Using Cooperative Adaptive Cruise Control (CACC) to Form High-Performance Vehicle Streams

Microscopic Traffic Model Calibration and Validation

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Xingan (David) Kan

Hao Liu

Steven E. Shladover

Xiao-Yun Lu

California PATH Program

Institute of Transportation Studies

University of California, Berkeley

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1 INTRODUCTION

This report documents the procedures the PATH team has implemented for calibrating the NGSIM oversaturated flow model in the Aimsun micro simulation environment. The goal of the calibration is to identify suitable driving behavior parameters of the NGSIM oversaturated flow model, which is used to describe the behaviors of human drivers. With these parameters, the model should be able to reproduce traffic flow patterns like those observed in the 18-km State Route 99 corridor to the south of Sacramento, CA. The Geoffrey E. Havers (GEH) statistic is adopted to quantify the relative difference between the simulated and observed flows. The calibrated model is considered satisfactory if the GEH statistic is less than 5 in at least 85% of the time intervals. In addition to the GEH statistic, the simulated traffic pattern must produce spatial and temporal speed distribution and flow-density relationships similar to those observed in the field.

This report contains 6 sections. In Section 2, we describe the background of the study site and the field data used as the benchmark data. Section 3 contains the detailed explanation of the metrics that depict the performance of the simulation model. In Section 4, we present the candidate behavior parameters to be calibrated and the calibration procedures. This section also includes the traffic modeling effort we have made for accurately reproducing the traffic operation at individual on/off-ramp bottlenecks. In Section 5, we present the identified behavior parameters and the calibration result analysis. In the final section, the recommendations for future research are proposed.

2 STUDY SITE AND BENCHMARK DATA

State Route (SR) 99 northbound was selected for model calibration. This section of freeway spans from the Elk Grove Blvd. interchange to the US-50 freeway interchange south of downtown Sacramento, CA. As indicated by the arrows in Figure 1, there are 9 interchanges with local arterial streets and an interchange with a major urban freeway. These interchanges include 4 partial cloverleaf interchanges, 3 full cloverleaf interchanges, 2 diamond interchanges with the local arterials, and a directional interchange with a freeway. Furthermore, there are 3 lanes (one high occupancy vehicle or HOV lane and two general purpose lanes) in each direction upstream of the Calvine Road interchange while an additional general purpose lane is added downstream of that interchange. The on-ramp merging and weaving sections located downstream of the Elk Grove Blvd. interchange, as well as the off-ramp at the US-50 freeway interchange, contribute to the recurrent traffic congestion observed during the morning peak in this corridor. This peak period typically begins at 6:30 AM and ends around 9:00 AM, and the morning congestion pattern is the result of the high demand for suburb to downtown trips during the morning hours. Currently, the on-ramps are metered using the local traffic responsive demand-capacity approach in order to control the flow of on-ramp traffic and mitigate the peak hour congestion.

