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Analysis of Bottleneck Traffic Capacity Drop using Aimsun Simulation

THESIS

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for the degree of

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by

Aaron Ka-Ho Chong

Thesis Committee:
Professor Wenlong Jin, Chair
Professor Stephen Ritchie
Professor R. (Jay) Jayakrishnan

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ABSTRACT OF THE THESIS

Analysis of Bottleneck Traffic Capacity Drop using Aimsun Simulation

By

Aaron Ka-Ho Chong

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Professor Wenlong Jin, Chair

Capacity must be understood to mitigate congestion and improve conditions on constrained road networks. Analytical and empirical studies have been used to examine capacity drop, but there are weaknesses for both; empirical work suffers from insufficient data and non-variable parameters while analytical research stems from simplified assumptions on the aggregate level. Simulation is often considered inaccurate, but can model microscopic properties and modify factors not possible with the other two. Therefore, this research serves to bridge the gap between analytical and empirical work by analyzing factors that affect capacity drop through microscopic simulation.

A short review was presented on the Gipps car following model and on Aimsun modelling. Simulation data for an activated bottleneck was obtained and plot against the triangular fundamental diagram to find initial capacity. Cumulative count curves were transformed using an oblique coordinate system to accurately observe capacity and determine average discharge flow-rate. Furthermore, speed profiles were obtained to assist in verifying system properties.

Capacity drop features were reproduced in simulation: a reduction in flow-rate was observed when upstream demand was higher than downstream capacity and discharge flow-rate was identified as upstream demand when less than downstream capacity. The difference between
the capacity and discharge rate yielded a capacity drop ratio of 5.52%. Speed profiles were generated to observe near stationary patterns as vehicles approached the bottleneck. Analytical and simulated results demonstrated that acceleration affects capacity to a higher degree, but merge distance is closer in magnitude between the two cases.
Chapter 1

Introduction

Congested traffic during peak periods is a growing concern for both drivers and engineers alike. Drivers are faced with increased commute times in addition to safety concerns from stop-and-go traffic such as cascading incidents from collisions. From the engineering viewpoint, there is the constant task of optimizing existing control with a space constraint and ever increasing population size. Congestion occurs when demand exceeds the capacity of a road section. Therefore, capacity must be understood before congestion can be remedied.

Capacity as a metric has had a variety of definitions ranging from the state of flow that can be achieved daily (Hall & Agyemang-Duah, 1991) to the maximum amount of vehicles that can pass through the system under prevailing roadway, traffic, and control conditions (National Research Council (U.S.), 2010). Common changes in roadway geometry involve bottlenecks, typically in the form of ramp merge locations or lane reductions. A bottleneck is considered activated when the upstream demand exceeds the downstream capacity. The capacity drop phenomenon often coincides with an activated bottleneck, where the capacity in a congested state is lower than that of an uncongested state. The difference between the congested and uncongested capacity is often referred to as the capacity drop magnitude. The capacity
drop magnitude has been observed to range from: 3% (Banks, 2010), 8-10% (Cassidy & Bertini, 1999), and 18% (Chung et al., 2007) as a non-exhaustive list of examples. Therefore, modeling capacity at bottlenecks is important because it allows engineers to optimize control for efficiency and improve disaster prevention strategies.

Analytical studies have used traffic flow models as a framework to examine capacity in depth. Most notably, the first order macroscopic model presented by (Lighthill & Whitham, 1955; Richards, 1956) laid the foundation for a majority of modeling that has followed. Microscopic models such as the one introduced by (Gipps, 1981) have been used in a similar fashion. Empirical work on capacity drop has been undertaken, from examples such as (Bertini et al., 2005) and (Ismail & Altun, 2008) with further analysis of vehicle properties' effects through (Yuan et al., 2015). However, there are weaknesses to both lines of research in the form of insufficient data and simplified assumptions for theoretical and empirical analysis. Thus, the research presented here is an attempt to bridge the gap between the two by examining how select vehicle behavior affects capacity drop through simulation.
Chapter 2

Literature Review

2.1 Gipps’ Car Following Model

Traditional car following models were variations of the form:

\[ a_n(t) = l_n \frac{[v_{n-1}(t) - v_n(t)]^k}{[x_{n-1}(t) - x_n(t)]^m} \]  \hspace{1cm} (2.1)

where \( n - 1 \) is the lead vehicle and \( n \) is the following vehicle. The following vehicle’s acceleration, \( a_n(t) \) is a function of the vehicle’s speeds \( v_n \) and locations \( x_n \) with calibrating parameters \( l, k, \) and \( m \).

