FREEWAY TRAVEL TIME ESTIMATION AS A FUNCTION OF DEMAND

By

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To those on whose shoulders I stand
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By

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Travel time is a fundamental measure of system effectiveness in transportation. It is used in evaluating the operating efficiency of traffic networks, in assessing the performance of traffic management strategies, and as input in various planning applications. For this reason extensive research has been performed on the subject of estimating travel time. Methods exist that directly derive travel time from field measurements, or use field data to make short-term estimations. Empirical equations and analytical models have been developed in order to enable long-term and macroscopic planning applications. Also, travel time is a standard output of all current traffic simulation models. However, despite the plethora of methods, techniques, equations and software that deal with travel time, there is a lack of a robust but simple to apply analytical relationship.

This thesis has the objective to fill in that gap. A new analytical travel time equation was developed based on results from simulation. A macroscopic (NETCELL) and a microscopic (CORSIM) simulation program were evaluated and in the end CORSIM was selected, due to the nature of its output. A number of variables that affect travel time were selected and 972 scenarios were implemented in the program. From
the results, with the use of linear regression, analytical travel time models were
developed and compared to the existing analytical models. It was concluded that the
models developed are consistent with queuing theory and with Akçelik’s speed-flow
equation and able to provide realistic results under a variety of conditions.
CHAPTER 1
INTRODUCTION

1.1 Background

Travel time is a fundamental measure of system effectiveness in transportation. It is used in evaluating the operating efficiency of traffic networks, in assessing the performance of traffic management strategies and as input in various planning applications and route choice models. Also, travel time is a standard output of all current traffic simulation models. For this reason extensive research has been performed on the subject of estimating travel time.

Methods exist that directly derive travel time from field measurements, or use field data to make short-term estimations. Direct measurement techniques can potentially measure travel time with high accuracy, however they are generally expensive, time consuming and labor intensive, as they require adequately large and unbiased samples of vehicles. On the other hand, short-term travel time estimation techniques, such as the extrapolation technique or methods based on statistical models or on traffic flow theory, are more cost efficient but have low accuracy or they are developed only for specific conditions and thus are applicable either only for free-flow conditions or only for congested conditions.

Empirical equations and analytical models have also been developed in order to enable long-term and macroscopic planning applications. These models have the advantage of being simple and easy to use, though they also have important limitations. Almost all of them are not time-dependent and thus unable to deal with varying flow periods or model periods of oversaturation. None of them consider the downstream
conditions, which can cause congestion upstream, nor do they take into account the segment length under developing saturated conditions.

Therefore, despite the plethora of methods, techniques, equations and software that deal with travel time, there is a lack of a robust but simple to apply analytical relationship for estimating freeway travel time, especially under congested conditions. Such a model should be able to estimate travel time with accuracy, under most conditions observed in the field, and also be simple and broad enough to be used cost-efficiently in planning applications.

1.2 Objectives

The purpose of this thesis is to develop analytical models for estimating travel time for freeway segments as a function of demand for a number of different scenarios and a variety of conditions. Those models should be easily applicable to large-scale – temporally and spatially – studies involving freeway corridor travel time and travel time reliability.

More specifically, the objectives of this study are the identification of variables which affect freeway corridor travel time as found in the literature, the development of appropriate scenarios that allow for a comprehensive coverage of observable conditions, the selection of the appropriate simulation software in order to generate data for the development of analytical models, the development of analytical travel time models based on the simulation results, the assessment of the analytical models developed and a comparison to existing travel time models.

1.3 Organization

The rest of this document is organized as follows. Chapter 2 discusses the literature relevant to the topic. The development of the methodology is described in
Chapter 3. The results are presented in Chapter 4 and are compared with other analytical models. Chapter 5 summarizes the conclusions and lists recommendations for future research.
CHAPTER 2
LITERATURE REVIEW

This chapter summarizes previous research related to the estimation of freeway travel time for various applications. Section 2.1 describes field-observed travel time applications while Section 2.2 deals with real-time travel time estimation based on field data. Section 2.3 reviews analytical travel time estimation models used for planning applications and section 2.4 presents various simulation models that can estimate travel time. Section 2.5 discusses the variables that have been found to affect travel time. The chapter concludes with a summary of the literature review findings (Section 2.6).

2.1 Field-Observed Travel Time Applications

In this section a number of techniques used to measure travel time directly are presented. Travel time can be measured directly with the use of techniques such as probe vehicles, license plate matching, electronic distance measuring instruments, automatic vehicle identification (AVI), automatic vehicle location (AVL), video imaging, and other. Direct measurement techniques can potentially measure travel time with a small margin of error if the sample of vehicles is adequately large and unbiased, however they have a number of disadvantages (Turner, 1996). They are generally more expensive, often require the installation of new types of sensors (AVI and AVL), may depend on voluntary public participation or are time consuming and labor intensive for collecting large amount of data (test vehicle method).

Alternatively, travel time can be measured indirectly by spot speed measurement techniques that measure only the instantaneous speed either at a fixed location (for example with road side radar sensors or double loop detectors), or at a fixed time over a wider area (for example by aerial photography) (Turner et al., 1998). Those techniques
are the most efficient ways, in terms of cost, to acquire significant amounts of speed
data, but the process of deriving the travel time from those measurements can reduce
the accuracy of the results.

2.2 Real-Time Travel Time Estimation Based on Observations

This section describes models used for real-time, short-term travel time prediction.
Most of these models use some of the techniques presented in the previous section to
estimate either travel times or their equivalent speeds.

Since most metropolitan areas in the United States and all around the developed
world have freeway networks already instrumented with inductive loop detectors (ILD), it
is likely that estimating travel time from data obtained through those systems is the most
widespread and cost-effective method, utilizing the already existing infrastructure. In
general, the data obtained from ILD include volume, lane occupancy and speed. In
order to estimate travel time from that data a number of methods have been proposed.

The simplest and most broadly used method is the extrapolation technique (Turner
et al., 1998), which is based on the assumption that speed can be considered constant
for the relatively small distance between two measurement points. Since this distance is
a known quantity the travel time can be simply calculated by dividing the distance with
the speed (Sisiopiku et al., 1994). However, a number of studies (Ferrier, 1999;
Quiroga, 2000; Li et al., 2006) have concluded that with increasing flows and congested
conditions, the performance of such extrapolation methods is much lower, as they fail to
capture the effects of congestion between adjacent detector stations.

Methods based on statistical models have been proposed to counter the
weaknesses of the extrapolation method. Dailey (1993) used cross-correlation
techniques to accurately measure the propagation time of traffic, while Van Arem et al.

Models based on traffic flow theory were also developed, applying the principle of vehicle conservation and comparing the inflow of one section during a previous time interval with its outflow during the current time interval (Bovy and Thijs, 2000). Nam and Drew (1996; 1998; 1999) developed a macroscopic model for real-time estimation of freeway travel time based on the area between the cumulative volume curves obtained from loop detectors at either end of a link. Rakha and Zhang (2005) pointed out three errors that Nam and Drew (1998; 1999) had made in their research referring to the comparison between shock wave theory and queuing models. They claimed that the area between the arrival and departure curves in queuing diagram is the total delay rather than total travel time and based on this point of view, the delay computations for shock wave analysis and queuing models are consistent. Lovell and Windover (1999) had also already pointed out the first error in the paper by Nam and Drew. Petty et al. (1998) proposed a model that was based on the assumption that the vehicles arriving at an upstream point during a given period have a common probability distribution of travel time to a downstream point. Coifman (2002) used a linear approximation of the flow-density relationship to estimate travel time from data obtained from dual loop detectors while Oh et al. (2003) suggested the estimation of travel time based on fluid model relations. Most of the above models were developed for specific traffic flow conditions and thus are applicable either only for free-flow conditions (Nam and Drew, 1996; Oh et
al., 2003) or only for congested conditions (Nam and Drew, 1998). Rilett et al. (2009) proposed several modifications to the traffic dynamic models suggested by Nam and Drew in order to be able to estimate travel time for varying traffic flow conditions, including transition periods, directly from the loop detector data.

2.3 Analytical Travel Time Estimation Models for Planning Applications

This section reviews analytical models that were developed for use in planning applications.

The analytical travel time estimation models found in the literature fall into two distinct categories, depending on the saturation conditions they can be applied for. Models such as the HCM speed flow curve can be used to estimate travel times for unsaturated conditions, but are not applicable in cases when demand exceeds capacity. This limitation is one of high importance, as the impact of oversaturated conditions on travel times and travel time reliability is very severe, and thus their usefulness in real-life planning applications is minor.

Three models were found in the literature to be applicable to oversaturated conditions: the Bureau of Public Roads (BPR) model (1964), the Metropolitan Transportation Commission (MTC) model (1995) and the Akçelik function (1991).

