Improving Average Travel Speeds Estimated by Planning Models

RICHARD DOWLING AND ALEXANDER SKABARDONIS

Modeling the impact of air quality requires estimating vehicle volumes and average travel speeds in order to estimate air pollutant emissions. Relatively accurate estimates of average vehicle travel speeds can be obtained from existing traffic operations models. However, when the travel demand impacts of the highway project extend over many miles of the freeway and arterial street network, it is not feasible to apply these detailed traffic operations models to the entire area of impact because of their extensive data requirements. Planners then must resort to areawide planning models to forecast vehicle volumes and average travel speeds. These traditional planning models, however, are not calibrated to produce accurate speed estimates. Free-flow speeds and a speed-flow curve are input into those models and adjusted as necessary to obtain calibrated volume estimates. Typically, the reasonableness of the final travel speeds is not checked once reliable volume forecasts have been achieved. A postprocessor methodology that can be applied at the end of a typical planning model forecast process to improve the estimates of travel speeds output by a planning model is proposed. The methodology uses an improved speed-flow curve and queueing analysis to obtain travel speed estimates that more closely approximate the average speeds estimated by typical operations models. The proposed methodology was applied to a real-life highway network and the results compared to FREQ and TRANSYT-7F simulations for a 5-mi section of freeway and a 3-mi section of a four-lane divided arterial street. The postprocessor significantly improved the original planning model estimates of average speed and delay.

The research herein came about as the direct result of new more stringent guidelines set by the San Francisco Metropolitan Transportation Commission (MTC) for evaluating the air quality impacts of new highway projects. These guidelines, developed in response to the Federal Clean Air Act, require project sponsors to demonstrate that each highway project to be added to the regional Transportation Improvement Program does not in and of itself increase air pollutant emissions for two critical pollutants: carbon monoxide and reactive organic gases. Before this, air quality impact analyses had focused on determining localized exceedances of ambient air quality standards rather than increases in pollutant emission burden. Now, project sponsors had to demonstrate not only that there would be no increases in localized exceedances but also that there would be no net increase in regional pollutant emissions either.

Detailed traffic operational analyses that had been sufficient if they focused on the proposed facility and a few parallel streets were no longer sufficient because any capacity improvement would naturally draw more traffic to the corridor and show a net increase in pollutant emissions within the corridor. It was now necessary to expand the analysis to include areawide coverage so that the reduction in travel in other corridors could be documented and included in the pollutant burden analysis. This requirement for broad coverage exceeds the capabilities of typical traffic operations models such as FREQ (*1*) and TRANSYT-7F (*2*). Thus, areawide planning models must be used, yet these large-scale planning models do not typically provide reliable estimates of operational speeds on a facility. The increased focus on the accuracy of the estimates of air pollutant emissions requires that these planning models be more accurate.

The proposed postprocessor methodology described in this paper was developed in response to these requirements for increased depth and breadth in the analysis of air quality impacts. The methodology seeks to obtain speed and congestion forecasts from areawide planning models that are closer to those that would be obtained if a more accurate operational model such as FREQ or TRANSYT-7F could be applied to each individual facility in a large study area.

R. Dowling, Dowling Associates, 180 Grand Avenue, Suite 995, Oakland, Calif. 94612. A. Skabardonis, Institute of Transportation Studies, University of California, Berkeley, Calif. 94720.

CRITIQUE OF CURRENT PLANNING MODELS' SPEED ESTIMATES

Planning models typically use a speed-flow curve such as the BPR curve to estimate the congested travel speed given the initial free-flow speed and the volume/capacity ratio (v/c) (*3*,p. 35). The standard equation for the BPR curve is

$$\text{congested speed} = \frac{\text{free-flow speed}}{(1 + 0.15 * v/c^4)} \tag{1}$$

As can be seen in Figure 1, the BPR curve generally overestimates actual average travel speeds for a freeway under all conditions [compared with the standard *1985 Highway Capacity Manual* (HCM) (*4*) speed-flow curve for an eight-lane freeway with a design speed of 70-mph].

The error in speed estimation is greatest at v/c ratios approaching 1.00, which is also where air pollutant emission rates are most sensitive to estimates of the average speed. A difference of 20 mph in the speed estimate (30 mph versus 50 mph at v/c ratios of 1.00) can double the emission rate.