The rationale behind this hypothesis was that the acceleration (or behavior) of the follower vehicle was determined by the difference between the spacing and speeds from the leader vehicle (Gazis et al., 1961). Gipps had created his new model on the idea that it is valuable to have iterations of acceleration, speed, and position be a fraction of reaction time (Gipps, 1981). Based on this principle, he had created a model based on three different aspects:
1. A vehicle $n$ should not exceed the desired speed $V_n$.

2. A variable acceleration rate such that vehicle $n$ should increase to its desired speed and then reduce to 0 as it approaches vehicle $n$’s desired speed.

3. Vehicle speed $n$ is controlled by the leader vehicle when decelerating.

The model focuses on two states, one as the uncongested state and the other as the congested state. The first equation of the model was obtained via combination of the first two constraints and the result of fitting to prior data:

$$v_n(t + \tau) \leq v_n(t) + 2.5a_n\tau(1 - \frac{v_n(t)}{V_n})\sqrt{0.025 + \frac{v_n(t)}{V_n}}$$

(2.2)

The second half is based around the third constraint and is applicable when under congested driving conditions. Stopping distances are calculated for both the leader and follower with the condition that the follower’s distance traveled cannot exceed the leader’s (with an additional safety margin $s_{n-1}$). A safety factor to $\tau$, $\theta$ is added such that the follower vehicle continues traveling at $v_n(t + \tau)$ before reacting to the leader vehicle.

$$v_n(t + \tau) \leq b_n\tau + \sqrt{b^2\tau^2 - b_n(2[x_{n-1}(t) - s_{n-1}(t)] - v_n(t)\tau - \frac{v_{n-1}^2(t)}{b})}$$

(2.3)

The final formulation is as follows:

$$v_n(t+\tau) = \min \begin{cases} v_n(t) + 2.5a_n\tau(1 - \frac{v_n(t)}{V_n})\sqrt{0.025 + \frac{v_n(t)}{V_n}} \\ b_n\tau + \sqrt{b^2\tau^2 - b_n(2[x_{n-1}(t) - s_{n-1}(t)] - v_n(t)\tau - \frac{v_{n-1}^2(t)}{b})} \end{cases}$$

(2.4)
2.2 Converting Microscopic to Macroscopic using Steady State Condition

The steady state condition describes that all vehicles travel at a constant velocity without accelerating. Therefore, the first half of Gipps' model equates to constant velocity as $a_n = 0$.

The second half of Gipps’ formulation is then taken:

$$v_n(t + \tau) = b_n\tau + \sqrt{b^2\tau^2 - b_n(2[x_{n-1}(t) - s_{n-1}(t) - x_n(t)] - v_n(t)\tau - \frac{v_{n-1}(t)^2}{b})} \quad (2.5)$$

Solving for the speed-headway function yields:

$$\left(\frac{b}{b'} - 1\right)u^2 - \frac{3}{2}\tau u - h_j = -h \quad (2.6)$$

And after converting to km/hr from m/s:

$$h = h_j + \frac{1}{2.4}\tau u - \left(\frac{b}{b'} - 1\right)\frac{u^2}{25.92b} \quad (2.7)$$

If assumed $b = b'$ (i.e. follower and lead vehicle deceleration is the same),

$$h = h_j + \frac{1}{2.4}\tau u \quad (2.8)$$

Setting $\frac{1}{h} = k$, yields the final equation:

$$k = \frac{1000}{h_j + \frac{1}{2.4}\tau u} \quad (2.9)$$

where density is a function of the jam spacing $h_j$, reaction time $\tau$, and speed $u$ (Rakha et al., 2009).


2.3 Capacity Drop Properties

Section capacity is defined as ”the maximum number of vehicles that can pass a given point during a specified period under prevailing roadway, traffic, and control conditions” (National Research Council (U.S.), 2010). Additionally, capacity may be reduced through temporary or permanent changes to a roadway section (e.g. traffic incident or lane reductions). A capacity drop of 5 - 6% was first observed by (Hall & Agyemang-Duah, 1991) based on empirical analysis. Furthermore, the magnitude of capacity drop has been estimated to range between 3% (Banks, 2010) to as high as 18% (Chung et al., 2007).