The BPR model is the oldest analytical model and has been traditionally used for planning models. It is similar to the 1965 HCM curve, which was parabolic in shape, and speed was fairly sensitive to increasing flows. The BPR curve is as follows:

\[ t = t_o \times (1 + 0.15 \times (d/c)^4) \]

Where:

- \( t \) = travel time per unit of distance
- \( t_o \) = minimum (zero flow) travel time per unit of distance
The main problem of the BPR curve is that it overestimates speeds for volume-to-capacity ratios in excess of 1.0 while underestimating speeds for volume-to-capacity ratios less than 1.0 (Singh, 1999).

Abu-Eisheh and Mannering (1986) calibrated the model parameters of the BPR function for 15 categories of roadway links, according to their speed limit and practical capacity. Of these categories only one (with speed limit 50+ mph and practical capacity 1000+ vphpl) is applicable for freeways and the modified BPR function that was proposed is the following:

\[ t = t_o \cdot (1 + 1.15 \cdot (d/c)^{6.87}) \]

Dowling and Skabardonis (1992) proposed another modification to the BPR curve as follows:

\[ t = t_o \cdot (1 + (d/c)^{10}) \]

This function had the same speed at volume-to-capacity ratio of 1.0 as the 1985 HCM curve. The results of the study were validated with results of operational models for test freeway sections.

The MTC model (Singh, 1995) was another update of the BPR model that was developed to account for the 1994 HCM free-flow relationship. It had a more gradual slope with constant speed for higher level of flows and the speed for volume-to-capacity ratio of 1.0 was only 5 mph less that the free-flow speed. The MTC curve is as follows:

\[ t = t_o \cdot (1 + 0.20 \cdot (d/c)^{10}) \]

Where:
t = travel time per unit of distance

\( t_0 \) = minimum (zero flow) travel time per unit of distance

d = demand

c = capacity

Akçelik (2003) argued that the MTC model is not theoretically justified from queuing theory, since while in oversaturated traffic conditions the travel time should increase linearly with flow; in the MTC model the increase is exponential.

Using concepts of queuing theory Davidson (1966; 1978) developed the following function to model flows near and above capacity:

\[ t = t_0 \times \left[ 1 + \frac{J_D x}{(1-x)} \right] \]

Where:

t = average travel time per unit of distance

\( t_0 \) = minimum (zero flow) travel time per unit of distance

\( J_D \) = a delay parameter (or \( 1-J_D \) = a quality of service parameter)

x = \( \frac{d}{c} \) = degree of saturation

d = demand (arrival) flow rate

c = capacity

The Akçelik (1991) speed-flow equation is a time-dependent modified form of the Davidson’s function that estimates average travel time over a given time period. It is as follows:

\[ t = t_0 + 900T\left\{ (x-1) + \left[ (x-1)^2 + \left( 8J_D x/cT \right) \right]^{0.5} \right\} \]

Where:

t = average travel time per unit of distance (s/km)
\[ t_0 = \text{free-flow travel time per unit of distance (s/km)} \]

\[ T = \text{duration of analysis period, i.e., the travel time interval during which an average arrival (demand) flow rate } v \text{ persists (h).} \]

\[ x = \frac{d}{c} = \text{degree of saturation} \]

\[ d = \text{demand (arrival) flow rate (veh/h)} \]

\[ c = \text{capacity (veh/h)} \]

\[ J_a = \text{Akçelik’s traffic delay/bunching parameter} \]

The equation is very similar to the Highway Capacity Manual 2000 Incremental Delay \((d_2)\) equation. In \(d_2\), the traffic delay/bunching parameter \((J_a)\) is replaced by the product of \(k^*l\), two factors that are dependent on controller settings and upstream filtering respectively.

The most appealing aspect of Akçelik’s model and its key difference with the previous models, is its ability to deal with varying flow periods and model periods of oversaturation. Another difference is that the BPR and MTC are more macroscopic and specifically geared towards planning applications, while Akçelik’s function is microscopic and focused on making realistic estimates of travel time. As far as the shape of the curve is concerned, the Akçelik curve has lower speed than the other models at a \(d/c\) ratio of 1.0 and higher sensitivity of speeds to increasing flows between \(d/c\) ratios of 1.0 and 1.5 compared to the MTC curve. Also, consistent with queuing theory, travel time increases linearly for the Akçelik curve for oversaturated conditions. A comparison of speeds and travel times between the three analytical models is shown in Figures 2-1 to 2-3.
A common limitation of all analytical models mentioned above is that travel time estimation is independent of the downstream conditions. A downstream bottleneck can cause upstream congestion and thus completely invalidate the results provided by these models.

In addition, another important factor, the length of the segment, is not taken into consideration. The length is an important parameter for travel time estimation under saturated conditions as it affects the percentage of the segment that experiences congestion and the T.T. per unit of distance within a link.

2.4 Travel Time Estimation using Simulation

This section discusses simulation models that can be used to estimate travel time. In regards to simulation, there are two different approaches. On the one end there is the macroscopic approach, where traffic flow is described by aggregate entities such as mean velocity or global density, while on the other end there is microscopic simulation, where each vehicle is modeled individually following a set of simple rules. A promising application of the macroscopic approach is cellular automata, that apply macroscopic rules on relatively small segments of the traffic network (Wahle et al., 2001). Examples of microscopic and macroscopic models will be discussed.

A prime example of microscopic simulation is CORSIM. Developed by the Federal Highway Administration (FHWA), it applies stochastic, interval-based simulation to describe traffic operations, where each vehicle is treated as a separate entity behaving according to a set of complex set of algorithms, such as car-following and lane changing. The output of CORSIM includes vehicle performance measures, such as average speed, delay and fuel consumption and network performance measures, such as d/c ratio and travel time for each roadway link. During each time step CORSIM
accumulates the distance traveled by all vehicles currently on the link and accumulates the travel time for all vehicles currently on the link. At the end of the reporting interval (time interval, time period or cumulative) it computes average speed by dividing total accumulated distance traveled by total accumulated travel time. Travel time per vehicle for the link is then computed by dividing the link length by the average speed.

The most promising macroscopic simulation model is the Cell Transmission model developed by Carlos Daganzo (1992; 1994). In this model, the traffic network is modeled by a set of cells (representing freeway segments) and their interconnections. The overall traffic state over time is determined by an algorithm consistent with the kinematic wave theory of traffic flow. It is as follows (Figure 2-4):

\[ q = \min\{v_k, q_{\text{max}}, w(k_j-k)\}, \text{ for } 0 \leq k \leq k_j \]

The method assumes that the road has been divided into homogenous sections (cells) \(i\), whose lengths equal the distance traveled by free-flowing traffic in one clock interval. The state of the system at \(t\) is defined by the number of vehicles contained in each cell \(n_i(t)\). Also the following parameters are defined for each cell:

\(N_i(t)\) = the maximum number of vehicles that can be present in cell \(i\) at time \(t\).

\(Q_i(t)\) = the maximum number of vehicles that can flow into cell \(i\) when the clock advances from \(t\) to \(t + 1\).

Thus, for consecutively numbered cells starting from \(i = 1\) for the upstream end of the road, the recursive relationship of the cell-transmission model can be expressed as:

\[ n_i(t+1) = n_i(t) + y_i(t) - y_{i+1}(t) \]

where \(y_i(t)\) = the inflow to cell \(i\) in the time interval \((t,t+1)\), given by:

\[ y_i(t) = \min\{n_{i-1}(t), Q_i(t), \delta[N_i(t)-n_i(t)]\} \]
where $\delta = \frac{w}{v}$

In this way, the model is able to keep track of the location of moving queues in the network and simulate queue spillback and dissipation. The University of California, as part of the PATH program, has developed NETCELL (Cayford et al., 1997), a deterministic freeway network simulation program based on the cell transmission model. NETCELL has lower computational requirements and running times than CORSIM, making it ideal for simulating large-scale networks or for quickly obtaining a significant amount of experimental data.

### 2.5 Variables Affecting Travel Time

This section identifies a preliminary list of variables that are considered to affect travel time. Investigating the exact impact of those variables on travel time will be the subject of this thesis.

For a freeway segment the Highway Capacity Manual 2000 identifies the following variables as affecting travel time:

1. **Free-flow Speed (FFS):** it determines the speed observed under uncongested conditions. Higher free-flow speed is equivalent to lower travel times for unsaturated conditions. Free-flow speed is usually determined by the geometric characteristics of the freeway segment, but can be affected by external factors, such as weather conditions (Stern et al., 2002).

2. **Demand-to-Capacity (d/c) ratio:** it is directly related to travel time since higher ratios indicate near congested or congested conditions and thus lower travel times. Its components are:
   a. **Demand:** it fluctuates daily, weekly and seasonally. Special events may also create non-periodic changes in demand. In freeway merge segments a distinction must be made between the main freeway demand and the ramp demand.
   
   b. **Capacity:** it is mainly a function of the geometric characteristics of the segment, but special events and conditions can have a significant impact on it.
3. Number of lanes: the number of lanes has a direct impact on the capacity of the freeway segment and also on the effect of ramps and special events, such as incidents and work zones.

