PROPOSED FRAMEWORK FOR EVALUATING SPILLBACK IN THE HCM

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ABSTRACT

The procedures detailed in the Highway Capacity Manual (HCM 2010) estimate capacity and several operational measures dictating level of service for freeway facilities and surface streets. However, these methods do not consider cases in which spillback occurs from one facility type to another. The queueing effects in oversaturated conditions as they propagate upstream – onto a freeway mainline or a surface street intersection – are not accounted for.

The objective of this paper is to propose a series of modifications to enhance the HCM 2010 methodologies in order to address spillback conditions. These modifications consider lane utilization and lane blockage under spillback conditions, and consist of restructuring existing equations and reference tables as well as development of new procedures. A four-regime methodology is proposed for evaluating spillback effects from urban streets to diverge and weaving segments. Also, a methodology is proposed to account for the spillback effects from freeway on-ramps, by reducing the effective green time as a proportion of the percent of time that the queue is expected to block the upstream signalized intersection. The framework developed here uses assumptions that should be further explored through an extensive, nationwide data collection effort.

Keywords: Spillback, Congested systems capacity analysis
INTRODUCTION
The procedures detailed in the Highway Capacity Manual (HCM 2010) estimate capacity and several operational measures dictating level of service for freeway facilities as well as surface streets (1). However, the existing methods do not consider cases in which spillback occurs from one facility type to another. Currently, the Signalized Intersections procedure (HCM 2010 Chapters 18 and 31) predicts the average expected queue length at an approach given any combination of geometric- or traffic-related inputs within the scope of the methodology. Similarly, the Freeway Facilities procedure (HCM 2010 Chapters 10 and 25) estimates the maximum expected queue length at an on-ramp in the case of oversaturated conditions on the freeway mainline. However, the effects of these queues as they propagate upstream are not examined.

The objective of this paper is to propose a series of modifications to existing HCM 2010 procedures provided in Chapter 14 (Freeway Merge and Diverge Segments), Chapter 13 (Freeway Weaving Segments), and Chapter 18 (Signalized Intersections) in order to address spillback conditions. These modifications consider lane utilization and lane blockage, and consist of restructuring existing equations and reference tables as well as development of new procedures. The framework developed here uses assumptions that should be further explored through an extensive, nationwide data collection effort.

This paper references the HCM 2010, as the upcoming HCM 6th edition has not been published as of the submission of this paper.

Section 2 provides an overview of the literature on spillback analysis. Section 3 describes the procedures to estimate the maximum expected queue length from surface street intersections, and its effects on diverge segments and weaving segments. Section 4 describes the procedures to estimate the maximum queue length from oversaturated freeway on-ramps, as well as the effects of this queue on signalized intersections. Conclusions and recommendations are provided in Section 5.

LITERATURE REVIEW
A review of the literature regarding operational effects of freeway-arterial interactions yielded a limited number of publications. Although much of the current research does not directly address performance measures in a manner consistent with HCM 2010 procedures, their concepts and framework are useful in developing the necessary procedural adjustments.

Spillback Effects from Surface Streets to Freeways
Shockwave analysis, which considers the effects of queueing upstream of a bottleneck on a freeway facility, has been used to evaluate operational impacts. Lighthill et al. (2) proposed a method to examine freeway operations in time and space for general types of bottlenecks. Several studies (3-5) have described methodologies for predicting the delay, cumulative queues and capacity deficiency originated from bottlenecks or downstream off-ramps to freeway mainline. They suggested a “2-pipe regime”, where vehicles are classified as either being free to choose any lane on the freeway mainline, or being restricted to the lane adjacent to the off-ramp. The restrictive “2-pipe” assumption assumes vehicles are not inhibited from switching between contiguous “pipes”. Driver behavior may render this assumption invalid.

Daganzo and Munoz (6) confirmed with field data that the discharge flows along the mainline are affected by the presence of an off-ramp queue, and observed an average discharge rate of 4,520 vph on a three-lane freeway. A similar study found that discharge rates along the mainline increase with decreasing off-ramp flows and decreasing queue lengths (7). For speed
reductions, it was found that rightmost lanes are more affected, whereas leftmost lane(s) are not
affected much (8). Queue presence at the off-ramp could reduce the mainline vehicles’ speed to
15.5 mph (7). Exiting drivers were found to sometimes obstruct through lanes by attempting to
force their way into the queue (7). However, no research has established any quantitative
measures for the probability of such blockage and its potential association with the off-ramp
queue length.

In conclusion, estimating operational measures in the case of spillback from an off-ramp
analytically is challenging, as it is very difficult to anticipate the wide variety of driver actions.
Trends and observations from previous research can be used to develop the methodology
framework for spillback effects from urban streets to freeways.

**Spillback Effects from Freeways to Surface Streets**

Limited research has been reported to address freeway spillback onto signalized intersections.
The HCM 2010 Merge/Diverge Segments methodology determines whether volume exceeds
capacity at any critical points along the segment, and estimates the maximum expected queue
along each on-ramp. However, the method does not consider the effects the resultant queue may
have on the upstream surface street. The HCM 2010 Interchange Ramp Terminals procedure
includes an adjustment to consider spillback from the downstream intersection to the upstream in
the form of additional lost time. This lost time is estimated for each upstream movement as a
function of the downstream queue length and storage availability. A similar logic can be applied
in the case of spillback from the on-ramp to a local signalized intersection, as it results in
additional lost time for some/all the signal phases that serve traffic movements destined for the
on-ramp.

Tian et al (9) analyzed the effects of ramp metering spillback onto a diamond interchange
using the simulator DRIVE. Capacity reduction and delay increase were found upstream from
the ramp meters due to discharging flow reductions resulting from queue spillback and
intersection blockage. The authors estimated the delay incurred by the affected movements with
a theoretical plot of demand over time.

