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GENERIC VEHICLE SPEED MODELS BASED ON TRAFFIC SIMULATION: DEVELOPMENT AND APPLICATION

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ABSTRACT

This paper summarizes the findings of a research project to develop new methods of estimating speeds for inclusion in the Highway Performance Monitoring System (HPMS) Analytical Process. The paper focuses on the effects of traffic conditions excluding incidents (recurring congestion) on daily average speed and excess fuel consumption. A review of the literature revealed that many techniques have been used to predict speeds as a function of congestion but most fail to address the effects of queuing. However, the method of Dowling and Skabardonis avoids this limitation and was adapted to the research. The methodology used the FRESIM and NETSIM microscopic traffic simulation models to develop uncongested speed functions and as a calibration base for the congested flow functions. The chief contributions of the new speed models are the simplicity of application and their explicit accounting for the effects of queuing. Specific enhancements include: (1) the inclusion of a queue discharge rate for freeways; (2) use of newly defined uncongested flow speed functions; (3) use of generic temporal distributions that account for peak spreading; and (4) a final model form that allows incorporation of other factors that influence speed, such as grades and curves. The main limitation of the new speed models is the fact that they are based on simulation results and not on field observations. They also do not account for the effect of incidents on speed. While appropriate for estimating average national conditions, the use of fixed temporal distributions may not be suitable for analyzing specific facilities, depending on observed traffic patterns. Finally, it is recommended that these and all future speed models be validated against field data where incidents can be adequately identified in the data.

DISCLAIMER

1. STUDY BACKGROUND

Introduction

This paper summarizes the findings of a research project to develop new methods of estimating speeds for inclusion in the Highway Performance Monitoring System (HPMS) Analytical Process.[1] The research project was concerned with developing speed models and fuel consumption relationships that account for a full range of highway and geometric conditions; both urban and rural conditions had to be covered as well as the effects of congestion, grades, and horizontal curves on speed and fuel consumption. However, this paper will focus on the effects of traffic conditions excluding incidents (recurring congestion) on daily average speed. (The effects of grades and curves on speed were assessed in the study but will not be reported here.) The effect of congestion on fuel consumption was also assessed. Because of budgetary limitations, field speed data were not collected, thereby restricting the study to adapting analytical methods to the problem. A limited test against data available from the Orlando freeway management system was performed against the final models, but more extensive field validation is still warranted.

Speed Estimation In The HPMS Analytical Process

The HPMS Analytical Process was developed to analyze sample section data submitted by the states. The Analytical Process relates the highway conditions in the sample data to user costs (vehicle operating costs, travel time, and accidents) and to other performance measures (fuel consumption and emissions). It is used to simulate highway performance given a set of highway and traffic conditions. Other parts of the Analytical Process are used to schedule and cost highway improvements. In this way it can be used to test the effect of alternative investment strategies. For estimating vehicle operating costs and travel time, the key link between highway conditions and user costs is speed: highway conditions affect speed and speed in turn is used to infer these costs. Within the Analytical Process, speeds are used to calculate travel times directly and are the basis for calculating fuel consumption and emissions. The current relationships of estimating speed are based on a sequential model. First, an initial speed is selected for the highway section based on facility type, average highway speed, speed limit, and volume-to-capacity (V/C) ratio. The initial speed is then adjusted downward in a series of separate steps accounting for (in order): pavement condition; curves; grades (trucks only); speed change and stop cycles; and idling time.

This process involves look-ups from multiple tables and slows the performance of the Analytical Process software. It also may involve "double counting" of some effects. Therefore, a goal of this study was to provide the improved speed relationships in equation form. Of the above adjustments, those that attempt to account for congestion (speed change, stop cycles, and idling time) appear to be the most simplistic. They are based on data that may not be representative of urban conditions throughout the country.

Review Of Recurring Congestion Literature

The *Highway Capacity Manual* (*HCM*; 1) serves as a focal point for research on highway capacity and the relationships between highway congestion and speed. It is the basic reference document used by traffic engineers in planning, designing, and analyzing highways. Researchers frequently compare their data on traffic volumes and speeds with the tables and curves in the *HCM*. Also, much past and ongoing

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[1]The study was funded by the Federal Highway Administration (FHWA), Office of Environment and Planning.

research by the National Cooperative Highway Research Program (NCHRP) and FHWA has as an explicitly stated objective the development of findings and procedures for inclusion in the next edition of the *HCM.*

HCM procedures for determining the capacity of multilane uninterrupted flow facilities, including freeways and multilane highways without access control, take into account the following highway characteristics: number of lanes; lane widths and lateral clearances; vehicle mix and the effects of grades on truck operating characteristics; and driver population. For highways with design speeds greater than 60 mph, the 1985 *HCM* recommends a value of 2,000 vehicles per hour per lane (vphpl) as the capacity of a freeway under the following ideal conditions: 12-foot lane widths; 6-foot lateral clearances between the edge of the travel lanes and the nearest obstacle or object on the roadside or in the median; all passenger cars in the stream; and driver characteristics typical of weekday commuter traffic streams in urban areas or regular users in other areas. The effects of deviations from these ideal conditions are taken into account through a series of adjustment factors that reduce capacity downward from the ideal value of 2,000 vphpl. HPMS submittal software uses procedures from the 1985 *HCM* to calculate the capacities of freeways and other multilane highways without traffic signals or stop signs. The HPMS Analytical Process also uses these procedures to calculate the capacities of highways following improvements. Once the capacity of a section is established, the *HCM* provides curves that can be used to calculate average travel speed depending upon traffic volume and design speed.

A review of the recent literature on traffic flow theory indicates several important areas where the speed-flow relationships for freeways and multilane highways presented in the 1985 *HCM* have been seriously challenged and either have been revised or are under revision:

- Capacity under ideal conditions (2,000 vehicles per hour per lane) appears to be understated. The recently revised Chapter 7 of the *HCM* (Multilane Rural and Suburban Highways) recommended that the theoretical ideal capacity of these highways be set at 2,200 passenger car equivalents per hour per lane (pcephpl). The recommended value of 2,200 is greater than the freeway capacity of 2,000 specified in the 1985 *HCM* for freeways. Thus, it is likely that the next edition of the *HCM* will specify a value of at least 2,200 for freeways.

- There is evidence suggesting that the decrease in speed as volumes approach capacity (prior to the development of queues) is much less than that shown in the speed-volume curves used by the *HCM.* Hall, Hurdle, and Bank (2) report findings from several recent studies regarding the size of the decrease in speed from uncongested to capacity flow conditions -- ranging from no observable decrease to a decrease of 25 percent. While the studies differ significantly in the size of the decrease, it should be noted that in no case are the speed decreases as great as those shown in the 1985 *HCM.* Recently approved revisions to Chapter 7 of the *HCM* contain speed-flow relationships that differ significantly from the 1985 version of the *HCM.* The new relationships keep speeds constant until 75 percent of capacity, and drop only 5 miles per hour between 75 and 100 percent of capacity.

- There is evidence for a "two-capacity" hypothesis under which the occurrence of bottlenecks on a section creates a metered flow that is less than the free-flow capacity of the section. For example, Hall and Agyemang-Duah (3) found that the queue discharge rate on a section of the Queen Elizabeth Highway in Ontario was 6 percent less than the free-flow capacity of the section.

- The Research Project Statement for NCHRP Project 3-45 (issued in Spring 1992) states that the material on basic freeway segments in Chapter 3 of the 1985 *HCM* is based on sparse databases and on information no longer current. According to that Statement, basic freeway flow characteristics and the impact of heavy vehicles and restricted lane/shoulder widths have not been comprehensively studied since the early 1960s. The Statement identifies several other factors that may affect speed-flow relationships as either ignored or not well specified in the *HCM.* These include driver experience with congested freeways, urban versus rural settings, horizontal and vertical geometry, the presence of ramp metering, and day-night differences.

The *HCM* provides procedures for estimating the capacities of signalized intersections, delays at intersections, and average speeds on arterials with signalized intersections. Chapter 11 of the *HCM* provides a procedure that can be used to calculate average travel speed and average running speed on arterials with signalized intersections. The procedure could work based on combining delay estimates at individual intersections with average running time on segments between intersections. When intersection demand volumes exceed capacity, it is difficult to accurately estimate delays since spillbacks may extend to adjacent intersections. The *HCM* states that the procedure for estimating intersection delays can be used with caution for ratios of demand volume to capacity of 1.2 or less. Other aspects of procedures for handling signalized intersections in the 1985 *HCM* have been challenged by researchers and are the focus of a current FHWA research project to update Chapters 9 (Signalized Intersections) and 11 (Urban and Suburban Arterials). Specifically, some weaknesses of the existing *HCM* have been noted.

