Session #6

SPEED AND DELAY PREDICTION MODELS FOR PLANNING APPLICATIONS

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ABSTRACT

Estimation of vehicle speed and delay is fundamental to many forms of transportation planning analyses including air quality, long-range travel forecasting, major investment studies, and congestion management systems. However, existing planning-level techniques do a poor job of estimating the duration and extent of congestion. To improve the state of the practice, a simplified queuing-based model, QSIM, was developed. QSIM incorporates several features including: the use of temporal distributions as a basis for developing hourly traffic estimates; estimation of "peak spreading"; accounting for daily variation in traffic by allowing hourly traffic estimates to vary stochastically; for freeways, the inclusion of a capacity drop after flow has broken down (i.e., after the onset of queuing) to model the growth and dissipation of queues; for arterials, considering the effects of signal density and progression; separate functions to estimate speeds in queuing and free-flow conditions based on relationships developed with microscopic traffic simulation models; use of the concept of highway capacity to determine when traffic operates under free-flow and queuing conditions as well as a basis for estimating free-flow speeds and the extent of queuing on the test link; and estimating delay rather than speed as the predictive variable. (Speed is then developed as a function of delay and free-flow speed.) The model was used to develop a dataset from which a series of predictive equations were developed. The equations use only a few, readily available independent variables. Application of the new procedure shows that under congested conditions, it predicts substantially more delay than do traditional methods. Future work includes field validation of the models and extending them to cover a variety of bottleneck conditions.

Introduction

Background

Estimating speeds for planning applications has become an important issue for transportation analysts to deal for a variety of reasons, including:

- **Long-Range Transportation Planning.** The foundation of long-range planning is the use of travel forecasting models which predict future traffic on a transportation network as a function of expected changes in land use and demographic patterns. Vehicle speeds on the network are used during the trip distribution and traffic assignment phases of the process.

- **Congestion Management Systems (CMS).** Performance measures are an integral part of a CMS and define the extent of congestion and permit the evaluation of the effectiveness of congestion reduction and mobility enhancement strategies. Thus, performance measures fall into two categories: those that relate to congestion (interference from traffic as it affects systems performance) and mobility (the ability to complete desired trips).

- **Major Investment Studies.** Major investment studies are in-depth investigations of large-scale transportation projects. They are an outgrowth of traditional benefit/cost (B/C) analyses conducted for highway projects but they are usually multimodal in nature. Also, the scope of a major investment study is larger and includes impacts beyond those that strictly accrue to the transportation system (e.g., financing and social issues). In both major investment studies and traditional B/C, determining vehicle speeds for each alternative is essential for:

    1. Estimating travel time savings, which can be half or more of the total transportation-related benefits of the project;

    2. Providing a basis for calculating vehicle operating costs; and

    3. Estimating the externalities of pollutant emissions and noise impacts.

- **Air Quality Planning.** For air quality planning, the MOBILE and EMFAC models require users to provide a variety of inputs, including vehicle speeds for the particular time period being analyzed. Much of the recent research activities in speed estimation techniques have been undertaken to improve the speed inputs to these models. These activities have used the post-processor approach, i.e., refining the speeds that are used to make assignments in travel forecasting models.

For undersaturated (i.e., uncongested) conditions on freeways, existing methods provide reasonable results. However, these methods deal with the consequences of oversaturated conditions by focusing only on specific links or highway segments for the peak hour only. Actually, the queue will not only affect speeds in the peak hour but it will also affect at least the next hour, and maybe more, depending on how long it takes to dissipate. Further, the queue may spill back onto upstream freeway sections that static analyses would indicate have V/C ratios less

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than 1.0. In this situation, traditional speed estimation methods will predict higher speeds but in reality these upstream sections are influenced by the queue that has formed downstream of them.

The situation is exacerbated when travel forecasting (network) models are used as the basis for estimating speeds. Within the structure of network models, the basic problem is illustrated in Figure 1. Network models routinely predict volumes well in excess of link capacity due to the way in which the internal algorithms work. While some traffic will be diverted off of congested facilities during the assignment process, it is still common for assigned volumes to exceed link capacities, sometimes by 50 percent or more when future year forecasts are made. (This is shown in Figure 1 as the divergent solid lines.) By definition, it is never possible to observe volumes in excess of capacity in the field; traffic will grow up to the physical capacity at which point flow will break down and a queue will form upstream (backward) from this location. (This is shown at Point A in Figure 1.) If the link is a freeway, prior research has shown that volumes will actually drop to less than capacity after flow has "broken down". Traffic may again build back up on the link due to volumes from intersecting links, but when it hits the physical capacity, a queue will form once again (Points B through D). It is precisely this condition that the speed estimation procedure developed here addresses.

In addition to the recurring congestion problem, the effect of incidents on speeds has not been addressed in the preparation of emissions inventories. Nonrecurring congestion accounts for 40 to 70 percent of all congestion in urban areas (Ref. 1). However, no methods have been developed to estimate what effect nonrecurring congestion has on speeds for air quality purposes. Addressing nonrecurring congestion's effects on speeds is not within the scope of this paper.

Requirements of Speed Estimation Procedures for Planning Applications

Based on the preceding discussion, speed estimation procedures for transportation planning applications should have the following characteristics:

- **Ease of use.** The procedure should be able to be applied in a simple fashion and require only a few, easily obtainable input data items. Ideally, the data items should be generally available to planners or collected as part of routine planning studies.

- **Consideration of traffic variability.** Traffic demand on a facility varies substantially day-to-day. For strictly recurring congestion, this means that for facilities that *on average* operate close to capacity, some days will see demand volumes in excess of capacity while others will operate below capacity.

- **Consideration of saturated conditions.** Nearly all of the highway capacity research to date has focused on parameters (speed, flow, and density) for unsaturated traffic conditions (the so-called "stable flow regime"). However, a major concern of practicing engineers and planners is what occurs after oversaturation (i.e., standing queues)[1] has occurred.

- **Use of *HCM* concepts and methods as a foundation.** The *HCM* provides a useful starting point not only because it is widely accepted but the delineation of capacity

[1] For the purpose of this research, the terms "oversaturation" and "congestion" are used equivalently and indicate what traffic engineers call "forced flow", that is, the traffic flow that occurs after capacity is exceeded (Level of Service F).

divides the two flow regimes (stable and unstable). Also, because many factors go into the determination of capacity (e.g., lane width and truck mix), it can be used as a single surrogate for many other factors influencing speed.

- **Application to travel forecasting models.** Travel forecasting (network) models are used to develop data for air quality estimation as well as many other planning applications. Therefore, the information provided by travel forecasting models should be the basis for the speed prediction procedures.

Methodology

The research presented here is an extension of three Federal Highway Administration (FHWA) projects (Refs. 2, 3, and 4). These studies were concerned with estimating the cumulative effects of congestion on vehicle speeds over the course of an entire day. Because most analytical methods consider only the effects of peak hour congestion (such as V/C ratio), a new measurement of daily congestion was used: the Average Annual Daily Traffic-to-Capacity (AADT/C) ratio, where capacity is the two-way capacity. For reference, the AADT values that result from various AADT/C ratios for typical situations are provided in Table 1.

A simple macroscopic simulation model was developed to study the effects of queuing on speeds. This model, named QSIM, was developed to integrate results obtained from simulation runs for congested and uncongested conditions and to produce estimates of the overall effect of AADT/C on average delays due to congestion over the course of a year. QSIM analyzes the effects of temporal variations in traffic and queuing on an hour-by-hour basis for weekdays and for weekends/holidays. Weekday travel is analyzed separately in each direction--the “home-to-work” peak direction, for which the peak occurs in the morning, and the “work-to-home” direction, for which the peak occurs in the afternoon. Both freeways and signalized arterials are considered by QSIM. Unsignalized streets were also considered by the study, but were not modeled with QSIM. Figure 2 shows the operation of QSIM and the subsequent analysis that was performed.

Set Test Section Capacity

The procedure starts by defining a test section for QSIM to analyze. The capacity of the section is determined using *HCM* procedures. For the research reported herein, the following basic capacity values were used:

- Freeways – 2,300 pcphpl, based on the 1994 *HCM* for 6+ lane facilities; and

- Signalized Arterials – 900 pcphpl, based on the *HCM’s* saturation flow rate of 1,800 pcphpl and a 50 percent green time.

The test section length is also set at this time; this is a key factor in QSIM as the speed and delay of vehicles are measured over the length of the section. For this study, segment length was fixed at 1.5 miles (7,920 feet) for freeways. For signalized arterials, the length of the segment is equal to the signal spacing. Setting a variable segment length for arterials is believed to capture the effect of queuing more realistically than a fixed one. Thus, high signal densities imply a shorter segment length, therefore, a higher percentage of the link will be consumed by queuing.