In addition to the above factors listed in the HCM 2000, two other factors can have a dynamic effect on the travel time of a freeway segment, especially under congested conditions:

4. The Period of Time during which the current demand to capacity ratio persists, since it allows for queues to accumulate and/or dissipate, consequently affecting the average travel time.

5. The Length of the segment, which under saturated conditions affects the percentage of the segment that experiences congestion and thus the T.T. per unit of distance within the link.

2.6 Literature Review Summary

Travel time estimation has been extensively researched due to its very important applications. Methods have been developed that directly measure travel time in the field or make real-time travel time estimations based on observations. A number of analytical models that estimate travel time as a function of demand have been proposed for use in macroscopic planning applications. These models are applicable to both unsaturated and saturated conditions. However, while their T.T. predictions at low demand levels are accurate and consistent with each other, at high levels of demand their predictions are no longer consistent with each other and are sometimes unrealistic. Also, none of the models consider the effects of downstream bottlenecking or the length of the freeway segment and only the Akçelik model is time-dependent. This emphasizes the need for a simple but dynamic analytical travel time equation that encompasses all the important parameters.

Simulation techniques, both microscopic and macroscopic have been successful in estimating travel times, however their use is time-consuming and requires knowledge
of the software used. Analytical models have the advantage of being simpler to use and more widely accessible. However, no analytical models of travel time estimation have been developed using results obtained from simulation.
Figure 2-1. Comparison of BPR, MTC, Akçelik and 1994 HCM speed-flow functions for freeways. (Source: http://www.mtc.ca.gov/maps_and_data/datamart/research/boston1.htm. Last accessed February, 2010)

Figure 2-2. Comparison of BPR, MTC, Akçelik and 1994 HCM travel times for freeways (v/c ratio up to 1.5) (Source: http://www.mtc.ca.gov/maps_and_data/datamart/research/boston1.htm. Last accessed February, 2010)

Figure 2-3. Comparison of BPR, MTC, Akçelik and 1994 HCM travel times for freeways (v/c ratio up to 2.0) (Source: http://www.mtc.ca.gov/maps_and_data/datamart/research/boston1.htm. Last accessed February, 2010)
Figure 2-4. The equation of state of the cell-transmission model.
CHAPTER 3
METHODOLOGY

The methodology developed in order to achieve the objectives mentioned in Chapter 1 is described in this section. It consists of seven steps:

1. Identification of possible variables and development of a conceptual test segment
2. Simulation model selection
3. Development of scenarios
4. Simulation runs and data extraction
5. Development of analytical travel time models
6. Comparison with existing analytical travel time models
7. Conclusions and recommendations

This chapter describes the first three of the above steps, while the remaining steps are discussed in chapters 4 and 5.

3.1 Identification of variables and development of a conceptual test segment

The travel time estimation models are developed for a typical freeway merge segment (Figure 3-1). The subject segment is the freeway segment for which travel times will be estimated and the “segment length” variable refers to the length of the subject segment only. The downstream bottleneck segment serves as a way to simulate downstream bottlenecks and capacity reductions (due to incidents, for example) in accordance to the various scenarios that will be tested.

Based on the conclusions of section 2.5, the variables investigated are:

- Free-flow speed (mph)
- Freeway demand (vphpl)
- Capacity (vphpl)
- Capacity reduction (%) due to downstream bottlenecks
- Number of lanes
- Period of time (for which the current demand-to-capacity ratio persists
- Segment Length

Special consideration must be given to the capacity reduction factor. In section 2.5 capacity, and its relation to demand, was identified as one of the important factors that
affect travel time. Under normal conditions, capacity can be mostly defined by some of the other factors already considered, such as free-flow speed and number of lanes, as well as by the geometric characteristics of the freeway segment. However, we are not only investigating the effect of capacity on travel time, but the effect of the reduction of capacity during special events, such as incidents and work zones, when compared to capacity under normal conditions.

For each of these variables, a possible range of values is selected within realistic field conditions. Within the range of possible values for each variable, a few values are selected and are considered for the development of scenarios. Once specific values are chosen for each variable, scenarios are developed using every possible combination of these values. A scenario represents a state where each variable is assigned a specific value among the possible values.

3.2 Simulation Model Selection

In order to arrive at a decision regarding the preferred simulation software for this project, a small number (45) of scenarios were selected and then implemented in NETCELL. Table 3-1 shows the range of values of the variables that were selected in order to develop the 45 scenarios, by combining the values in all possible ways, while Table 3-2 shows the characteristics of the network and the input parameters used in the NETCELL implementation, that remain the same in all scenarios. The freeway segment and on-ramp capacities were obtained from CORSIM simulation runs. In addition to that, a custom flow-density curve was used in this implementation instead of NETCELL’s default triangular one. The custom curve was defined by 6 xy coordinate pairs that were also obtained from CORSIM simulation runs. Figure 3-2 shows the flow-density relationship used ("CORSIM" curve) compared to the default one.
The output obtained from the NETCELL simulation runs is the travel time $T_i(t)$ of the subject link for each time interval, where $T_i(t)$ is the time it takes to traverse link $i$ when the link was entered at time $t$. NETCELL calculates the travel time of a roadway link by using the cumulative vehicle counts for that link (Cayford et al., 1997). Under the assumption of FIFO, the time at which a vehicle will exit a link entered at time $t$ is the time $d_i(t)$ at which the cumulative number of departures from the link equals the sum of the initial link occupancy and the cumulative count of arrivals to the arc in $[0,t)$. Thus, $T_i(t) = d_i(t) - t$ (Figure 3-3). This calculation applies to travel times for individual cells as well. The only difference in the treatment of cells and links is the placement of the counting locations. The average travel time over the entire study period is then calculated by averaging the travel times of all intervals. The results of the simulation runs are shown in Figures 3-4 to 3-11.

For the shortest time period (30 min), increasing the length of the subject segment led to a decrease in the average travel times, both for congested and uncongested conditions (Figure 3-4). This is expected due to the increase in the percentage of the subject segment that experiences uncongested conditions as the segment length increases. In addition to that, the 1-mile long segment reached its maximum average travel time on lower demand levels than the other two segments.

However, for the longest time period (90 min), the shortest segment (1-mile long) had the lowest average travel times for congested conditions (Figure 3-6). The 5-mile and 10-mile segments had almost identical travel times for congested conditions, both higher than the travel times of the 1-mile segment. For uncongested conditions, the 1-mile and 10-mile segments had similar travel times, though very different curve
geometry, as the 1-mile segment reached its maximum average travel time on lower demand levels than the 10-mile segment. The 5-mile long segment had higher travel times for uncongested conditions than both the 1-mile and the 10-mile segment.

For the 60-minute period, the 5-mile long segment had the highest average travel times for congested conditions (Figure 3-5). The 1-mile long segment had the lowest travel times, but they were comparable to the travel times of the 10-mile long segment. For uncongested conditions, the 10-mile length segment had the lowest travel times, while the 1- and 5-mile long segments had almost identical travel times. Again the 1-mile long segment reached its maximum value on lower demand levels.

For a 1-mile long segment, the shortest time period (30 minutes) had the highest travel times and the longest time period (90 minutes) had the lowest travel times (Figure 3-7). The curves were also similar in shape, reaching their maximum values at almost the same demand levels.

For a 5-mile long segment, the shortest time period (30 minutes) had the lowest travel times both for congested and uncongested conditions (Figure 3-8). The 60- and 90- minutes periods had nearly identical travel times for congested conditions, both higher than those of the 30-minute period. For uncongested conditions the 90-minute period had higher travel times.

For a 10-mile long segment, the shortest time period (30 minutes) had the lowest travel times and the longest time period (90 minutes) had the highest travel times, both for congested and uncongested conditions (Figure 3-9).

For congested conditions (2300 vphpl) the travel times for the 1-mile long segment decreased as the period of time increases (Figure 3-10). For the 5-mile long segment
the travel times increased as the period of time increases and the for the 10-mile long segment the travel times as a function of the period of time increased with a greater rate.

For congested conditions (2300 vphpl) the travel times for the 30-minute period decreased as the segment length increases (Figure 3-11). However, the travel times for the 60- and 90-minute periods first increased with the increase in segment length and then decreased, showing a peak at the 5-miles.

These results indicate a complex and disorderly interaction between the “segment length” and “period of time” variables, which does not follow the expected trends. The inconsistency of the results can be explained by the nature of the output that is provided by NETCELL. While the methodology may still be applicable in determining the travel time within a single cell, it is concluded that using the cumulative counts curve to derive single vehicle travel times over an entire link may lead to errors, especially in long segments where there is a greater possibility for vehicle passing, and thus the assumption of FIFO cannot be applied. An alternative way to obtain the average travel time over the entire study period would be to divide the entire area between the cumulative arrivals curve and the cumulative departures curve with the total number of vehicles. However, we are not only interested in the average travel time, but also in individual one-minute slices, that show how travel time changes with time for a fixed demand. This cannot be obtained with this methodology. For this reason, CORSIM was chosen over NETCELL as the preferred simulation software, despite having higher run times and complexity.
3.3 Development of Scenarios

In order to develop the final list of scenarios that would be tested, the conceptual test segment was implemented into CORSIM and an initial series of extensive preliminary runs were conducted. This procedure is described below.