Capacity drop for a lane drop scenario typically contains three characteristics:

1. The maximum discharge flow-rate should be equal to the downstream capacity when the bottleneck is deactivated.
2. Capacity drop occurs when the upstream flow-rate is higher than downstream capacity.
3. There are some unobservable stationary states between upstream and downstream sections (Jin et al., 2015).

Empirical studies on capacity drop are often limited by lack of environments under which it can be clearly observed. Additionally, work is often restricted to simple parameters (e.g. occupancy) that make it difficult to obtain accurate results. Furthermore, capacity drop may be affected by time of day and weather conditions which also complicates gathering data. Analytical models do not contain these weaknesses, but may not be completely accurate either as they typically stem from simplified assumptions of reality. Adding in factors to compensate commonly results in larger and inefficient calculation times. Although simulation is also often inaccurate, it can occasionally be used to acquire data not present in empirical and analytical work.
Chapter 3

Simulation Setup

3.1 Aimsun Modeling

3.1.1 Vehicle Loading

Aimsun loads vehicles if there is space available for the next vehicle to enter the section, defined as $PH$ or Possible Headway.

$$PH = x_{leader} + d_{leader} - l_{leader} - d_{follower}$$  \hspace{1cm} (3.1)

$PH$ is calculated by the $x$ position, $l$ vehicle length, and $d$ braking distance of both leader and follower vehicles for a simulation time $t$. Within Aimsun, the braking distance is defined as the distance required to stop using a predefined max deceleration parameter.

Vehicles can enter the network if $PH$ is greater than the braking distance for the follower vehicle.
Once Aimsun determines an open space is available, entrance time and position for vehicles are calculated.

The time needed to travel a distance $x$ is a function of the $PH$, $d_{\text{follower}}$ braking distance of the follower vehicle, and $V_n$ maximum desired speed. Maximum desired speed is a function of the max section speed defined, the mean max desired speed of vehicles, and a speed acceptance parameter.

$$t_x = \frac{PH - d_{\text{follower}}}{V_n} \quad (3.2)$$

The real entrance time ($t_{\text{real}}$) is the maximum value between $t_e$ theoretical entrance time and $t_s$ simulation time subtracted by $t_x$. The entrance position is then defined as:

$$x_n = V_n \ast (t_s + t_{\text{real}}) \quad (3.3)$$

### 3.1.2 Arrival Algorithms

By default, Aimsun uses an exponential distribution to determine the time interval between two consecutive vehicle arrivals with a mean input flow-rate $\lambda$ in $\frac{\text{veh}}{\text{sec}}$ and mean time headway $\frac{1}{\lambda}$. Other options include uniform, normal, constant, and ASAP distributions.
For the constant distribution, all headways between vehicles are constant as $\frac{1}{\lambda}$ sec, where $\lambda$ is the mean input flow-rate in $\frac{veh}{sec}$.

The ASAP distribution sets vehicles to enter into the network as soon as space is available within the road segment. No headway is generated and the total input flow-rate is set onto the entrance section at the beginning of each time step.

### 3.1.3 Car Following Model

Aimsun uses a modified version of the Gipps car following model. In addition to the formulation in Eq. 2.4, there is an additional bound on the second half: $v_b \geq MH$, where $MH$ is the minimum headway between the leader and follower vehicles.

The two-lane car-following model has two definitions: absolute and relative. Both options attempt to model the effect that slower moving vehicles in the right lanes may have on faster vehicles in the left lanes.

![Figure 3.2: Aimsun Car Following Modes](image)

Relative: Blue, Absolute: Red

Absolute mode focuses on determining the number of vehicles within the maximum distance from the merge point and calculating their mean speed to compare to the adjacent lane. If the adjacent lane is an on-ramp, Aimsun determines a maximum speed difference between the two lanes and sums them together as the maximum speed for the section.
Relative mode assumes that a vehicle driving in the fast lane will reduce speed in the presence of slow vehicles to anticipate a slower vehicle pulling in front of it. Aimsun calculates the number of slower vehicles that may be assumed by the faster vehicle to not change lanes; the maximum speed is then calculated dependent on the leading adjacent slow vehicle so as to avoid collision.