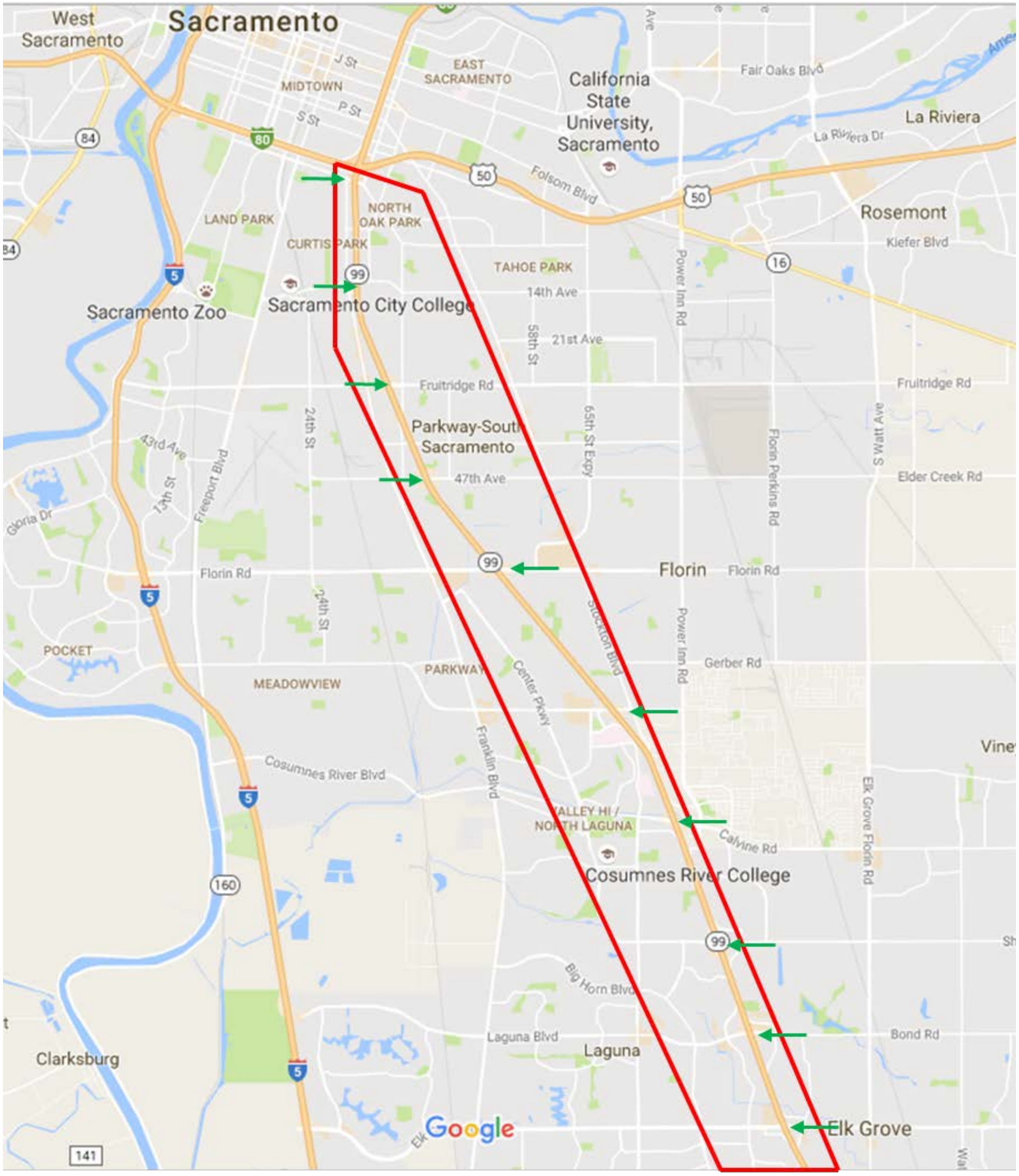

Figure 1. Study Site: SR-99 Freeway South of Downtown Sacramento, CA.

The freeway mainline and on-ramp vehicle counts obtained from an 8-hour period (4:00 AM to 12:00 PM) of October 6, 2015 were used in our simulation experiment as the traffic inputs. Particularly, the 5-minute interval loop detector data for vehicle counts were obtained from PeMS (1) and used as the demand input at the most upstream location of the simulation network and the entry points of the on-ramps, and as the turning percentages at any applicable mainline-off-ramp split.

The 5-minute interval vehicle count and speed data obtained from 16 loop detector stations within the study corridor were adopted as the benchmark data for model calibration. The modeled vehicle counts and speeds were compared with the benchmark data for determining the model performance. The locations and IDs of the detector stations are shown in Figure 2. Detectors with good data quality are shown in blue, those with less acceptable data quality, shown in red, were not used to collect field data for calibration and validation.