It is also fairly common in planning models to have estimated demands in excess of the capacity of individual facilities. However, there are no observed speed-flow curves for v/c ratios in excess of 1.00. Consequently, the BPR curve (or some similar curve) is applied in situations in which volumes exceed the facility's capacity.

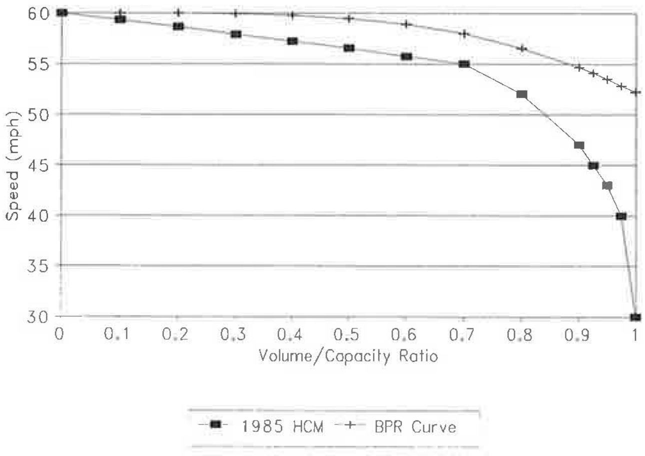

FIGURE 1 HCM and BCR speed-flow curves.

The BPR curve does not account for the effects of queueing on travel speeds and demand. Planning models consequently will significantly overestimate speeds of facilities near, at, or over capacity.

PROPOSED APPROACH

The discussion has identified two basic problems with the speed estimation procedures in typical planning models:

1. The standard BPR speed-flow curve overpredicts congested travel speeds, and
2. The BPR curve does not accurately represent speed-flow conditions when queueing is present.

The proposed methodology for improving planning model speed estimates therefore would replace the standard BPR speed-flow curve with an improved curve based on actual field observations, and it would add queueing analysis techniques to the calculation of travel speeds for situations in which the model-predicted volumes exceed capacity.

It would be most desirable to input the corrected speed-flow relationship directly into the planning model, but many software packages allow the user to specify only minor variations on the BPR curve when equilibrium assignment is being used (software packages typically allow a great deal of latitude in specifying the speed-flow curve, except when equilibrium assignment is used). For example, users of the MINUTP software package can change the coefficient but not the power of the BPR curve (*5*). It is thus not always possible to use a completely new speed-flow curve in the actual assignment process of many planning models.

Planning models are typically calibrated against traffic counts. This calibration process must compensate (at least partly) for the inaccuracies typically contained in the speed-flow curve. If the model's traffic volume forecasts are presumed to be reliable, then as a "second choice" we can choose to accept the model's traffic forecasts and merely correct the resulting speed estimates output by the model for congested situations. These speed estimates can be corrected not only by using a better speed-flow curve but also by considering queueing that arises in over-capacity situations.

It is consequently proposed that the improved speed-flow curves discussed be applied after the traffic assignment stage to improve the estimate of actual average travel speeds on the presumption that the planning model's volume forecasts are accurate (even if they used a different speed-flow curve).

Because the HCM speed-flow curve does not deal with v/c ratios in excess of 1.00 (conditions that can't occur in real life, but that can occur in planning models), another method involving queueing analysis must be used to estimate speeds for v/c's in excess of 1.00. Queues would be estimated for the peak period by dividing the peak period into hour-long intervals and performing a queueing analysis for each 1-hr time slice. The average speed over all 1-hr time slices is then calculated for the peak period.

The resulting planning model postprocessor methodology was tested against simulation results using traffic operations models for freeway and arterial street sections. The resulting speed and delay estimates were then compared to determine the effect of the postprocessor on planning model speed estimates.

PROPOSED SPEED-FLOW CURVE

Several investigators have tried different speed-flow curves to improve speed estimates and the estimated traffic volumes output by planning models. Most of these efforts have focused on changing the parameters of the the BPR curve while retaining the basic form of the equation. Different equations have been developed for freeway and arterial street facilities.