3.3.1 Final Test Segment and CORSIM implementation

The test segment that was implemented in CORSIM follows the shape depicted in Figure 3-1 with one exception. The program sets a limit of 9999 vehicles per hour to the total entry volume input for a single direction. Thus, for the scenarios where the total entry volume had to be greater than that, as for example in scenarios with 4 lanes and a volume greater than 2500 vphpl, an additional merge segment was used prior to the entry segment (Figure 3-12). In these scenarios, 75% of the total volume comes from the pre-entry freeway lanes and 25% from the pre-entry on-ramp to give the total input of the entry segment. The geometric characteristics and the network properties (when they differ from the default values) of the final test segment are shown in Table 3-3. The “minimum separation of generation of vehicles” parameter was changed to 1 second from the 1.4 seconds that is the default, because early test runs showed that otherwise, for high demands, the simulation failed to emit the specified number of vehicles into the network.

3.3.2 Preliminary Runs

For each of the six variables (freeway demand, free-flow speed, number of lanes, segment length, time period and downstream capacity reduction) a number of values corresponding to commonly found values in the field were selected (Table 3-4). On-ramp demand was set to 50% of the corresponding freeway demand per lane. No values for capacity were selected as it would be obtained for each scenario from the
results of the simulation runs. Downstream capacity reduction is simulated by reducing the free-flow speed of the downstream bottleneck and the exact value of this parameter for each scenario will also be calculated from the data obtained from the runs. Also, for the preliminary runs, CORSIM’s default initialization period (with a maximum duration of 15 minutes unless the system reaches equilibrium earlier) was used. From all the possible combinations of variables, 810 scenarios were created. Each of these scenarios was simulated 10 times and the average travel time per vehicle (sec/vehicle) as well as the volume per lane (vehicles/hour/lane) were computed. That number of simulations was chosen in order to obtain a sample size with tolerable error 5%. The computation of travel time per vehicle in CORSIM is performed as follows. During each time step CORSIM accumulates the distance traveled by all vehicles currently on the link and accumulates the travel time for all vehicles currently on the link. At the end of the reporting interval (time interval, time period or cumulative) it computes average speed by dividing total accumulated distance traveled by total accumulated travel time. Travel time per vehicle for the link is then computed by dividing the link length by the average speed.

The results of the CORSIM simulation runs were deemed consistent with the expected trends. Average travel time per vehicle increased with the increase in demand, slightly at first (uncongested conditions) and more sharply after the threshold to congested conditions was crossed (Figure 3-13). Reducing the downstream capacity increased travel time and caused congested conditions to occur for lower demands (Figure 3-14). Increasing the free-flow speed resulted in lower travel times, with the effect being more pronounced in uncongested conditions (Figure 3-15). Travel time also
decreased in scenarios with 4 lanes when compared to their equivalent with only 3 lanes, especially in congested conditions (Figure 3-16). Increasing the segment length had no effect in average travel time for uncongested conditions but resulted in significant decrease in the average travel times of congested scenarios, as was expected, because a greater percentage of the segment was operating in uncongested conditions (Figure 3-13). For the same reasons, increasing the time period led to increased average travel times for congested conditions (Figure 3-17).

3.3.3 Final Runs

The preliminary runs showed that CORSIM simulation can be used effectively in determining the average travel time under a variety of conditions. However, two issues were identified that made it necessary to perform a revised set of simulation runs.

First, a change was implemented in the initialization period, in order to be more consistent across the various scenarios. Instead of having an initialization period that differs both in duration (as it was terminated as soon as the system reached equilibrium) and in the entry volumes (as CORSIM uses for the initialization period the entry volumes of the first time period) between each scenario, it was decided that all scenarios would have a uniform 15-minute initialization period with a demand of 1000 vphpl in order to fill up the network before the specific scenario demands are applied.

The second issue is clearly depicted in Figure 3-14. There, we can see that all scenarios where the downstream free-flow speed is 15 mph experience congested conditions, and there is no uncongested scenario. The same problem occurs for a few scenarios where the downstream free-flow speed is 25 mph. That problem could be solved by including scenarios with lower demands for those cases. At the same time, we see that most of the scenarios without a downstream bottleneck fall under
uncongested conditions; therefore the scenarios with the lowest demands could be omitted. From the preliminary results we used the throughput volumes to determine the new ranges for the freeway demand parameter. The throughput for scenarios with downstream free-flow speed of 15mph varied from 1376 vphpl to 2381 vphpl, thus a range of 1200 vphpl to 2600 vphpl was chosen for that case. In the same way, the throughput for scenarios with downstream free-flow speed of 25mph varied from 1734 vphpl to 2442 vphpl, thus a range of 1600 vphpl to 2600 vphpl was chosen. The scenarios with no downstream bottleneck were never congested with demands lower than 2000 vphpl, while the maximum throughput was 2539 vphpl, so a range of 2000 vphpl to 2600 vphpl was chosen (Table 3-5). The resulting number of scenarios for the final runs is 972.
Table 3-1. Test variables and their range of values for simulation model selection (45 scenarios)

<table>
<thead>
<tr>
<th>Variables</th>
<th>Range of values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Freeway demand (vphpl)</td>
<td>1700 2000 2300 2600 2900</td>
</tr>
<tr>
<td>Downstream capacity (% of maximum capacity)</td>
<td>100 - -</td>
</tr>
<tr>
<td>Main segment length (miles)</td>
<td>1 5 10</td>
</tr>
<tr>
<td>Period of time (min)</td>
<td>30 60 90</td>
</tr>
<tr>
<td>FFS (mph)</td>
<td>- 70</td>
</tr>
<tr>
<td># of lanes</td>
<td>- 3</td>
</tr>
</tbody>
</table>

Table 3-2. Network characteristics and input parameters of the NETCELL implementation

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Simulation control parameters</td>
<td></td>
</tr>
<tr>
<td>Clock tick length (sec)</td>
<td>5</td>
</tr>
<tr>
<td>Roadway geometry</td>
<td></td>
</tr>
<tr>
<td>Entry segment length (ft)</td>
<td>2000</td>
</tr>
<tr>
<td>Merge segment length (ft)</td>
<td>2000</td>
</tr>
<tr>
<td>Downstream bottleneck length (ft)</td>
<td>2000</td>
</tr>
<tr>
<td>On-ramp length (ft)</td>
<td>1000</td>
</tr>
<tr>
<td>Flow characteristics</td>
<td></td>
</tr>
<tr>
<td>Freeway segments capacity (veh/h/lane)</td>
<td>2415</td>
</tr>
<tr>
<td>On-ramp capacity (veh/h)</td>
<td>1770</td>
</tr>
<tr>
<td>Freeway jam density (veh/mile)</td>
<td>190</td>
</tr>
<tr>
<td>On-ramp jam density (veh/mile)</td>
<td>144</td>
</tr>
<tr>
<td>On-ramp demand (vphpl)</td>
<td>1000</td>
</tr>
</tbody>
</table>
Table 3-3. Network characteristics and input parameters of the CORSIM implementation

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Simulation parameters</strong></td>
<td></td>
</tr>
<tr>
<td>Time interval duration (sec)</td>
<td>60</td>
</tr>
<tr>
<td>Minimum separation for generation of vehicles (sec)</td>
<td>1.0</td>
</tr>
<tr>
<td><strong>Roadway geometry</strong></td>
<td></td>
</tr>
<tr>
<td>Entry segment length (ft)</td>
<td>2640</td>
</tr>
<tr>
<td>Merge segment length (ft)</td>
<td>2640</td>
</tr>
<tr>
<td>Downstream bottleneck length (ft)</td>
<td>5280</td>
</tr>
<tr>
<td>On-ramp length (ft)</td>
<td>424</td>
</tr>
<tr>
<td>Acceleration lane length (ft)</td>
<td>770</td>
</tr>
<tr>
<td>Pre-entry segment length (ft)</td>
<td>100</td>
</tr>
<tr>
<td>Pre-entry on-ramp length (ft)</td>
<td>141</td>
</tr>
</tbody>
</table>

Table 3-4. Test variables and their range of values for the CORSIM preliminary runs (810 scenarios)