### 3.1.4 Lane Changing Model

Lane changing is determined based on three basic rules:

- **Is it necessary to change lanes? (Zone X)**
  - Parameters involve the traffic conditions of the lanes involved (e.g. turning options, distance to next turn).

- **Is it desirable to change lanes? (Zone Y)**
  - The intermediate zone in which vehicles will try to position themselves in lanes closest to their turn. Also dependent on improvement of traffic conditions for a driver (i.e. faster speed, shorter queue).

- **Is it possible to change lanes? (Zone Z)**
  - The urgent zone in which vehicles are trying to get into their respective lane option and reducing speed to a stop if the target lane lacks a big enough gap to make a lane change (Transport Simulation Systems, 2014).

![Figure 3.3: Lane Changing Zones](image)
### 3.1.5 Miscellaneous Parameters

Other parameters are described in this section as they have been modified within the network, but do not fit within the previous sections.

- **Cooperation**: The percentage of upstream vehicles that will create gaps for vehicles in adjacent lanes to change lanes.

- **Aggressiveness**: Controls size of acceptable gaps for lane changing vehicles to change lanes (e.g. 0% is the default gap size, whereas 100% is vehicles closely fitting the gap).

- **Imprudent Lane Changing**: Allows vehicles to enter gaps small enough that do not respect system stability; vehicles involved may need to brake at a rate double the amount of their max deceleration.

- **FIFO**: Defines whether or not the first vehicle is allowed to merge onto the main stream first, or if following vehicles are allowed to merge first.

- **Merge Distance**: The distance from which vehicles are allowed to merge onto the mainline.

- **Cooperation distance**: The distance from which vehicles consider to be a side adjacent lane.

![Figure 3.4: Merge and Cooperation Distance](image)

### 3.2 Network Setup

The network created in Aimsun consists of a 1600 m section of road with a physical bottleneck in the form of a lane drop approximately 1350 m downstream. Capacity of the road section
was set to be 2100 vphpl and have a speed limit of 110 kmph. The road section was chosen to be homogeneous (i.e. 100% car usage, 0% all other vehicle types) to study the effects without temporary bottlenecks. Detectors were placed 50 m apart as shown in Fig. 3.5 and were set to poll every 30 seconds with a total simulation time of 5 hours per experiment. Information gathered from detectors included vehicle speed, density, flow-rate, counts, and headways.

Additionally, the following parameters were set constant throughout all experiments:

- Merge distance set to 300 m.
- Cooperation distance set to 330 m.
- Aggressiveness percentage changed to 40%.
- Cooperation percentage left default at 80%.
- Reaction time changed from default 0.8 s to 1.10 s.
- Arrival distribution changed from exponential to constant.
- Imprudent lane changing not allowed.
- No FIFO restriction to lane changing in place.

The merge/cooperation distance effectively acts as the signpost where vehicles are notified of a change in roadway section. Aggressiveness was changed to 40% in addition to a constant arrival distribution with no FIFO restriction on lane changing to closer reflect standard driving conditions. A reaction time of 1.1s was chosen based on (Gipps, 1981). Unless previously stated, all other variables were left as default.
Chapter 4

Macroscopic Features

4.1 Methodology

Simulation data was plotted against the triangular fundamental diagram using Matlab to provide further information on the traffic state of the system. Subsequently, curves of cumulative vehicle count were obtained for each detector and transformed using an oblique coordinate system to magnify its features.

4.1.1 Fundamental Diagram of Traffic Flow

Eq. 2.9 yields the triangular fundamental diagram by setting all individual vehicles’ headway, $h_j$ as constant and multiplying the resultant $k$ density function by $u$ speed to obtain $q$ flow-rate.

\[
q = Q(k) = \min\{u_f k, w(k_j - k)\}
\]

\[
u = U(k) = \min\{u_f, w\left(\frac{k_j}{k} - 1\right)\}
\] (4.1)
Detector flow-rate, density, speed, and headway were plotted against the triangular fundamental diagram for the initial capacity of the section, $q_c$ as shown in Fig. 4.1a.