For freeways without ramp metering, the queue discharge rate depends on freeway merge
operations. While arrival rates at the back of the on-ramp queue are an input to HCM 2010
procedures, departure rates into the mainline during congested conditions are currently not
available, and no guidance was found in the literature to provide such estimates. This is a critical
aspect of evaluating spillback conditions at a merge ramp, as the discharge rate of the on-ramp
traffic onto the freeway is a key parameter to calculate the queue length along the ramp over
time.

In conclusion, there are tools available to estimate the lost time incurred at the upstream
intersections as a function of a downstream queue length and storage availability. Still, extensive
field observations are necessary to document discharge rates at congested on-ramps and based on
these estimate the resultant queue length along the on-ramp.

**SPILLBACK FROM SURFACE STREETS TO FREEWAYS**

Spillback may occur due to either inadequate capacity of the ramp proper, or the ramp terminal.
The capacity of the ramp proper is defined as the off-ramp’s maximum allowable hourly flow
rate based on its geometric characteristics. The capacity of the ramp terminal is defined as the
capacity of the signalized or unsignalized approach to the surface street. Four steps are proposed
for estimating queue length and determining spillback occurrence.
• **Step 1 – Capacity Checks**
Determine whether capacity is exceeded at any of the critical points along the diverge section:

a) Demand at the study diverge ramp is compared against the capacity of the ramp proper using HCM 2010 Exhibit 13-10.

b) Demand at the study diverge ramp is compared against the estimated capacity of the downstream intersection approach. Depending on the intersection type, the respective capacities can be obtained from HCM 2010 Chapters 18 to 21.

• **Step 2 – Queue Length Estimation**
Estimate the expected queue length for any conditions where demand exceeds capacity. Three scenarios may occur.

*Scenario A:* Queue forms as a result of demand exceeding capacity at the ramp proper. In this case, it is assumed that the available ramp storage is completely occupied (queue storage ratio ≥ 1) and the queue \( Q \) will be located along the mainline as vehicles attempt to enter the ramp. The queue is estimated as:

\[
Q = (v_R - c_{rp}) * f_{HV} * PHF * f_p * t
\]  

Where \( v_R \) is off-ramp demand for the (pc/h), \( c_{rp} \) is capacity of the ramp proper (pc/h), \( f_{HV} \) is adjustment factor for heavy vehicle presence, \( PHF \) is peak hour factor, \( f_p \) is adjustment factor for driver population, \( t \) is analysis time period (h).

*Scenario B:* Queue forms as a result of demand exceeding capacity at the downstream signalized ramp terminal. The HCM 2010 Chapter 31 describes a method for determining the maximum queue length based on geometric and operational characteristics at the approach.

*Scenario C:* Queue forms as a result of demand exceeding capacity at the downstream unsignalized ramp terminal. The 95th percentile queue length at Two-Way-Stop-Controlled intersections, All-Way-Stop-Controlled intersections and roundabouts are estimated using HCM 2010 Equations 19-68, 20-33, and 21-20, respectively.

• **Step 3 – Queue Storage Ratios and Spillback Checks**
Estimate the queue storage ratio \( R_Q \) for the ramp proper queues as follows (for scenario C, the HCM 2010 calculates 95th percentile queue storage ratio with 95th percentile queue length):

\[
R_Q = \frac{L_h Q}{L_a N}
\]  

Where \( L_a \) is available queue storage distance (ft/ln), and \( L_h \) is average vehicle spacing in stationary queue (ft/veh).

If \( R_Q \) exceeds 1, spillback is expected to occur.

• **Step 4 (Optional, if \( R_Q \geq 1 \)) – Queue Length Upstream of the Ramp Proper/Downstream Ramp Terminal**
Determine the queue length upstream of the ramp proper or downstream ramp terminal \( Q_{SP} \) if queue storage ratio exceeds 1.

*Scenario A:* \( Q_{SP} \) is estimated based on the additional demand not served by the off-ramp’s available capacity:
\[ Q_{SP} = (v_R - c) \times f_{HV} \times PHF \times f_p \times L_h \]  

Where \( v_R \) is demand for the off-ramp (pc/h), \( c \) is capacity of the off-ramp (pc/h), \( L_h \) is average vehicle spacing in stationary queue (ft/veh).

Scenario B and C: \( Q_{SP} \) is estimated as:

\[ Q_{SP} = (R_Q - 1)L_a \]

Spillback Effects on Isolated Diverge Junctions
This section provides procedures for evaluating traffic operations at a diverge segment when there is spillback from the off-ramp. First, four queue regimes are introduced to classify spillback conditions. Then, methodologies to estimate several operational effects are presented, followed by a numerical example.

Queue Regimes
When spillback occurs, one of four possible queue regimes affect the influence area (i.e., the physical area affected by queueing). These are distinguished as a function of the estimated queue length \( Q_{SP} \) compared to the ramp deceleration lane length \( L_D \) and the available queue storage distance upstream of the diverge \( L_E \), as illustrated in Figure 1 and discussed below.

Regime 1 \((Q_{SP} \leq L_D)\): The queue ends within the deceleration lane and does not spill back into the mainline freeway (Figure 1a). When spillback occurs, the available deceleration distance is reduced compared with undersaturated conditions. Minimal turbulence is expected along the freeway mainline, and it is likely that only the deceleration lane is affected.

Regime 2 \((L_E \geq Q_{SP} > L_D)\): The queue of vehicles extends upstream beyond the deceleration lane, but sufficient lateral clearance on the right-hand shoulder of the mainline allows for additional queue storage distance (Figure 1b). In this case, it is expected that exiting drivers will decelerate and join the back of the queue, potentially causing turbulence in Lane 1 (the rightmost through lane). Although the spillback influence area is identical to that of Regime 1, the average travel speed and lane-changing behavior of non-exiting vehicles in Lane 1 can be expected to differ from that of Regime 1.