Temporal Distributions and Peak Spreading

Once the AADT/C level is set, AADT is determined by multiplying AADT/C by the (two-way) capacity. Average Weekday Daily Traffic (AWDT) and Average Weekend/ Holiday Daily Traffic (AWEDT) are determined by applying factors to AADT: 1.0757 for AWDT and 0.8393 for AWEDT (Ref. 3). From these daily volumes, temporal distributions are used to determine "target" volumes by hour (Table 2). Separate distributions exist for freeways and nonfreeways; three AADT/C ratios (AADT/C less than or equal to 7, AADT/C between 7 and 11, and AADT/C greater than 11); and peak direction (morning and afternoon). Direct application of these distributions would lead to problems for AADT/C ratios on the boundary values. For example, the high AADT/C range's distribution is flatter than the middle range; this could possibly lead to predicting congestion in an hour for the middle range while not predicting congestion in the same hour for the high range. Therefore, a smoothing procedure was used as described in Table 3. This procedure accounted for problems at the boundary values and further spread out traffic throughout the day as AADT/C ratios increased above 13. This additional peak spreading feature was deemed necessary since the data for developing the distributions (713 urban ATRs [Ref. 3]) did not have a large number of sites where AADT/C was greater than 13. The formula for smoothing the distributions of AADT/C ratios greater than or equal to 13 is basically a weighted average computation between the actual hourly percentage derived in Ref. 3 and a perfectly flat daily temporal distribution. The first term in the equation represents the weight applied to the actual percentage. The second term is the weight applied to a perfectly flat distribution, which has an hourly percentage equal to (1/48), since there are two directions being considered. An example will clarify the process. For an AADT/C of 14, the "weight" on the actual percentage is 10 and the weight on the "flat" percentage (1/48) is 2. For an AADT/C of 16, these weights are 8 and 4, respectively. Note that the procedure adds volumes to off-peak hours as it subtracts from peak hours. It should be pointed out that this procedure is not based on observed data but has been instituted as a purely mechanical procedure to account for two conditions: 1) counterintuitive results at the AADT/C boundaries of the original temporal distributions; and 2) excessive queuing that would result at very high AADT/C ratios if the temporal distributions were unaltered. This second condition is really the phenomenon of peak spreading.

Stochastic Variation in Traffic Volumes

In order to account for day-to-day variability in traffic flows, QSIM stochastically determines what the test volume in a given hour should be from the "target" hourly volume (determined above) and information on hourly variability (Table 4), where the "target" volumes are the mean of normal distribution and the variance is defined as:

$$\text{Variance} \quad = \quad (\text{Coeff. of Variation} * \text{Mean})^2 \tag{1}$$

Random sampling is then used to select the test volume from this distribution.

Uncongested Speed Functions

If the test volume is less than the section's capacity, newly developed uncongested speed functions are applied. The uncongested speed functions were determined by running FRESIM and NETSIM in a series of experiments to gauge the effects of various highway and traffic conditions (Ref. 2). The results were then analyzed with multiple and nonlinear regression analysis to develop equations that predict delay. (Speed was calculated as a function of delay, as described below.) The relationships used were as follows:

Freeways

$$d_{vc} = 4.46 \text{ V/C} - 1.55 \text{ (V/C)2} - 0.05 \text{ sff V/C} + 0.044 \text{ sff (V/C)2} \quad (2)$$

Where: d_{vc} is delay due to congestion in vehicle-hours per 1,000 vehicle-miles; and

s_{ff} is free-flow speed in miles per hour.

Signalized Arterials

$$d_{sig} = (32.6 + 30.0 \text{ (V/C)2}) (1 - \text{e-.3n}) \quad (3)$$

Where: d_{sig} is delay in vehicle-hours per 1,000 vehicle-miles; and

n is the number of signals per mile, assuming an "intermediate" signal progression case (see below).

In both of these formulations, delay is the additional travel time beyond that which would result if all vehicles could traverse the section at the free-flow speed. Delay includes not only the time spent sitting at traffic control devices, but also the time lost while decelerating to a stop and then accelerating back to the free-flow speed. The second term of the equation varies between 0 and 1 and is essentially an adjustment factor for the delay due to the V/C ratio; for high values of signals per mile the term is close to 1.

The NETSIM experiments conducted to produce Eq. (3) were based on assuming a "fixed time" scenario in a simple network of multiple signals where zero offsets in the signal timing were used. (All signals in the test network were also assumed to have the same volumes and capacities.) Since the signals were closely spaced, a degree of progression existed within the system. To gauge the effect of progression, a second set of NETSIM runs was made with the same network and data, but this time with "ideal" progression in the study direction. The results show that progression can have a substantial improvement in arterial speeds (Table 5). A method for incorporating progression effects into Eq. (3) is discussed in a later section.

Perform Queuing Analysis

1. **Determine percent of link under queuing**. If test volume exceeds capacity, a queue is assumed to form. For simplicity, the program assumes that the bottleneck point from which the queue builds is at the downstream end of the segment. The program accumulates total travel time on the segment. If the length of the queue exceeds the length of the segment, total delay due to the bottleneck will naturally exceed total delay on the segment itself. (This additional delay can be estimated by increasing segment length.) For freeways, once volumes exceed capacity, vehicles are assumed to move through the bottleneck point at a flow rate less than capacity. Therefore, two basic freeway capacity values are used: 2,300 pcphpl (the new *HCM* capacity for six or more lanes) for unsaturated conditions and 2,000 pcphpl for oversaturated conditions (Ref. 6).

   Queues are estimated for the beginning and ending of each hour. If the demand volume plus any leftover queue is greater than the capacity of the section, the queue at the end of the hour is calculated by:

$$Q2 = Q1 + V - C \tag{4}$$

Where:
- Q1 = queue at the beginning of the hour (vehicles);
- Q2 = queue at the end of the hour (vehicles);
- V = demand (test) volume for the hour (vehicles); and
- C = bottleneck capacity of the section (vehicles).

2. Calculate Queue Speed. For both freeways and signalized arterials, if the V/C ratio is greater than 1.0, queuing is assumed to take place. Queuing will also affect traffic if there is a standing queue at the end of the preceding hour. If travel in the hour under consideration is affected by queuing, the program analyzes the growth (or decline) in queue length over the hour. Vehicle-hours of travel are estimated separately for those portions of the segment that are affected by queuing and those that are not. The approach developed by Dowling and Skabardonis to combine speeds for queued and unqueued conditions was modified for use here (Ref. 3). Their formulation is:

$$\text{Link Speed} = [\text{Queue Speed} * (\text{Queue Length}/\text{Link Length})] + [\text{Nonqueue Speed} * (1 - \text{Queue Length}/\text{Link Length})] \tag{5}$$

In the current formulation, the speed on the segment is based on estimating total vehicle-hours of travel (VHT) and vehicle-miles of travel (VMT) first, then computing speeds as VMT/VHT; this avoids the computational problems of the Dowling and Skabardonis approach. VMT and VHT are tracked separately for queued and unqueued portions of the test segment:

$$VMT = \{(UQL * DVOL) + (AQL * CAP)\}/5280 \tag{6}$$

Where:
- UQL = length of the segment that is not queued, in feet;
- DVOL demand volume for this hour (determined stochasti-cally from the temporal distributions), in vehicles;
- AQL = average queue length during the hour, in feet; and
- CAP = the bottleneck capacity, in vehicles.

The first term counts the number of vehicles that are entering the segment at the back of the queue. When the entire segment is consumed by a queue, this term becomes zero. The second term counts the number of vehicles in the queue that are processed through the bottleneck. Queue length is found by multiplying the number of queued vehicles by the calculated queue spacing (Equation 8).

$$VHT = (UQL * DVOL * UQDEL) + (AQL/QSPACE) \tag{7}$$

Where:
- UQDEL = unqueued delay, in hours per vehicle-foot, calculated using the uncongested delay function; and
- QSPACE = spacing of vehicles in the queue, in feet per vehicle = Queue Speed/CAP. (8)

The first term is the number of vehicle-hours experienced by vehicles on the unqueued portion of the segment. The second term calculates the number of vehicles that (on average) are in the queue during the hour. Note that QSPACE depends on the assumed queue speed, which for freeways was determined empirically from freeway data to be 15.5 mph (Table 6). For arterials, queue speed was determined analytically as capacity (vehicles per hour) times vehicle spacing (feet per vehicle) and is roughly 8-9 mph. The second term is equivalent to estimating queued VHT (QVHT) as a function of queued VMT (QVMT) and queue speed:

$$\text{QVHT} = \text{QVMT/Queue Speed} \qquad (9)$$

Letting:

$$\text{QVMT} = \text{AQL} * \text{CAP (the second term in Equation (6)), and}$$
$$\text{Queue Speed} = \text{CAP} * \text{QSPACE},$$

produces:

$$\text{QVHT} = (\text{AQL} * \text{CAP})/(\text{CAP} * \text{QSPACE}) = \text{AQL/QSPACE}. \qquad (10)$$

Note that in the methodology, the traditional speed/flow/density relationships are used. There is some evidence that in the congested (unstable) traffic flow regime these relationships do not apply. Additional microscopic simulation carried out by the authors indicates that the *number of vehicle merging operations over a given space* significantly influences speed, flow, and density (Ref. 7). Vehicle merges are in turn a function of the type of bottleneck and traffic demand. The assumed freeway queue speed of 15.5 mph is felt to be representative of an on-ramp bottleneck, therefore, the current research applies to this situation only. (However, on-ramps are the most common form of bottlenecks in most areas.)

