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Conference Paper · October 2014

DOI: 10.1109/ITSC.2014.6958199

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2014 IEEE 17th International Conference on
Intelligent Transportation Systems (ITSC)
October 8-11, 2014. Qingdao, China

A Methodology for Calibrating Microscopic Simulation for Modeling Traffic Flow under Incidents

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***Abstract* - Incidents cause significant delays and their impacts on traffic flow need to be modeled to evaluate and mitigate them. The objective of this paper is to provide a methodology for modeling incidents in microscopic traffic simulation environment. Queue lengths and shockwave propagation speeds resulting from incidents obtained from analytical kinematic wave theory (Lighthill – Witham – Richard or LWR) were used to calibrate incident model in microscopic traffic simulation (VISSIM). A two lane freeway along the HRBT corridor was taken as a test bed. Fundamental diagram corresponding to the corridor was developed using Van-Aerde traffic mode and was used as an input to LWR. Queue length as a result of incidents blocking only one lane and both lanes were analyzed. Reduction in capacity, as suggested in HCM, was considered in the process of incident model calibration. The results suggest that a well calibrated incident model in VISSIM is capable of reproducing the queue lengths and shockwaves obtained from LWR and thus can be used for understanding the impact of incidents on freeways and testing alternative traffic management strategies, e.g., minimize delay or emissions.**

I. INTRODUCTION

Incidents negatively impact freeway operations by blocking a shoulder, a lane or the entire road resulting in degradation of freeway throughput. Highway Capacity Manual (HCM) states that more than 50% of congestion and delays are caused by incidents [1]. Considering the impact of incidents on freeway operations and the magnitude of delay they are responsible for, it remains one of the least investigated topics in the domain of Traffic Engineering. Several analytical models of traffic theory have been proposed to outline incident-induced shockwave profiles. Such profiles are useful to describe key variables required for freeway performance evaluation such as queue length, delay and total time needed for the queue to dissipate.

Given the fact that the majority of shockwave profiles are developed based on analytical models, they are deterministic in nature. This makes them prone to several uncertainties related to traffic dynamics, e.g., variation in arrival pattern of vehicles, flow rates and the presence of on- and off-ramps and auxiliary lanes. Microscopic traffic simulation models are proposed to overcome this shortcoming and have gained popularity in the field of Transportation Engineering because such models provide a flexible environment for assessing alternative traffic management strategies. Many researchers have investigated the potential of microscopic traffic simulation models to address the challenges related to mobility and efficiency of transportation systems [2]–[4]. However, even though considerable progress has been made on micro-simulation models, there seems to be lack of standard methodology of modeling incidents in microscopic traffic simulation models.

Accurate estimation of shockwave profiles and queue propagation during incidents are vital for incident management strategies, traffic rerouting and assisting travelers in making informed decisions. However, successful representation of incidents in microscopic traffic simulation models requires them to be rigorously calibrated against valid measurements.

Modeling incidents is important as it provides the basis for analyzing the impact of the incidents. Keeping this in mind, the prime objective of this study is to develop a methodology for simulating incidents on freeways in microscopic traffic simulation environment. The secondary objective is to bridge the gap between analytical and microscopic traffic simulation models of incidents in a complementary way. This approach can be used to locate the end of a queue which, in turn, can be used as an offline decision support tool to practitioners of traffic management and operations.

The central theme of this study is to reproduce shockwave profiles in a microscopic traffic simulation. For this purpose, the Lighthill – Witham – Richard (LWR) kinematic wave theory [5] [6] is used as a ground truth for calibrating shockwave profile and queue length as a result of lane closure (incidents) on freeways in micro-simulation. However, successful representation of shockwaves in LWR requires reliable input variables from the fundamental traffic diagram. In this paper, a four-parameter single-regime fundamental diagram known as Van-Aerde traffic model is used. Moreover, in the process of modeling incidents in VISSIM, an effort was made to reproduce the capacity reduction of freeways due to incidents as stated in HCM.

II. LITERATURE REVIEW

A. LWR Macroscopic Model

The first dynamic traffic model that used the fundamental diagram and traffic conservation law was proposed by Lighthill, Whitham [5] and Richards [6] and

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is known as LWR-model. This traffic flow model is based on fluid dynamics continuity equation and is known as the first order LWR traffic flow model.