- procedures for analyzing signalized intersections in Chapter 9 are restricted to undersaturated conditions, are limited to only one period of analysis, and totally neglect actuated control;

- procedures for analyzing signalized arterials overstate speeds; and

- the effects of metering by upstream signals, platoon dispersion, and queue overflow on arterial delays are not well documented.

Based on our review of past and current *HCM* activities, it became apparent that the most important issue to be addressed in developing improved speed estimation procedures for HPMS was how to estimate the consequences of **queuing.** (Queuing results when the number of vehicles attempting to use a section exceeds its capacity.) Dowling and Skabardonis (4) developed a procedure for predicting the effects of queues on travel speeds. Their procedure was developed as part of a traffic assignment output "post-processor" to provide improved speeds for air quality impact analysis and was based on adding queued vehicles in one time period to the adjacent time period:

$$\text{Ave. Link Speed} = \text{Ave. Queue Speed} * (\text{Ave. Queue Length/Link Length}) + \text{Non-Queue Speed} * (1 - \text{Ave. Queue Length/Link Length}) \quad (1)$$

Where:

- Ave. Queue Speed = Capacity/Lane * Vehicle Spacing in Queue.

- Ave. Queue Length = Ave. Queue * Vehicle Spacing in Queue.

- Ave. Queue = $(Q_1 + Q_2)/2$.

- Q_1 = Queue at start of time slice.

- Q_2 = Queue at end of time slice.

Dowling and Skabardonis note that, while their procedure assumes that the queue will occur on the same segment where volume is predicted to exceed capacity, in reality the queue would form upstream of the bottleneck section. However, since the purpose of the analysis is to predict the effect of queuing on speed, this mislocation of the queue may not be a problem. Further, it should be noted that this procedure does not account for the consequences of a queue extending upstream and causing additional queues due, for example, to the blocking of intersections and ramps. The effects of this shortcoming are at least partially offset by the fact that the procedure does not account for the improved speeds on links downstream from the bottleneck.

Simpler procedures also exist for representing speeds under queuing. Many relationships have been developed over the years for estimating speeds in the traffic assignment component of travel forecasting. These relate the V/C ratio to speed in a mathematical formulation originally developed by the Bureau of Public Roads (BPR). As shown in Table 1, estimates over speeds in congestion (V/C > 1.0) vary widely based on the formula. The chief problem with developing such equations is that V/C ratios greater than 1.0 cannot theoretically be observed: by definition, queuing begins at a V/C of 1.0 causing the observed volumes to drop well below capacity. Thus, this family of curves is of limited usefulness in addressing the problem of queuing.

In the past decade, traffic simulation models have been developed to overcome many of the problems noted above. A battery of traffic simulation models (referred to as the TRAF models) have been developed by FHWA. The TRAF system is comprised of a family of simulation models integrated within a single system. The TRAF system is designed to provide a single simulation modeling system for all applications in traffic operations. The system allows simulation of virtually any highway network at different levels of detail. The system can simulate traffic flow on urban streets (local road, arterials), freeways and interchanges, and rural roads. The user can simulate traffic flow microscopically (individual vehicles) or macroscopically (vehicle groups). The microscopic simulation models have been shown to handle congested (oversaturated) conditions much more efficiently (9). Given the highway characteristics, traffic demand and flow patterns, driver/vehicle characteristics, and prevailing traffic control, microscopic simulation is performed using well-established theories of traffic flow (e.g., car-following and platoon dispersion). Separate microscopic simulation models have been developed for freeways (FRESIM) and urban arterials (NETSIM). Both FRESIM and NETSIM have been subjected to a degree of validation and calibration by FHWA. However, as some researchers have reported discrepancies in their application, a pending contract will more perform more extensive validation and calibration. For the purposes of this research, since the scope was broad (the effects of grades, curves and congestion in both urban and rural areas were examined) and the level of funding prohibited extensive field data collection, the simulation approach was chosen. Preliminary testing of FRESIM and NETSIM was performed to identify potential problems with their use; these tests are reported under Results.

Table 1

Speeds Produced by the BPR Function and Its Variants

FFS = 60 mph

V/C	BPR	Ruiter (S_c=25mph)	Ruiter (S_c=50mph)	CSI/JHK	Mod BPR4	Mod BPR10	Davidson (J=0.04)
0.10	60.00			60.00	59.99	60.00	59.73
0.50	59.44			59.97	56.47	59.94	57.69
0.75	57.28			59.27	45.58	56.80	53.57
0.90	54.62			56.99	36.23	44.49	44.12
0.95	53.47			55.49	33.07	37.53	34.09
1.00	52.17	24.98	49.70	53.45	30.00	30.00	N/A
1.05	50.75	23.46	46.68	50.80	27.08	22.82	
1.10	49.20	22.21	44.18	47.52	24.35	16.70	
1.15	47.53	21.17	42.10	43.64	21.83	11.89	
1.20	45.77	20.30	40.35	39.30	19.52	8.34	
1.30	42.00	18.93	37.60	30.01	15.56	4.06	
1.40	38.07	17.92	35.59	21.37	12.39	2.00	
1.50	34.10	17.16	34.08	14.49	9.90	1.02	
1.60	30.26	16.58	32.92	9.58	7.94	0.54	
1.70	26.63	16.13	32.02	6.29	6.42	0.30	
1.80	23.30	15.78	31.31	4.14	5.22	0.17	

Notes: (1) FFS = free-flow speed
(2) S_c = speed at capacity
(3) Functions

BPR: $FFS/(1 + (0.15 * (V/C)^4))$ (5)
Ruiter: $S_c * (0.555 + (0.444 * (V/C)^{-3}))$ (6)
CSI/JHK: $FFS/(1 + (0.1225 * (V/C)^8))$ (7)
Mod BPR4: $FFS/(1 + (V/C)^4)$ (8)
MOD BPR10: $FFS/(1 + (V/C)^{10})$ (8)
Davidson: $FFS/(1 + (0.04 * (V/(C - V))))$ (8)

2. METHODOLOGY

Overview

The researchers decided that the microscopic simulation models would be the most appropriate tool to use because of their ability to capture queuing effects. However, because their data requirements are very intensive, the approach depicted in Figure 1 was pursued. The chosen approach relied on developing a new model, dubbed "QSIM" (for its queue-handling abilities), that allows more extensive testing than otherwise would be possible given the data intensiveness and run times of the microscopic simulation models.

Uncongested Conditions

A series of experimental designs were developed to which FRESIM and NETSIM were applied. The freeway experimental design is shown in Figure 2. V/C ratio was used as the main determinant of freeway speeds in urban areas and is computed from the volume per lane and truck percentage. (Since FRESIM doesn't account for lane width and lateral clearance, these items are assumed to be at their "ideal" levels.) To reduce the number of runs, note that not all possible levels of the factors were studied. For urban signalized highways, a variation of the V/C ratio was used: V_m/C_m, the volume of all lane groups on the main approach divided by the capacity of all lane groups on the main approach. A full factorial design was run with the following factors and their levels:

- V_m/C_m (0.2, 0.4, 0.6, 0.75, and 0.85);
- free-flow speed (FFS; 40 and 50 mph);
- left turn bay and phase (present/absent);
- signals per mile (0.33, 0.5, 1, 3, 6, and 10); and
- signal progression (fixed time and progressive).

Where more than one signal was included in the test network, all signals on the section were assumed to have the same V_m/C_m ratio. For unsignalized arterials, a full factorial design was also examined:

- volume per lane (300, 600, 900, and 1,200);
- number of lanes (2 and 4);
- stop signs per mile (0, 1, and 2);
- intersections per mile (0, 1, 5, and 10); and
- free-flow speed (40 and 50 mph).