Hourly Speed Estimates

The strategy of predicting delay rather than speed was taken for two reasons. First, it is computationally more efficient because it avoids the calculation of average speeds weighted by volumes. Second, it allows the inclusion of multiple sources of delay (such as delay attributable to grades and curves) in the final speed estimation. Since delay is defined as the time that is experienced by vehicles in excess of what it would have been at the free-flow speed, the final speed on the segment can be computed as:

$$S = \frac{1}{\frac{1}{S_{ff}} + \frac{\text{Delay}}{1000}} \qquad (11)$$

Where: S is overall average speed, reflecting the combined effects of all sources of delay;
Sff is free-flow speed; and
Delay is the sum of delay from all sources (including congestion) in hours per 1,000 vehicle-miles, as computed above.

QSIM Output Data Set

QSIM completes its analysis for each of 24 hours in a day for the weekday/inbound direction, weekday/outbound direction, and weekend/holiday with both directions combined. The results are stored in a data set and the process is repeated by again randomly determining test volumes for another "day." A total of 1,000 "days" were so treated. From these 1,000 observations, delays and volume-weighted speeds were computed for each hour and test direction. For some

AADT/C levels, the speeds in a particular hour represent a combination of "days" when queuing did and did not occur due to the stochastic determination of test volumes.

Curve Fitting

The data in the tables may be used directly by analysts if desired. However, equations were developed from the data in the tables to facilitate application of the procedure. The curve fitting process was as follows. First, three separate time periods were defined: peak hour, peak period, and daily. Second, three analysis periods were defined: weekday, weekend/holiday, and combined. All hours of a day were used in the daily analyses. The peak period for both freeways and arterials included three hours in the morning (7-10 a.m.) and three hours in the afternoon (4-7 p.m.). This choice is something of a compromise since for high AADT/C levels all six hours will be congested while for low AADT/C levels fewer hours will be congested. Consideration was given to varying the length of the peak period as a function of AADT/C but was dismissed in favor of consistency. From the QSIM output, the peak hour for freeways was between 4 p.m. and 5 p.m. for AADT/C <= 10 and between 5 p.m. and 6 p.m. for AADT/C > 10. For arterials, the peak hour was between 4 p.m. and 5 p.m. Due to the nature of the polynomial models used, *the equations are valid for AADT/C levels up to 18; higher AADT/C levels are very unlikely to occur in practice.*

2. Study Results

Field Validation

A limited amount of field validation was conducted. For uncongested conditions on freeways, comparisons of QSIM's speed function were made against data from the Denver and Orlando freeway surveillance systems. QSIM's uncongested freeway speed function was found to follow the field data closely. For V/C ratios below 0.80-0.85, QSIM has a slight tendency to underpredict speeds while at V/C levels between 0.85 and 1.00 it slightly overpredicts speed.

Validation under congested conditions is more difficult using freeway surveillance data alone. The problem is that QSIM (or any other analytical method) must use the demand volume as input, even if the volume is over capacity. Such conditions are not possible to observe directly in the field since under forced flow, observed volumes will be less than the ideal capacity. A larger problem with using the freeway surveillance data in this manner is the large amount of variation that exists in both speeds and the physical capacity of the section under study. Procedures in the *HCM* would indicate that capacity is fixed. However, several important factors are either not considered in the calculation of capacity or can not be distinguished from our data. In the latter category, the field data in hand do not break out the number of trucks in the traffic stream; only total vehicles are included. Use of a fixed percent trucks in the capacity calculation does not allow for the fact that the observed number of trucks in a given hour can vary. Therefore, even accepting that *HCM* procedures account for every factor influencing capacity, our data will naturally show that physical capacity varies from day-to-day because of the fluctuation in the truck mix. Beyond this, other factors not considered by the *HCM* also may affect capacity. Traffic engineers have observed that when traffic flow approaches the theoretical capacity, it becomes highly sensitive to external influences. Under extremely dense conditions, a single aberrant driver may be all that is needed to shift the traffic stream into forced flow. Other factors that may affect capacity include weather, light conditions, and pavement condition.

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The determination of capacity is extremely important for the QSIM model because it uses capacity to determine whether traffic is operating in either the free-flow or forced flow regimes. However, given the problems with capacity noted above, comparisons between the field data (which in effect have a variable capacity) and QSIM results (based on a fixed capacity) are probably inappropriate. To get a more "fair" comparison, QSIM's capacity was allowed to vary stochastically. That is, for each hour of comparison, QSIM was run several times where the capacity value for a particular run was sampled from a normal distribution with a mean of 2,300 pcphpl with a standard deviation of 50 for Denver and 90 for Orlando. The mean value of 2,350 pcphpl was selected using the following procedure. First, daily capacity was assumed to vary both above and below the theoretical capacity. To capture those cases higher than the theoretical capacity, any hours where volume was greater than or equal to the theoretical capacity were identified. For those below, the V/C ratio had to be between 0.9 and 1.0 plus the density of vehicles had to be between 40 and 42 vehicles per lane-mile in order for the data to qualify. (This is the density at capacity suggested by the *HCM*.)

Speeds for each QSIM run were then averaged; these were used as the basis of comparison. On average, the hour-by-hour comparisons showed that QSIM overpredicted speeds by about seven percent. For entire peak periods, QSIM's performance improves: about four percent high on average. Overall, we believe that QSIM's predictions are in line with the field data.

Final Speed Equations

Freeways

Curve fitting for freeways was straightforward. Plots of the data indicated that polynomial forms of AADT/C could be easily fit to the data. The results appear in Table 7 and Figure 3. Not that the minimum speed is equal to the 15.5 mph queue speed used in QSIM. This speed is indicative of the type of bottleneck assumed, namely, an on-ramp. More onerous bottlenecks, such as lane drops and incident-caused lane blockages, would have lower speeds.

The dependent variable in these equations is weighted average hourly delay measured in hours per 1,000 vehicle-miles. For example, consider the Peak Period. The equation for predicting weekday delay (in hours per 1,000 vehicle-miles) is:

$$\text{Delay} = 0.0001732632 * (\text{AADT/C})^5 - 0.0000116968 * (\text{AADT/C})^6 + 0.0000001974 * (\text{AADT/C})^7.$$

Speed is calculated by using the formula:

$$\text{Speed} = \frac{1}{1/\text{FFS} + \text{Delay}/1000}$$

Where: FFS is the free-flow speed.

As a complete example, assume a six-lane freeway with a calculated one-way (*HCM*) capacity (based on three lanes in one direction) of 6,600 vphpl and a free-flow speed of 60 mph. Its AADT is 158,400 vpd; therefore, its AADT/C is: 158,400/(2 * 6,600) = 12.0. Delay for Weekday Peak Period is calculated using the above formula and is 15.2602 hours per 1,000 vehicle miles. Weekday Peak Period Speed (i.e., the weighted average speed for the six-hour period defined as the Peak Period) is:

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$$\text{Speed} = \frac{1}{1/60 + 15.2602/1000}$$

$$= 31.32 \text{ mph.}$$

Signalized Arterials

Curve fitting for arterials was far more complex due to the extra influence of signal density, in addition to AADT/C. The same basic functional form was assumed as for the uncongested delay function as described previously. Two separate models were fit from the data with the cutoff point being an AADT/C value of seven (Table 8 and Figure 4).

AADT/C <= 7 (Arterials)

$$\text{Delay} = (1 - e\text{-}.3n) * (NOQ + Q)$$

Where: n = signals per mile;
NOQ = predicted value from the "no queue equations";
and
Q = predicted value from the "queue equations".