### 4.1.2 Capacity Drop Ratio Definition

Cumulative count curves were obtained by plotting the detector input count $N(x, t)$ at location $x$ and time $t$. The slope of the curve given by approximations between points of the stepwise function is the flow-rate between periods $t_i$ and $t_{i+1}$ as in Fig. 4.2a.

![Figure 4.1: Triangular Fundamental Diagram](image)

(a) Flow-rate - Density  
(b) Speed - Density  
(c) Speed - Flow-rate

![Figure 4.2: Properties of Cumulative Vehicle Count](image)

(a) Flow-rate approximation  
(b) Queue Features

Figure 4.2: Properties of Cumulative Vehicle Count
Additionally, curves generated from consecutive detector locations $N(x_i, t)$ and $N(x_{i+1}, t)$ demonstrate queue propagation via vertical and horizontal separation indicating queue length and trip time respectively (Moskowitz & Raff, 1954; Newell, 1982).

An oblique coordinate system was used to magnify the features of the curves by subtracting $N(x_i, t)$ by a background flow $q_0(t - t_0)$ where $t_0$ is the curve’s starting time (Cassidy, 1998).

![Oblique Coordinate System](image1.png)

![Capacity Drop](image2.png)

**Figure 4.3: Derived Features of Cumulative Vehicle Count**

For a lane drop scenario, network demand is defined by an upstream flow-rate $C_1$ immediately prior to a queue forming (Cassidy & Bertini, 1999). Capacity $C_2$, is defined as the average discharge rate downstream after the queue has formed. The difference between the two values represents the dropped capacity $C_*$ of the road section (Jin et al., 2015). A queue begins forming at time $t_1$ with a corresponding $C_1$ as shown in Fig. 4.3b. The average discharge rate is $C_2$ at $t_2$ after the vehicle trajectories have stabilized.

Furthermore, the capacity drop ratio is defined as:

$$\Delta = 1 - \frac{C_*}{C_2}$$

(4.2)
4.2 Results

4.2.1 Capacity Drop Ratio For Simulation

Speed, flow-rate, density, and headway were obtained for each detector and plot against the triangular fundamental diagram for a theoretical one and two lane scenario. For all scenarios below, vehicles were set to use a constant acceleration of 2.0 m/s$^2$ and speed of 110 kmph.

![Detector 14 Diagrams](image)

Figure 4.4: Detector Halfway Through Section with a Demand Level of 4000 vph

The figure above was obtained from a detector approximately 650 m upstream from the bottleneck with a demand level of 4000 vph. Vehicles enter the section at 110 kmph, but quickly
decelerate as they reach the bottleneck location. Further evidence of this phenomenon is indicated by the diminishing headway between vehicles and average discharge rate of 2000 vph.

Fig. 4.5 displays flow-rates and densities for detectors spaced 400 m apart. Vehicles immediately slow down as they approach the bottleneck as shown in Detectors 6 and 14. As vehicles approach the bottleneck via Detector 22, the flow-rate curve shifts to the left towards the theoretical one lane capacity of 2000 vph as shown in Detector 30.

The definition supplied by (Jin et al., 2015) is considered to gain further supporting evidence
on the existence of capacity drop within a lane drop scenario; if upstream demand is lower than downstream capacity, throughput should be preserved. Therefore, upstream demand was reduced to below the previous experiment’s average discharge rate of 2000 vph (i.e. 1900 vph) in order to observe the difference in flow-rate upstream and downstream of the bottleneck. The flow-rate-density graphs for the detector before and after the bottleneck are shown below.

![Flow-rate-density graphs](image)

(a) Before bottleneck  
(b) After bottleneck

Figure 4.6: Detectors Before and After Bottleneck With Demand Level of 1900 vph

As indicated by Fig. 4.6, the flow-rates are nearly identical upstream and downstream of the bottleneck which further supports the earlier definition of no reduction in flow-rate given an upstream demand lower than downstream capacity.