Congested Conditions

Because of the inadequacy of single hour-based procedures (such as the *HCM*) to account for the effects of queuing, a different measure of congestion was used: AADT/C (average annual daily traffic divided by two-way capacity. This measurement is not in wide use by the profession but to provide the reader with a better sense of what the AADT/C ratios mean in terms of facility operation, the following translation is provided assuming a four-lane (two lanes each direction) freeway section and 10 percent trucks:

AADT/C	AADT
11.0	88,000
12.0	96,000
14.0	112,000
16.0	128,000

Since AADT is a daily rather than hourly concept, it was necessary to simulate traffic operations over an entire day. (Both FRESIM and NETSIM allow for this feature; the user can specify different volumes for different time periods.) To do so, a method of determining hourly volumes from AADT had to be derived. Initially, results from a completed FHWA study were used (10). This study developed temporal distributions for urban freeways and nonfreeways for each direction by peak orientation. (A particular direction can have its peak in either the morning or afternoon time periods.) The initial calibration of the QSIM model occurred with these runs. However, a more recent FHWA study updated the temporal distributions to include the effect of peak spreading (11); these new distributions were distinguished by freeway/nonfreeway and three AADT/C ranges. The final speed models were developed using the latest temporal distributions.

The QSIM Model

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 and 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.

For each hour analyzed, QSIM determines whether or not travel in that hour will be affected by queuing. Queuing will affect traffic if the volume attempting to use the segment is greater than its capacity or if there is a standing queue at the end of the preceding hour. If travel in the hour under consideration is not affected by queuing, then the program simply applies relationships between travel time and V/C ratios for unqueued conditions to calculate vehicle hours of travel for the segment. 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 used by Dowling and Skabardonis is used to combine speeds for queued and unqueued conditions (4). The one significant departure from the Dowling and Skabardonis procedure is the addition of a queue discharge (dissipation) rate: with the onset of congestion, vehicles are assumed to move through the bottleneck point at a flow rate less than capacity. For freeways, the capacity is assumed to be 2,300 pcphpl and the queue discharge rate is 2,000 pcphpl. The capacity for signalized arterials is 900 pcphpl and is based on the *HCM's* saturation flow rate of 1,800 pcphpl and a 50 percent green time. The capacity for unsignalized arterials is 600 pcphpl. Since arterials are interrupted flow facilities, their queue discharge rate is the same as their capacity.

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 rerunning the program with an increased segment length.

QSIM is capable of producing outputs for segments with AADT/C ratios from 0 to 24. However, for high AADT/C ratios, the program predicts very long queues, with the duration of queuing extending past midnight in the home-to-work direction. While the program produces a realistic representation of the delays which would occur under very high AADT/C ratios, in reality drivers would, if possible, shift to alternative facilities rather than wait in these queues to use the segment under consideration.

3. RESULTS

Initial FRESIM and NETSIM Testing
Both FRESIM and NETSIM were tested against *HCM* procedures for uncongested conditions as a validity check. FRESIM was used to estimate freeway capacity by coding a network with a long entrance ramp and acceleration lane onto a basic freeway segment. Volumes (all cars) were then increased incrementally until a maximum throughput volume was obtained. Results show that based on FRESIM, the capacity of a freeway lane is about 2,260 pcphpl for 70 mph free-flow speed and about 2,230 for a 60 mph free-flow speed. These values are in line with the new Chapter 7 of the *HCM*. Another test compared the speed-V/C curves obtained with FRESIM with those of the new Chapter 7 of the *HCM*. The new Chapter 7 shows that the capacity of an ideal lane is 2,200 pcphpl, a 10 percent increase over the old value of 2,000, and that the shape of the speed/volume curve is much flatter than previously estimated. As shown in Figure 3, FRESIM results for zero and 10 percent trucks were compared with curves from the new *HCM* for various free-flow speeds. Several observations may be made from these curves:

- For free-flow speeds of 50 mph and 60 mph, the FRESIM results showed that as congestion built, speeds decreased more than suggested by the new *HCM*. However, the nature of this speed drop was much smaller than would be predicted using the old (1985 version) speed/volume relationship.

- Under high V/C levels for free-flow speeds of 60 mph and 70 mph, a substantial deviation existed between FRESIM runs for zero and 10 percent trucks in the traffic stream. The two curves were coincident for 60 mph up to a V/C ratio of 0.8 and up to 0.7 for 70 mph. However, above these levels speeds dropped off sharply for the 10 percent truck curve. If the *HCM* method is adequately adjusting for the effect of trucks in the traffic stream, the curves should have been coincident over the entire range of V/C.[1] What these results suggest is that the influence of trucks is not constant over all V/C ranges but rather increases with increasing congestion. Put another way, the passenger equivalent values for trucks did not remain a constant 1.5 as suggested by the *HCM* but increase at higher congestion levels. If, in fact, these results are verified in the future, the most likely reason is that at higher congestion levels trucks take a longer time to go through speed changes, thus slowing down themselves and other vehicles in the traffic stream.

The basic approach for studying urban signalized arterial speeds was based on using the V_m/C_m ratio as a surrogate for other signal-related factors. Various combinations of signal phasing, lane configurations, and turning movements can then be developed to produce a particular V_m/C_m level. Several tests were conducted to determine if V_m/C_m could be used to control for the full array of signal characteristics. Three approach designs were used:

- Method 1 -- one exclusive left turn bay and phase; one exclusive through lane; and one shared through and right turn lane;

[1]FRESIM input volumes were reduced for the 10 percent truck case in accordance with the *HCM* procedures. For this analysis a pce of 1.5 was assumed, as specified in Table 7-8 of the *HCM*.

- Method 2 -- one shared left turn and through lane (totally permissive left turns) and one shared though and right turn lane; and

- Method 3 -- one exclusive left turn bay and phase; two exclusive through lanes; and one exclusive right turn lane.

Based on these designs, volumes and phasing were adjusted to reach V_m/C_m levels of 0.43, 0.68, 0.85, and 0.96. If V_m/C_m is adequately accounting for the various combinations of signal factors, speeds on the approach links in the networks should be roughly the same.

However, the results of the experiment showed that speeds were not the same for all three methods. The two methods with exclusive left turn bays and phases (Methods 1 and 3) showed about the same approach speeds (Figure 4). Method 2 showed almost no drop in approach speeds over the range of V_m/C_m levels from 0.43 to 0.96. Likewise, Methods 1 and 3 showed excellent correspondence with the *HCM* when predicted stopped delays were compared; but under Method 2 the *HCM* predicted far greater stopped delay. Assuming that NETSIM is correctly simulating permissive left turns, a possible explanation for this discrepancy is that the *HCM* method is not sufficiently adjusting for permissive left turns. As described in the *HCM*, the adjustment is made through a complicated series of calculations. The most important consideration is the opposing flow -- as opposing flow increases, the capacity decreases. Without field data it is not possible to select one method over another. For the purposes of this study, it was decided that only results from networks with a separate left turn bay and phase would be considered.

FRESIM and NETSIM Experiment Results

For congested conditions, freeway profiles showing volumes and speeds by hour were created (Figure 5). These profiles are useful in examining the duration and extent of congestion. (Congested locations are shaded.) The building of congestion as well as queue dissipation can also be traced over time and space. It is exactly these conditions that are of interest in speed modeling. As can be seen, congestion spills over into adjacent hours as AADT/C increases. Performing analysis on strictly peak hour traffic would not capture these effects. For uncongested conditions on signalized arterials, signal density was found to be a strong determinant of vehicle speeds in addition to V_m/C_m for fixed time signals (Figure 6). Curiously, the results for progressive signal timing were essentially the same as for fixed time. However, the test networks were very short in length (1 to 1/2 miles) and, in the fixed time, case all signal phases exactly coincided. In such a short network if signals all turn at the same time, a large degree of progression exists. Therefore, the similarity of fixed time and progressive schemes is to be expected.

The results of the FRESIM and NETSIM experiments were used in two ways. First, they served as the basis for developing the uncongested flow relationships that were built into QSIM (refer to the next section. Second, the congested experiments served as a check for preliminary QSIM results. Based on these comparisons, QSIM was calibrated by adjusting the queue discharge rate and assumed queue speed so that the average speed results were in agreement.

QSIM Application

The development of QSIM was based on a slightly different strategy than estimating speeds directly. Rather, so that the effects of congestion as well as other geometric features (grades and curves) could be jointly considered, each case was analyzed in terms of the <u>delays</u> that they produced, where delay is additional travel time (actually measured in hours per vehicle-mile) beyond that at the free-flow speed. Hence. total travel time included travel time at the free-flow speed plus delays due to congestion, traffic control devices, grades, and curves. Speed could then be derived as the inverse of delay.