<table>
<thead>
<tr>
<th>Variables</th>
<th>Range of values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Freeway demand (vphpl)</td>
<td>1800 2000 2200 2400 2600</td>
</tr>
<tr>
<td>Downstream capacity (downstream free-flow speed in mph)</td>
<td>Same as FFS 25 15</td>
</tr>
<tr>
<td>Main segment length (miles)</td>
<td>1 5 10</td>
</tr>
<tr>
<td>Period of time (min)</td>
<td>30 60 90</td>
</tr>
<tr>
<td>FFS (mph)</td>
<td>50 60 70</td>
</tr>
<tr>
<td># of lanes</td>
<td>3 4</td>
</tr>
<tr>
<td>On-ramp demand (vphpl)</td>
<td>50% of the corresponding freeway demand</td>
</tr>
</tbody>
</table>
Table 3-5. Test variables and their range of values for the CORSIM final runs (972 scenarios)

<table>
<thead>
<tr>
<th>Variables</th>
<th>Range of values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Freeway demand (vphpl) for no downstream bottleneck</td>
<td>- - 2000 2200 2400 2600</td>
</tr>
<tr>
<td>Freeway demand (vphpl) for 25mph downstream FFS</td>
<td>- 1600 1800 2000 2200 2400 2600</td>
</tr>
<tr>
<td>Freeway demand (vphpl) for 15mph downstream FFS</td>
<td>- 1200 1400 1600 1800 2000 2200 2400 2600</td>
</tr>
<tr>
<td>Downstream capacity (downstream free-flow speed in mph)</td>
<td>Same as FFS</td>
</tr>
<tr>
<td>Main segment length (miles)</td>
<td>1 5 10</td>
</tr>
<tr>
<td>Period of time (min)</td>
<td>30 60 90</td>
</tr>
<tr>
<td>FFS (mph)</td>
<td>50 60 70</td>
</tr>
<tr>
<td># of lanes</td>
<td>3 4</td>
</tr>
</tbody>
</table>

On-ramp demand (vphpl) 50% of the corresponding freeway demand
Figure 3-1. Conceptual test segment

Figure 3-2. Comparison of the default NETCELL flow-density curve and the one obtained through CORSIM, which was used in the NETCELL implementation
Figure 3-3. The computation of cell travel time $T_i(t)$ in NETCELL. This calculation applies to travel time for an entire link as well.

Figure 3-4. Travel time as a function of freeway demand and segment length (30 min time) – NETCELL preliminary results
Figure 3-5. Travel time as a function of freeway demand and segment length (60 min time) – NETCELL preliminary results

Figure 3-6. Travel time as a function of freeway demand and segment length (90 min time) – NETCELL preliminary results
Figure 3-7. Travel time as a function of freeway demand and period of time (1 mile length) – NETCELL preliminary results

Figure 3-8. Travel time as a function of freeway demand and period of time (5 miles length) – NETCELL preliminary results
Figure 3-9. Travel time as a function of freeway demand and period of time (10 miles length) – NETCELL preliminary results

Figure 3-10. Travel time as a function of period of time and segment length (2300 vphpl Freeway Demand) – NETCELL preliminary results
Figure 3-11. Travel time as a function of segment length and period of time (2300 vphpl Freeway Demand) – NETCELL preliminary results

Figure 3-12. Test segment for scenarios with entry volume higher than 10000 vph
Figure 3-13. Travel time as a function of freeway demand and segment length (No downstream bottleneck, 3 lanes, 60mph FFS, 30 min time) –CORSIM preliminary results

Figure 3-14. Travel time as a function of freeway demand and downstream capacity reduction (4 lanes, 70mph FFS, 60 min time, 1 mile main segment length) –CORSIM preliminary results
Figure 3-15. Travel time as a function of freeway demand and free-flow speed (No downstream bottleneck, 3 lanes, 10 miles main segment length, 90 min time) –CORSIM preliminary results

Figure 3-16. Travel time as a function of freeway demand and number of lanes (25mph downstream free-flow speed, 5 miles main segment length, 30 min time, 50mph FFS) –CORSIM preliminary results
Figure 3-17. Travel time as a function of freeway demand and time period (No downstream bottleneck, 3 lanes, 1 mile main segment length, 60mph FFS) – CORSIM preliminary results
CHAPTER 4
SIMULATION RESULTS AND MODEL DEVELOPMENT

From all possible combinations of variables of Table 3-5, 972 scenarios were created. Each of these scenarios was simulated 10 times and the average travel time per vehicle (sec/vehicle) as well as the volume per lane (vehicles/hour/lane) over the entire study period were computed. For 324 of these scenarios that had a time period of 90 minutes, the average travel time per vehicle (sec/vehicle) for each 1-minute long interval was also obtained. The results of these runs were used to develop three types of analytical models:

1. A model for estimating the one-minute interval travel time for congested conditions as a function of time.

2. A model for estimating the average travel time over the entire study period for either congested or uncongested conditions.

3. A model for estimating the excess or unserved volume accumulation per hour (in vehicles/hour/lane) over the entire study period.

For all models regression was used to obtain the final analytical relationship.

Section 4.1 describes how the capacity reduction parameter is computed from the data, while section 4.2 provides the methodology used to distinguish between congested and uncongested conditions. Sections 4.3 to 4.5 present the development of the three analytical model types mentioned above.

4.1 Computing Capacity Reduction

Before delving into the results of the simulation runs, the issue of the capacity reduction parameter must be addressed. As mentioned, the reduction in capacity is simulated by reducing the free-flow speed of the downstream bottleneck. However, this reduction leads to different capacity reductions (%) across the various scenarios and needs to be computed in order to be part of the analytical models.
4.1.1 Capacity Definition

Before computing the reduction in capacity we must first define capacity itself. In this study capacity is defined as the throughput of the scenarios with the maximum demand (2600 vphpl) when there is no downstream bottleneck. Free-flow speed and number of lanes have a significant impact on capacity and thus 6 different capacities were computed, for each combination of free-flow speed and number of lanes, by averaging the throughput of all scenarios under each condition (for a demand of 2600 vphpl and no downstream bottleneck). The results are shown in Table 4-1.

4.1.2 Capacity Reduction

For each scenario where there is a downstream bottleneck (free-flow speed reduction) the capacity reduction (%) was calculated with the following equation:

\[
\text{Capacity Reduction} = \frac{(\text{Capacity} - \text{Throughput})}{\text{Capacity}}
\]

Where:

- Capacity = The Capacity that corresponds to this scenario, based on the number of lanes and the free-flow speed, as calculated above
- Throughput = Average volume per lane (veh/hour/lane) of the subject segment

4.2 Defining Congested and Uncongested Conditions

The analytical model for estimating the one-minute interval travel time as a function of time will only deal with congested conditions, and also there will be two analytical models for estimating the average travel time over the entire study period, one for uncongested and one for congested conditions. For this reason it is important to define when congestion exists. It was determined that conditions will be considered congested when the “excess or unserved volume accumulation” in the entire simulation time exceeds 20 vehicles/hour, where the latter is defined as the difference between the
demand and the throughput of the subject segment. This result was obtained by correlating the excess volume accumulation with the “additional travel time” variable which is the travel time (in sec/mile) minus the minimum (zero flow) travel time. It was determined that when the excess volume accumulation was less than 20 vehicles/hour, the average additional travel time was 3.88 seconds and only in 3 out of 118 cases it was greater than 10 seconds, thus the 20 vehicles/hour excess volume accumulation was selected as the threshold value that divides congested and uncongested conditions. Using this definition, 118 scenarios (12% of the total) were uncongested conditions and 854 (88% of the total) were congested.

4.3 Development of Analytical Model for Estimating the One-minute Interval Travel Time for Congested Conditions as a Function of Time

A total of 268 scenarios corresponded to congested conditions and thus were used for the development of this model.

4.3.1 Model Development

Figure 4-1 shows the shape of the one-minute interval travel time curve as a function of time. The same shape applies to all of the congested scenarios. Figure 4-1 demonstrates that the curve is composed of 3 parts: a relatively flat part at the beginning, a significant and relatively steady slope in the middle part, and a flat part at the end. The flat part at the beginning represents the time required for the system to react to the increased demand occurring after the initialization period. The second part represents the period where the queue in the subject segment progressively increases, and the flat part at the end is the period when the subject segment is fully congested.

It was determined that the curve can be fully defined by specifying the maximum travel time (where the curve flattens), the average slope with which the travel time
increases until it reaches the flat point (making the reasonable assumption that the curve is a straight line during the period of increase), as well as the beginning of congested time. For each of these dependent variables a linear regression was performed. For all cases, explanatory variables that proved to be insignificant for a 95% level of confidence were dropped and the regression was performed again without them, until all remaining variables were statistically significant.