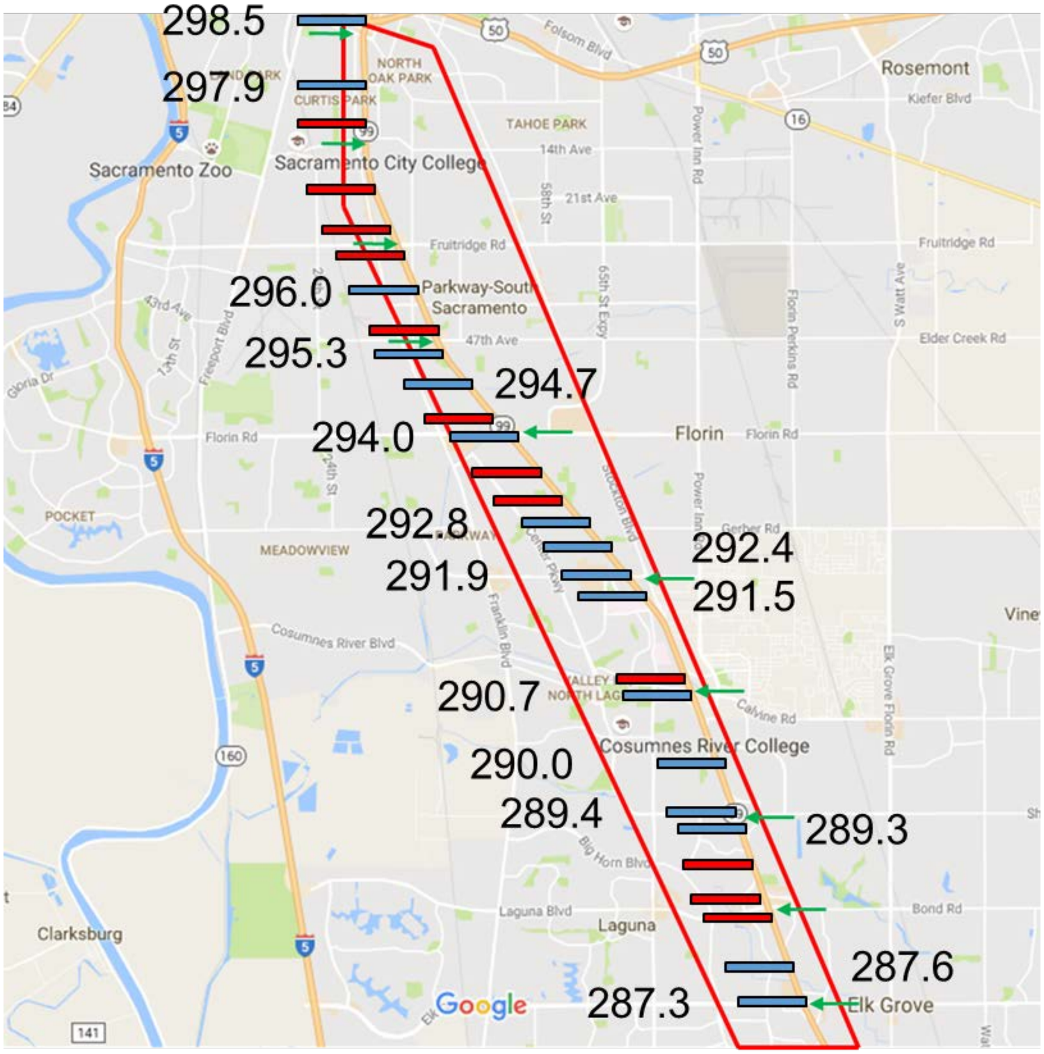

Figure 2. Locations and IDs of Detector Stations.

3 METRICS OF MODEL PERFORMANCE

The modeled vehicle counts and speeds at the locations shown in Figure 2 were compared with real traffic measurements in every 5-minute interval to assess the accuracy of the simulation model in representing the observed conditions. The GEH statistic is used to determine the accuracy of the simulated vehicle counts:

$$GEH(k) = \sqrt{\frac{2[M(k) - C(k)]^2}{M(k) + C(k)}} \quad \text{(Eq. 1)}$$

where $M(k)$ are the simulated vehicle counts during the k-th time interval (veh); and $C(k)$ are the vehicle counts measured in the field during the k-th time interval (veh).

A satisfactory calibration requires that on average of all detectors, for at least 85% of all 5-minute time intervals, the vehicle count is to satisfy the condition $GEH(k) < 5$. In addition, the contour plots of the field observed and simulated speeds at all 5-minute intervals should exhibit similar trends over the length of the corridor as well as the duration of the study period. Lastly, the simulated and observed flow-density relationships should show similar patterns and trends.

4 Calibration Procedure

This section is divided into two subsections. Section 4.1 describes the car following and lane changing parameters selected for adjustment and fine tuning in the calibration process. The recommended values and their effect on the macroscopic traffic performance are discussed in detail. Section 4.2 documents the modifications made to the NGSIM car following and lane changing models, as well as adjustments made to account for the HOV traffic characteristics, truck volumes, ramp metering, and geometric changes throughout the corridor.