Regime 3 \((Q_{SP} > L_E, \text{ Lane 1 blocked})\): The queue occupies all available storage distance along the shoulder of the mainline and subsequently blocks Lane 1 (Figure 1c). This may occur either when there is no shoulder available for additional queue storage, or when drivers choose Lane 1 rather than use the shoulder once the deceleration lane is entirely occupied. Non-exiting vehicles using Lane 1 are likely to be forced to change lanes to avoid delay, causing increased lane-changing and turbulence in Lanes 1 and 2. Average speed in Lane 1 is expected to be significantly lower.

Regime 4 \((Q_{SP} > L_E, \text{ Lanes 1 and 2 blocked})\): The queue occupies all available storage distance along the shoulder of the mainline and blocks Lane 1. Drivers may use Lane 2 and force their way into the queue, blocking an additional lane (Figure 1d). This condition likely occurs as a result of excessive volumes of exiting vehicles and/or insufficient queue storage distance upstream of the diverge area. Under these conditions, additional lane-changing activity and
turbulence are expected in Lanes 2 and 3, and thus the average travel speed and capacity of the freeway segment are expected to be significantly reduced. Figure 2 provides a sketch of the potential relationship between queue distance on Lane 1 and blockage probability of Lane 2. It is assumed that a queue length shorter than 100 ft in Lane 1 does not affect Lane 2 blockage probability, while queue length longer than 500 ft is assumed to result in 100% blockage probability for Lane 2. Further empirical research is needed to determine which factors cause the transition between Regime 3 and 4 blockage to occur (queue length, prevailing driver behavior, etc.) and to solidify the relationship illustrated in Figure 2.

**FIGURE 1** Regimes at a diverge segment.

- a) Regime 1, b) Regime 2, c) Regime 3, d) Regime 4.

**FIGURE 2** Queue distance on Lane 1 and blockage probability of Lane 2.

**Operational Effects**
Due to the potential differences in operations across the mainline lanes, operational measures are estimated on a lane-by-lane basis, considering: lane(s) that are blocked (spillback lanes); lane(s) within the influence area; and lane(s) outside the influence area. Average travel speed and capacity estimation based on the expected queue regimes are discussed below.

**Average Speeds in Diverging Segments**
It is assumed that spillback that does not involve full
blockage of mainline lanes (Regimes 1 and 2) will affect the average travel speed in the adjacent (influence) lane. Speeds in the remaining lanes are assumed not to be affected, and thus the speed estimation equations provided in the HCM 2010 Equation 13-12 are still valid. In cases involving lane blockage (Regimes 3 and 4), increasing lane-changing within the spillback area is likely. Therefore, in addition to the factors that influence travel speed in undersaturated conditions (free-flow speed and outer lanes’ demand flow rate), the proportion of the exiting flow rate compared to the entire freeway flow rate also affects lane changing and speeds in the leftmost lanes (6). Table 1 summarizes the factors likely affecting average travel speed, by regime, and by lane. For illustration purposes, several assumptions (Table 1) are made to calculate average travel speed\(^1\). Once the speed for each lane is determined, the average speed across all lanes \(S\) is computed as follows:

\[
S = \frac{v_{SA} + v_{IA} + v_{O}}{\left(\frac{v_{SA}}{S_{SA}} + \frac{v_{IA}}{S_{IA}} + \frac{v_{O}}{S_{O}}\right)}
\]

Where \(v_{SA}\), \(v_{IA}\), and \(v_{O}\) are the flow rate at spillback lane(s), influencing area and outer lane(s), respectively. \(S_{SA}\), \(S_{IA}\), and \(S_{O}\) are the speed at spillback lane(s), influencing area and outer lane(s), respectively.

### TABLE 1 Factors and Equations to Estimate Average Travel Speed

<table>
<thead>
<tr>
<th>Diverge Segments</th>
<th>Spillback Regime</th>
<th>Spillback Lane(s)</th>
<th>Influence Area</th>
<th>Outside Spillback Area</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1, 2</td>
<td>• Factors: Exiting demand flow rate, and User-estimated stopped threshold speed • Estimates: 20 (mph)</td>
<td>• Factors: Length of influence area, Proportion of exiting demand flow rate, and Demand flow rate of spillback area • Estimates: FFS – 15 (mph)</td>
<td>Refer to HCM 2010 Exhibit 13-12</td>
</tr>
<tr>
<td></td>
<td>3, 4</td>
<td>• Factors: Exiting demand flow rate and User-estimated stopped threshold speed • Estimates: 20 (mph)</td>
<td>• Factors: Proportion of weaving demand flow rate and Available queue storage distance within the base length • Estimates: FFS – 15 (mph)</td>
<td></td>
</tr>
</tbody>
</table>

| Weaving Segments | 1                | • Factors: Exiting demand flow rate and User-estimated stopped threshold speed • Estimates: 20 (mph) | • Factors: Demand flow rate of vehicles within the spillback area • Estimates: FFS – 15 (mph) | Refer to HCM 2010 Equation 12-20 |
|------------------|------------------|• Factors: Further empirical research needed • Estimates: FFS – 5 (mph) |
|                  | 2                | • Factors: Exiting demand flow rate and User-estimated stopped threshold speed • Estimates: 20 (mph) | |
|                  | 3                | • Factors: Exiting demand flow rate and User-estimated stopped threshold speed • Estimates: 20 (mph) | |
|                  | 4                | • Factors: Exiting demand flow rate and User-estimated stopped threshold speed • Estimates: 20 (mph) | |

**Capacity Checks and Adjustments** When spillback occurs, the capacity of diverge segments is determined on a lane-by-lane basis. The base value of the entire diverge segment \(c_{d}\) is determined based on HCM 2010 Equation 13-8. Adjustments are made on the base capacity depending on prevailing regime.