The final results of the three regressions are shown in Tables 4-2 to 4-4. It must be noted that the regression for the maximum travel time actually estimates the “additional travel time” and that the minimum (zero flow) travel time needs to be added. The regressions eventually lead to the three following analytical equations that describe the behavior of travel time as a function of time in congested conditions:

Maximum Travel Time (sec/mile) = \((\frac{3600}{FFS}) + 0.3344*d - 0.2577*c + 1006*cr - 7.9*L - 22.8*N\)

Travel Time Slope (sec/min/mile) = \(0.00605*d - 0.0038*c - 63.48*cr + 0.06742*d*cr - 0.663*L - 2.646*N_{cr}\)

Beginning of congested time (min) = \(-0.01343*d + 0.01698*c - 15.708*cr + 0.904*L + 1.55*N_{cr}\)

Where:

FFS = free-flow speed (mph)

d = freeway demand (vphpl)

c = capacity (vphpl) when there is no downstream bottleneck

\(cr\) = capacity reduction (\% of c) due to downstream bottleneck

L = segment length (miles)
\[ N = \text{number of lanes} \]
\[ N_{cr} = \text{number of lanes (when cr}>0\%\) and 0 otherwise (when there is no downstream bottleneck and thus cr=0\%).} \]

The correlation factor for all three models is very strong, as indicated by the adjusted \( R^2 \) that are 0.872, 0.877 and 0.902 for the three models respectively. The signs of all parameters are reasonable and correspond to their expected effect on the dependent variables, with the exception of the capacity reduction sign on the travel time slope equation. This is because capacity reduction is also included in the composite parameter \( d^\ast cr \) that has a positive sign and thus the total effect of an increase in capacity reduction is an increase of the travel time slope, as expected.

### 4.3.2 Sensitivity Analysis

Sensitivity analysis is performed on the three models. To test each variable’s sensitivity, the values of all the other variables are fixed to an average value across all scenarios, with the exception of number of lanes, where 3 lanes are chosen as the default. Then the variable that is to be tested takes a higher and a lower value within its range and the results are tabularized. The sensitivity analysis for the three models is shown in Tables 4-5 to 4-7. Freeway demand appears to have the greatest relative impact on both the maximum travel time and the travel time slope, while capacity has the greatest relative impact on the beginning of congested time.

### 4.4 Development of Analytical Model for Estimating the Average Travel Time

Figure 4-2 shows the results of 18 scenarios, performed with different freeway demands and different downstream bottleneck conditions. The figure shows that travel times have the following pattern. At low demands, where uncongested conditions are prevalent, travel time as a function of demand increases with a very small slope.
However, after congested conditions are reached, travel time increases with a significant slope and almost linearly with demand. Additionally, reducing the downstream capacity increases travel time and causes congested conditions to occur for lower demands. Similar graphs were created for the other scenarios that indicated the following trends: Increasing the free-flow speed results in lower travel times, with the effect being more pronounced in uncongested conditions. Travel time also decreased in scenarios with 4 lanes when compared to their equivalent with only 3 lanes, especially in congested conditions. Increasing the segment length had no effect in average travel time for uncongested conditions but resulted in significant decrease in the average travel times of congested scenarios, as was expected, because a greater percentage of the segment was operating in yet uncongested conditions. For the same reasons, increasing the time period led to increased average travel times for congested conditions. From these results it was also obvious that the behavior of the travel time equation depended heavily on whether the conditions were congested or uncongested. For these reasons, two distinct models were developed for the two conditions.

4.4.1 Model for Uncongested Conditions

A total of 118 scenarios corresponded to uncongested conditions. A linear regression was performed with the additional travel time as the dependent variable. Explanatory variables that proved to be insignificant for a 95% level of confidence were dropped and the regression was performed again without them, until all remaining variables were statistically significant. The regression result is shown in Table 4-8. As was expected, the only factor that was statistically significant is the freeway demand (while the effect of the free-flow speed is taken into consideration in the minimum travel
time that occurs at zero flow). Thus the equation for the average travel time for uncongested conditions is the following:

\[
\text{Travel Time}_{\text{unc}} \text{ (sec/mile)} = \frac{3600}{\text{FFS}} + 0.00258d
\]

Where:

- FFS = free-flow speed (mph)
- d = freeway demand (vph\(pl\))

The correlation factor was significant, with the adjusted \(R^2\) at 0.750.

4.4.2 Model for Congested Conditions

A total of 854 scenarios corresponded to congested conditions. A linear regression was performed with the additional travel time as the dependent variable. Explanatory variables that proved to be insignificant for a 95% level of confidence were dropped and the regression was performed again without them, until all remaining variables were statistically significant. The equation for the average travel time for congested conditions is the following:

\[
\text{Travel Time}_{\text{con}} \text{ (sec/mile)} = \frac{3600}{\text{FFS}} + 0.1238d - 0.1243c - 3.46L + 0.67T - 15.24N_{cr} + 0.3964d*cr - 21.524L*cr
\]

Where:

- FFS = free-flow speed (mph)
- d = freeway demand (vph\(pl\))
- c = capacity (vph\(pl\)) when there is no downstream bottleneck
- cr = capacity reduction (% of c) due to downstream bottleneck
- L = segment length (miles)
- T = time period (min)
\text{N}_{cr} = \text{number of lanes (when cr>0\%) and 0 otherwise (when there is no downstream bottleneck and thus cr=0\%).}

The correlation factor was significant, with the adjusted R$^2$ at 0.932. The signs of all parameters are reasonable and correspond to their expected effect on travel time.

\textbf{4.4.3 Sensitivity Analysis}

Sensitivity analysis is performed on the average travel time model for congested and uncongested conditions. To test each variable’s sensitivity, the values of all the other variables are fixed to an average value across all scenarios, with the exception of number of lanes, where 3 lanes are chosen as the default. Then the variable that is to be tested takes a higher and a lower value within its range and the results are tabularized. The sensitivity analysis for the two models is shown in Tables 4-11 and 4-12. Free-flow speed has the greatest impact on travel time for uncongested conditions, while freeway demand has the greatest impact on travel time for congested conditions.

\textbf{4.4.4 Comparison with other Analytical Models}

In order to compare our model with the analytical models that were discussed in the literature review, it is necessary to make a number of assumptions for the variables that are not shared by all the models. The variables common in all models are free-flow speed, freeway demand and capacity. A free-flow speed of 60 mph for all models is used in the comparison, while freeway demand varies in range from 1400 to 2200 vehicles per hour. Also, capacity reduction is being considered in all models, directly as an independent variable in our model, or indirectly in the other analytical models, by modifying the demand to capacity ratio. The average capacity reduction of 25\% that was computed for the sensitivity analysis will be used. Akçelik’s model shares an additional variable with our model, the time period. For both models a time period of 30
minutes is used. The variables that are exclusively considered in our model are number of lanes and segment length. For these variables, the following values were used: 3 lanes and 5 miles. Finally, for Akçelik’s model, the traffic delay parameter $J_a$ takes a value of 0.1, that is typical for freeways (Akçelik, 1991). The resulting travel times of the four models are shown in Figure 4-3. The CORSIM-derived analytical model differs in shape than all the other models, because of the abrupt “step” that occurs when the conditions change from uncongested to congested. This is due to the use of two different equations for uncongested and congested conditions. However, it is arguably more realistic than the other analytical models, as speeds tend to drop abruptly when a breakdown in flow occurs. Another conclusion of the comparison is that in our model and in Akçelik’s model travel time under congested conditions increases linearly, which is consistent with queuing theory, while in the MTC and BPR models it increases exponentially. The above conclusion indicates that our model and Akçelik’s are compatible with each other and can match to a significant degree under the appropriate conditions. The difference between the proposed model and Akçelik’s model can be attributed to the fact that Akçelik’s model does not consider the segment length and thus it does not take into account the travel times of vehicles that are still travelling in the non-congested portion of each segment. It can be inferred that the equivalent segment length for Akçelik’s model under congested conditions increases with time to encompass all vehicles that are queued. Therefore, to be able to compare it to our model, Akçelik’s model is adjusted to take into account the percentage of the segment length that is congested or uncongested. The adjusted Akçelik model is as follows:

$$t_{\text{Akçelik}_{\text{adj}}} = t_{\text{Akçelik}_{\text{con}}} * (p\%_{\text{con}}) + t_{\text{Akçelik}_{\text{unc}}} * (1 - p\%_{\text{con}})$$
Where:

\( t_{\text{Akçelik, adj}} \) = travel time per unit of distance in the adjusted Akçelik model

\( t_{\text{Akçelik, con}} \) = travel time per unit of distance under congested conditions in the original Akçelik model

\( t_{\text{Akçelik, unc}} \) = travel time per unit of distance under uncongested conditions in the original Akçelik model

\( p_{\%\text{con}} \) = percentage of the segment length that is congested

The percentage of the segment length that is congested is estimated as the ratio of the average travel time over the entire study period divided by the maximum travel time that was obtained from the one-minute interval travel time equations.

Figure 4-4 shows the comparison between the adjusted Akçelik model and the proposed model, which shows that the two of them now yield more similar results.

### 4.5 Development of Analytical Model for Estimating the Excess Volume Accumulation

For the development of this model there is no distinction made between congested and uncongested conditions. Therefore, all 972 scenarios were used in the regression.