LWR model has been employed in many traffic engineering applications, e.g., to analyze traffic flow dynamics [7], and establish a shockwave profile model for traffic flow on congested urban arterials [8]. Cetin [9] developed a methodology based on kinematic wave model to estimate the queue dynamics at signalized intersection. Lu and Skabardonis [10] formulated a numerical algorithm to estimate the shockwave speed on freeway based on vehicle trajectory data. Moreover, LWR model has been successfully applied in quantifying incident-induced delay [11], analyzing traffic perturbations and maximum queue lengths [12] and estimate the time-space domain of the influence of incidents on freeways [13]. All these works show that LWR model has been identified as an important tool in analysis of incidents and queue lengths. Its simplicity and ability to reproduce freeway shockwaves makes LWR attractive in the field of traffic engineering.

However, some properties of observed traffic pattern cannot be explained by the kinematic wave theory, e.g., the start-stop phenomena in congested traffic and the heterogeneous traffic composition [14].

B. Microscopic Simulation Model

Microscopic simulation models have been widely used in the area of Transportation Engineering. However, the reliability of these models depends on how well they are calibrated. Many studies were focused on calibration of such models , for example [2]–[4]. Some of them proposed methodology for calibrating microscopic traffic simulation models [15]–[17]. However, relatively little effort was made to represent congestion induced by incidents in micro-simulation.

III. Methodology

In this study, VISSIM was used as a microscopic traffic simulator. A VISSIM model is calibrated for normal traffic conditions using volume and speed of traffic as measures of effectiveness. The model is validated under incidents scenarios and queue length estimated from the LWR model was used as a measure of effectiveness in the validation process. To determine the input variables for LWR, the Van Aerde traffic model is used to develop a fundamental diagram for the HRBT corridor. The methodological approach of this study is summarized in Figure 1.

A. Fundamental Diagram - Van Aerde Traffic Flow Model

Fundamental diagram (FD) attempts to formulate valid relationships between volume, speed and density of traffic field is an important input for LWR model. In this study Van Aerde model, which is a single-regime model, is used [18]. It is a combination of the Greenshield's and Pipe's models which was proposed by Van Aerde and Rakha [19] and [20].

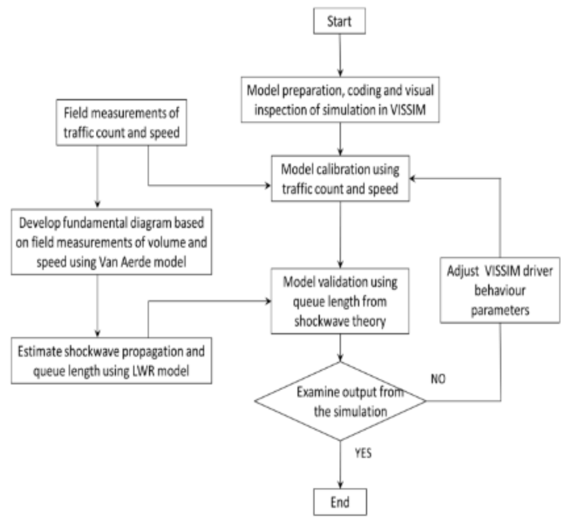

Figure 1. Methodological approach of the study

It is a four-parameter model which gives it an additional degree of freedom to be calibrated to different traffic conditions, as opposed to Greenshield which has only two parameters [18]. Rakha [21] compared the Van Aerde model with other single- and multi-regime models and demonstrates that it provides superior fit to field data corresponding to different road types and aggregation intervals. Several authors have used the Van Aerde model, e.g., [22], [23], [24] and [25]. Mathematical formulation of the Van Aerde model is discussed in [20].

The Van Aerde model has been implemented in heuristic SPD_CAL solver. It computes the free-flow speed, speed at capacity, capacity and jam density to fit a model relating speed, volume and density corresponding to a given traffic data. In this paper, SPD_CAL is used to formulate the fundamental diagram for the HRBT corridor.

B. LWR Shockwaves

Figure 2 (a) shows the fundamental diagram and the shockwave speed due to incident. When an incident happens (point A), the capacity will drop to point B. Point C represents the capacity of the link. Figure 2 (b) shows the shockwave profile of the link due to the incident. Assuming r denotes the duration of incident, from Figure 2, $r = t_F - t_E$, while $t_G - t_F$ is the total time from lane opening to the time of last vehicle joining the queue, $t_H - t_F$ is the total time from lane opening to normal condition and Q_m is the queue length.