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\(^{1}\) Further empirical data collection and research are necessary to develop equations predicting travel speeds by regime by lane.
Regime 1 and 2 A capacity adjustment factor (CAF) accounting for the presence of a queue in an adjacent lane can be estimated based on the expected lower lane utilization by mainline vehicles of Lane 1. This CAF is likely to be influenced by the proportion of exiting vehicles \((v_R/v_F)\) and the distance to the closest adjacent downstream off-ramp \((L_{DOWN})\) \(^{(6)}\). This study assumes 0.85 as the CAF value\(^{2}\). The adjusted diverge segment capacity \((c)\), is thus estimated as:

\[
c = \frac{c_d}{N} \{N_O + CAF \times N_{IA}\}
\]

Where \(c_d\) is base capacity as determined by HCM 2010 Equation 13-8 (pc/h), \(N\) is number of unblocked freeway mainline lanes in study segment (ln), \(N_O\) is number of lanes outside of the influencing area (ln), \(N_{IA}\) is number of lanes in the influence area (ln).

Regime 3 and 4 The capacity of all lanes that are expected to be blocked by exiting vehicles is subtracted from the base value. A CAF is also applied to adjust the capacity of lanes within the influence areas; 0.7 is assumed here\(^{3}\). The capacity of the spillback lanes is adjusted based on the probability of a lane being blocked at any time during the 15-minute analysis period \((P_B)\). It is estimated based on the length of the queue beyond the off-ramp storage distance \((Q_{SP})\), as shown in Figure 2. Empirical data collection at sites experiencing Regime 3 and 4 spillback with various geometric characteristics is necessary to establish prediction models for \(P_B\). The adjusted diverge segment capacity \((c)\), is estimated as:

\[
c = \frac{c_d}{N} \{N_O + CAF \times N_{IA} + (1 - P_B) \times CAF\}
\]

Where CAF is capacity adjustment factor, 0.7, \(P_B\) is probability of blockage in the influence area, between 0 and 1.

Numerical Example
To illustrate the proposed methodology, a numerical example is provided for a six-lane freeway segment with a one-lane right-hand off ramp, with the following characteristics:

- Freeway FFS = 60 mi/h
- Ramp FFS = 40 mi/h
- No shoulder along mainline freeway available for additional queue storage (i.e., \(L_E = L_D\))
- Freeway demand \((v_F)\) = 4500 veh/h
- Ramp demand \((v_R)\) = 1000 veh/h
- Ramp deceleration lane length \((L_D)\) = 400 ft
- Capacity of ramp proper = 1500 pc/h
- Capacity of downstream intersection approach = 600 pc/h
- Available queue storage distance = 300 \((L_A)\) ft/ln
- Average stored passenger car length = 25 \((L_{pc})\) ft
- Average stored heavy-vehicle length = 45 \((L_{HV})\) ft

\(^{2}\) Empirical data collection at sites experiencing Regime 1 and 2 spillback with a variety of geometric characteristics would be necessary to establish accurate prediction models for CAFs.

\(^{3}\) Empirical data collection with a variety of geometric characteristics would be necessary to establish accurate prediction models for CAF.
- Peak hour factor = 0.95
- Rolling terrain for freeway and ramp
- 5% trucks on freeway and both ramps
- 15-minute period analysis

The results are shown in Figure 3. As shown, the exiting demand (1132 pc/h) exceeds the capacity of the downstream intersection (600 pc/h). Queue formation follows Scenario B and the queue length (30 veh) is estimated based on HCM 2010 Chapter 31. Queue storage ratio is calculated as $2.5 > 1$. The optional Step 4 is needed, and the length of additional queue is estimated as 450 ft, which is longer than the ramp deceleration lane length (400 ft) by 50 ft (less than 100 ft assumption in Figure 2), indicating this condition falls in Regime 3. Since there is no shoulder available for additional queue storage, and the queue length exceeds the deceleration lane by 50 ft, Lane 1 is blocked and the probability of blockage on Lane 2 is 10% (Figure 2). The final diverge segment capacity is thus calculated as 3058 pc/h, with a v/c ratio of 1.67. The level of service (LOS) is determined to be F.

<table>
<thead>
<tr>
<th>Geometric Data</th>
<th>Ramp</th>
<th>Freeway</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lanes</td>
<td>1</td>
<td>3</td>
</tr>
<tr>
<td>Free-Flow Speed (FFS), m/s/h</td>
<td>35</td>
<td>60</td>
</tr>
<tr>
<td>Deceleration Length/Segment Length (L), ft</td>
<td>800</td>
<td>5280.00</td>
</tr>
<tr>
<td>Terrain</td>
<td>Rolling</td>
<td>Rolling</td>
</tr>
<tr>
<td>Grade</td>
<td>0</td>
<td>0.00</td>
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<tr>
<td>Right Side</td>
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<td></td>
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<table>
<thead>
<tr>
<th>Demand and Capacity</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Demand, veh/h</td>
<td>1000</td>
<td>4500</td>
</tr>
<tr>
<td>Peak Hour Factor (PHF)</td>
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<td>0.95</td>
</tr>
<tr>
<td>Trucks and Buses (PT), %</td>
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<td>5</td>
</tr>
<tr>
<td>RVs (PR), %</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Passenger Car Equivalent Truck (Er)</td>
<td>2.5</td>
<td>2.5</td>
</tr>
<tr>
<td>Passenger Car Equivalent RV (Er)</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Heavy Vehicle Factor (Ev)</td>
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<td>0.93</td>
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<tr>
<td>Driver Population Factor (Fr)</td>
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<td>1</td>
</tr>
<tr>
<td>Flow Rate, pc/h</td>
<td>1132</td>
<td>5993</td>
</tr>
<tr>
<td>Capacity, pc/h</td>
<td>1500</td>
<td>3058  (Eq. 7)</td>
</tr>
<tr>
<td>Volume-to-Capacity Ratio (v/c)</td>
<td>0.75</td>
<td>1.67</td>
</tr>
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</table>

<table>
<thead>
<tr>
<th>Spillback Results</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Regimes</td>
<td>3</td>
<td>Lanes Outside Influence Area, In</td>
</tr>
<tr>
<td>Average Vehicle Length (L), ft/veh</td>
<td>25.01</td>
<td>Queue Storage Ratio (RQ)</td>
</tr>
<tr>
<td>CAF</td>
<td>0.7</td>
<td>PB for Regimes 3 and 4</td>
</tr>
<tr>
<td>Queue Storage Length Ramp Lane (Lr), ft</td>
<td>300</td>
<td>Shoulder Length (L r), ft</td>
</tr>
<tr>
<td>Queue Storage Length Lane 1, ft</td>
<td>500</td>
<td>Queue Length (Qop), ft</td>
</tr>
<tr>
<td>Average Speed (S), m/s/h</td>
<td>46.31</td>
<td>Average Density, pc/mi/ln</td>
</tr>
<tr>
<td>LOS</td>
<td>F</td>
<td></td>
</tr>
</tbody>
</table>

FIGURE 3 Example results (spillback on isolated diverge junctions).