For example, consider the Peak Period. The equation for predicting weekday delay for an AADT/C value of 5, free-flow speed of 40 mph, and three signals per mile is:

$$\text{Delay} = (1 - e\text{-}.^{3*3}) * \{32.6326 + (0.27187282 * 5^2) - (0.01054104 * 5^3) + (0.0000288004 * 5^6) - (0.0000013948 * 5^7)$$

$$= (0.59343) * (38.4530)$$

$$= 22.819 \text{ hours per 1000 vehicle-miles.}$$

Speed is calculated by using the same formula as for freeways above, except that a different free-flow speed is used:

$$\text{Speed} = \frac{1}{1/40 + 22.819/1000}$$

$$= 20.9 \text{ mph.}$$

AADT/C > 7 (Arterials)

$$\text{Delay} = (EQ{>}_7) + \{(1 - e\text{-}.^{3n}) * (NOQ)\}$$

Where: $EQ{>}_7$ is the predicted value from the "AADT/C > 7" Equations.

For example, consider a segment where AADT/C = 12, free-flow speed = 40 mph, and signals per mile = 4. Delay for the peak period on weekdays is calculated as:

$$EQ{>}_7 = \{2.789265513 * (12 - 7)\} + \{0.259827 * (12 - 7)^2 * (1 - e\text{-}.^{3*4})\}$$

$$= 13.9463 + 4.5392$$

$$= 18.4855$$

$$NOQ = 32.6326 + (0.27187282 * 12^2) - (0.01054104 * 12^3)$$
$$= 53.5667$$

Therefore:

$$Delay = 18.4855 + \{(1 - e\text{-}.^{3*4}) * 53.5667\}$$
$$= 55.9182.$$

Speed is then calculated as:

$$Speed = \frac{1}{1/40 + 55.9182/1000}$$
$$= 12.4 \text{ mph.}$$

Adjustments for the Effects of Progression

All of the QSIM analysis – including its uncongested delay function and the final equations – are based on the NETSIM results for the no-progression case. (This was defined as zero offsets in the timing of the signals in the test networks.) However, as shown in Table 5, progression results in significantly higher arterial speeds. Based on these data, it was observed that progression effectively acts to “reduce” the signal density. That is, with signal progression, the system behaves as if signal density was actually lower. Therefore, the problem was to develop an adjustment factor to reduce signals per mile in the delay prediction equations. The first step in estimating the adjustment factor was to examine the data in Table 5. For each level of signals per mile (6 and 10) and V/C ratio (0.2, 0.4, 0.6, 0.75, 0.85, and 0.95), the uncongested delay function was used to predict speeds. The next step was to add a factor to the signals per mile variable in the $(1 - e\text{-}.^{3n})$ term of the delay function to see how close the predicted speed could get to the actual speed under progression. The adjustment factor was varied iteratively until the average percent error was minimized; the factor was found to be approximately 0.167. This value implies that with progression the signal density can be thought of as one-sixth the signal density for no progression. For the high signal densities on which the factor was developed (Table 5), this ratio makes sense. However, for lower signal densities, the authors thought that the ratio was too high. To compensate for this, several mathematical functions were tested to see if the ratio could be lowered for low signal densities while still using the empirical information tested above. The form finally selected was:

$$n^{/} = \frac{2n}{n + 2} \tag{12}$$

Where:

$n^{/}$ = signals per mile, adjusted for ideal progression
n = actual number of signals per mile.

This functional form produces more reasonable results. For example, at a signal density of 10 per mile, $n^{/}$ is 1.67 which corresponds to the empirical information and at low signal densities $n^{/}$ is also low. However, note that the NETSIM experiments on which the adjustment factor is based (Table 5) are for ideal progression conditions. Also, the original (fixed time) case assumes some degree of progression since signal spacing is relatively small and the timing offsets are zero. One way to think of the situation is to consider that the base models (fixed time) assume intersection arrival flows similar to the *HCM* Arrival Type 3 and the progression case arrival flows similar to

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Arrival Type 6. It is left up to the analyst to decide which case is most appropriate for the particular situation being studied. If the analyst believes the progression situation to be less than the pure ideal, but more favorable than the base case, an intermediate value can be chosen. Therefore, the recommendation is to consider progression in some fashion when using any of the delay equations.

3.0 Application of the New Speed Estimation Methodology

Introduction

The new speed estimation methodology is applied to each segment individually. By comparison, many forms of transportation planning analyses are typically based on network (travel forecasting) models where many hundreds, sometimes thousands, of individual links exist. Speeds on these individual links are estimated during the traffic assignment process, usually with some variant of the Bureau of Public Roads (BPR) function. It is believed by many analysts that the speeds resulting from the final assignment are too crude for use in most analyses, especially air quality analysis; speeds are used to achieve realistic traffic assignments (link volumes) rather than to match any observed or theoretical norms. Therefore, many urban areas use a "post-processing" approach where the final assignments and basic link data are used to estimate speeds external to the travel forecasting process. Most areas base their post-processors on standard and well-known procedures, such as the *HCM*, and estimate speeds link-by-link. The purpose of this Section is to show how the new speed equations may be used as the basis for such post-processing of network model output.

Comparisons to Standard Practice

The new speed equations are based on roadway AADT because of its widespread availability. When the QSIM model is applied, AADT is converted to AWDT and Average Weekend/Holiday Daily Traffic (AHDT) depending on the case being studied. The final equations, however, relate AADT/C to speed so the effect of varying traffic on weekdays and weekends/holidays is automatically considered. Output from traffic forecasting models is almost always considered to be AWDT due to the way in which trip generation procedures are constructed. Therefore, analysts should first convert the models' predicted volumes (AWDT) to AADT values. The adjustment factor reported in Section 2 (i.e., the inverse of 1.0757) provides a default value in lieu of locally developed factors.

Freeways

Comparisons were made for both congested and uncongested links on a hypothetical urban freeway section as presented in Figure 5 and Table 8. The so-called "standard practice" scenario is based on applying the original BPR function for $V/C < 1.0$ and the modified BPR4 function for $V/C >= 1.0$ as post processors to travel forecasting network data. [2] The reasons for these choices

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[2] Original BPR equation speed $= FFS/(1 + (0.15 * (V/C)^4))$

Modified BPR4 equation speed $= FFS/(1 + (V/C)^4)$

Where: FFS = free-flow speed; and
V/C = volume-to-capacity ratio.

are that the original BPR function closely follows the latest freeway speed/flow curves from the 1994 *HCM*, and the BPR4 curve shows a reasonable degradation of speeds for V/C ratios over 1.0. In practice, most agencies would probably use a single function so that the comparison is geared towards what is felt to be the best procedure.[3] The BPR functions and *HCM* procedures are applied on a hourly basis, usually the "peak hour for the peak direction." Because the new speed equations consider both directions simultaneously, the offpeak direction must be estimated separately for the standard practice case. Note that the hourly V/C ratio used for standard practice is based on the K- and D-factors rather than the temporal distributions on which the new speed equations are based. Note also that the volume differences along the hypothetical segment are large and probably would not be found in the field; the authors are using these for example purposes only. Because of the volume difference, two sets of K- and D-factors (derived from HPMS data) are used. The smaller K- and D-factors for the higher congestion case indicates that at least a small degree of peak spreading is accounted for, something that may not normally be done in practice unless states or local agencies also differentiate these factors as a function of volume/ congestion.

The results, as presented in Table 9, demonstrate the implications of using standard practice versus the new speed equations. Considering only the peak hour, the new equations predict lower speeds with the gap between the procedures widening as congestion builds. Additionally, the effect of stochastic variation in traffic from day-to-day is also apparent: for an AADT/C of 9.0, the standard practice peak direction has a V/C ratio of 0.99, right at the boundary between stable and forced flow. The BPR function predicts relatively high speeds because the V/C ratio indicates that speeds are always in the stable regime. In contrast, the new equations do not assume that volumes will be constant from day-to-day for the peak hour, but on some days volumes will be high enough to kick flow over into the unstable regime.

Although not directly compared, the peak period speeds predicted using the new equations are likely to be a lot lower than those predicted using standard practice, depending on how local agencies address other hours around the peak hour. If they assume that the peak period is six hours per day (as assumed by the new equations) and if they also assume that speeds in all of these six hours are equivalent to speeds in the peak hour, then their BPR-predicted peak period speeds will be close, but still consistently higher, than those predicted by the new equations. If they assume a shorter peak period, then the volume (VMT) "exposed" to low speeds is decreased, a potentially important fact for air quality analyses. If they assume lower volumes in hours around the peak, then predicted speeds will increase. Overall, even if state and local planners followed the absolute best practices, they would still predict higher speeds than speeds predicted by the new equations developed by this research.