Speed-Flow Curve for Freeways

Several researchers have shown that the idealized freeway speed-flow curve in the HCM may not accurately reflect actual freeway operational conditions (*6*). The proposed postprocessor methodology is flexible enough to allow the researcher to input any desired speed-flow curve, as long as it can be specified in an equation using function available in most planning model software packages. However, for purposes of illustrating the approach, we will confine ourselves to the basic BPR equation and modify the parameters to best fit the freeway speed-flow curve in Figure 3-4 of the HCM. It does not particularly matter which curve (the HCM curve or another curve based on local data) is assumed to best represent actual operational conditions. The intent here is to demonstrate the improvements in estimated speeds that can result when a superior speed-flow curve is used to estimate speeds.

Figure 2 shows the results for two curves visually fitted to the HCM curve. The first curve (labeled "Mod.BPR4" in the figure) is

$$\text{congested speed} = (\text{free-flow speed})/[1 + (v/c)^4] \qquad (2)$$

This curve changes only the coefficient for the curve from 0.15 to 1.00. This change forces the BPR curve to drop more rapidly to 30 mph at v/c of 1.00, as does the HCM curve. This simple change in the coefficient can be implemented in many available software packages such as MINUTP.

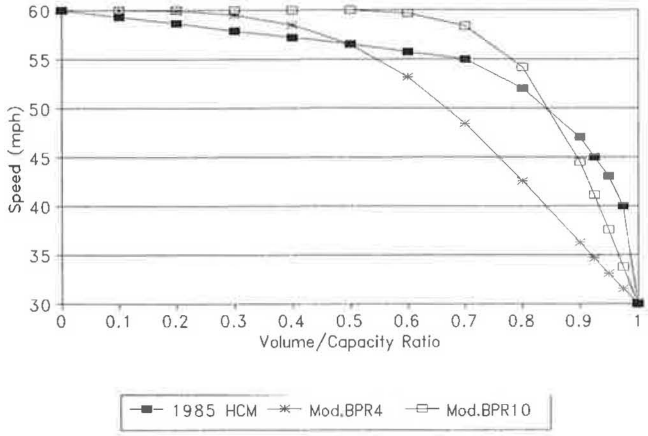

FIGURE 2 Fit of modified BPR curves to HCM curve.

The second curve seeks to correct the underprediction of speed by the first curve for values of v/c above 0.5. The curve (labeled "Mod.BPR10" in the figure) is as follows:

$$\text{congested speed} = (\text{free-flow speed})/[1 + (v/c)^{10}] \quad (3)$$

This latter curve provides a much better fit to the HCM curve, especially at the higher values of v/c ratios.

Speed-Flow Curves for Arterial Streets

Whereas operational models such as TRANSYT-7F can estimate average running speeds for an arterial street using free-flow speeds and queueing analysis at intersections, planning models must make their estimates solely on the basis of flow and capacity data for each link of the network. The development of satisfactory and relatively simple speed-flow curves for arterials has been particularly complex.

Several investigators have developed speed-flow curves for arterials (*6*). Chapter 11 of the 1985 HCM contains a method for determining average speeds on arterials on the basis of free-flow speed, intersection spacing, signal timing, and arterial class. From this information, the running speed between intersections is determined along with the average delay per intersection. The running speed and total intersection delay are then combined to determine average speed for the arterial.

$$\text{average speed} = \frac{\text{segment length}}{\text{run time} + \text{intersection delay}} \quad (4)$$

$$\text{run time} = \frac{\text{segment length}}{\text{running speed}} \quad (5)$$

A Class I arterial is a principal arterial primarily serving through traffic with speed limits of between 40 and 45 mph. A segment is generally the distance between signals on an arterial street. The running speeds are computed from the running times given in Table 11-4 of the HCM.

Intersection delay is calculated according to Equations 11-2 and 11-3 of the HCM. These equations require assumptions of the green time per cycle (g/c) for the arterial and the cycle length. The through-lane capacity and v/c ratios are also used in these delay equations.

The estimated signal delay varies for different g/c ratios, cycle lengths, and v/c ratios for an arterial with a single through-lane approach. The g/c ratio has as great an effect on intersection delay as the v/c ratio. Even for a known g/c ratio and v/c ratio, the assumed cycle length (60 or 180 sec) can double the estimated delay per intersection.

