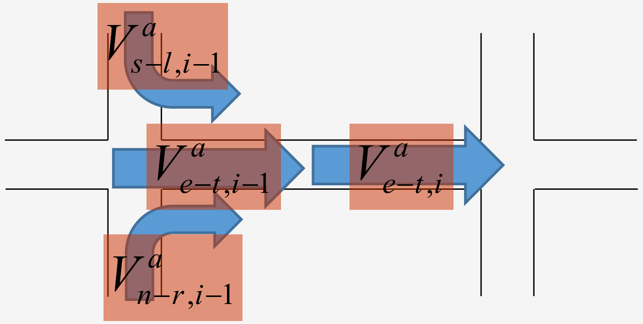
Actual outflow to an intersection is dependent on actual inflow:

$$V^a_{m',i} = r_{m',i} V^a_{m,i-1}$$



Actual inflow to an intersection is dependent on actual outflow from the upstream intersection, and has to be discharged:

$$V^a_{m,i} = r_{m,i} \sum_{m'} V^a_{m',i-1} \qquad V^a_{m,i} \le s_{m,i} g_{m,i}$$



Green time for all the phases and lost time:

$$\sum_p g_{p,i} + t_l \xi = 1$$

Appendix: Formulations

Constraints --- boundary intersections

At the boundary intersections, the actual flow entering the target area may be limited by the green duration

$$V^{a}_{m',i} = \min\left(r_{m',i} V_{m,i}, g_{m',i} s_{m',i}\right) \quad i \in \Phi$$

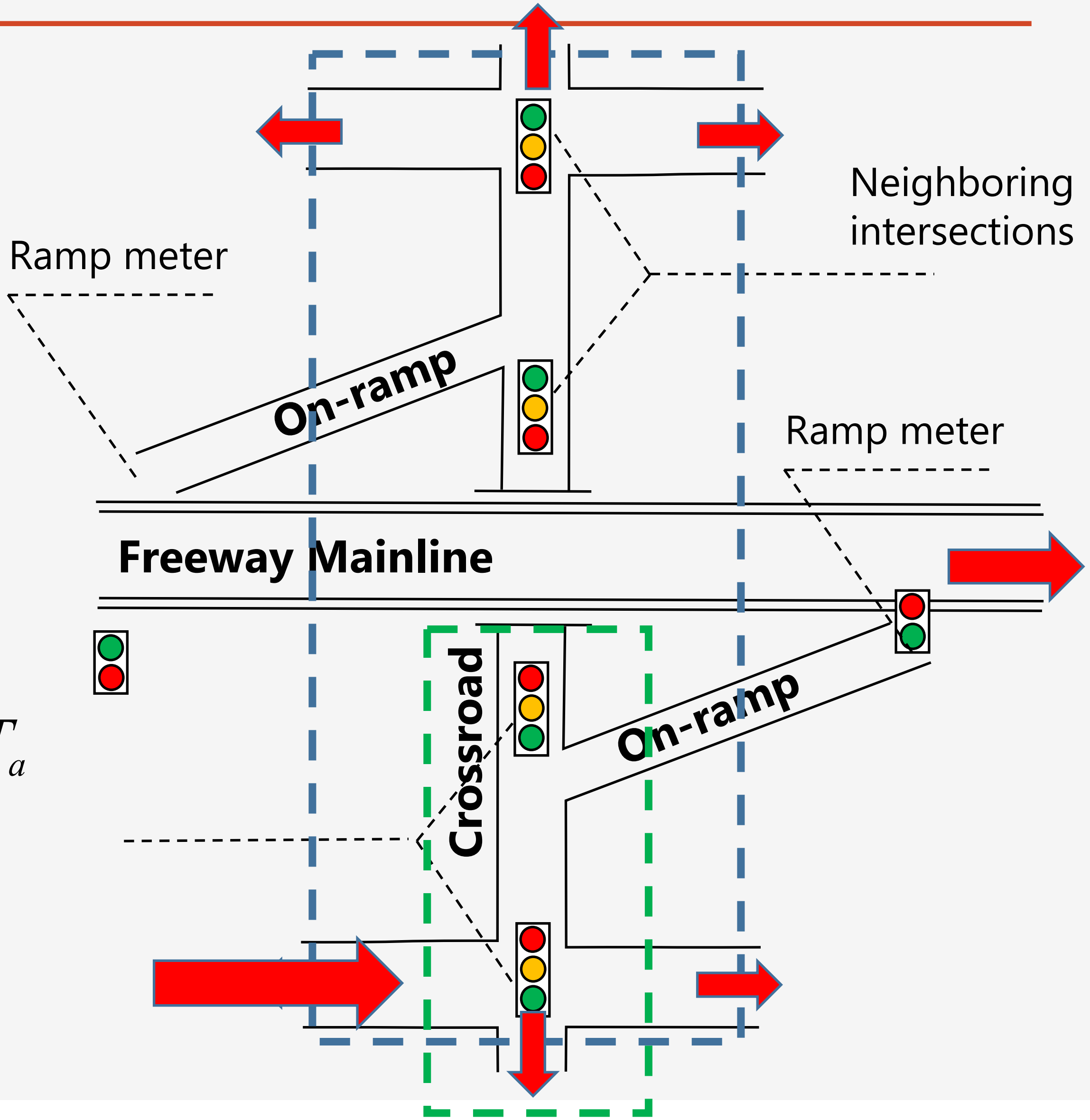
Arterial throughput:

$$\boxed{T_a = \sum_{i \in \Phi} V^{a}_{m',i}}$$

To be used here

$$\max \; T_f + k \times T_a$$

Φ : Boundary intersections

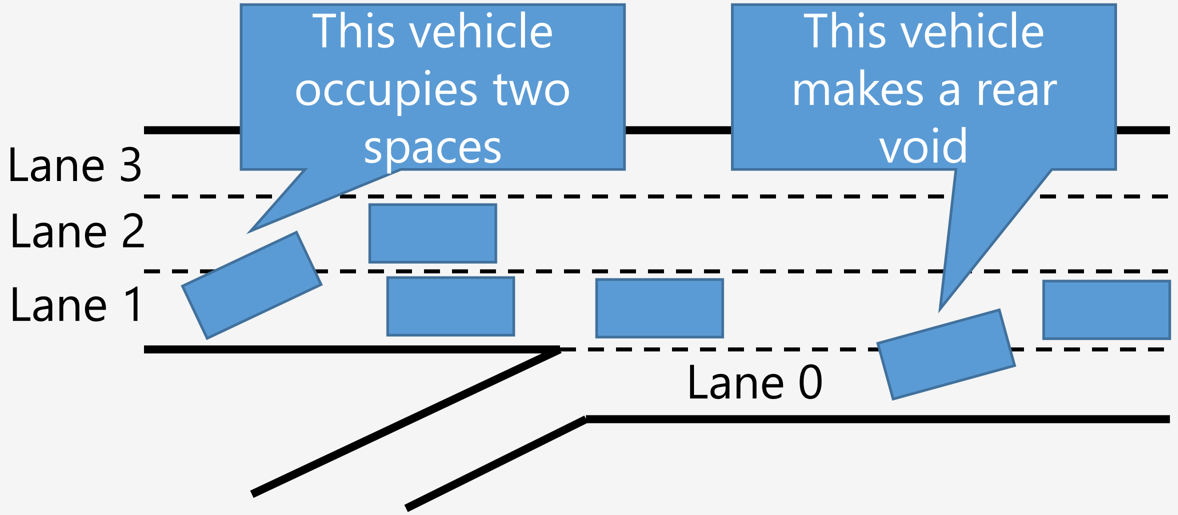


Appendix: Formulation

Assuming mainline vehicles would decelerate to the same speed as the merging vehicle

Rear void when merging is complete: $\frac{v_1^2 - v_0^2}{2a}$

Reduced capacity due to rear void: $\frac{v_1^2 - v_0^2}{2al} S_o r_o$



Assuming the density ratio between Lanes 1 and 2 keeps constant, for each on-ramp vehicle, $\frac{o_2}{o_1 + o_2}$ vehicles would perform pre-allocation lane change. Therefore, the resulted capacity drop is $\frac{o_2}{o_1 + o_2} S_o r_o$

$$C_w = C_b - \gamma S_o r_o \left(\frac{v_1^2 - v_0^2}{2al} + \frac{o_2}{o_1 + o_2} \right)$$

a: Deceleration for cooperative lane change

l: Average vehicle length

v_i: Speed on lane i

o_i: Detected occupancy on lane i

γ: Calibrated parameter

Appendix: Formulation

The formulation in non-linear due to variable cycle length.

- Use discrete value for the cycle length and introduce 0-1 variables to indicate the actual value.
- Discrete possible values for the cycle length (60,65,70,……,180)
- $\xi \rightarrow \sum_i y_i \frac{1}{C_i}$ $\quad\quad \sum_i y_i = 1$

Binary variable

$$\boxed{y_1 \times \frac{1}{60} + y_2 \times \frac{1}{65} + \ldots + y_{25} \times \frac{1}{180}}$$

- With this formulation, each quadratic term includes at least one binary variable and is easy to solve with many solvers.