In applying QSIM to estimate the effects of AADT/C on average speeds for freeways, the following equation was used to estimate the effects of V/C ratio on travel time for unqueued conditions:

$$d_{vc} = 4.46\ V/C - 1.55\ (V/C)^2 - 0.05\ s_{ff}\ V/C + 0.044\ s_{ff}\ (V/C)^2 \qquad (2)$$

where:

- d_{vc} is delay due to congestion in hours per 1,000 vehicle-miles
- s_{ff} is free-flow speed in miles per hour
- V/C is volume-to-capacity ratio (based on an assumed capacity of 2,200 passenger cars per hour per lane).

This relationship between congestion delay, free-flow speed, and V/C was estimated using regression analysis from FRESIM results for uncongested conditions as per the design shown in Figure 2. If the above equation is transformed to predict speed instead of delay (see the method shown in equation 12), the predicted values of speed are very similar to those for the BPR curve in Table 1. Since d_{vc} is the <u>added</u> travel time due to congestion, it is equal to zero when the V/C ratio is zero.

For AADT/C ratios from 1 to 18, QSIM produced estimates of average delay on the segment due to congestion including both the delay due to the gradual decrease in speed with increasing volumes (as volumes approach capacity) and the delay due to queuing (when the number of vehicles attempting to use the segment exceeds its capacity).

The following curve was then fit to the computer program outputs:

$$D_{cong} = 0.0797\ x + 0.00385\ x^2 \qquad \text{for } x <= 8 \qquad (3)$$

$$D_{cong} = 12.1 - 2.95\ x + 0.193\ x^2 \qquad \text{for } x > 8 \text{ and } x <= 12 \qquad (4)$$

$$D_{cong} = 19.6 - 5.36\ x + 0.342\ x^2 \qquad \text{for } x > 12 \qquad (5)$$

where:

- D_{cong} is average congestion delay in hours per 1,000 vehicle-miles
- x is the AADT/C ratio for the segment.

The effects of signalized intersections on travel times were estimated using regression analysis on NETSIM results for uncongested facilities with fixed signal systems and left turn bays and phases. The following equation was developed:

$$d_{sig} = (\ 68.6 + 297.7\ V/C\ )\ (\ 1 - e^{-n/24.4}\ ) \qquad (6)$$

where:

- d_{sig} is delay in hours per 1,000 vehicle-miles

- V/C is volume-to-capacity ratio
- n is the number of signals per mile.

Here, delay is additional travel time beyond that which would result if all vehicles could traverse the section at the free-flow speed. It includes not only the time spent sitting at red lights, but also the time lost while decelerating to a stop and then accelerating back to the free-flow speed. It should be noted that the above equation does not accurately reflect the effects of congestion when the V/C ratio equals or exceeds 1.0. To analyze such conditions it was necessary to use QSIM to model the temporal distribution of travel and the growth and decline of queues over time. The following equation was developed from QSIM outputs:

$$D_{cong} = (1 - e^{-n/24.4}) (68.7 + 17.7 x) \quad \text{for } x <= 7 \text{ and } n < 16 \qquad (7)$$

$$D_{cong} = (1 - e^{-n/24.4}) (192.6 + 14.4 (x - 7) - 1.16 (x-7)^2) + 0.160 (x-7)^2 \quad \text{for } 7 < x <= 18 \text{ and } n < 16 \qquad (8)$$

where:

D_{cong} is average congestion delay in hours per 1,000 vehicle-miles

x is AADT/C

n is the number of traffic signals per mile.

Delays due to stop signs were estimated using the following equation from the uncongested unsignalized NETSIM runs:

$$d_{ss} = n \left(1.9 + 0.067 s_{ff} + \frac{1000 v}{c (c - v)} \right) \qquad (9)$$

where:

- d_{ss} is delay in hours per 1,000 vehicle-miles
- s_{ff} is free-flow speed
- V/C is the volume-to-capacity ratio
- n is the number of stop signs per mile.

This estimate of delay includes not only the time spent stopped but also the time lost while decelerating to a stop and then accelerating back to the free-flow speed. The first two terms of the above equation were estimated based on NETSIM outputs for highways with unsignalized intersections. The third term represents the additional delay due to other vehicles. This delay was estimated based on the assumption that congestion effects at stop signs can be modeled as a single server queue with random arrivals and service times. By using linear regression analysis to fit curves to the computer program outputs, the following relationship was developed for estimating average delays due to congestion and traffic controls on highways with stop signs:

$$D_{cong} = n\,(1.90 + 0.067\,s_{ff} + 0.0939\,x + 0.0211\,x^2), \qquad (10)$$

for $x <= 6$

$$D_{cong} = n\,(3.22 + 0.067\,s_{ff} + 1.02\,(x-6) - 0.131\,(x-6)^2) + 1.48\,(x-6)^2, \qquad (11)$$

for $6 < x <= 15$

where:

- D_{cong} is average delay in hours per 1,000 vehicle-miles
- x is AADT/C
- n is the number of stop signs per mile
- s_{ff} is free-flow speed in miles per hour.

Once the delay is estimated for congestion it can be combined with information concerning delay from other geometric features to produce an overall estimate of speed. The research project also produced procedures for estimating delay due to grades and horizontal curves which are primarily a concern in rural areas (12). The formulation for estimating delay due to grades and curves is not presented here but can be combined with the congestion delay to estimate speed:

$$S = \left(\frac{1}{s_{ff}} + \frac{D_{cong}}{1{,}000} + \frac{D_{grade} + D_{curve}}{L} \right)^{-1} \qquad (12)$$

where

- S is overall average speed, reflecting the combined effects of AADT/C, grades, and curves
- s_{ff} is free-flow speed assuming no congestion, grades, or curves
- D_{cong} is congestion delay in hours per 1,000 vehicle-miles
- D_{grade} is added delay due to grades in hours per vehicle
- D_{curve} is added delay due to horizontal curves in hours per vehicle
- L is segment length in miles

If the delay due to grades and curves is assumed to be negligible for urban applications the D_{grade} and D_{Curve} terms can be dropped from equation 12.

Excess Fuel Use Due to Congestion

Estimates of the added fuel consumption due to congestion in gallons per hour of congestion delay were developed from the simulation models and reference 13 and are summarized in Table 2. Given the relatively small differences between the estimates from different sources, particularly for automobiles, we recommend using the FRESIM-based estimates for all types of facilities. Excess fuel use due to congestion is estimated as a function of the added travel time due to congestion:

$$G = 0.620\ D_{cong} \quad \text{for autos} \qquad (13)$$

$$G = 1.607\ D_{cong} \quad \text{for single unit trucks with six or more tires} \qquad (14)$$

$$G = 1.934\ D_{cong} \quad \text{for combinations} \qquad (15)$$

Where:

- G is excess fuel consumption due to congestion in gallons per 1,000 vehicle-miles
- D_{cong} is congestion delay in hours per 1,000 vehicle-miles.

Table 2

Additional Fuel Consumption Rates

	Added Gallons Per Hour of Congestion Delay		
	Automobiles	Heavy Single-Unit Trucks	Combinations
FRESIM	0.620	1.607	1.934
NETSIM (unsignalized intersections)	0.653	1.511	
NETSIM (signalized intersections)	0.631	1.254	
ITE Handbook (13) (Freeway Fuel Consumption Rates)	0.589	2.100	

Comparison With Other Speed Methods
Table 3 presents the results of applying the new speed models with some of the methods listed in Table 1 for freeway conditions. For the other methods, speeds for each hour of the day in each direction were estimated by using the same volumes (based on the temporal distributions in reference 11) that were input to QSIM. They were then weighted by the hourly volumes to compute the daily average. The modified BPR equations come the closest to the QSIM results even though all the methods are commonly criticized for underestimating delay under queuing (14). As congestion worsens (i.e., AADT/C increases) the gap between QSIM and the modified BPR equations widens. Closer examination revealed that these methods underestimate speeds for uncongested conditions. For example, the data in Table 1 show that both modified BPR equations predicted a speed of 30 mph at capacity, surely a desired quality by their developers since the previous *HCM* indicated the same. As discussed earlier, the new *HCM* will show a much higher speed at capacity, probably somewhere near 50-55 mph for a facility

designed to 60 mph. Therefore, although the modified BPR curves closely match the QSIM results for the moderately congested AADT/C levels, application of the curves on an hourly basis will still not account for queuing effects.