#### 4.5.1 Model Development

A linear regression was performed with the excess volume accumulation as the dependent variable (Table 4-10). Explanatory variables that proved to be insignificant for a 95% level of confidence were dropped and the regression was performed again without them, until all remaining variables were statistically significant. The equation for the excess volume accumulation is the following:

\[
\text{Excess Volume Accumulation (veh/h)} = 0.7156 \times d - 0.6529 \times c + 769 \times cr + 0.5884 \times d \times cr - 33.68 \times N_{cr}
\]
Where:

d = freeway demand (vphpl)
c = capacity (vphpl) when there is no downstream bottleneck
cr = capacity reduction (% of c) due to downstream bottleneck
Ncr = number of lanes (when cr>0%) and 0 otherwise (when there is no downstream bottleneck and thus cr=0%).

The correlation factor was significant, with the adjusted R² at 0.958. The signs of all parameters are reasonable and correspond to their expected effect on excess volume accumulation.

4.5.2 Sensitivity Analysis

Sensitivity analysis is performed on the excess volume accumulation model. To test each variable’s sensitivity, the values of all the other variables are fixed to an average value across all scenarios, with the exception of number of lanes, where 3 lanes are chosen as the default. Then the variable that is to be tested takes a higher and a lower value within its range and the results are tabularized. The sensitivity analysis for the excess volume accumulation model is shown in Table 4-13. Freeway demand appears to have the greatest relative impact on excess volume accumulation.
### Table 4-1. Capacity as a function of number of lanes and free-flow speed

<table>
<thead>
<tr>
<th># Lanes</th>
<th>FFS (mph)</th>
<th>Capacity (veh/h)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>50</td>
<td>2240</td>
</tr>
<tr>
<td>3</td>
<td>60</td>
<td>2299</td>
</tr>
<tr>
<td>3</td>
<td>70</td>
<td>2323</td>
</tr>
<tr>
<td>4</td>
<td>50</td>
<td>2312</td>
</tr>
<tr>
<td>4</td>
<td>60</td>
<td>2335</td>
</tr>
<tr>
<td>4</td>
<td>70</td>
<td>2339</td>
</tr>
</tbody>
</table>

### Table 4-2. Maximum additional travel time regression

<table>
<thead>
<tr>
<th></th>
<th>Coefficients</th>
<th>t Stat</th>
</tr>
</thead>
<tbody>
<tr>
<td>Capacity Reduction %</td>
<td>1006.01850</td>
<td>18.92660</td>
</tr>
<tr>
<td>Lanes #</td>
<td>-22.82736</td>
<td>-2.03833</td>
</tr>
<tr>
<td>Freeway Demand (vphpl)</td>
<td>0.33436</td>
<td>17.52120</td>
</tr>
<tr>
<td>Length (miles)</td>
<td>-7.90305</td>
<td>-5.03678</td>
</tr>
<tr>
<td>Capacity</td>
<td>-0.25773</td>
<td>-8.75346</td>
</tr>
<tr>
<td>Adjusted R Square</td>
<td>0.872</td>
<td></td>
</tr>
</tbody>
</table>

### Table 4-3. Travel time slope regression

<table>
<thead>
<tr>
<th></th>
<th>Coefficients</th>
<th>t Stat</th>
</tr>
</thead>
<tbody>
<tr>
<td>Capacity Reduction %</td>
<td>-63.48146</td>
<td>-4.24908</td>
</tr>
<tr>
<td>Freeway Demand (vphpl)</td>
<td>0.00605</td>
<td>3.25631</td>
</tr>
<tr>
<td>Length (miles)</td>
<td>-0.66309</td>
<td>-8.22034</td>
</tr>
<tr>
<td>Demand*Cap.Red.</td>
<td>0.06742</td>
<td>9.73657</td>
</tr>
<tr>
<td>LanesCR #</td>
<td>-2.64588</td>
<td>-9.53096</td>
</tr>
<tr>
<td>Capacity</td>
<td>-0.00380</td>
<td>-1.97981</td>
</tr>
<tr>
<td>Adjusted R Square</td>
<td>0.877</td>
<td></td>
</tr>
</tbody>
</table>

### Table 4-4. Beginning of congested time regression

<table>
<thead>
<tr>
<th></th>
<th>Coefficients</th>
<th>t Stat</th>
</tr>
</thead>
<tbody>
<tr>
<td>Capacity Reduction %</td>
<td>-15.70824</td>
<td>-2.91820</td>
</tr>
<tr>
<td>Freeway Demand (vphpl)</td>
<td>-0.01343</td>
<td>-10.11474</td>
</tr>
<tr>
<td>Length (miles)</td>
<td>0.90404</td>
<td>8.26893</td>
</tr>
<tr>
<td>LanesCR #</td>
<td>1.54782</td>
<td>4.20942</td>
</tr>
<tr>
<td>Capacity</td>
<td>0.01698</td>
<td>11.09789</td>
</tr>
<tr>
<td>Adjusted R Square</td>
<td>0.902</td>
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</table>
Table 4-5. Sensitivity analysis of maximum travel time (MTT) model

<table>
<thead>
<tr>
<th>Variables</th>
<th>Average Values</th>
<th>Minimum Values</th>
<th>Maximum Values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Variable Value</td>
<td>MTT (sec/mile)</td>
<td>Variable Value</td>
</tr>
<tr>
<td>Freeway Demand (vphpl)</td>
<td>2200</td>
<td>346.57</td>
<td>1800</td>
</tr>
<tr>
<td>Capacity (vphpl)</td>
<td>2300</td>
<td>346.57</td>
<td>2200</td>
</tr>
<tr>
<td>Capacity Reduction (%)</td>
<td>25</td>
<td>346.57</td>
<td>0</td>
</tr>
<tr>
<td>Length (miles)</td>
<td>5</td>
<td>346.57</td>
<td>1</td>
</tr>
<tr>
<td>FFS (mph)</td>
<td>60</td>
<td>346.57</td>
<td>50</td>
</tr>
<tr>
<td># of lanes</td>
<td>3</td>
<td>346.57</td>
<td>4</td>
</tr>
</tbody>
</table>

Table 4-6. Sensitivity analysis of travel time slope (TT slope) model

<table>
<thead>
<tr>
<th>Variables</th>
<th>Average Values</th>
<th>Minimum Values</th>
<th>Maximum Values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Variable Value</td>
<td>TT Slope (sec/min/mile)</td>
<td>Variable Value</td>
</tr>
<tr>
<td>Freeway Demand (vphpl)</td>
<td>2200</td>
<td>14.53</td>
<td>1800</td>
</tr>
<tr>
<td>Capacity (vphpl)</td>
<td>2300</td>
<td>14.53</td>
<td>2200</td>
</tr>
<tr>
<td>Capacity Reduction (%)</td>
<td>25</td>
<td>14.53</td>
<td>0</td>
</tr>
<tr>
<td>Length (miles)</td>
<td>5</td>
<td>14.53</td>
<td>1</td>
</tr>
<tr>
<td># of lanes</td>
<td>3</td>
<td>14.53</td>
<td></td>
</tr>
</tbody>
</table>

Table 4-7. Sensitivity analysis of beginning of congested time (BoCT) model

<table>
<thead>
<tr>
<th>Variables</th>
<th>Average Values</th>
<th>Minimum Values</th>
<th>Maximum Values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Variable Value</td>
<td>BoCT (min)</td>
<td>Variable Value</td>
</tr>
<tr>
<td>Freeway Demand (vphpl)</td>
<td>2200</td>
<td>14.75</td>
<td>1800</td>
</tr>
<tr>
<td>Capacity (vphpl)</td>
<td>2300</td>
<td>14.75</td>
<td>2200</td>
</tr>
<tr>
<td>Capacity Reduction (%)</td>
<td>25</td>
<td>14.75</td>
<td>0</td>
</tr>
<tr>
<td>Length (miles)</td>
<td>5</td>
<td>14.75</td>
<td>1</td>
</tr>
<tr>
<td># of lanes</td>
<td>3</td>
<td>14.75</td>
<td>4</td>
</tr>
</tbody>
</table>
Table 4-8. Average additional travel time for uncongested conditions regression

<table>
<thead>
<tr>
<th>Coefficients</th>
<th>t Stat</th>
</tr>
</thead>
<tbody>
<tr>
<td>Freeway Demand (vphpl)</td>
<td>0.00258</td>
</tr>
<tr>
<td>Adjusted R Square</td>
<td>0.75</td>
</tr>
</tbody>
</table>

Table 4-9. Average additional travel time for congested conditions regression

<table>
<thead>
<tr>
<th>Coefficients</th>
<th>t Stat</th>
</tr>
</thead>
<tbody>
<tr>
<td>Freeway Demand (vphpl)</td>
<td>0.12383</td>
</tr>
<tr>
<td>Length (miles)</td>
<td>-3.46475</td>
</tr>
<tr>
<td>Time (min)</td>
<td>0.67342</td>
</tr>
<tr>
<td>Demand*Cap.Red.</td>
<td>0.39641</td>
</tr>
<tr>
<td>Lanes_{CR} #</td>
<td>-15.24007</td>
</tr>
<tr>
<td>Capacity</td>
<td>-0.12430</td>
</tr>
<tr>
<td>Adjusted R Square</td>
<td>0.932</td>
</tr>
</tbody>
</table>