A graduated distribution from 1750 vph to 4000 vph was created to more closely examine the capacity of the lane drop simulation and is detailed in Appendix A.1. A cumulative count curve for all detectors was plotted and subtracted by a background flow of 1750 vph to magnify the curve’s features (Cassidy, 1998). As stated previously, the capacity of a road section is the flow-rate before queue formation. From Fig. 4.7a, the queue begins forming at 60 minutes which corresponds to a flow-rate of approximately 2000 vph. The average discharge rate is 1935.77 vph at 98 minutes of the simulation.
A second set of cumulative count curves was created from a demand distribution of 2000 vph to 2250 vph and subtracted by a background flow of 1950 vph to obtain a more accurate value of downstream capacity for the segment. From Fig. 4.7b, a capacity of 2048.89 vph at 43.5 minutes was obtained. Compared to the order of magnitude in (Cassidy & Bertini, 1999), 43.5 minutes may be unreasonable. However, when considering the demand pattern, the relative time until queue formation becomes 13.5 minutes. A capacity drop ratio of 5.52% was determined from the difference between the flow-rate before queue formation and average discharge rate. This result further coincides with dropped capacity when upstream demand is higher than downstream capacity.
Chapter 5

Microscopic Features

5.1 Methodology

Vehicle position and speed was extracted from the simulations via Aimsun API and plot with Matlab to obtain vehicle trajectory plots and speed profiles. Near stationary states were identified from the information gathered to further support the capacity drop phenomenon.

5.1.1 Near Stationary Conditions

Time periods that exhibit near stationary properties were identified from intervals where vehicle arrivals displayed a linear trend such as in Fig. 4.7a (Cassidy, 1998). For example, linearity is shown in the periods between 0 - 30 minutes, 30 - 60 minutes, and from 98 minutes onwards. The period between 98 minutes to the end of simulation yields the average discharge rate after a queue has formed from the bottleneck.
Given a period where linearity is exhibited on cumulative count curves, it is possible to identify near stationary states by observing near constant vehicle speeds as shown in the figure below.

![Position x vs Time t](image)

**Figure 5.1: Vehicle Trajectories in Near Stationary State**

The existence of near stationary states identified from the cumulative count curves and vehicle trajectory plots provide additional observations on capacity drop. Vehicle speed profiles can be used to determine the queue formation time by demonstrating a steep drop in velocity through a given vehicle and can be used to verify merge distance as vehicles begin to accelerate.

### 5.2 Results

#### 5.2.1 Speed Profile For Simulation

Additional data was generated using the same demand pattern of 2000 vph to 2250 vph in Appendix A.1, constant vehicle acceleration of 2.0 \( m/s^2 \) and maximum speed of 110 kmph. Every 100\(^{th}\) vehicle was obtained from the Aimsum API and the speed profiles of five vehicles
spaced equally throughout the entire simulation period were chosen.

From the figure above, vehicles begin to decelerate when reaching the merge distance at approximately 1000 m downstream of the start of the segment. As Vehicle 100 passes the bottleneck at 1350 m, it begins accelerating in a range between $1.7 \, m/s^2$ and $2.1 \, m/s^2$ for the last 250 m. Vehicle 2300 corresponds to the period between 30 - 60 minutes, which indicates that the queue is beginning to form. Subsequently, Vehicle 4500 onwards corresponds to the time period past 98 minutes which is indicated by vehicles entering the section at less than 110 kmph and continuing at that speed until moving past the bottleneck. Additionally, it can be said that Vehicles 4500 - 8900 are relatively stationary as their velocity does not change outside of noise generated from a stop-and-go pattern induced by the bottleneck.

Figure 5.2: Speed Profile for Demand Distribution of 2000 vph to 2250 vph
Chapter 6

Analytical vs. Simulation Results on Capacity Drop

This chapter serves to compare the differences in the capacity drop ratio between an analytical behavioral model and the Aimsun simulation using identical properties. The factors to be changed are constant maximum vehicle acceleration and merge distance.

6.1 Methodology

6.1.1 First-Order Behavioral Model

A new first-order behavioral model was introduced in (Jin, 2017a). This model is based off of the third multilane LWR model for a continuous lane-drop bottleneck from (Jin, 2017b) with an added bounded acceleration constraint. By introducing bounded acceleration, capacity drop can be modeled based on the kinematic wave model.
The final formulation is as follows:

\[ f(L; C_*) = \frac{\Delta l}{L \cdot (l_2 q_0 - C_*)^3} (C_* W)^2 q_0 - \alpha \left( \frac{C_*}{l_2 q_0 - C_*} W \right) = 0 \]  

(6.1)