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4 The segment capacity without off-ramp spillback is 6900 pc/h, indicating spillback results in 55% capacity decrease.
Spillback Effects on the Diverge Ramp of a Weave

This section provides procedures for evaluating traffic operations at a weaving segment when there is spillback from the off-ramp. This study focuses primarily on adjustments and recommendations made to one-sided weaving segments.

Queue Regimes

When spillback occurs, drivers are likely to use any available space despite pavement markings so as to avoid potential high-speed collisions with incoming mainline vehicles at the rear of the queue. Therefore, the weaving segment length used in the proposed methodology is the base length ($L_B$), i.e., the distance between points in the respective gore areas where the left edge of the ramp-traveled way and the right edge of the freeway-traveled way meet (Figure 4). The spillback area boundary is defined as belonging to one of four regimes, based on the estimated queue length compared to the weaving segment’s geometry. Performance measures and capacity are estimated based on these regimes and their respective boundaries.

Regime 1 ($Q_{SP} < L_B$) The queue occupies a portion of the auxiliary lane within the base length. Both weaving and non-weaving flow rates are able to complete their intended maneuvers (Figure 4a).

An additional check should be conducted to compare the calculated maximum weaving length ($L_{MAX}$ – the length at which weaving turbulence no longer has an impact on operations within the segment) to the available weaving distance after taking into consideration the presence of the queue, which is $(L_B – Q_{SP})$. $L_{MAX}$ is estimated using HCM 2010 Equation 12-4.

- If $(L_B – Q_{SP}) < L_{MAX}$, the weaving segments procedure is valid
- If $(L_B – Q_{SP}) \geq L_{MAX}$, the merge and diverge junctions must be analyzed independently

Regime 2 ($Q_{SP} \geq L_B$, upstream lateral clearance available) The queue extends along the entire length of the auxiliary lane within the base length. Two O-D demand flow rates ($v_{RF}$ and $v_{RR}$) are restricted from executing their intended maneuvers at the merge point, and a separate queue will likely form at the on-ramp. This regime occurs when there is sufficient lateral or shoulder clearance available for additional queue storage immediately upstream of the merge point (Figure 4b). Exiting vehicles ($v_{FR}$) are likely to decrease their speed to join the back of the queue on the shoulder. Although the spillback area boundaries are identical to that of Regime 1, it is likely that average travel speed of vehicles within the spillback area are different.

It is not necessary to compare the maximum weaving length and available weaving distance, since the queue in the auxiliary lane blocks $v_{RF}$ and $v_{RR}$ from executing their intended maneuvers and it no longer operates as a weaving segment.

Regime 3 ($Q_{SP} \geq L_B$, no upstream lateral clearance available, Lane 1 blocked) The queue extends beyond the auxiliary lane, and two O-D demand flow rates ($v_{RF}$ and $v_{RR}$) are restricted from executing their intended maneuvers at the merge point. This regime occurs when there is no lateral clearance available immediately upstream of the merge point, and mainline vehicles wishing to exit are forced to block the rightmost freeway lane (Lane 1) (Figure 4c). Similar to Regime 2, this segment no longer operates as a weaving segment.

Regime 4 ($Q_{SP} \geq L_B$, no upstream lateral clearance available, Lanes 1 and 2 blocked) The prevailing conditions under this regime are identical to those of Regime 3, except that Lane 2 (in
addition to Lane 1) becomes partially or fully blocked by queued vehicles (Figure 4d). Under this regime, the relationship between queue distance on Lane 1 and blockage probability of Lane 2 is similar to that shown in Figure 2 for diverges. Further empirical research is needed to solidify these relationships.

Similar to Regime 2, this segment no longer operates as a weaving segment.

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**Operational Effects**

Operational measures for weaving segments with spillback are estimated on a lane-by-lane basis, considering spillback lane, influence lane and lane(s) outside the influence area.

**Average Speeds in Weaving Segments** Past studies have shown that spillback from an off-ramp can cause a reduction in speeds along the mainline in diverge segments, and it is likely that similar effects would be observed at weaving segments. However, empirical research is needed to better understand the factors that influence average travel speeds within and outside the spillback area during each of the regimes.

For traveling in lanes within the influence area, speeds are expected to be influenced by the proportion of weaving demand flow ($v_{w}/v_{F}$), available queue storage distance within the base length ($L_{B} - Q_{SP}$), and demand flow of vehicles within the spillback area.

For lanes outside the spillback area and in cases of spillback not involving mainline lane blockage (Regimes 1 and 2), it is unlikely that average travel speeds will be affected. Therefore, the speed prediction model for non-weaving vehicles in the current HCM 2010 Weaving Segments procedure is valid, assuming that non-weaving mainline vehicles tend to use the outer lanes. However, in cases involving mainline lane blockage (Regimes 3 and 4), there is no empirical evidence to suggest which factors affect average speed. While more abrupt lane-changing within the spillback area is likely, the restricted O-D flows ($v_{RR}$ and $v_{RF}$) may result in a decrease in lane-changing activity and overall turbulence. The factors and assumptions used to calculate average travel speed by lane, by regime, are provided in Table 1. Once the speed for each lane is determined, the average speed across all lanes is computed by Equation (5).