4.1 Calibration Parameters

Table 1 lists the driving behavior parameters that are targeted for calibration. A more detailed description regarding the model parameters can be found in a separate companion report entitled *Using Cooperative Adaptive Cruise Control (CACC) to Form High-Performance Vehicle Streams: Microscopic Traffic Modeling*. The table also summarizes the possible ranges of the parameters and the qualitative relationships between individual parameters and the macroscopic traffic performance.

Table 1. Driving Behavior Parameters Involved in Calibration.

Parameter	***Range***	***Description***
Mean reaction time (s)	0.4 – 1.2	The reaction time is used in the Gipps acceleration term of the NGSIM model. The Gipps term specifies the maximum safe acceleration rate a driver can achieve in an update interval. With a lower reaction time, a driver will apply a higher acceleration rate when speeding up, and a higherer deceleration rate when slowing down. In this case, the driver responds quickly to the speed and headway variations of the front vehicle. If the mean reaction time of the driver population reduces, the traffic flow will become more stable and the traffic flow at a bottleneck segment is less likely to break down. Once it does break down, it can recover quickly from the congested state.
Maximum acceleration (m/s^2)	1.5 – 2.5	The maximum acceleration is used in the IDM free flow term of the NGSIM model. The IDM term determines a driver's acceleration rate as the driver accelerates from a low speed state (e.g., exiting a queue). With a higher maximum acceleration, the IDM term will yield a greater acceleration rate for vehicles under the congested state. As a result, the queue in the congested region can discharge more quickly.
Maximum deceleration (m/s^2)	2.0 – - 5.0	The maximum deceleration is used in the Gipps acceleration term of the NGSIM model. With a higher maximum deceleration, a driver can drive more aggressively while avoiding the crash with the front vehicle. In this case, the driver will accept larger acceleration when speeding up and ***smaller*** deceleration when slowing down. In the systematic point of view, a larger maximum deceleration will lead to higher queue discharging flow, but less stable traffic. In addition, a driver with a larger maximum deceleration will brake harder when yielding to an on-ramp merging vehicle. Such a behavior can intensify the disturbances caused by the on-ramp traffic.
Mean headway (s)	1.0 – 2.0	The headway is used in the Newell term of the NGSIM model. The Newell term determines the acceleration of a driver in most of the car-following cases (e.g., except the case when the driver accelerates from a low speed state). With a lower headway, the modeled driver will become more aggressive. As a result, the queue discharging rate will be higher but the traffic flow will be less stable.
DLC desire threshold	0 – 1	When a driver's DLC desire is larger than the threshold, the driver will start searching for merging gaps in the adjacent lane. A high DLC desire threshold will reduce the chances of the discretionary lane change maneuver. It may lead to flow and speed heterogeneity among different lanes—one lane is congested but its adjacent lane is free flowing. When the parameter is at a low level, the modeled drivers will frequently change lanes, leading to frequent traffic disturbances and an unstable traffic flow.
Lane changing anticipatory	> 0	This parameter represents the average distance upstream from the off-ramp at which the drivers start to make lane changes toward the freeway exit. When the distance is large, the drivers will have more

distance for exiting (km)	chances to find large gaps in the target lane instead of forcing into short gaps. In this case, the exiting drivers' lane changing behaviors will have a smaller impact on the traffic flow stability.

4.2 MODIFICATION OF SIMULATION MODELS

Application of Local Behavior Parameters

In the preliminary calibration, we simulated the freeway corridor under the assumption that drivers adopt a single set of parameter values for all locations along the corridor. However, such assumption cannot accurately reproduce the temporal and spatial distributions of speed at the two most important bottlenecks, the weaving section at the US-50 interchange and the on-ramp merging area at the Sheldon Road interchange.

The weaving section at the US-50 bottleneck consists of 5 lanes, with the three left lanes designated to SR-99 through traffic and the two right lanes for US-50 access. During the morning peak hours, 50% of the traffic at this location exits SR-99 to access US-50, which requires lane change maneuvers towards the right lanes. Additionally, the 12^{th} Ave. interchange is about 600 meters upstream of the US-50 interchange. Thus, large on-ramp demand from 12th Ave entails additional lane change maneuvers in this area. Furthermore, some traffic entering SR-99 may not intend to exit and access US-50, which results in further lane changes towards the left lanes. Surprisingly, such high demand for lane change maneuvers in a 600-meter freeway segment does not cause capacity and speed reductions in the morning peak. This can be explained by the presence of frequent last-minute and aggressive lane changes in this section of SR-99, which is typical for morning peak commuters who frequently travel on this corridor and are typically willing to accept shorter gaps during lane changes. Simulating such traffic flow pattern requires adopting lower than normal lane changing anticipatory distance for exiting and shorter reaction time.