Signalized Arterials

A similar comparison was conducted for signalized arterials (Table 10). Because the current research verified that signal density is an extremely important factor in determining arterial speeds, the *HCM* (Chapter 11) methodology was considered to be the most desirable "standard practice" for post-processing speeds from network model output. (The BPR4 equation was also applied since it depends solely on V/C ratio and is easier to apply on a network basis.) For the peak hour (both directions), both the *HCM* and the BPR4 equation predict higher speeds than the new

[3] Even if local agencies choose not to apply the new speed equations developed in this research, they should consider using these two post-processor functions for saturated and unsaturated conditions rather than a single function.

equations, although the *HCM* results are closer to the new equation results. At the highest congestion level compared (AADT/C = 13), the predicted speeds for the *HCM* and the new equations are closer due to the nonlinear nature of the *HCM* delay equations. The six-hour peak period speeds predicted by the new equations are very close to those of the (single) peak hour, indicating that speeds are relatively insensitive to signal V/C ratio for the ranges studied. The same observations made for freeways regarding speed prediction for hours around the peak also apply to arterials.

4. Summary of the Research

This research has developed a new procedure for estimating vehicle speeds. From a theoretical standpoint, it has several advantages over other planning methods now in use:

- It is based on predicting the effects of traffic over an entire day by taking each hour in sequence. Hourly volumes are developed using temporal distributions developed from a large (713) set of urban ATRs from around the country. The effects of peak spreading as congestion builds are explicitly accounted for in the distributions.

- Day-to-day variations in traffic are considered by the model. The ATR data were used to determine the variability in hourly volumes. This information was used to vary volumes stochastically within the QSIM model, resulting in a more reliable estimate of speeds than would be obtained if only averages were used. The reason for this is the nonlinear nature of the delay. Consider a volume very near to capacity; this is the average volume that might occur on the facility over the course of a year. Only a small increase in volume will kick the flow over into the unstable speed regime. This effect, which goes unnoticed if the average is used, is addressed by allowing volumes to vary stochastically.

- Uncongested speed functions based on applying the FRESIM and NETSIM microscopic simulation models were developed. The arterial speed function contains a term for signal density, a highly important determinant of arterial speeds. The positive effects of progression are also accounted for by the arterial equations.

- The effects of congestion are incorporated by combining queuing analysis with the hourly volume estimation discussed above. Queues are built and dispersed over time and their effects on speeds are computed. The model uses an assumed speed for queued vehicles on freeways based on extensive data from freeways in Orlando and Denver. Also, a freeway queue discharge rate lower than capacity is used to dissipate queues.

- Both tables and equations are developed that allow the user flexibility in how to apply the research. Equations are developed that predict speeds for both directions on a link for the peak hour (4-5 p.m. for AADT/C <= 10 and 5-6 p.m. for AADT/C > 10), the peak period (7-10 a.m. and 4-7 p.m.), and the entire day. If only one direction is required, the data tables in Reference 4 should be used. Separate equations were developed for weekdays, weekends/holidays, and all days combined, but most analyses will probably focus on the weekdays.

- Based on the above points, the main finding of the research is that under congested conditions (queuing), traditional methods of speed estimation overestimate speeds (i.e., produce higher speed estimates than should be obtained) by a substantial margin. This overestimation is directly related to the level of congestion. For extremely congested networks, such as those encountered when performing 20-year traffic forecasts, the speed overestimation by traditional methods can be inordinately large. Conversely, for uncongested facilities, speed estimation is much simpler and a variety of methods can be used, subject to the observations in Point 7 above. Finally, the overestimation of speeds in congested conditions has ramifications for air quality planning, congestion management, and other transportation planning applications. If speeds are predicted to be higher than observed in the field, CO and VOC estimates will be lower than they should be and NO_X emissions will (usually) be higher. Other implications of using higher-than-expected speed estimates include underestimating the extent of congestion in an urban area and underestimating the benefits of highway improvements and transportation control strategies.

Because project research resources were limited, only limited field validation of the models was accomplished. Also, extension of the work to cover additional types of bottlenecks besides on-ramps should be undertaken.

References

1. Grenzeback, L. R., W. R. Reilly, P. O. Roberts, and J. R. Stowers, "Urban Freeway Gridlock Study: Reducing the Impacts of Large Trucks on Peak-Period Urban Freeway Congestion," paper prepared for Transportation Research Board, 69th Annual Meeting, January 1990.

2. Margiotta, R., H. Cohen, G. Elkins, A. Rathi, and M. Venigalla, "Speed Determination Models for HPMS: Final Report," prepared for FHWA, October 31, 1993.

3. Margiotta, R., H. Cohen, R. Morris, A. Dixson, and J. Trombly, "Roadway Usage Patterns: Urban Case Studies: Final Report," prepared for VNTSC and FHWA, July 22, 1994.

4. COMSIS Corporation, et al., "Development of Diurnal Traffic Distribution and Daily, Peak, and Offpeak Vehicle Speed Estimation Procedures for Air Quality Planning," prepared for FHWA, Work Order B-94-06, April 1996.

5. Dowling, R. and A. Skabardonis, "Improving the Average Travel Speeds Estimated By Planning Models," presented at 71st Annual Meeting of the Transportation Research Board, January 1992.

6. Hall, F.L. and K. Agyemang-Duah, "Freeway Capacity Drop and the Definition of Capacity", paper presented at the Transportation Research Board Annual Meeting, 1991.

7. Margiotta, R., A. Dixson, and M. Penic, "Examination of Freeway Bottleneck Traffic Parameters Using FRESIM", paper submitted to 77th Transportation Research Board Annual Meeting, 1997.

Table 1. AADT/C Levels and Corresponding AADT Values

	Freeways (10% Trucks)		
	4-Lanes[1]	6-Lanes[2]	4-Lane Signalized Arterials (8% Trucks)[3]
AADT/C	**AADT[4]**	**AADT[4]**	**AADT[4]**
9	72,000	113,000	30,000
10	80,000	126,000	33,000
11	88,000	138,000	37,000
12	96,000	151,000	40,000
13	104,000	163,000	43,000
14	112,000	176,000	47,000
15	120,000	188,000	50,000

[1]Ideal Capacity = 2,200 passenger cars per hour per late (pcphll)
[2]Ideal Capacity = 2,300 pcphpl
[3]Ideal Capacity = 900 pcphlp (based on a saturation flow rate of 1,800 pcphpl and 50 percent green time
[4]Rounded to nearest 1,000

Table 2. Weekday Temporal Distributio

Type of Facility: Freeway

	AADT/C						AADT/C			
	LE 7.0		7.1 - 11.0		GT 11.0		LE 7.0		7.1 - 11.	
	Peak Direction		Peak Direction		Peak Direction		Peak Direction		Peak Direct	
	AM	PM	AM	PM	AM	PM	AM	PM	AM	
	Pct. of	Pct. of	Pct. of	Pct. of	Pct. of	Pct. of	Pct. of	Pct. of	Pct. of	Pct
Hour	Daily Volume	Daily Volume	Daily Volume	Daily Volume	Daily Volume	Daily Volume	Daily Volume	Daily Volume	Daily Volume	Da Vo
1	0.42	0.58	0.44	0.57	0.47	0.54	0.34	0.47	0.37	0
2	0.27	0.33	0.27	0.34	0.27	0.32	0.21	0.28	0.23	0
3	0.23	0.25	0.22	0.26	0.20	0.24	0.15	0.18	0.17	0
4	0.23	0.22	0.21	0.21	0.18	0.18	0.14	0.14	0.16	0
5	0.38	0.29	0.35	0.28	0.31	0.25	0.24	0.18	0.28	0
6	1.17	0.68	1.12	0.69	1.06	0.72	0.74	0.42	0.81	0
7	3.26	1.75	3.16	1.90	2.86	2.18	2.23	1.19	2.35	1
8	4.83	2.90	4.59	3.05	3.90	3.27	4.11	2.28	3.85	2
9	3.56	2.57	3.80	2.76	3.66	3.04	3.45	2.33	3.42	2
10	2.58	2.24	2.75	2.30	2.94	2.53	2.64	2.29	2.69	2
11	2.46	2.33	2.50	2.34	2.68	2.49	2.64	2.56	2.65	2
12	2.56	2.56	2.61	2.61	2.73	2.69	2.90	3.02	2.90	2
13	2.65	2.71	2.68	2.75	2.75	2.78	3.20	3.35	3.17	3
14	2.70	2.77	2.75	2.81	2.82	2.86	3.14	3.24	3.14	3
15	2.93	3.12	2.93	3.15	2.97	3.15	3.18	3.44	3.116	3
16	3.26	4.01	3.21	3.87	3.21	3.60	3.40	4.13	3.35	3
17	3.47	4.81	3.38	4.43	3.28	3.82	3.46	4.78	3.49	4
18	3.42	4.85	3.32	4.39	3.29	3.77	3.31	4.83	3.45	4
19	2.66	3.23	2.66	3.20	2.82	3.22	2.68	3.23	2.75	3
20	1.95	2.23	1.97	2.25	2.12	2.36	2.14	2.41	2.18	2
21	1.54	1.78	1.54	1.79	1.62	1.86	1.73	1.97	1.75	2
22	1.40	1.63	1.44	1.69	1.54	1.74	1.49	1.71	1.50	1
23	1.14	1.30	1.19	1.39	1.27	1.46	1.10	1.26	1.11	1
24	0.79	0.98	0.83	1.05	0.89	1.07	0.74	0.94	0.75	0
TOTAL	49.87	50.13	49.92	50.08	49.84	50.16	49.36	50.64	49.67	50

Source: Ref. 5.