Using the fundamental diagram, the shockwave formation in the time-space diagram is drawn (see Figure 2). The shockwave speed w_{AB} is the queue formation speed as a result of freeway lane closure due to incident. Therefore w_{AB} can be defined as queuing shockwave speed and is calculated as shown in equation (1).

$$w_{AB} = \frac{q_B - q_A}{k_B - k_A} \tag{1}$$

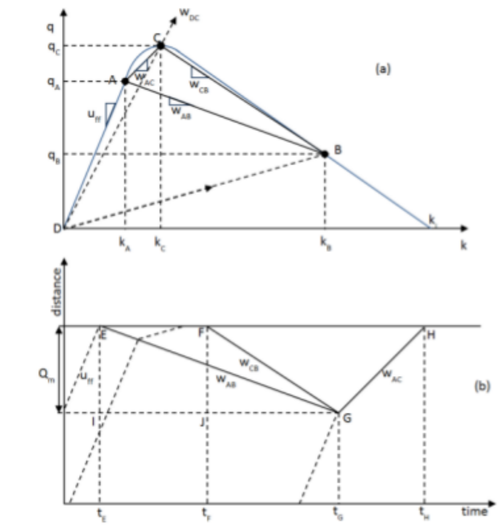

Figure 2. The fundamental diagram and incidents shockwave profile

As the incident lane clears, vehicles begin to discharge at saturation flow rate forming another shockwave with a speed of w_{CB} moving upstream. This shockwave is defined as discharge shockwave speed, formulated as equation (2).

$$w_{CB} = \frac{q_B - q_C}{k_B - k_C} \tag{2}$$

The position of the last vehicle in the queue defines the maximum queue length. At this point the queuing and discharge shockwave meet and a third shockwave with shockwave speed w_{AC} is generated propagating downstream. This shockwave is formed when the upstream vehicle reaches the vehicles that are just leaving the queue with speed at capacity. This shockwave speed is defined as departure shockwave, shown by equation (3)

$$w_{AC} = \frac{q_C - q_A}{k_C - k_A} \tag{3}$$

With the known shockwave speeds w_{AB} and w_{CB} and the length of lane closure duration (r), by utilizing Δ EGI and Δ FGJ in Figure 2, yields

$$w_{CB} = \frac{Q_m}{t_{G-}t_F} \tag{4}$$

$$w_{AB} = \frac{Q_m}{r + (t_{G-}t_F)} \tag{5}$$

By equating the Q_m from equation (4) and (5), the total time from lane opening to the time of last vehicle dissipating from the queue, t_G-t_F (minutes) can be determined as shown by equation (6),

$$t_{G-}t_F = \frac{w_{AB}.r}{w_{CB} - w_{AB}} \tag{6}$$

The queue length can be determined by the following procedure,

$$Q_m = w_{CB}.t_{G-}t_F \tag{7}$$

Substituting $t_{G-}t_F$ from the equation (6) into equation (7), the queue length (kilometers) can be determined as,

$$Q_m = \frac{r}{60}\frac{|w_{CB}|.|w_{AB}|}{|w_{CB}| - |w_{AB}|} \tag{8}$$

IV. Case Study

A. Network description

The Hampton Roads Bridge Tunnel (HRBT) corridor, connecting Norfolk and Hampton in Virginia, was taken as test bed for this study. This corridor was selected because vehicle count is conserved for a considerable length, i.e., there are no on- or off-ramps.

B. Traffic Flow Characteristic

Traffic data collected by three sensors was used to build the fundamental diagram corresponding to the HRBT corridor. The data was comprised of traffic speed and volume. Measurements that were affected by incidents were removed from the analysis. A factor of two was used to convert heavy goods vehicles (HGV) to the equivalent number of passenger cars.

The fundamental diagram corresponding to the HRBT corridor was developed using the Van Aerde traffic model and demonstrated that free-flow speed is 98 km/h, the speed at capacity is 83 km/h, the capacity is 1650 veh/lane/hour and the jam density is 150 veh/km/lane as shown in Figure 3.