On the other hand, drivers are more cautious and less aggressive at the Sheldon Road interchange. Such behavior is explained by the poorer visibility in this area, mainly due to the presence of a vertical curve and an overpass. As a result, the bottleneck quickly forms at the beginning of the peak, when the demand is relatively lower.

Due to the contrasting traffic characteristics, applying a single set of global parameters would not accurately represent the real world traffic conditions. Selecting conservative parameters will reproduce the bottleneck at the Sheldon Road interchange but simulate a significantly more congested US-50 bottleneck. Conversely, aggressive parameters will replicate the US-50 bottleneck but result in a less congested freeway for the rest of the corridor. Therefore, more aggressive parameters were applied to the US-50 bottleneck while more conservative parameters were applied to the Sheldon Road bottleneck.

Deactivation of HOV Rules

The leftmost lane of the modeled freeway corridor is the HOV lane. Nonetheless, we did not implement the HOV rule in the simulation experiments due to insufficient data. For example, many on-ramps and all off-ramps do not have designated HOV bypass lanes, thus making the HOV percentage difficult to estimate. This prevents accurate estimate of how many and where the HOVs entered and exited the freeway. Despite this limitation, speed contour plots from PeMS suggest that the spatial and temporal distributions of speed of the HOV and general purpose lanes are similar throughout the study period, due to the high utilization of the HOV lane. The HOV lane does not remain in free flow during the peak period and exhibits the same congestion pattern as the general purpose lanes, so it was not necessary to model it differently from the general purpose lanes.

Modeling Truck Demand

Due to the low truck volume and lack of appropriate driving behavior model for trucks, truck traffic was not explicitly modeled in this simulation. Instead, we applied the passenger car equivalent factor to translate the truck demand into the equivalent passenger car demand. The HCM2010 suggests that a factor of 2.0 should be used for trucks on freeways with level terrain. As the truck traffic is on average 6.5% of the total demand in the study network, we adjusted the input demand as:

$$Vehicles_{Input} = Vehicles_{Observed} \cdot (93.5\% \cdot 1 + 6.5\% \cdot 2.0) = 1.065 \cdot Vehicles_{Observed}$$

Ramp Metering

Ramp metering is active from 6:00 AM to 9:00 AM at all of the on-ramps in the study corridor. During the time, an on-ramp vehicle needs to make a full stop at the beginning of the acceleration lane prior to merging into the freeway mainline. In the simulation experiments, we adopted Aimsun's Traffic Condition function to model the required stop at metered on-ramps during the peak hours.

Car-Following and Lane-Changing Behavior Modifications

In the original NGSIM model, the lane changing desire threshold for mandatory lane changes is set as a constant value for all modeled drivers. It means that drivers that need to make a mandatory lane change will start to search for the gap at the same location. It does not capture the random variation of drivers' gap searching behavior—some may start looking for gaps at further upstream locations and others at downstream locations. To address this issue, we make the lane change desire threshold a normally distributed random variable. Each driver is assigned a random threshold value based on the user-specified mean and standard deviation.

In addition, the original NGSIM model does not consider the cooperative lane change behaviors when freeway drivers are approaching a merging on-ramp. Drivers in the mainline lanes typically move toward the median lane on the left in anticipation of potential conflicts from merging traffic. The existing discretionary or mandatory lane-changing algorithm cannot simulate such behaviors. Hence, we have added a new lane-changing model to simulate such cooperative behavior. Under the updated simulation, the driver in the right-most mainline lane will start to continuously examine the gaps in the left adjacent lane when the driver is 100 meters upstream of a merging on-ramp. If the current gap is not acceptable, the driver will skip that gap and examine the next downstream gap. This process will continue until the driver successfully makes a lane change or passes through the merging area if the driver cannot successfully make the lane changes.