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Table 3. Peak Spreading Modification Procedures for Temporal Distributions

AADT/C Range	Modifications
1-7	None; low range used
8	(1/3 of low range) + (2/3 of middle range)
9	None; middle range used
10	(2/3 of middle range) + (1/3 of high range)
11	(1/3 of middle range) + (2/3 of high range)
12	None; high range used
13+	[{pct* (24 - AADT/C)} + (1/48) * (AADT/C - 12}]/12 where: pct = hourly distribution of traffic from (5)

Table 4. Average Coefficients of Variation from Urban ATR Sites

	Functional Class			
	Urban Freeway		Urban Principal Arterial	
	Day of Week		Day of Week	
	Weekend	Weekday	Weekend	Weekday
	Coeff.	Coeff.	Coeff.	Coeff.
Hour	VAR	VAR	VAR	VAR
1	26.83	26.78	28.59	33.37
2	27.32	27.34	31.79	39.77
3	26.82	25.63	33.95	42.66
4	28.59	23.29	37.59	41.05
5	34.54	22.01	42.47	34.57
6	44.64	19.12	47.37	25.77
7	51.23	14.59	50.03	17.42
8	54.66	11.85	53.36	14.42
9	41.18	11.37	41.54	13.12
10	29.77	11.71	31.63	13.08
11	25.05	12.69	28.80	13.86
12	22.18	13.14	26.52	13.74
13	18.56	13.07	21.27	13.40
14	17.47	13.05	20.23	13.62
15	17.68	12.69	20.01	13.34
16	18.89	11.75	21.01	12.62
17	20.40	11.44	21.89	12.28
18	21.20	12.48	22.62	13.35
19	21.01	16.15	23.74	16.79
20	22.29	19.66	25.78	20.32
21	23.96	20.05	28.19	22.55
22	25.38	19.94	31.49	24.18
23	29.58	23.96	36.04	29.40
24	36.33	29.32	43.51	36.35

Table 5. Effect of Progression on Vehicle Speeds

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Signals per Mile	Signalization	V/C					
		0.2	0.4	0.6	0.75	0.85	0.95
6	Progression	28.8	29.7	28.8	28.5	27.3	24.2
	Limited Progression	22.2	20.2	18.6	17.4	16.5	14.8
10	Progression	24.5	25.2	24.1	23.2	22.6	17.4
	Limited Progression	20.0	18.4	17.0	15.9	17.4	12.5

Table 6. Speeds in Queues from Freeway Field Data

City	No. 5-minute periods	Average Speed	Coefficient of Variation
Orlando (1-4)	**2,407**	**16.1 mph**	**30.3%**
Denver (1-25)	**1,263**	**15.0 mph**	**33.3%**

Source: Analysis of Freeway Management Center Data

Table 7. Coefficients for Freeway Delay Equations

PEAK PERIOD

	Weekday	Weekend	Combined
AADT/C			
$AADT/C^2$			
$AADT/C^3$			0.0347966374
$AADT/C^4$			-.0122499794
$AADT/C^5$	0.0001732632	0.0000253342	0.0016577982
$AADT/C^6$	-.0000116968		-.0000862934
$AADT/C^7$	0.0000001974	-.0000000318	0.0000015096

PEAK HOUR

	Weekday	Weekend	Combined
AADT/C			
$AADT/C^2$			
$AADT/C^3$	0.1055828979		0.0824450974
$AADT/C^4$	-.0417711090		-.0316382045
$AADT/C^5$	0.0059009773	0.0000256501	0.0043851790
$AADT/C^6$	-.0003319769		-.0002436727
$AADT/C^7$	0.0000064684		0.0000047234

DAILY

	Weekday	Weekend	Combined
AADT/C			
$AADT/C^2$			
$AADT/C^3$	0.0551483782	0.0096559689	0.0461854203
$AADT/C^4$	-.0189486676	-.0023347525	-.0154380323
$AADT/C^5$	0.0023287974	0.0001925990	0.0018559670
$AADT/C^6$	-.0001133801	-.0000044347	-.0000887095
$AADT/C^7$	0.0000018954		0.0000014614

Note: The equations should be applied up to an AADT/C value of 18.

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Table 8. Coefficients for Arterial Delay Equations

PEAK PERIOD

NO QUEUE EQUATIONS

	Weekday	Weekend	Combined
INTERCEPT	32.6326	32.8082	32.6270
AADT/C			
$AADT/C^2$	0.27187282	0.10420309	0.23911219
$AADT/C^3$	-0.01054104		-0.00898736
$AADT/C^4$		-0.00012311	
$AADT/C^5$			
$AADT/C^6$			
$AADT/C^7$			

QUEUE EQUATIONS

	Weekday	Weekend	Combined
AADT/C			
$AADT/C^2$			
$AADT/C^3$			
$AADT/C^4$			
$AADT/C^5$			
$AADT/C^6$	0.0000288004	0.0000025067	0.0000225491
$AADT/C^7$	-0.0000013948		-0.0000010684

AADT/C > 7 EQUATIONS

	Weekday	Weekend	Combined
(AADT/C - 7)	2.789265513		
$(AADT/C - 7)^2$			0.265213232
$(AADT/C - 7)^3$		0.0142379985	
$(AADT/C - 7)(1 - e^{-.3n})$			1.445427904
$(AADT/C - 7)^2(1 - e^{-.3n})$	0.259827162	0.1060087507	
$(AADT/C - 7)^3(1 - e^{-.3n})$			

Note: n = signals per mile. The equations should be applied up to an AADT/C value of 18.

Table 8. (Continued)

PEAK HOUR

NO QUEUE EQUATIONS

	Weekday	Weekend	Combined
INTERCEPT	32.9859	32.6524	32.8591
AADT/C			
$AADT/C^2$	0.36235337	0.13826170	0.31492585
$AADT/C^3$	-0.01545356		-0.01297063
$AADT/C^4$		-0.00019683	
$AADT/C^5$			
$AADT/C^6$			
$AADT/C^7$			

QUEUE EQUATIONS

	Weekday	Weekend	Combined
AADT/C			
$AADT/C^2$			
$AADT/C^3$			
$AADT/C^4$			
$AADT/C^5$	0.0004278173	0.0000639922	0.0003209231
$AADT/C^6$	-0.0000010179		
$AADT/C^7$			-0.0000007210

AADT/C > 7 EQUATIONS

	Weekday	Weekend	Combined
(AADT/C - 7)	4.935624291		
$(AADT/C - 7)^2$			0.430066318
$(AADT/C - 7)^3$		0.0213118297	
$(AADT/C - 7)(1 - e^{-.3n})$			1.916741055
$(AADT/C - 7)^2(1 - e^{-.3n})$	0.191184548	0.1612475797	
$(AADT/C - 7)^3(1 - e^{-.3n})$			

Note: n = signals per mile. The equations should be applied up to an AADT/C value of 18.

Table 8. (Continued)

DAILY

NO QUEUE EQUATIONS

	Weekday	Weekend	Combined
INTERCEPT	32.9015	32.8089	32.5177
AADT/C			
$AADT/C^2$	0.15119788	0.10661207	0.19583856
$AADT/C^3$			-0.00728030
$AADT/C^4$		-0.00015241	
$AADT/C^5$			
$AADT/C^6$	-0.00000333		
$AADT/C^7$	0.00000014		

QUEUE EQUATIONS

	Weekday	Weekend	Combined
AADT/C			
$AADT/C^2$			
$AADT/C^3$			
$AADT/C^4$			0.0007935231
$AADT/C^5$		0.0000325589	
$AADT/C^6$	0.0000144058		
$AADT/C^7$	-0.0000006601		

AADT/C > 7 EQUATIONS

	Weekday	Weekend	Combined
(AADT/C - 7)			
$(AADT/C - 7)^2$	0.1852764500		0.1586415772
$(AADT/C - 7)^3$		0.0102536589	
$(AADT/C - 7)(1 - e^{-.3n})$			
$(AADT/C - 7)^2(1 - e^{-.3n})$	0.1346060360		0.1211710141
$(AADT/C - 7)^3(1 - e^{-.3n})$		0.0097281924	

Note: n = signals per mile. The equations should be applied up to an AADT/C value of 18.