Comparison With Freeway Field Data

A test was run to check the new speed models against actual speed data from the Orlando freeway surveillance system. The system covers about 11 miles of Interstate 4 equally split north and south of the Central Business District. Volume, occupancy, and direct speed measurements are taken at 25 stations along this segment, many of which routinely experience queuing in peak hours. (AADT/C ranged from 8.6 to 13.9.) Data were available at 5-minute intervals. For each day of data at each station, the harmonic average daily speeds were calculated along with ADT (24-hour volume). ADT/C was then used as a surrogate for AADT/C, and the speed models were applied using this number. For each day, the observed and predicted speeds were calculated and the percent error noted. The results are shown in Table 4. The average and median errors are very small (1.1 and 0.6 percent, respectively), but the data show very large variations. Given the wide variation in day-to-day traffic patterns as well as the fact that incidents are included in the Orlando data, this is to be expected. While this is only a simple check on the model's performance, the fact that the speed models, on average produce, a small error is encouraging for the project's purposes since national averages are adequate for the HPMS models.

Table 3

Comparison of Speed Models for Freeways (Average Daily Speeds)

AADT/C	# Hrs. With V/C > 1.0[1]	BPR	CSI/JHK	BPR4	BPR10	New HPMS Models
11	1	58.4	59.4	51.7	57.2	50.8
12	2	57.7	58.9	49.4	54.7	47.2
13	5	57.0	58.0	46.9	51.3	41.0
14	5	56.1	56.7	44.5	47.4	35.4
15	11	55.0	54.9	42.1	43.3	30.5
16	14	53.9	52.7	39.8	39.3	26.3
17	21	52.6	50.2	37.6	35.8	22.7

[1]Weekday hours in each direction. For highly congested facilities, both directions will experience V/C > 1.0 in each direction during peak hours. Hourly volumes were developed from temporal distributions in (11). Speeds are the weighted average of weekday and weekend speeds.

Table 4

Distribution of Error Rates for Test of HPMS Speed Models vs. Orlando Data

No. of Station-Days	((Predicted - Observed)/Observed) * 100							
	Mean	Std	P10	P25	P50	P75	P90	P95
647	1.1	10.4	-10.9	-6.3	0.6	6.5	14.1	20.1

Note: P10 to P95 are the 10th to 95th percentiles.

4. CONCLUSIONS

The main limitation of the new speed models is the fact that they are based on simulation results and not on field observations. Both FRESIM and NETSIM have undergone some field validation by FHWA, but much work remains to be done in this area. For example, in the preliminary testing of FRESIM we noted a discrepancy between traffic streams with no trucks and mixed cars/trucks at V/C levels close to 1.0. Also, with NETSIM a major discrepancy exists between its results and *HCM* results for permissive left turns. Whether these simulation results are verified or refuted by field data remains to be seen. For the current study, given the limitations in project scope, simulation appeared to be the only viable approach to developing new speed models for HPMS.

Another shortcoming of the new models -- which is also true of nearly all the speed estimation methods in use -- is that they address only recurring congestion. Nonrecurring (incident-related) congestion is a major contributor to delays in urban areas. Future speed modeling efforts should consider the treatment of incidents. In particular, the presence of incidents in data that are used to develop or calibrate speed models should be explicitly addressed. Ideally, incidents should be treated separately from other speed-flow relationships.

For congested conditions, the models are based on a set of pre-defined temporal distributions. If a particular facility's traffic patterns vary much from these, the nature of queuing will be different. Since the purpose of the research was to develop models for HPMS -- which does not have facility-specific temporal traffic data -- the use of average conditions is appropriate. For other applications, this assumption may not be valid.

The chief contributions of the new speed models are the simplicity of application and their explicit accounting for the effects of queuing. The basis for their development (QSIM) is a direct outgrowth of the work of Dowling and Skabardonis (4) with several enhancements:

- *The inclusion of a queue discharge rate for freeways.* Since capacity has been observed to drop as a result of a bottleneck, this feature will capture delay more fully.

- *Use of newly defined uncongested flow speed functions.* These functions were developed by applying the FRESIM and NETSIM microscopic traffic simulation models and include not only a congestion term (V/C) but terms for traffic control device density (signals per mile and stop signs per mile) as well.

- *Use of generic temporal distributions that account for peak spreading.* While the temporal distributions on which the models are based are fixed, they were developed from national data and are felt to be representative of average conditions. Further, the temporal distributions used are distinguished by congestion level and facility type: as AADT/C increases, the distributions become less pronounced in their peaks.

- *A final model form that allows incorporation of other factors that influence speed, such as grades and curves.* Speed is not estimated directly in the new models. Rather, delay (measured as deviation from the free-flow speed) as a function of highway and traffic conditions is first estimated, combined, and converted to speed. The model form can, therefore, be applied to facilities with different free-flow speeds and can incorporate future delay terms (such as pavement condition and incidents) in the future.

- *Validation against microscopic traffic simulation models.* Given the problems noted above with the simulation models, this validation is useful but not sufficient: validation against field data is preferable. Although a limited check was performed with freeway data from Orlando, a more intensive effort is warranted, especially for signalized arterials.

In their current form, the models predict daily average speed and excess fuel consumption due to congestion. For HPMS's purposes, these measurements are valuable since the HPMS models are concerned with estimating annual user costs on the highway system. For other applications, hourly or peak period speeds are of greater interest. (The daily speeds are "diluted" with speeds from many uncongested hours.) FHWA is now sponsoring work to adapt the new models to estimate hourly and peak period speeds. As part of this effort, more intensive field validation will also be performed.

REFERENCES

1. *Highway Capacity Manual: Special Report 209*, Transportation Research Board, Washington, D.C., 1985.

2. Hall, F.L., V.F. Hurdle, and J.H. Banks, "A Synthesis of Recent Work on the Nature of Speed-Flow and Flow-Occupancy (or Density) Relationships on Freeways", paper presented at TRB Annual Meeting, January 1992.

3. Hall, F.L. and K. Agyemang-Duah, "Freeway Capacity Drop and the Definition of Capacity", paper presented at TRB Annual Meeting, January 1991.

4. Dowling, Richard and Alexander Skabardonis, "Improving the Average Travel Speeds Estimated by Planning Models", paper presented at TRB Annual Meeting, January 1992.

5. *Traffic Assignment*, Federal Highway Administration, August 1973.

6. Ruiter, Earl R., "Highway Vehicle Speed Estimation Procedures for Use in Emission Inventories", prepared for U.S. Environmental Protection Agency, Contract 68D90073, September 1991.

7. Cambridge Systematics, *Analysis of Temporal Demand Shifts to Improve Highway Speed Modeling*, Arizona Department of Transportation, Phoenix, AZ, April 1988.

8. Davidson, K.B., "A Flow-Travel Time Relationship for Use in Transportation Planning", *Proceedings of the Third Conference of Australian Research Board*, Vol. 3. Part 1.

9. Wang, Shui-Ying, "TRAF-NETSIM: What It Is, What It Does", *Institute of Transportation Engineers Journal*, April 1990, pg. 22-27.