5 CALIBRATION RESULTS

After the model fine tuning by trial and error, we have determined the appropriate values of the behavior parameters as shown in Table 2. As suggested earlier, the parameter values were adjusted to accommodate the unique downstream weaving bottleneck at US-50 and the geometric differences at the upstream Sheldon Road bottleneck. The bottleneck at US-50 required lower reaction time to account for the last-minute and aggressive lane changes while the bottleneck at Sheldon Road required slightly longer reaction time to account for the poor visibility.

Table 2. Calibrated Behavior Parameters.

Parameter	*US-50 Bottleneck*	*Sheldon Rd Bottleneck*	*Rest of Network*
Mean reaction time (s)	0.4	1.0	0.8
Maximum acceleration (m/s^2)	2.0	2.0	2.0
Maximum deceleration (m/s^2)	-4	-4	-4
Mean headway (s)	1.4	1.4	1.4
DLC desire threshold	0.15	0.15	0.15
Lane changing anticipatory distance for exiting (km)	0.6	1.5	1.5

We have carried out 10 simulation replications with the identified behavior parameters. Each simulation replication covers the 8-hour period from 4:00 AM to 12:00 PM. Vehicle count and mean speed at each detector were recorded every 5 minutes. Comparison of the observed and simulated vehicle counts is summarized in Table 3. It can be seen that on average, the simulated flows satisfied the calibration criteria.

Table 3. Calibration of Freeway Flows.

Freeway: 5-min flows of SR-99 Northbound					
Detector Location (post-mile)	Target	Cases	Cases Met	% Met	Target Met?
287.3	GEH < 5 for > 85% of k	960	951	99.1%	Yes
287.6	GEH < 5 for > 85% of k	960	934	97.3%	Yes
289.3	GEH < 5 for > 85% of k	960	945	98.4%	No
289.4	GEH < 5 for > 85% of k	960	944	98.3%	Yes
290.0	GEH < 5 for > 85% of k	960	951	99.1%	Yes
290.7	GEH < 5 for > 85% of k	960	950	99.0%	Yes
291.5	GEH < 5 for > 85% of k	960	891	92.8%	Yes
291.9	GEH < 5 for > 85% of k	960	872	90.8%	Yes
292.4	GEH < 5 for > 85% of k	960	897	93.4%	Yes
292.8	GEH < 5 for > 85% of k	960	907	94.5%	Yes
294.0	GEH < 5 for > 85% of k	960	909	94.7%	Yes
294.7	GEH < 5 for > 85% of k	960	935	97.4%	Yes
295.3	GEH < 5 for > 85% of k	960	929	96.8%	Yes
296.0	GEH < 5 for > 85% of k	960	866	90.2%	Yes
Overall	GEH < 5 for > 85% of k	13440	12881	95.8%	Yes

Figure 3 summarizes the simulated vs. observed queue propagation. The contour plots show the 5-minute average speeds at the detectors throughout the selected peak period for the field data (left) and the aggregate of all 10 replications (right). The simulation reproduced the field observed peak duration and the length of queue fairly accurately, with the exception of the most upstream bottleneck at Sheldon Rd., which showed slightly shorter simulated duration and slightly less simulated queue propagation. Despite these minor discrepancies, the overall congestion pattern and the presence of severe speed reduction were accurately replicated.