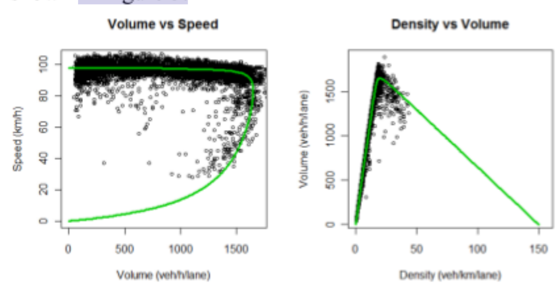

Figure 3. Van Aerde Fundamental Diagram using the empirical data

C. Scenario

In this study, two incident scenarios were setup. The first scenario is an incident blocking only one lane of a two-lane freeway and the second is an incident blocking all lanes. These scenarios were tested under relatively congested traffic conditions with volume-to-capacity ratio of 0.7, or level of service C, which is assumed to be representative of all traffic conditions as it lies in the transition from heavily congested to non-congested. Each scenario was tested for multiple incident durations ranging from 5 to 40 minutes with an increment of 5 minutes.

D. Calibration of the base model

Averages of speed and vehicle count aggregated every 5-minute, from the simulation and observed measurements from three detectors were used as measures of effectiveness in the calibration process. Considering the objective of the study is to setup a methodology for incident modeling in VISSIM, queue lengths as a result of incidents are used in the model validation process. The process of calibration and validation is shown in Figure 1.

VISSIM provides a set of adjustable car-following driver behavior parameters [26]. In this study, VISSIM driver behavior parameters CC0, CC1 and CC2 are adjusted to meet the measures of effectiveness for calibration and validation as they are the most influential car-following parameters for freeway traffic operations [27]. CC0 (Standstill distance) is the desired distance between two stopped vehicles. This parameter is useful for adjusting the queue length. CC1 (Headway Time) is the desired time headway between the leading and following vehicles in seconds and is used to adjust freeway capacity. CC2 (Following Variation) is the additional safety distance and impacts capacity of roadways.

As a performance indicator of the calibration process, Geoffrey E. Heavers (GEH), which considers both the relative and absolute differences between the simulated and observed datasets, is used as shown in Equation (9), [28]. A micro-simulation model with GEH value less than 5 is considered to be a good fit between simulated and observed measurements, value in the range 5-10 requires further investigation while value above 10 is assumed to be a bad fit [28].

$$GEH = \sqrt{\frac{(simulated - observed)^2}{0.5 * (simulated + observed)}} \qquad (9)$$

E. Modeling incidents and capacity reduction in VISSIM

In this study, traffic light with a single-control fixed time signal (red and green sequence) was used to simulate incidents. This allows simulating exact incident duration. Traffic signals were installed in one or both of the lanes according to the incident scenario. VISSIM feature 'queue counters' were used to determine the queue length from the location of traffic signal.

According to HCM, the effect of incidents on freeways capacity depends on the number of lanes closed and the total number of lanes the freeway comprises. The proportion of the available capacity for a two-lane freeway under incident conditions blocking one lane is 35% due to rubbernecking effect [29]. As discussed in Section IV.B, the capacity of the modeled freeway corridor was estimated to be 1650 veh/h/lane. Therefore, the capacity of the section affected by one lane closure will be nearly 580 veh/h/lane. However, when incident blocks the entire road, the available capacity is obviously 0%. In this study, capacity reduction was modeled by using the VISSIM feature called 'Reduced Speed Areas' which temporarily decreased the speed of vehicles. The speed on the 'reduced speed area' was set to be 23 km/h so that the maximum count of vehicles downstream of the incident is 580 veh/h/lane, i.e., 35% of the capacity.

V. Results and Discussion

For a microscopic model to be useful, it has to be calibrated to replicate the actual traffic conditions. Therefore, CC0 was set to be 3.8 so that the queue length obtained from the VISSIM model was nearly the same as what is suggested by the LWR model. This was followed by adjusting CC1 and CC2 parameters (1.5 sec and 7.5 m respectively) so that the actual operations of the freeway traffic are replicated. The goodness-of-fit between simulated and observed measures provided average GEH values of 1.175, 0.989 and 0.828 for traffic counts as well as 2.104, 1.299 and 0.173 for traffic speeds corresponding to three detectors on the corridor. The overall GEH values of the model were 0.997 for traffic count and 1.192 for traffic speed with standard deviations of 0.711 and 1.150 respectively.