**Capacity Checks and Adjustments** When spillback occurs, the capacity of weaving segments is determined on a lane-by-lane basis. The base capacity value of the entire diverge segment ($C_{w}$) is

---

5 Further empirical data collection and research are necessary to develop equations for travel speeds by regime by lane.
determined as the smaller value of the two weaving base capacities – HCM 2010 Equation 12-6 and 12-8. Adjustments are made on the base capacity differently by regime.

- **Regime 1 and 2** A CAF is used to account for the weaving demands \( v_w/v_F \), and the distance from the upstream end of the queue to the closest upstream on-ramp \( L_{UP} - Q_{SP} \). We assume 0.85 as the CAF value\(^6\). The adjusted weaving segment capacity \( c \) is determined as:

\[
c = \frac{c_w}{N} \{ N_O + CAF \} \tag{8}
\]

Where \( c_w \) is base capacity as determined by HCM 2010 Equation 12-6/12-8 (pc/h).

- **Regime 3 and 4** Similar to spillback onto diverge segments, CAF is also applied to adjust the capacity of lanes within the influence areas; 0.7 is assumed here\(^7\). The capacity of the spillback lanes is adjusted based on the blockage probability within the 15-minute analysis period \( P_B \), which is estimated from Figure 2. The adjusted weaving segment capacity \( c \), is determined as follows.

\[
c = \frac{c_w}{N} \{ N_O + CAF + (1 - P_B) * CAF \} \tag{9}
\]

**Numerical Example**

To illustrate the proposed methodology, a numerical example is provided for a four-lane mainline with a one-sided (right-hand), one-lane ramp-weaving segment, with the following characteristics:

- Freeway FFS = 65 mi/h
- Interchange density (ID) = 1 int/mi
- Length of deceleration lane and extension along shoulder \( L_E \) = 2000 ft
- Freeway-to-freeway demand \( v_{FF} \) = 3500 veh/h
- Ramp-to-freeway demand \( v_{RF} \) = 600 veh/h
- Freeway-to-ramp demand \( v_{FR} \) = 700 veh/h
- Ramp-to-ramp demand \( v_{RR} \) = 300 veh/h
- Weaving segment base length \( L_B \) = 1600 ft
- Capacity of ramp proper = 1000 pc/h
- Capacity of downstream intersection approach = 600 pc/h
- Peak hour factor = 0.95
- Level terrain for freeway and ramp
- 5% trucks on freeway and both ramps
- 15-minute period analysis

The results are shown in Figure 5. The exiting demand (1079 pc/h) exceeds the capacity of ramp proper (1000 pc/h). Queue formation follows Scenario A and queue length (30 veh) is estimated using Equation (1). Queue storage ratio is calculated as 2.5 > 1. The optional Step 4 is

\(^6\) Empirically based research is needed to fully understand which factors affect CAF in weaving segments, and to what extent.

\(^7\) Empirical data collection is necessary to establish validate prediction models for CAFs.
needed, and the length of additional queue is 1832 ft, which is between the segment base length (1600 ft) and the extended queue storage distance (2000 ft). Regime 2 is prevailing, and the capacity is calculated as 5934 pc/h, with a v/c ratio of 0.93. Since v/c is less than 1 and does not exceed capacity, the procedure continues into LOS determination. The speed for each lane is estimated as follows: spillback lane – 20 mph; influence area – 50 mph; outer lanes – 48.3 mph. Then the average speed across all lanes is computed on a lane-by-lane basis (Equation (5)) as 38.5 mph. Density is estimated as 35.7 pc/m/ln, indicating LOS E.

**Geometric Data**

<table>
<thead>
<tr>
<th>Mainline Lanes (N), Ln</th>
<th>4</th>
<th>Weaving Lanes (Nw), Ln</th>
<th>2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length (L), ft</td>
<td>1300</td>
<td>Weaving Configuration</td>
<td>One-Sided Weave</td>
</tr>
<tr>
<td>Ramp to Freeway Lanes Changes (LC), ft</td>
<td>1</td>
<td>Ramp to Ramp Lanes Changes (LC), ft</td>
<td>0</td>
</tr>
<tr>
<td>Interchange Density (ID), in/ft</td>
<td>1.0</td>
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<td></td>
</tr>
</tbody>
</table>

**Terrain Type**

<table>
<thead>
<tr>
<th>Level</th>
<th>Grade, %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.00</td>
</tr>
</tbody>
</table>

**Demand and Capacity**

<table>
<thead>
<tr>
<th>Volume (V), veh/h</th>
<th>FF</th>
<th>FR</th>
<th>RF</th>
<th>RR</th>
</tr>
</thead>
<tbody>
<tr>
<td>3500</td>
<td>700</td>
<td>600</td>
<td>300</td>
<td></td>
</tr>
<tr>
<td>Heavy Vehicle Factor (FHV)</td>
<td>0.976</td>
<td>0.976</td>
<td>0.976</td>
<td></td>
</tr>
<tr>
<td>Flow Rate (v), pc/h</td>
<td>3776</td>
<td>755</td>
<td>647</td>
<td>324</td>
</tr>
<tr>
<td>Peak Hour Factor (PHF)</td>
<td>0.95</td>
<td>Driver Population Factor (b)</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>Weaving Demand (W), pc/h</td>
<td>1402</td>
<td>Non Weaving Demand (ww), pc/h</td>
<td>4100</td>
<td></td>
</tr>
<tr>
<td>Total Demand (D), pc/h</td>
<td>5502</td>
<td>Weaving Volume Ratio (WR)</td>
<td>0.255</td>
<td></td>
</tr>
<tr>
<td>Minimum Lane Change Rate (LC), ft</td>
<td>1402</td>
<td>Maximum Weaving Length (Lw), ft</td>
<td>5106</td>
<td></td>
</tr>
<tr>
<td>Capacity (Cw), pc/h</td>
<td>8328</td>
<td>Volume-to-Capacity Ratio</td>
<td>0.93</td>
<td></td>
</tr>
</tbody>
</table>