Table 4-10. Excess volume accumulation regression

<table>
<thead>
<tr>
<th>Coefficients</th>
<th>t Stat</th>
</tr>
</thead>
<tbody>
<tr>
<td>Capacity Reduction %</td>
<td>768.99635</td>
</tr>
<tr>
<td>Freeway Demand (vphpl)</td>
<td>0.71558</td>
</tr>
<tr>
<td>Demand*Cap.Red.</td>
<td>0.58843</td>
</tr>
<tr>
<td>Capacity</td>
<td>-0.65293</td>
</tr>
<tr>
<td>Lanes_{CR} #</td>
<td>-33.68058</td>
</tr>
<tr>
<td>Adjusted R Square</td>
<td>0.958</td>
</tr>
</tbody>
</table>
Table 4-11. Sensitivity analysis of average travel time for uncongested conditions (TTU) model

<table>
<thead>
<tr>
<th>Variables</th>
<th>Average Values</th>
<th>Minimum Values</th>
<th>Maximum Values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Variable Value</td>
<td>TTU (sec/mile)</td>
<td>%</td>
</tr>
<tr>
<td>Freeway Demand (vphpl)</td>
<td>2200</td>
<td>65.68</td>
<td>-18.18%</td>
</tr>
<tr>
<td>FFS (mph)</td>
<td>60</td>
<td>65.68</td>
<td>-16.67%</td>
</tr>
</tbody>
</table>

Table 4-12. Sensitivity analysis of average travel time for congested conditions (TTC) model

<table>
<thead>
<tr>
<th>Variables</th>
<th>Average Values</th>
<th>Minimum Values</th>
<th>Maximum Values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Variable Value</td>
<td>TTc (sec/mile)</td>
<td>%</td>
</tr>
<tr>
<td>Freeway Demand (vphpl)</td>
<td>2200</td>
<td>214.77</td>
<td>-18.18%</td>
</tr>
<tr>
<td>Capacity (vphpl)</td>
<td>2300</td>
<td>214.77</td>
<td>-4.35%</td>
</tr>
<tr>
<td>Capacity Reduction (%)</td>
<td>25</td>
<td>214.77</td>
<td>0</td>
</tr>
<tr>
<td>Length (miles)</td>
<td>5</td>
<td>214.77</td>
<td>-80.00%</td>
</tr>
<tr>
<td>Time Period (min)</td>
<td>60</td>
<td>214.77</td>
<td>-50.00%</td>
</tr>
<tr>
<td>FFS (mph)</td>
<td>60</td>
<td>214.77</td>
<td>-16.67%</td>
</tr>
<tr>
<td># of lanes</td>
<td>3</td>
<td>214.77</td>
<td></td>
</tr>
</tbody>
</table>

Table 4-13. Sensitivity analysis of excess volume accumulation (EVA) model

<table>
<thead>
<tr>
<th>Variables</th>
<th>Average Values</th>
<th>Minimum Values</th>
<th>Maximum Values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Variable Value</td>
<td>EVA (veh/h)</td>
<td>%</td>
</tr>
<tr>
<td>Freeway Demand (vphpl)</td>
<td>2200</td>
<td>487</td>
<td>-18.18%</td>
</tr>
<tr>
<td>Capacity (vphpl)</td>
<td>2300</td>
<td>487</td>
<td>-4.35%</td>
</tr>
<tr>
<td>Capacity Reduction (%)</td>
<td>25</td>
<td>487</td>
<td>0</td>
</tr>
<tr>
<td># of lanes</td>
<td>3</td>
<td>487</td>
<td></td>
</tr>
</tbody>
</table>
Figure 4-1. One-minute interval travel time as a function of time (15 mph downstream free-flow speed, 2200 vphpl freeway demand, 4 lanes, 5 miles main segment length, 50mph FFS) –CORSIM final results

Figure 4-2. Travel time as a function of freeway demand and downstream capacity reduction (3 lanes, 70mph FFS, 60 min time, 5 miles main segment length) – CORSIM final results
Figure 4-3. Comparison of BPR, MTC, Akçelik and the model that was derived from CORSIM simulation runs

Figure 4-4. Matching results for the adjusting Akçelik model and the model that was derived from CORSIM simulation runs
CHAPTER 5
CONCLUSIONS AND RECOMMENDATIONS

5.1 Summary and Conclusions

A review of the existing literature indicated that while many methods, techniques, analytical equations and software deal with the issue of estimating travel time, there is a lack of a robust but simple to apply analytical relationship. For this reason, a new analytical travel time equation was developed based on results from simulation. A macroscopic (NETCELL) and a microscopic (CORSIM) simulation program were evaluated and in the end CORSIM was selected, due to the nature of its output. A number of variables that affect travel time and their range of values were selected and 972 scenarios were implemented in the program. From the results, with the use of linear regression, analytical travel time models were developed and compared to the existing analytical models. It was concluded:

- The following parameters have a significant impact on freeway travel time: freeway demand, capacity reduction, segment length, time period, free-flow speed and number of lanes.

- Analytical models were developed to estimate the one-minute interval travel time as a function of time, the average travel time over the entire study period and the excess or unserved volume accumulation.

- Different models were developed for congested and for uncongested conditions.

- The final set of analytical models is presented in Table 5-1.

- Sensitivity analysis of the models showed that freeway demand has the greatest relative impact on the results of all models except two. The beginning of congested time is only related to segment length, while free-flow speed is the most prominent factor affecting average travel time under uncongested conditions.
5.2 Model Applications and Further Research

The models developed in this study can be used to estimate freeway travel time easily and quickly, especially compared to a full-scale simulation of the corridor. For this reason they can be applied in the following:

- Planning applications over entire freeway corridors. For example, our travel time model could be incorporated into FREEPLAN, a software analysis tool developed by the Florida Department of Transportation for conceptual planning and preliminary engineering applications concerning freeways. Currently, FREEPLAN only calculates level of service and determines maximum service volumes, but lacks the tools to estimate travel time.

- Travel time reliability applications. For example, Florida’s Department of Transportation has been developing a procedure for assessing travel time reliability on a yearly basis for the entire freeway system of the state, a key component of which is the estimation of travel time for a number of different scenarios, including congestion, rainfall, incidents, work zones and combinations of them. Our model is capable of providing the travel time for all of these varying sets of conditions and for different hourly demand volumes.

- Incident and work zone impact studies. Due to the way our model handles capacity reduction for downstream bottlenecks, it is ideal for use in determining the impact of incidents as well as work zones on travel time and level of congestion.

Suggestions for future research include expanding the list of scenarios to include all different freeway segments (diverging, weaving, etc.), as well as a variety of starting conditions. For example, it would be useful to evaluate scenarios were the starting conditions are congested, and that congestions either persists or dissipates. Studies using field data should also by conducted in order to validate the results of the simulation runs and calibrate the parameters to better emulate the observable conditions. Finally, a more sophisticated definition of congested conditions could be investigated. The use of shockwave analysis to determine congestion, instead of using the excess volume accumulation as a threshold, should be explored.
Table 5-1. Analytical models developed and the resulting equations

Analytical model for estimating the one-minute interval travel time curve as a function of time

Maximum Travel Time (sec/mile) = \( \frac{3600}{FFS} + 0.3344d - 0.2577c + 1006cr - 7.9L - 22.8N \)

Travel Time Slope (sec/min/mile) = \( 0.00605d - 0.0038c - 63.48cr + 0.06742d*cr - 0.663L - 2.646Ncr \)

Beginning of congested time (min) = \( -0.01343d + 0.01698c - 15.708cr + 0.904L + 1.55Ncr \)

Analytical model for estimating the average travel time over the entire study period

Travel Time for Uncongested Conditions (sec/mile) = \( \frac{3600}{FFS} + 0.00258d \)

Travel Time for Congested Conditions (sec/mile) = \( \frac{3600}{FFS} + 0.1238d - 0.1243c - 3.46L + 0.67T - 15.24Ncr + 0.3964d*cr - 21.524L*cr \)

Analytical model for estimating the excess volume accumulation over the entire study period

Excess Volume Accumulation (veh/h) = \( 0.7156d - 0.6529c + 769cr + 0.5884d*cr - 33.68Ncr \)
LIST OF REFERENCES


BIOGRAPHICAL SKETCH

Georgios Chrysikopoulos was born in Athens, Greece in 1983. After completing his secondary education at the 1st Public High School of Argiroupolis, Athens, he entered the National Technical University of Athens in 2001. He received the degree of Bachelor of Science in civil engineering in 2008. In August 2008, Georgios began his graduate studies as a student at the University of Florida at Gainesville and in 2010 he earned the degree of Master of Science in civil engineering.