Matching the queue lengths obtained from the VISSIM model and LWR model was performed by changing the value of the stand still distance (CC0) parameter in VISSIM. The changes in absolute percentage difference between the queue lengths from VISSIM model and LWR model is shown in Figure 4 and the most suitable value for CC0 was found to be 4.0. The model was once again checked for any changes in the speed and volume outputs from the simulation and there were no significant changes.

In order to have representative data, each simulation was run 10 times. The objective of calibrating and validating the VISSIM model is that the queue length obtained from LWR is within the 95% confidence interval (CI) of the queue length obtained from the VISSIM.

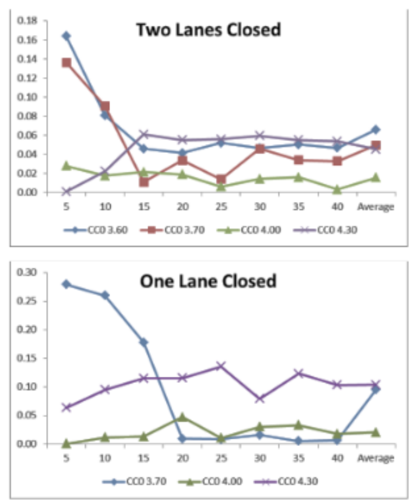

Figure 4. Absolute percentage difference between queue lengths from LWR model and VISSIM model

Considering the output from VISSIM is stochastic, the queue length corresponding to each simulation will be different. The average of the queue lengths obtained from all simulation runs was taken to be the estimated queue length from the simulation. The standard deviation of the queue lengths was used to estimate the upper and lower bounds of the 95% CI as shown in Equations (10) and (11).

$$\text{Upper CI bound} = \bar{x} + t_{\frac{\alpha}{2}} \frac{\sigma}{\sqrt{n}} \qquad (10)$$

TABLE 1. THE RESULT OF ANALYTICAL LWR MODEL AND SIMULATION MODEL

A. Two Lanes Closed

Incident duration minute	w_{AB} kph	w_{CB} kph	w_{AC} kph	$Q_{max\text{-}LWR}$ km	$Q_{max\text{-}VISSIM}$ km	Std of $Q_{max\text{-}VISSIM}$	Lower bound of 95% CI of $Q_{max\text{-}VISSIM}$	Upper bound of 95% CI of $Q_{max\text{-}VISSIM}$	$Q_{max\text{-}LWR}$ between CI bounds of $Q_{max\text{-}VISSIM}$
5	-8.36	-12.68	61.88	2.05	1.99	0.46	1.66	2.32	Yes
10	-8.36	-12.68	61.88	4.09	4.02	0.34	3.78	4.27	Yes
15	-8.36	-12.68	61.88	6.14	6.27	0.38	6.00	6.54	Yes
20	-8.36	-12.68	61.88	8.19	8.34	0.47	8.00	8.77	Yes
25	-8.36	-12.68	61.88	10.23	10.30	0.36	10.04	10.56	Yes
30	-8.36	-12.68	61.88	12.28	12.45	0.28	12.25	12.65	Yes
35	-8.36	-12.68	61.88	14.33	14.55	0.31	14.33	14.77	Yes
40	-8.36	-12.68	61.88	16.37	16.42	0.29	16.21	16.63	Yes

B. One Lane Closed

Incident duration minute	w_{AB} kph	w_{CB} kph	w_{AC} kph	$Q_{max\text{-}LWR}$ km	$Q_{max\text{-}VISSIM}$ km	Std of $Q_{max\text{-}VISSIM}$	Lower bound of 95% CI of $Q_{max\text{-}VISSIM}$	Upper bound of 95% CI of $Q_{max\text{-}VISSIM}$	$Q_{max\text{-}LWR}$ between CI bounds of $Q_{max\text{-}VISSIM}$
5	-6.17	-12.52	61.88	1.01	1.01	0.11	0.93	1.09	Yes
10	-6.17	-12.52	61.88	2.03	2.00	0.25	1.82	2.18	Yes
15	-6.17	-12.52	61.88	3.04	3.08	0.24	2.91	3.24	Yes
20	-6.17	-12.52	61.88	4.05	4.24	0.30	4.03	4.46	Yes
25	-6.17	-12.52	61.88	5.06	5.12	0.31	4.90	5.34	Yes
30	-6.17	-12.52	61.88	6.08	6.26	0.36	6.00	6.52	Yes
35	-6.17	-12.52	61.88	7.09	7.33	0.41	7.04	7.62	Yes
40	-6.17	-12.52	61.88	8.10	8.25	0.35	8.00	8.50	Yes

$$\text{Lower CI bound} = \bar{x} - t_{\frac{\alpha}{2}} \frac{\sigma}{\sqrt{n}} \tag{11}$$

