Type-1 freeway on-ramp

- Freeway Mainline
- Ramp meter
- Target arterial area

Diagram showing a Type-1 freeway on-ramp with surrounding intersections and target arterial areas.
Type-2 freeway on-ramp
Impacts by a local-ramp metering control

More freeway throughput
- Lower metering rate
- Lower green ratio for the ramp signal
- Less arterial throughput
- Less inflows through the control area

Longer on-ramp queue
- Shorter cycle length to the ramp signal
- Shorter cycle lengths at the upstream intersections
- Lower green ratios at the upstream intersections

Neighboring intersections
- Ramp meter
- Freeway Mainline
- On-ramp
- Crossroad
- Target arterial area for 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

Traffic State Detection/Prediction

Freeway Input Data
- Freeway flow rate
- Ramp flow rate
- Freeway base capacity
- On-ramp length

Arterial Input Data
- Flow rate by turning movement
- Volume to on-ramp
- Number of lanes in each approach
- Bay length
- ...

Local RM activation

Signal Optimization:
RM and neighboring intersections

Objective
- Maximize total throughput for the freeway and the local arterials
- Minimize queue vehicles

Constraints
- Freeway on-ramp segment capacity
- Ramp maximum queue length
- Intersection queue constraints

Output

Freeway control Output
- Metering rate

Arterial control Output
- Cycle length
- Green splits
- Offsets
- Phase sequence

Traffic Conditions
- On-ramp queue length
- Freeway throughput
- Arterial throughput

Execution

Constraints
- Local RM activation
  - Maximize total throughput for the freeway and the local arterials
  - Minimize queue vehicles

Output
- Freeway on-ramp segment capacity
- Ramp maximum queue length
- Intersection queue constraints

Arterial Input Data
- Flow rate by turning movement
- Volume to on-ramp
- Number of lanes in each approach
- Bay length
- ...

Freeway Input Data
- Freeway flow rate
- Ramp flow rate
- Freeway base capacity
- On-ramp length
Methodology

**Objective**

Maximize Freeway throughput

\[ + \alpha \times \text{Arterial throughput} \]

\[ - \beta \times \text{total queue vehicles} \]

- 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)

**Diagram**

- Freeway Mainline
- Crossroad
- On-ramp
- Neighboring intersections
- Ramp meter
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
- Number of lane-changing vehicles due to ramp flows
- Capacity reduction due to vehicle weavings

This vehicle occupies two spaces
This vehicle creates a rear void

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

**Constraints**

- **On the on-ramp**
  - Queues caused by excessive volumes
  - Queues caused by vehicles per cycle from the upstream intersections

- **On the arterial links**
  - Vehicle queues on an arterial link
  - Vehicles entering a link
  - Vehicles not experiencing local progression

On-ramp queue ≤ On-ramp length

Bay length/ link length ≤ Discharging capacity

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

### Constraints

**Local progression bands**
- Offsets between intersections
- Phase sequence
- Local progression bands
- Vehicles not experiencing progression
- Vehicle queue on an arterial link
- Other upstream movements
- Local bandwidth

These vehicles form the queue

\[ u_{m,i+1} \leq u_i \leq v_{m,i+1} \]

\[ u_{m,i} \leq v_{m,i} \leq v_{m,i+1} \]

\[ u, v: \text{start, and end of a green phase} \]
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 lane

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)

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

Optimization results

Turning ratios

RM Green ratio = 0.22
Illustration of model Application

- **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.

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

*When* 1.3 *volume*, ramp length = 972 ft

*When* 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 minimize 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**

\[
\max \ T_f + k \times T_a \\
T_f = \min \left( V_f + V_o^a, C_w \right) \\
k = \frac{\sum_{i \in \Phi} V_i / sN_\phi}{(V_f + V_o) / C_b N_d}
\]

\(S_o\) : 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

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

\(V_o^a\) : Actual flow onto the on-ramp

\(\Phi\) : Boundary intersections

\(N_d\) : Number of lanes at the downstream of the on-ramp
Appendix: Formulations

Constraints --- on-ramp

\[ l_p + l_c \leq L_o \]

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

\[ l_c = \frac{V_o^a}{3600 \xi} \]
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

\[ l_p : \text{Queues caused by excessive demand (veh)} \]
\[ V_o^a : \text{Actual flow onto the on-ramp} \]

\[ l_c : \text{Queues caused by arrivals from upstream intersection in every cycle (veh)} \]

\[ s_o : \text{Saturation flow rate at the on-ramp metering signal (veh/h)} \]

\[ r_o : \text{Metering rate} \]

\[ T : \text{Time horizon of study (hr)} \]

\[ \xi : \text{Reciprocal of cycle length (1/C)} \]

\[ \Omega : \text{Set of movements entering the on-ramp} \]

\[ g : \text{Green ratio} \]

\[ s_m : \text{Saturation flow rate for movement } m \text{ (veh/h)} \]
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:

\[ l_{a,i} = \sum_{m} V_{m',i-1} r_{m,i-1} \left( g_{m,i-1} - b_{m,i-1} \right) \frac{C}{3600} \]

- Applied to all pairs of adjacent intersections

Link queue smaller than the bay length/link length:

\[ l_{a,i} \leq L_{b,i}, l_{a,i} \leq L_{l,i} \]

- \( b_{m,i} \): local bandwidth of movement \( m \)
- \( l_{a,i} \): sum of vehicles not enjoying progression from all movements \( m \)
- \( L_{b,i} \): 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:

\[ V_{m',i}^a = r_{m',i} V_{m,i-1}^a \]

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

\[ V_{m,i}^a = r_{m,i} \sum_{m'} V_{m',i-1}^a \]

\[ V_{m,i}^a \leq s_{m,i} g_{m,i} \]

Green time for all the phases and lost time:

\[ \sum_p g_{p,i} + t_i \xi = 1 \]
Appendix: Formulations

Constraints --- boundary intersections

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

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

Arterial throughput:

\[ T_a = \sum_{i \in \Phi} V_{m',i}^a \]

To be used here

\[ \max \ T_f + k \times T_a \]

\[ \Phi : \text{Boundary intersections} \]
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 \( i \)
- \( \gamma \): Calibrated parameter
Appendix: Formulation

The formulation is 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)

\[ \xi \rightarrow \sum_i y_i \frac{1}{C_i} \quad \sum_i y_i = 1 \]

Binary variable

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

\[ y_1 \times \frac{1}{60} + y_2 \times \frac{1}{65} + ... + y_{25} \times \frac{1}{180} \]