The equation above is the result of two acceleration parameters \( a^*(v) \) and \( \alpha^*(v) \) which represent the acceleration rate in the standing wave and the maximum acceleration rate as a function of speed. \( \Delta l \) is the difference between the upstream \( (l_1) \) and downstream \( (l_2) \) number of lanes. Furthermore, \( W \) is the wave speed and \( q_0 \) is the background flow-rate as shown on Fig. 4.1a. \( L \) is the distance of the lane drop zone, which will be assumed analogous to the merge distance in Aimsun. \( C_* \) is the dropped capacity which can be solved by using the bisection method with bounds starting at 0 and the downstream capacity \( (l_2 C) \); the capacity drop ratio can be calculated from Eq. 4.2.

The factor impact analysis presented utilizes a constant maximum acceleration rate and the TWOPAS model. However, for comparison to the Aimsun simulation, the bounded acceleration \( \alpha^*(v) \) is replaced with the Gipps car following model.

### 6.2 Results

#### 6.2.1 Capacity Drop Ratio for Analytical and Simulation Models

The default values for the factors were set to be identical for both models: \( V = 30.56 \) \( m/s \), \( W = 4.73 \) \( m/s \), \( k_j = 0.14 \) \( veh/m \), \( l_1 = 2 \), and \( l_2 = 1 \); speed limit of 110 kmph, acceleration of \( 2.0 \) \( m/s^2 \), and merge distance of 300 m. Two tests were performed to observe the impact of acceleration and merge distance on capacity drop: vehicle accelerations from 1.0 to 3.0 \( m/s^2 \) at a merge distance of 300m and merge distances of 0 m to 1400 m with a constant vehicle acceleration of \( 2.0 \) \( m/s^2 \). Simulation factors are listed in detail in Appendix A.2.
The analysis from (Jin, 2017a) provides insight on acceleration and merge distance affecting capacity drop to a significant degree. The paper states that for a lane drop zone length of 0 m, when the bottleneck is discontinuous, the capacity drop ratio is 100%. For increasingly small values of acceleration (i.e. when vehicles cannot accelerate), the capacity drop ratio also trends towards 100%. Furthermore, for larger values of acceleration and merge distance, vehicles have greater knowledge of the road section that they can use to merge more efficiently which results in a lowered capacity drop ratio.

Simulation also exhibited a general downward trend to the capacity drop ratio when vehicle acceleration was increased; intuitively, as vehicles are able to speed up after leaving the bottleneck section, average discharge rate is increased. Additionally, when merge distance is increased, more vehicles can exist in the upstream X/Y zones which allows for increased time for decision making and allows vehicles to systematically change into appropriate lanes when necessary.
6.2.2 Speed Profiles of Simulation Experiments

Speed profiles of select Aimsun experiments were obtained to more closely examine capacity drop for simulation. The processes in Sec. 4.2.1 and 5.2.1 were repeated with the above factors to obtain the figure below.

![Speed Profiles of Vehicles With Modified Acceleration and Merge Distance](image)

(a) Acc: 1.5 m/s²  
(b) Acc: 1.0 m/s²  
(c) Merge Dist.: 200 m  
(d) Merge Dist.: 100 m

Figure 6.2: Speed Profiles of Vehicles With Modified Acceleration and Merge Distance

At an acceleration of 1.5 m/s², vehicles enter the section progressively slower than at an acceleration of 2.0 m/s² from Fig. 5.2. Furthermore, the queue forms earlier than the baseline, evidenced by Vehicle 2000 exhibiting the same properties of vehicles in the congested state. Fewer vehicles enter the section as well, which indicates a reduced flow-rate when
compared to the baseline. The vehicle count is reduced by almost half when reducing the acceleration even further to 1.0 m/s$^2$, which results in an even larger reduction in flow-rate as Vehicle 4900 does not leave the section before the simulation ends.

Vehicles begin slowing down further upstream when decreasing merge distance and are given less space to accelerate past the bottleneck. As a result, the lead vehicle decreases in speed more rapidly than the baseline counterpart. Subsequent vehicles begin decelerating earlier in the segment as the queue forms as shown by Vehicle 2300 in the 200 m experiment; Vehicle 1800 in the 100 m experiment is unable to reach the desired 110 kmph before having to decelerate.