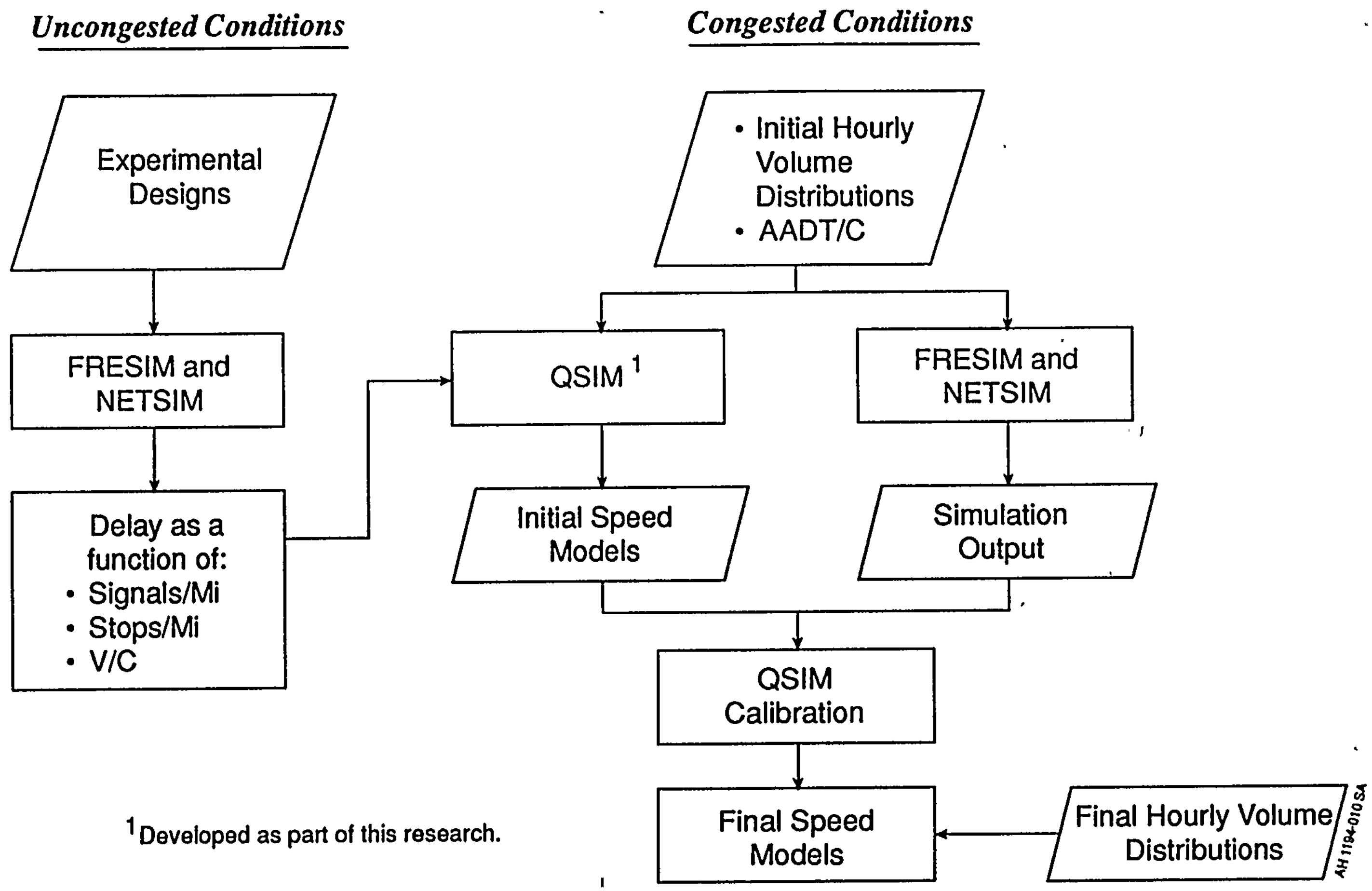
10. Margiotta, R.A. and H.S. Cohen, *Statistical Analysis of Continuous Traffic Counter Data: Vol. 2*, prepared for VNTSC and FHWA, August 1992.

11. Margiotta, R.A., H.S. Cohen, R. Morris, and J. Trombly, *Roadway Usage Patterns: Urban Case Studies*, prepared for VNTSC and FHWA, July 1994.

12. Margiotta, R.A., H.S. Cohen, G. Elkins, R. Morris, A. Rathi, and M. Venigalla, *Speed Determination Models for the Highway Performance Monitoring System*, prepared for FHWA, October 1993.

13. *Transportation Planning Handbook*, Institute of Transportation Engineers, 1992.

14. Helali, Khaled and Bruce Hutchinson, "Improving Road Link Speed Estimates for Air Quality Models", paper presented at TRB Annual Meeting, January 1994.



[1] Developed as part of this research.

Figure 1. Overview of Methodology

For FFS=50/60/70

Volume/Lane	* Avg. Curve (0)	0% (1/2 mi)				2% (1/4 mi)				4% (1/4 mi)				2% (1 mi)				4% (1 mi)			
Avg. Grades/Length → Truck Percentage ↓		0%	5%	15%	25%	0%	5%	15%	25%	0%	5%	15%	25%	0%	5%	15%	25%	0%	5%	15%	25%
500	0	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●
	5	●		●								●								●	
	7	●		●								●								●	
1,000	0	●	●	●	●	●				●		●		●				●		●	
	5	●		●								●								●	
	7	●		●								●								●	
1,500	0	●	●	●	●	●				●				●				●			
	5	●																			
	7	●																			
2,000	0	●	●	●	●	●				●				●				●			
	5	●																			
	7	●																			
2,200	0	●	●	●	●	●				●				●				●			
	5	●																			
	7	●																			

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* Radius= $\frac{5729.58}{\text{Degree}}$
* Superelevation = 8%
* FFS = 50, 5° and 7° not run; FFS = 60, 5° not run } No effect (speeds in FRESIM are effect based on the AASHTO design speed equations).

Figure 2 **Experimental Design For Freeways and Rural Multilane Divided Highways: Uncongested Conditions**

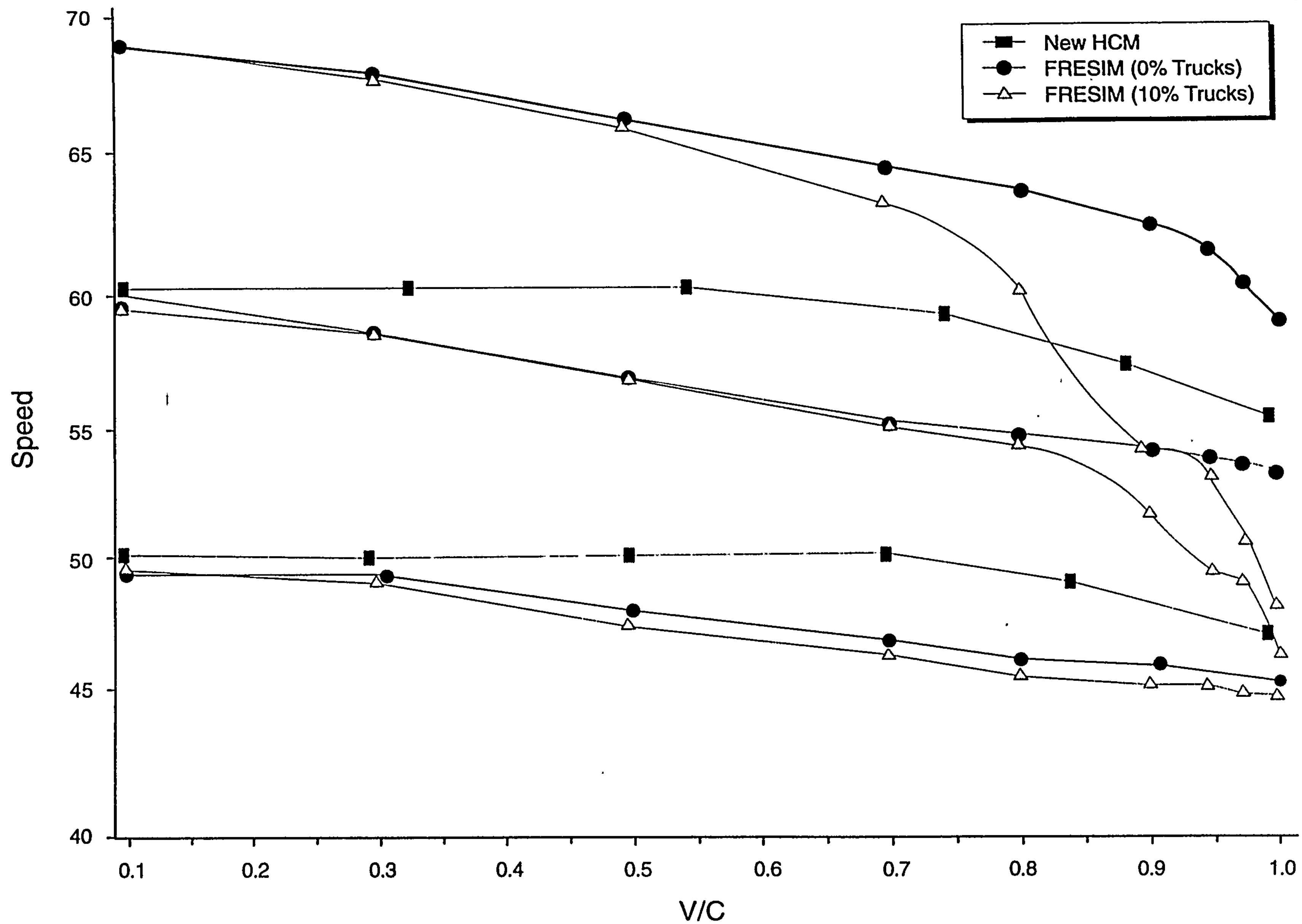

Figure 2-4 Comparison of Speed vs. V/C by Different Methods

Figure 3 **Maximum service flow rates are 2,200 pcphpl for 60 mph and 70 mph, and 2,000 pcphpl for 50 mph.**