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Table 9. Comparison of New Speed Equations to Standard Practice,

		Standard Practice: Weekday Peak Hour								
		Peak Direction			Offpeak Direction			Combined Directions		Pea Hou
Link	AADT/C	Volume	V/C	Speed[2]	Volume	V/C	Speed[2]	Volume	Speed	Spe
(1,2)	7.0	3,390	0.77	57.0	2,650	0.60	58.9	6,040	57.8	55.
(2,3)	8.0	3,870	0.88	55.0	3,030	0.69	58.0	6,900	56.3	51.
(3,4)	9.0	4,355	0.99	52.4	3,410	0.78	56.8	7,765	54.3	44.
(4,5)	11.0	4,440	1.01	29.4	3,690	0.84	55.8	8,130	41.4	29.
(5,6)	13.0	5,250	1.19	20.0	4,360	0.99	52.4	9,610	34.7	20.

AADT/C	K-factor[4]	D-Factor[4]
< 11	10.4%	58.7%
>= 11	8.1%	56.1%

[1]The new equations predict speeds for both directions simultaneously.
[2]Calculated using the original BPR function for V/C < 1.0; modified BPR4 for V/C >= 1.0 (see previous footnote)
[3]The peak period is from 7-10 a.m. and 4-7 p.m.; volumes are from the Appendix.
[4]K- and D-factors developed from 1993 HPMS data.

Table 10. Comparison of New Speed Equations to Standard Practice, Sigr

AADT/C	AADT	HCM Methodology								BPR Speed
		Peak Direction			Offpeak Direction			Total Volume	Wtd Avg Speed	
		Volume	V/C	Speed	Volume	V/C	Speed			
8.0	28,800	1,622	0.90	24.6	1,229	0.68	28.0	2,851	26.1	30.2
9.0	32,400	1,825	1.01	19.1	1,382	0.77	27.1	3,208	22.6	26.6
11.0	39,600	1,942	1.08	14.7	1,582	0.88	25.2	3,524	19.4	22.5
13.0	46,800	2,295	1.20	8.7	1,870	1.04	17.4	4,165	12.6	16.8

Arrival Type = 3 (minimal progression)
m = 16 (incremental delay factor)
C = 90 sec
g/C = 0.5
FFS = 40 mph
Sig/Mile = 2.0
Capacity = 1,800 (4-lane arterial)

AADT/C	K-factor	D-factor
< 11	9.9%	56.9%
>= 11	8.9%	55.1%

Note: K- and D-factors developed from 1993 HPMS data.

Figure 1. Forecasted Network Link Volumes vs. Reality

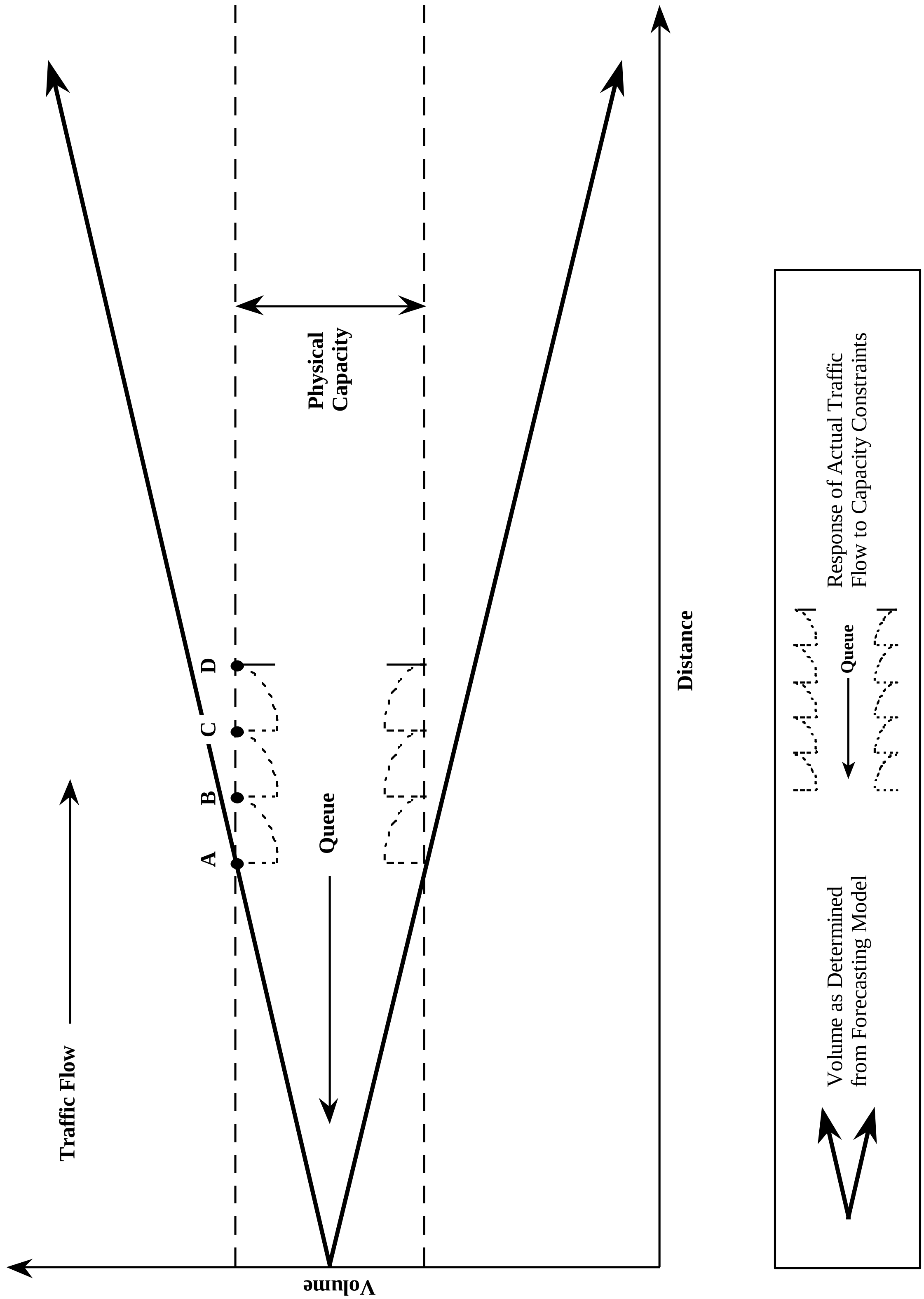

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Figure 2. Overview of QSIM Methodology