6.3 Comparison of Analytical and Simulation Results

The analytical results provide a consistent downward trend in the capacity drop ratio when compared to the simulation results. The simulation capacity drop magnitude tends to be lower than the theoretical which may be a result of the modified Gipps car following model that Aimsun uses. A discontinuity exists in the simulation data which may indicate that there are additional factors which affect capacity drop in Aimsun. The discontinuity is evidenced by the large reduction in discharge flow-rate as shown in Fig. 6.2, which may further suggest that there may be network instability for extreme values of certain parameters.

Given the information from both scenarios, vehicle acceleration most likely affects capacity drop to a higher degree than the merge distance. This is further exhibited by the queue formation process in Fig. 6.2 which occurs faster in the acceleration scenario. The analytical data displays a higher overall capacity drop ratio that decreases at a slower rate than the merge distance. However, the effect on merge distance in simulation is closer in magnitude and more consistent with the theoretical results.
Chapter 7

Conclusion

The research presented analyzed conditions upstream and downstream of a simulated lane drop bottleneck in an attempt to examine factors affecting the capacity drop phenomenon. The triangular fundamental diagram was derived from the Gipps car following model using the steady state condition. Additionally, capacity drop was defined as having no reduction of throughput given a lower demand than capacity and capacity drop otherwise through bottleneck activation.

Simulation data was plot against the triangular fundamental diagram for both a one lane and two lane scenario and curves of cumulative vehicle count were generated. Cumulative count curves were transformed to use an oblique coordinate system to improve visibility of features. Additionally, speed profiles of every 100th vehicle was obtained to verify the results of the other figures.

When plotting the data against the fundamental diagram at a demand rate of 4000 vph, the discharge flow-rate results in approximately 2000 vph. On further examination of several detectors along the length of the road section, the data results in a congested upstream and trends towards the left to an uncongested downstream. Upon setting the demand level
to 1900 vph, there exists no difference between detectors upstream or downstream of the bottleneck.

A graded distribution from 1750 vph to 4000 vph was created to determine the downstream capacity of the section. Examination of the transformed cumulative count curve yields a more accurate average discharge flow-rate of 1935.77 vph. A second transformed cumulative count curve was created with a distribution from 2000 vph to 2250 vph and further examination yielded a more accurate capacity of 2048.89 vph. The difference produced a capacity drop ratio of 5.52%. Speed profiles using the 2000 vph to 2250 vph distribution confirmed the previous results of the cumulative vehicle curves; the decreased flow-rate is evidenced by reduced total vehicle count.

Additionally, a comparison between an analytical first-order behavioral model and the simulation results was conducted. The theoretical results yielded a more consistent downward trend than the experiment results. Furthermore, the simulations contained a discontinuity in the graph which was accounted for as an extreme reduction in flow-rate as vehicles began decelerating earlier when the acceleration and merge distance was decreased. For both models, decreasing acceleration and merge distance increased the magnitude of capacity drop. The theoretical and simulation results demonstrated that vehicle acceleration has a greater influence on capacity drop than merge distance.

Further elements to discuss include the last characteristic of capacity drop (i.e. there are some unobservable stationary states upstream and downstream of the bottleneck) and further calibration of the model indicated when using sufficiently small values of vehicle acceleration and merge distance. However, this research can be used as a supplement to pre-existing materials on the effects of capacity drop between empirical and analytical work.


Appendix A

Experiment Parameters

A.1 Cumulative Count Flow-rate Distribution

<table>
<thead>
<tr>
<th>Time (hr)</th>
<th>Flow-rate (vph)</th>
<th>Time (hr)</th>
<th>Flow-rate (vph)</th>
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</table>

(a) 1750 vph - 4000 vph  
(b) 2000 vph - 2250 vph
### A.2 Speed Profile Acceleration & Merge Distance

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<thead>
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<th>Acceleration ($m/s^2$)</th>
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(a) Constant Merge Distance: 300 m  
(b) Constant Acceleration: 2.0 $m/s^2$
## Appendix B

### Capacity Drop Ratio Results

<table>
<thead>
<tr>
<th>Accel. $(m/s^2)$</th>
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<th>Simulation (%)</th>
<th>Merge Distance $(m)$</th>
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<td></td>
<td></td>
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</table>

(a) Acceleration Parameter  
(b) Merge Distance Parameter