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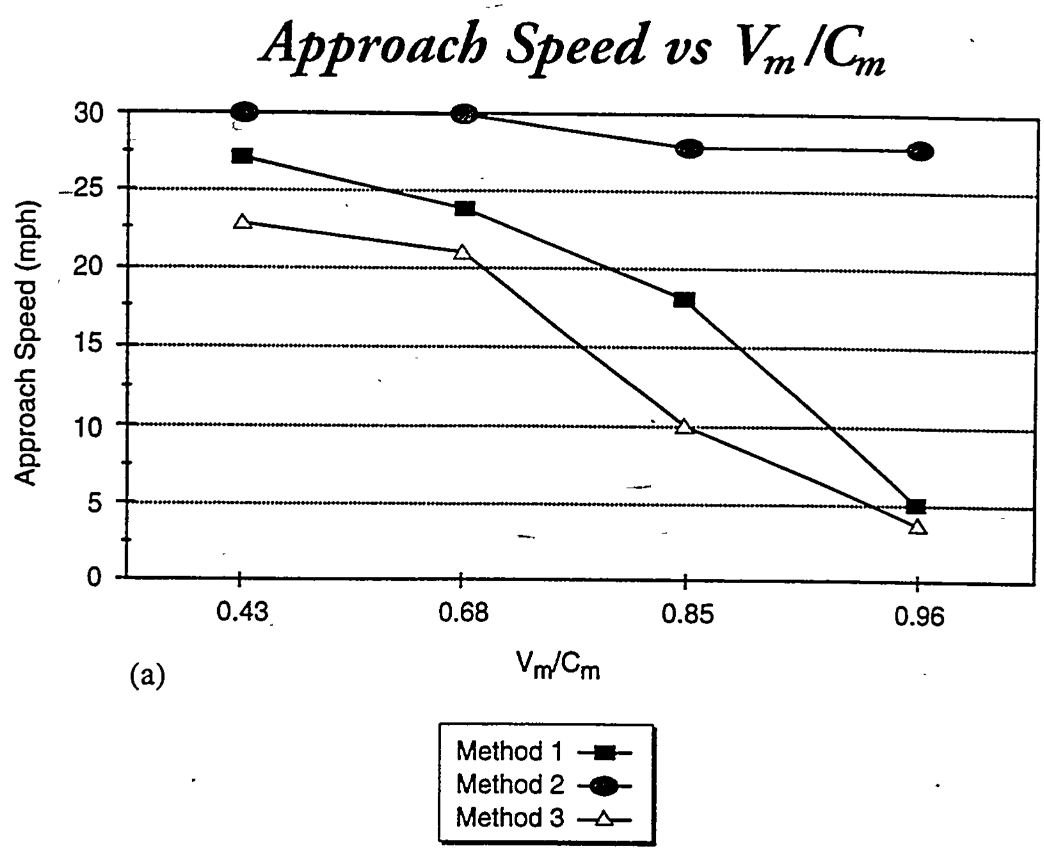
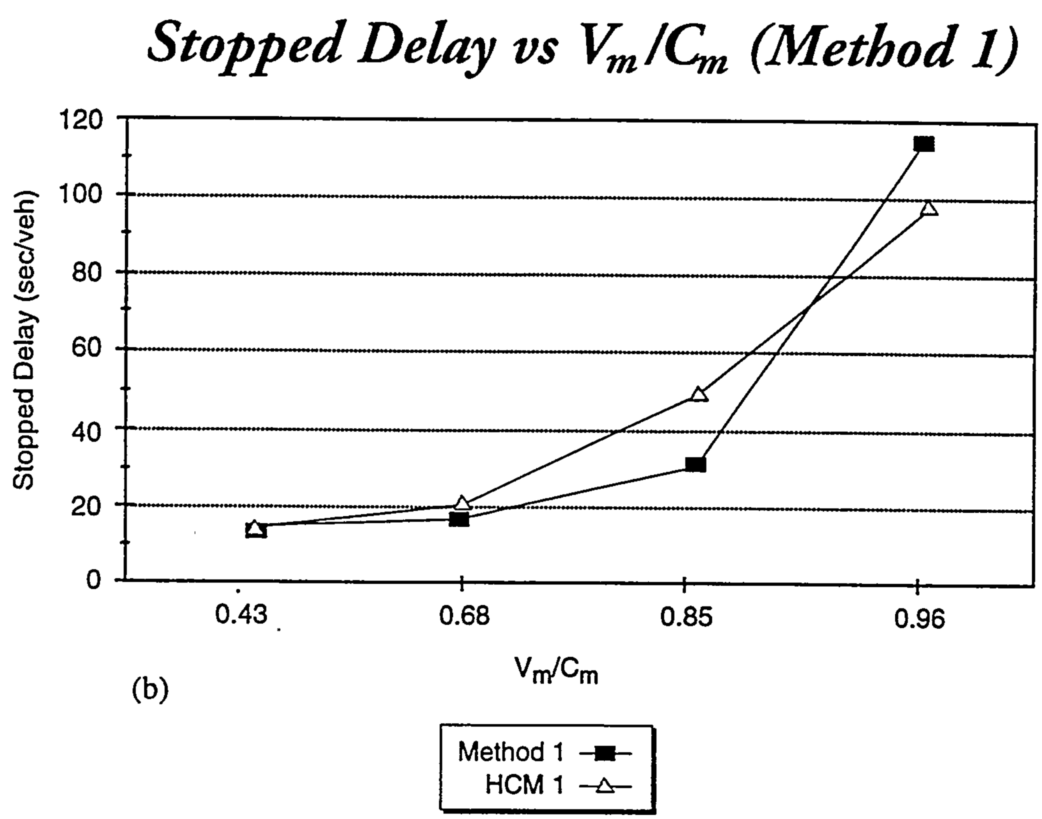


(a)



(b)