Where $\bar{x}$ is average queue length, σ is the standard deviation of the queue length, α is 1 - confidence level, $t_{\frac{\alpha}{2}}$ is t value of $\frac{\alpha}{2}$ and n is the number of simulation runs.

TABLE 1 shows the queue length obtained from the analytical LWR and VISSIM models for both scenarios, i.e., when an incident closes one or both lanes. The result shows that the queue lengths from the analytical LWR model for both scenarios and for all incident durations are within the 95% confidence interval of the result from simulation model. This shows that the VISSIM model is reproducing the shockwave propagations as desired.

A regression line was fitted to resulting queue lengths of different incident durations and the dotted lines are the upper and lower bounds of the confidence interval. The results suggested that for traffic flow of 1155 veh/h/lane (i.e., volume-to-capacity ration of 0.7 or level of service of C) the rate of increase in queue length as a result of an incident blocking two lanes of the freeway is 0.415 km for every minute increase in incident duration. Similarly, when an incident closes only one lane of the freeway, the rate of increase of queue length for every minute increase in incident duration is 0.209 km. However, it should be noted that these rates are applicable only when vehicles are conserved or there is no on- or off-ramp.

Comparing the methodological approach and the results of this study with the works of other researchers is difficult because of the fact that most of the studies that aimed at modeling incidents on microscopic traffic simulation focused only in controlled closure of freeway lanes, e.g., work zones [30]. Other studies [31] focused on simulating the reduction in capacity due to incidents but did not reproduce the shockwave propagation and queue length following the incident events. The study of [32] suggested that VISSIM lacks the ability to model the rubber necking behavior of drivers; however, as shown in this study, rubbernecking effect can be modeled by using reduced speed areas.

VI. Summary and Conclusion

This study introduced a methodology for developing incident-specific models in a microscopic traffic simulation environment. Moreover, it showed how capacity reduction due to rubbernecking can be modeled in VISSIM and how incident models can be calibrated. In addition, this study showed how analytical models and microscopic traffic simulation models can complement each other for better analysis of traffic flow theory. The results indicate that queue length can be correctly estimated in simulation models during both partial and complete closure of freeways for multiple durations.

The advantage of this study is that it coupled analytical models of traffic theory with microscopic traffic simulation models. This approach is beneficial for a quick and immediate estimation of shockwave propagation, queue length and location as well as queue dissipation time, e.g., when there is no immediate loop detector to locate the end of the queue. This in turn provides background conditions for implementation of alternative traffic management strategies in microscopic traffic simulation models. This allows experimenting the feasibility of alternative traffic management strategies under controlled environment without the effect of extraneous components affecting the flow of traffic, e.g., weather conditions, mechanism of road-rule enforcement and transportation policy measures.

The limitation of this study is that queue lengths obtained from analytical models were used as ground truth for calibrating the VISSIM model under incident scenario. For example, in LWR vehicle acceleration and deceleration are not considered in estimating the speed of propagation of shockwaves. Moreover, the objective of this study is limited to primary incident only and no attention was given to secondary incidents. Another limitation of this study is that lane changing behavior of drivers around the site of incidents is not considered as this requires intensive data, e.g., vehicle trajectories.

Future works related to this study will focus on comparing queue lengths obtained from VISSIM model and what is observed in the field. In addition, modeling of secondary incidents will be attempted for complete analysis of incidents on microscopic traffic simulation models. Moreover, this study will be coupled with optimal traffic management strategies with the objective of minimizing the impact of incidents, e.g. ramp metering and variable speed limits.

VII. Acknowledgment

The authors would like to thank Transportation Research Institute of Old Dominion University (TRI, ODU), Virginia Department of Transportation (VDOT) and Mr. Seth Schipinski for their support to this study.

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