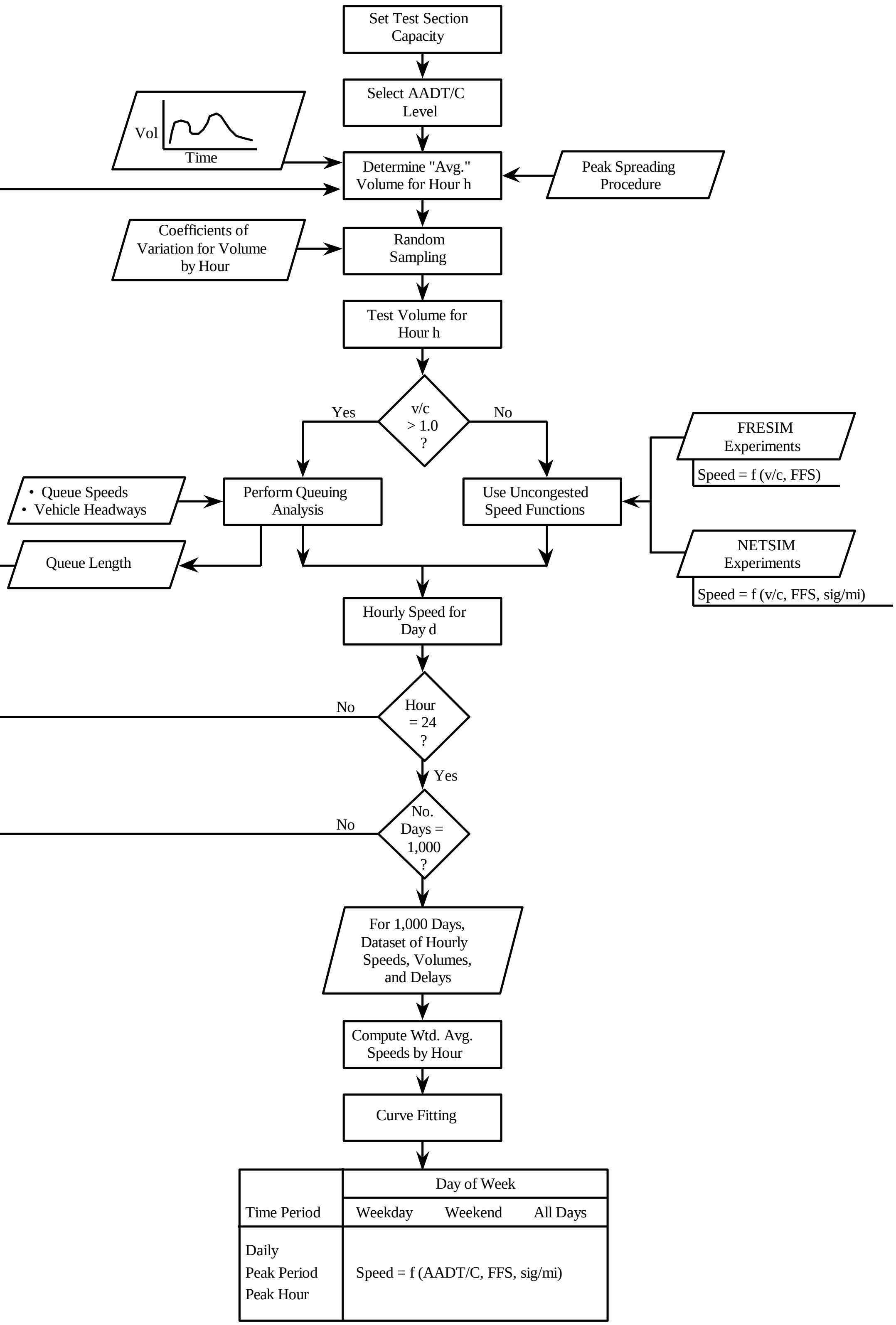
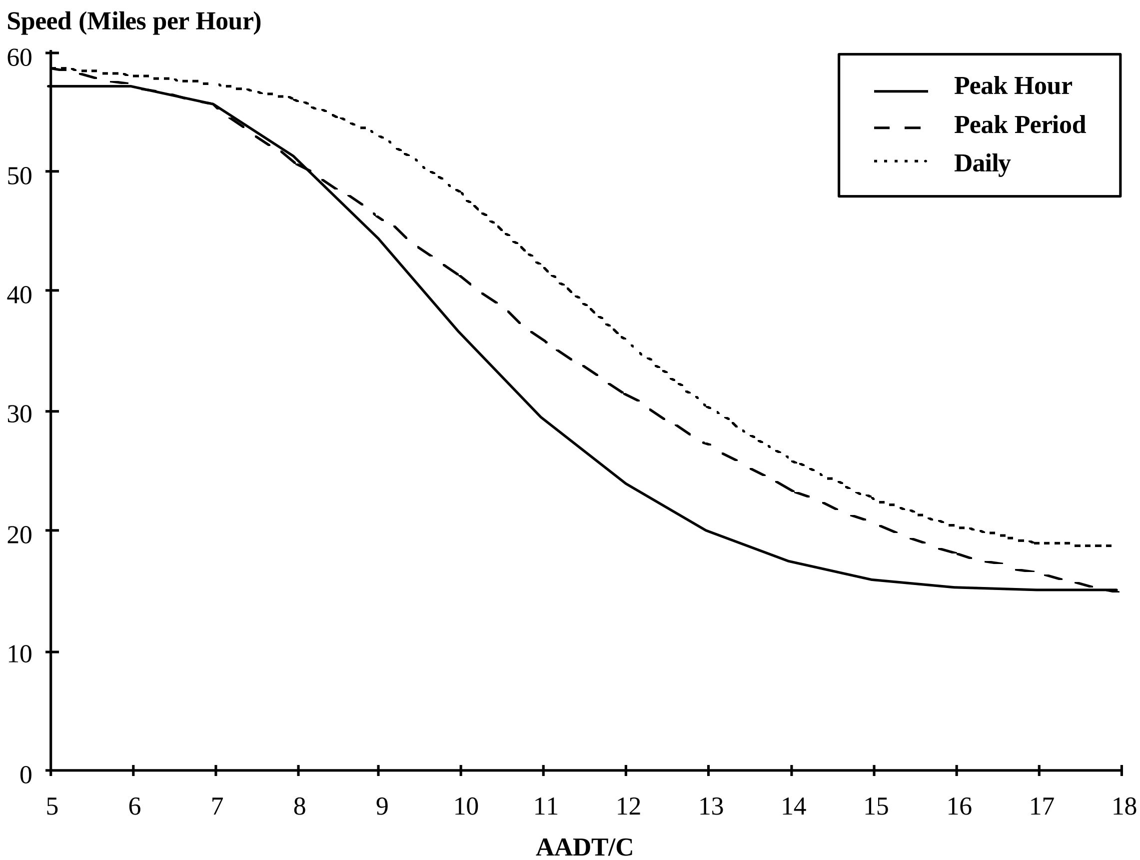


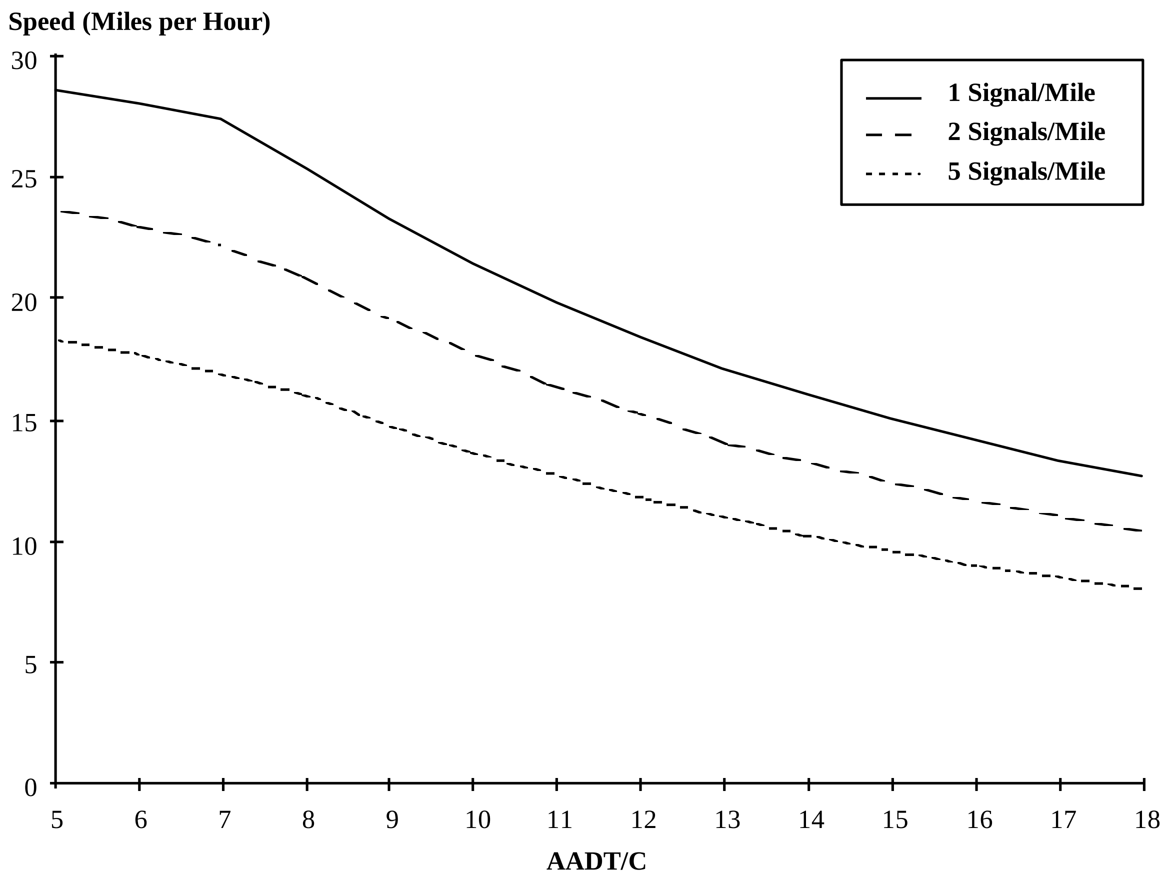
Figure 3. Speed Curves for Freeways
Weekdays



Note: FFS = 60 mph

Figure 4. Speed Curves for Arterials

Peak Period Weekdays



Note: FFS = 40 mph