Figure 4 Tests of V_m/C_m

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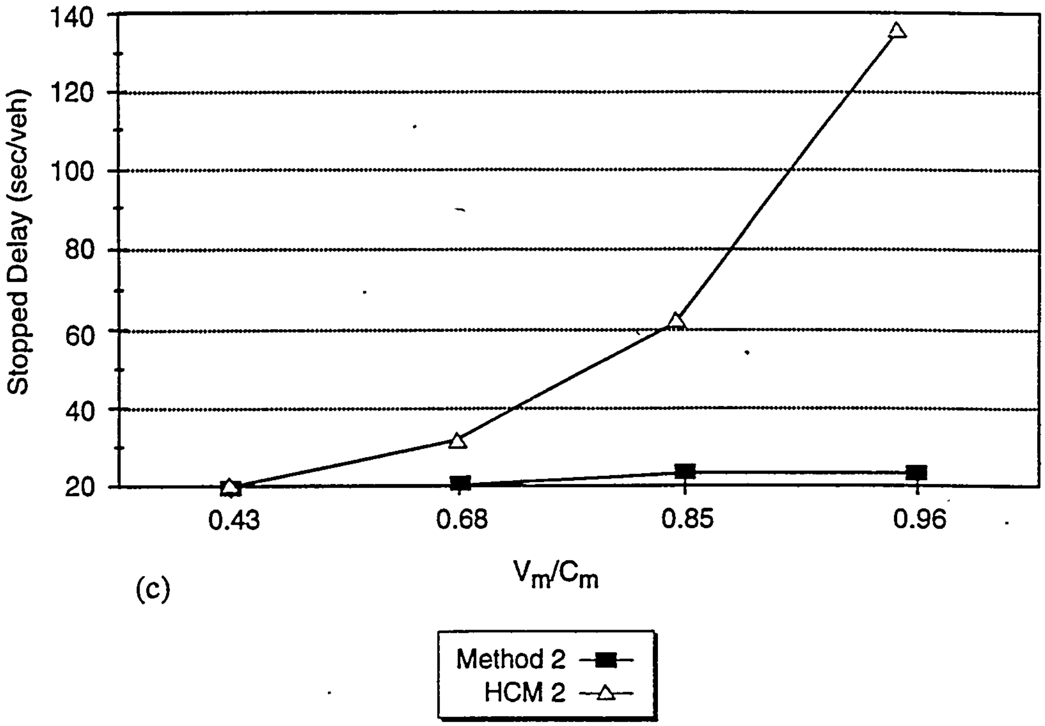
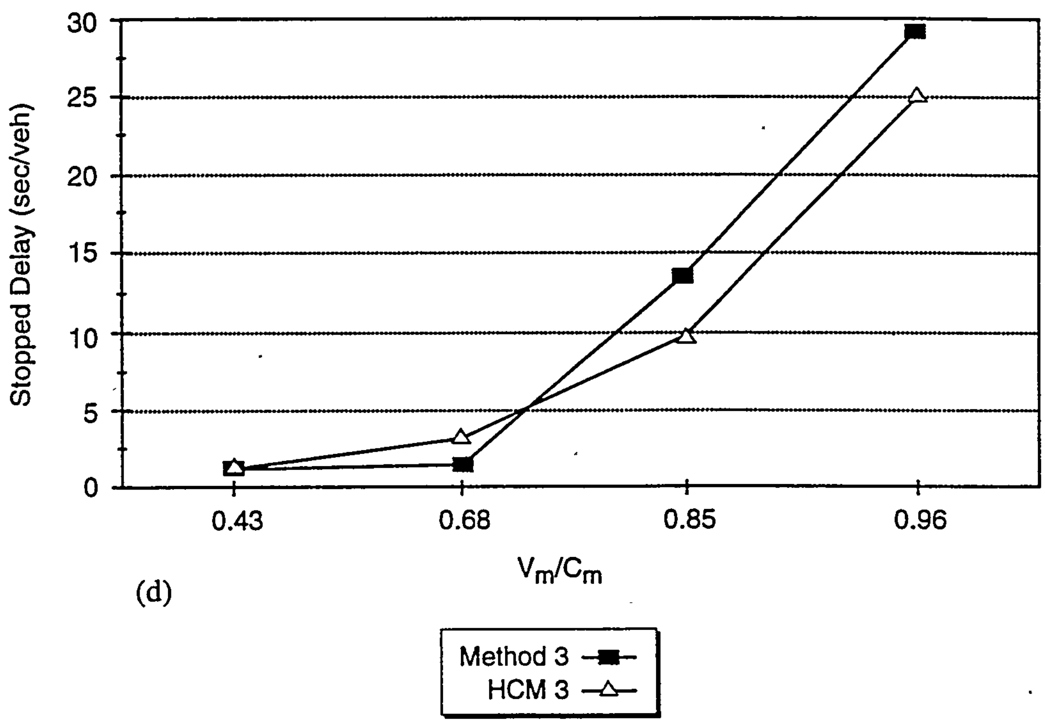
Stopped Delay vs V_m/C_m (Method 2)



(c)

Stopped Delay vs V_m/C_m (Method 3)



(d)

Figure 4 Tests of V_m/C_m (continued)

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AADT/C = 14.0
Direction = Inbound

Node:		20–22		22–25		25–27		27–30		30–40		40–50		50–60	
		5000'		5000'		5000'		5000'		5300'		5300'			
Hour	TargetV/C[1]	Speed[2]	Vph[3]	Speed	Vph	Speed	Vph	Speed	Vph	Speed	Vph	Speed	Vph	Speed	Vph
6	0.36	68.8	719	68.6	719	68.9	721	68.8	719	68.6	719	66.2	721	65.0	719
7	0.92	67.7	1,787	67.6	1,774	67.7	1,757	67.7	1,742	66.4	1,727	58.8	1,706	59.7	1,685
8	1.34	67.0	2,645	66.7	2,627	66.8	2,616	35.0	2,543	10.6	2,186	11.7	1,905	49.9	1,852
9	1.03	67.7	2,076	67.5	2,089	23.9	2,069	4.8	1,840	4.6	1,823	8.8	1,828	50.8	1,836
10	0.81	68.3	1,638	68.1	1,639	15.0	1,674	4.4	1,792	4.6	1,798	9.2	1,814	50.4	1,818
11	0.77	68.3	1,545	68.0	1,548	67.9	1,550	6.7	1,695	4.7	1,787	9.2	1,815	50.7	1,825
12	0.80	68.4	1,588	68.3	1,586	68.4	1,593	38.6	1,607	5.1	1,752	8.8	1,792	47.7	1,813

[1] Based on capacity of link (50,60) = 2,002 vplph [2] Auto speeds [3] Total vehicles entering link, including 10% trucks.

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Figure 5 **Speed Profile for Congested Freeway Experiment (continued)**

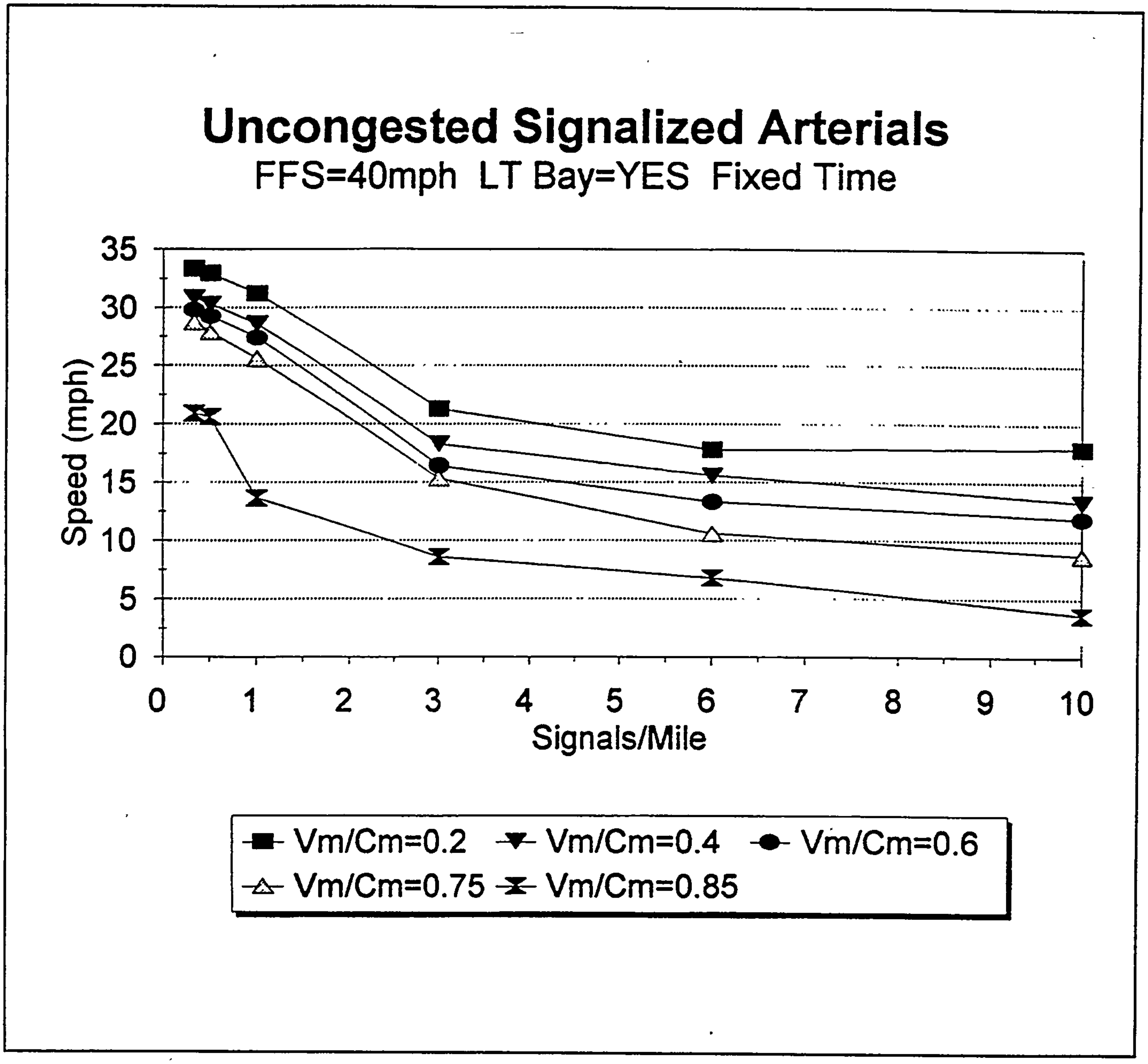

Figure 6

Speeds vs. Signals per Mile, Uncongested Arterials

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