**Weaving and Non Weaving Speeds**

| Total Lane Change Rate (LC), kc/h | 1402 | Weaving Lane Change Rate (Lw), kc/h | 1402 |
| Nonweaving Lane Change Rate (LC), kc/h | 0 | Non Weaving Index (W) | 0 |
| Weaving Intensity Factor (W) | 0.000 | Speed Weaving (Sw), mi/h | 65.0 |
| Speed Non Weaving (Sw), mi/h | 48.3 |

**Spillback Results**

<table>
<thead>
<tr>
<th>Regimes</th>
<th>2</th>
<th>Lanes Outside Influence Area, ln</th>
<th>2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Influence Area Lanes, ln</td>
<td>1</td>
<td>Spillback Lanes, ln</td>
<td>1</td>
</tr>
<tr>
<td>Queue Storage Length: Ramp Lane (La), ft</td>
<td>300</td>
<td>Shoulder Length (Ls), ft</td>
<td>2000</td>
</tr>
<tr>
<td>Queue Storage Length Lane 1, ft</td>
<td>500</td>
<td>Queue Length (Qd), ft</td>
<td>1832.00</td>
</tr>
<tr>
<td>Average Vehicle Length (Lh), ft/veh</td>
<td>25.010</td>
<td>Queue Storage Ratio (QSR)</td>
<td>2.50</td>
</tr>
<tr>
<td>Capacity Adjustment Factor</td>
<td>0.850</td>
<td>Probability Blockage on Lane 2</td>
<td>0.000</td>
</tr>
<tr>
<td>Adjusted Capacity, pc/h</td>
<td>5934</td>
<td>(Eq. 8)</td>
<td>Proportion of Lane 1 Weaving Flow</td>
</tr>
<tr>
<td>Spillback Lanes Flow, pc/h</td>
<td>1000</td>
<td>Influence Area Lanes Flow, pc/h</td>
<td>318</td>
</tr>
<tr>
<td>Outer Lanes Flow, pc/h</td>
<td>4184</td>
<td>Spillback Lanes Speed, mi/h</td>
<td>20.0</td>
</tr>
<tr>
<td>Influence Area Lanes Speed, mi/h</td>
<td>31.0</td>
<td>Outer Lanes Speed, mi/h</td>
<td>48.3</td>
</tr>
<tr>
<td>Average Speed (S), mi/h</td>
<td>38.5</td>
<td>(Eq. 5)</td>
<td>Average Density, pc/mi/ln</td>
</tr>
</tbody>
</table>

**FIGURE 5** Example results (spillback on weaving segments).

**SPILLBACK FROM FREEWAYS TO SURFACE STREETS**

8 The segment capacity without spillback is 8328 pc/h, indicating spillback results in 29% capacity decrease.
This section discusses the effects of spillback from merge ramps on signalized surface street intersections. The method assumes that arrival rates greater than ramp throughput cause spillback at the upstream signalized intersections. As a result, the effective green time for a specific phase may be reduced, affecting the capacity of the intersection. The computational steps of the methodology are described below followed by a numerical example. The last section discusses important aspects of the method.

Methodology

- **Step 1: Perform steps 1 through 6 of the HCM Signalized Intersections procedure**
  This analysis determines which movements lead to the merge. Lane group flow rates are defined and intermediate outputs calculated as usual. Finally, the effective green times and capacities are obtained.

- **Step 2: Define the departure rate \( \mu \)**
  As discussed in the literature review, there is no information on departure rates from a congested merge junction; thus this quantity is user-defined in our methodology.

- **Step 3: Determine arrival rate for the lane groups approaching the on-ramp**
  The arrival rate for each phase \( (\lambda_i) \), in seconds, is:
  \[
  \lambda_i = \min \left( \frac{s_i N_i v_i}{3600 \cdot 3600} \right)
  \]
  Where \( s_i \) is the saturation flow rate for the approach, \( N_i \) is the number of lanes for the approach, and \( v_i \) is the flow rate of phase \( i \).

  Note that the arrival rates for simultaneous movements must be considered together.

- **Step 4: Determine the queue length at the end of each phase**
  It is critical for the proposed method to estimate the formation and discharge of queues throughout the period of analysis. If one cycle is considered, then the difference between the queue length \( (\Delta Q) \) at the beginning and the end of the cycle is calculated as:
  \[
  \Delta Q = \sum _{i}^n [ (\lambda_i - \mu) g_i + (-\mu Y_i) ] L_h
  \]
  Where \( L_h \) is the average length of a vehicle, \( g_i \) is the green time for each phase \( i \), and \( Y_i \) is the yellow/clearance time for each phase \( i \).

  The queue length at the end of the cycle \( (Q_{SP1}) \) is given by:
  \[
  Q_{SP1} = Q_{SP0} + \Delta Q
  \]
  where \( Q_{SP0} \) is the initial queue length (ft), which corresponds to the average queue during the previous analysis interval, or is input by the user.

  If an analysis period greater than one cycle is used, an adjusted initial queue is estimated and then an average for the analysis period is obtained. This step ensures that the proposed method is compatible with the 15-minute period used by the HCM, as well as the timing variations resulting from the fully or semi actuated analysis procedure. However, the methodology does not account for this variability cycle-by-cycle, as this is a limitation of the HCM procedures. First, the number of cycles \( (nC) \) is determined:
The difference between the queue length in the beginning and the end of the period ($\Delta Q_T$) is then estimated by multiplying the number of cycles by $\Delta Q$ for each cycle, noting that, the average queue length $Q_{SP0}'$ should not become negative:

$$Q_{SP0}' = \max \left\{ Q_{SP0} + \frac{nC \times \Delta Q}{2} \right\}$$

**Step 5: Reduced capacities and modified effective green times**

The reduced capacity of each lane group is determined as:

$$c'_i = N \times s \frac{g'_i}{C}$$

$$g'_i = \frac{L_M - Q_{SP_{i-1}}}{(\lambda_i - \mu)L_h}, \text{if } Q_{SP_{i-1}} < L_M$$

$$g'_i = g_i - \frac{L_M - Q_{SP_{i-1}}}{(\lambda_i - \mu)L_h}, \text{if } Q_{SP_{i-1}} > L_M$$

Where $g'_i$ is the reduced effective green time, $L_M$ is the storage distance, and $Q_{SP_{i-1}}$ is the queue length in the end of the previous phase.