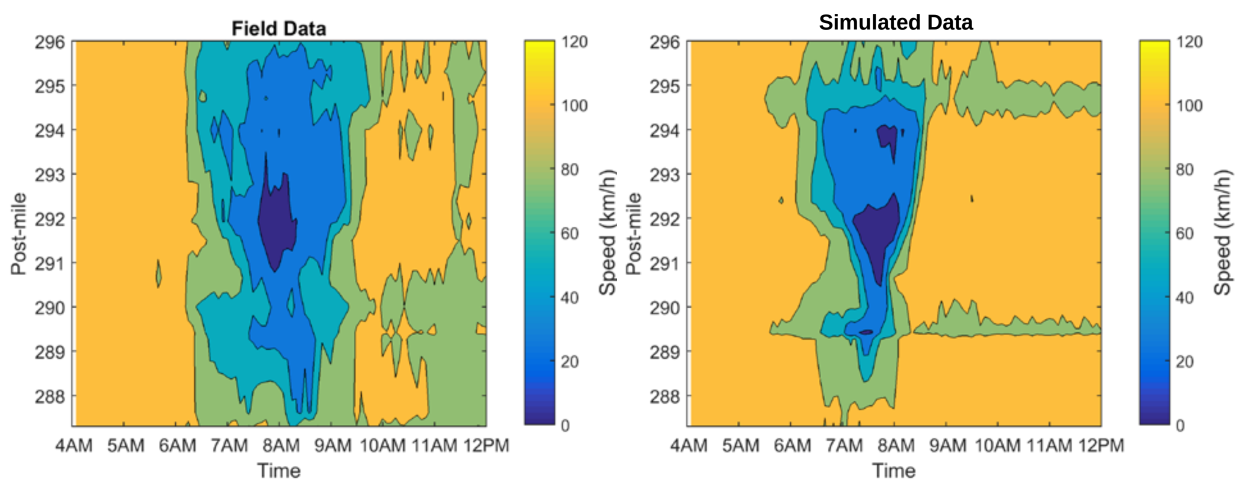

Figure 3. Field and Simulated Speed Contour Plots.

Figure 4 shows the field observed and simulated flow-density relationships of a four-lane mainline section at the most important bottleneck, the Florin Rd. on-ramps at milepost 294. In both replications, the simulated data provided near-perfect match in the uncongested state, as well as good representation of the congested state. The fundamental diagram shows that the simulated maximum capacity is slightly lower than the observed maximum capacity, otherwise, the simulation provided a good representation of the overall macroscopic traffic behavior.

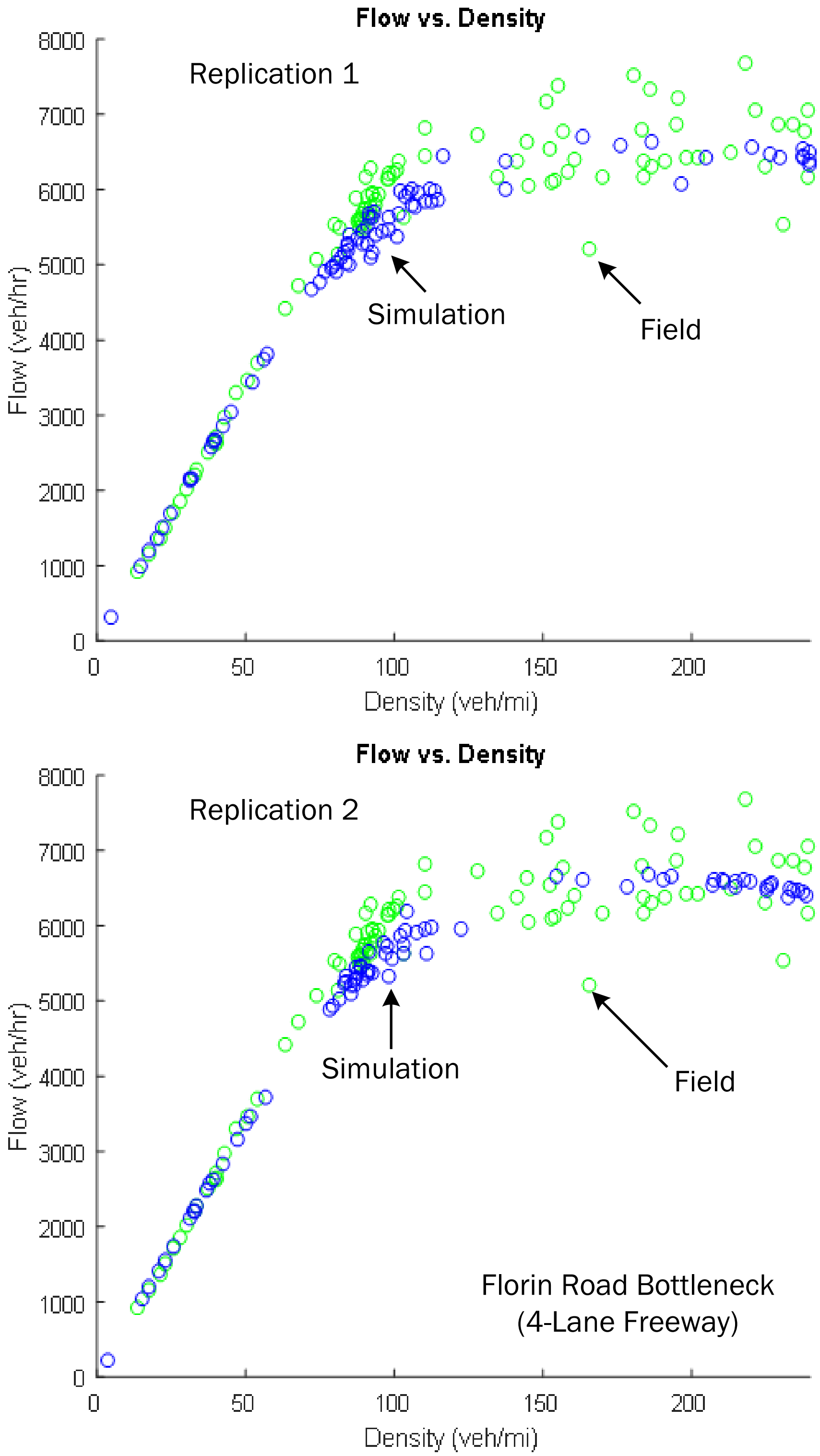

Figure 4. Fundamental Diagram for A Typical Bottleneck Plotted for Two Different Simulation Replications (Different random number sequences).

6 CONCLUSIONS

The calibration effort reported here has produced a model of a complicated freeway corridor that has sufficient fidelity to be usable for the intended purpose of the current project, which is to provide a basis for evaluating the traffic impacts of ACC and CACC vehicles when they are operated under a variety of management strategies and as varying fractions of the vehicle population. This is already at the frontier of the current state of the art in traffic micro-simulation modeling.

We have recommended some future improvements to the NGSIM model, despite its ability to reasonably reproduce the macroscopic traffic performance of a complex freeway corridor. For example, the NGSIM model has separate algorithms for simulating movements of the on-ramp merging traffic and off-ramp diverging traffic. It was expected that the model could depict the traffic dynamics at the weaving area by activating the merging and diverging algorithms simultaneously. This cannot be accomplished because these algorithms currently could not model the complicated driver interactions unique to the weaving sections. For example, there are no rules to determine the lane-changing priority for the diverging vehicles, merging vehicles, and mainline vehicles trying to create gaps for the merging and diverging traffic. Absence of such rules can lead to very conservative gap acceptance and yielding behaviors of the modeled drivers. Consequently, the modeled traffic flow may exhibit unrealistically low speeds.

The current NGSIM model can simulate freeway drivers' yielding behavior as they cooperatively accommodate the merging and diverging vehicles. Nonetheless, the same algorithm is adopted to capture the yielding behavior in both the merging and diverging cases. In real-world traffic, the drivers in the mainline lanes are usually more likely to yield to the merging vehicles than the diverging vehicles. The timing of the cooperation and the deceleration applied by drivers in the mainline lanes might be different as well. Without a proper algorithm to describe the yielding behavior, the simulation model sometimes creates unrealistic bottlenecks close to the end of the off-ramps. The model may also generate unrealistically long queues on the on-ramps.

To address the above limitations, we propose the following for future research:

- Develop a function to model the interactions of merging, diverging and mainstream vehicles within the weaving area. This function will be able to determine vehicles' priority of using the time and space resource of the weaving segment based on their speed, relative location and the traffic condition.
- Create separate functions to depict the cooperative driving behaviors for the merging and diverging case. This function will capture the negotiation process between the merging/diverging vehicle and the mainline vehicle. This function will be able to allow a merging/diverging vehicle to identify an acceptable gap without waiting for an unrealistically long time at the end of the on/off-ramps.
- If there are resources to perform field or simulator-based tests in the future, experiments can be designed to measure the actual driving behaviors in the above scenarios. The

measured data can help to determine the behavior parameters used in various car-following and lane-changing cases.

If the above tasks are successfully completed, the resulting models are expected to even more realistically reproduce the traffic flow dynamics (especially the congestion states) at a freeway site. The work will not only supplement the development of microscopic traffic flow theory, but also lay a solid ground for designing a simulation testbed for testing various advanced transportation technologies such as Connected and Automated Vehicles.

References

1. Caltrans PeMS. http://pems.dot.ca.gov/. Accessed on October 24, 2016.