**Numerical Example**

To illustrate the proposed methodology, a numerical example is provided for a four-leg, two-phase intersection. The NB movement leads to the on-ramp. The input data are shown in Table 2.

**TABLE 2 Inputs and Signalized Intersection Analysis Results (HCM 2010)**

<table>
<thead>
<tr>
<th>Data Element</th>
<th>Eastbound</th>
<th>Westbound</th>
<th>Northbound</th>
<th>Southbound</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lane group</td>
<td>L T T+R</td>
<td>L T T+R</td>
<td>L T T+R</td>
<td>L T T+R</td>
</tr>
<tr>
<td>Phase number</td>
<td>2 2 2 2</td>
<td>6 6 6 6</td>
<td>3 8 8 8</td>
<td>7 4 4 4</td>
</tr>
<tr>
<td>Number of lanes $N$ (ln)</td>
<td>1 1 1 1</td>
<td>1 1 1 1</td>
<td>1 1 1 1</td>
<td>1 1 1 1</td>
</tr>
<tr>
<td>Flow rate $v$ (veh/h)</td>
<td>71 239 185</td>
<td>118 337 287</td>
<td>133 870 963</td>
<td>194 513 497</td>
</tr>
<tr>
<td>Adjusted saturation flow rate $s$ (veh/h/ln)</td>
<td>702 1643 1201 825 1643 1398 1903 1683 1683 1683 1683 1683 1630</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Effective green time $g$ (s)</td>
<td>30.0 30.0 30.0 30.0 30.0 30.0 6.2 50.0 50.0 9.3 53.6 53.6</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Capacity $c$ (veh/h)</td>
<td>149 484 354 206 484 412 328 827 909 225 887 899</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Volume-capacity ratio $X$</td>
<td>0.47 0.49 0.52 0.57 0.70 0.70 0.41 1.05 1.07 0.86 0.58 0.58</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The first phase serves the NBT movement. The arrival rate is $\lambda_{NT} = 0.457$. For the EB and WB directions, the EBL and WBR movements must be added, for a total flow of $71 + 24 = 95$ veh/h, yielding $\lambda_{ELWR} = 0.026$.

Since a 15-minute period of analysis is used, it is necessary to adjust the initial queue length to a representative average by using the equations on step 4. The results are:

- $\Delta Q = -5$;
- $nC = 10.23$;
- $Q_{SP0}' = 273$.

The average queue of 300 ft was reduced to 273 ft during the analysis period. Finally, following the equations shown on Step 5, the adjusted effective green times per phase are determined and the results are shown on Table 3.

Table 3 also compares the final results for this example for a cycle and a 15-minute
period. As shown, the effective green times decrease as a result of queing. When the 15-min period is considered, this effect is slightly reduced, reflecting the process of queue discharge along the period for this specific example.

**TABLE 3 Intermediate Results, Queues and Adjusted Effective Green Times**

<table>
<thead>
<tr>
<th>Time period</th>
<th>Data Element</th>
<th>Northbound</th>
<th>Eastbound/Westbound</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Lane group</td>
<td>Number of Lanes</td>
</tr>
<tr>
<td></td>
<td></td>
<td>TH</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>LT</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>RT</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Flow rate</td>
<td>1644</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>71</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>24</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Adjusted saturation flow rate</td>
<td>1683</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>702</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1398</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Arrival rate</td>
<td>0.457</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.020</td>
</tr>
<tr>
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<td></td>
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<td>Arrival rate</td>
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<td></td>
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<td>g'_{i}</td>
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<td>13</td>
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<td>15-minute period analysis</td>
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<td>506</td>
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<tr>
<td></td>
<td></td>
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<td>17</td>
</tr>
</tbody>
</table>

A limitation of the proposed method is that it does not account for free-flowing right turn movements that could potentially affect queue accumulation and discharge on the ramp. This effect has been historically a limitation of the HCM signalized intersection method.

**CONCLUSIONS AND RECOMMENDATIONS**

This paper proposes a series of modifications to the existing HCM 2010 procedures in order to address spillback from one facility to another. In the absence of nationwide field data, the methodology uses assumed values to implement the methods.

For spillback from surface streets to freeways, the procedure first estimates the maximum expected queue length. Four regimes are defined as a function of queuing conditions, and the method adjusts speeds and capacities on a lane-by-lane basis. The procedures consider spillback for both diverge and weaving segments. This procedure can be implemented as an enhancement of the Diverge Segment procedure, Weaving Segments procedure and/or the Freeway Facilities procedure once suitable nationwide data are obtained. When the off-ramp queue is found to extend beyond the ramp proper, a Freeway Facilities analysis should be conducted for multiple periods of analysis and for the entire distance of the area affected, in order to consider effects on upstream segments.

For spillback from freeways to surface streets, the procedure estimates queuing by first obtaining the rate of departures from the on-ramp and the rate of arrivals from the intersection. By comparing the queue length and the storage space in each phase within a cycle, the reduction on the effective green is computed.

Several assumptions are used to implement the proposed procedures. Nationwide data collection and calibration on the following parameters are recommended:
- Percentage of freeway mainline traffic in each lane, as a function of spillback regime and queue length
- Speed along each freeway mainline lane, as a function of spillback regime and queue length
- Lane-by-lane capacity adjustments as a function of spillback regime and queue length
- Discharge rates for on-ramps during congested conditions, as a function of freeway and ramp geometry and demand.

**ACKNOWLEDGEMENT**

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REFERENCES