The estimated intersection delay will be quite unstable when the actual g/c ratios and cycle lengths for signals along an arterial are unknown.

Figure 3 shows the range in estimated average speeds for a Class I arterial, with a 40-mph free-flow speed, and signals at 1-mi spacing, assuming a certain range in g/c ratios and cycle lengths. As can be seen in this figure, the range in speeds is quite large, even for a given v/c ratio.

Figure 3 also compares the modified BPR curve developed for freeway links with the average speed estimates resulting from the method in Chapter 11 of the HCM. The modified BPR curve results in speed estimates higher than the 60- and 180-sec cycle estimates derived from Chapter 11. At v/c ratios in excess of 0.85, the modified BPR curve tends to overestimate the impact of congestion on arterial speeds.

It is apparent that a separate, flatter speed-flow curve could be developed to better match the Chapter 11 HCM estimates of average arterial travel speeds. However, the high variation in average speeds for a given v/c ratio appears to imply that little accuracy would be gained by such an effort. The variation in speeds for a given v/c ratio is greater than the variation in speeds across v/c ratios.

Therefore, for demonstrating the postprocessor methodology we will use the same speed-flow curve for both freeway and arterial street facilities. This curve is a modified BPR curve with a coefficient of 1.00 and with v/c raised to a power of 10. The more data-intensive method contained in Chapter 11 of the HCM is too volatile and is too dependent on data not generally available in planning studies for effective use in a planning model.

PROPOSED QUEUEING ANALYSIS TECHNIQUE

For v/c ratios in excess of 1.00 it is necessary to develop a simple queueing process that can work with the relatively limited data that would be available to a postprocessor after the planning model has completed its assignment process.

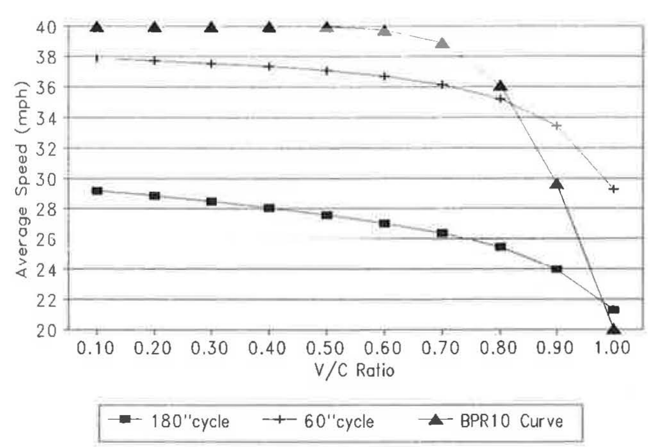

FIGURE 3 Modified BPR curve versus Chapter 11 HCM.

Queueing is a complex process that requires the determination of the variation of demand over time and consideration of the spatial characteristics of the queue itself.

Spatial Treatment of Queueing

The proposed treatment of the spatial nature of queueing is discussed for two situations: a freeway and an arterial street.

For a freeway, bottlenecks occur when there is a sudden increase in demand or a sudden drop in capacity. Figure 4 shows a typical queueing situation that could arise when an on-ramp merges with the freeway. The sudden increase in demand contributed by the on-ramp exceeds the capacity of the downstream segment of the freeway and causes traffic to back up on the ramp and on the upstream portion of the freeway itself. The congestion also reduces the traffic volumes able to continue downstream on the freeway, thus reducing the net demand on the downstream segments.

The postprocessor (since it must function after the assignment stage has been completed) must work with the individual link data from the loaded highway network. Consequently, it is not feasible for the postprocessor to track the upstream or downstream impact of queueing. The queueing analysis must be confined to the specific link at which it is detected.

The postprocessor therefore must assume that the queue on a freeway segment will occur in the segment in which the demand exceeds capacity. The effect of this assumption (shown in Figure 5) is similar to that of physically shifting the mainline and ramp queues from upstream of the bottleneck to the bottleneck itself. All vehicles entering the bottleneck section from the freeway mainline and the on-ramp would be presumed to share equally in the total congestion; in the real world, however, drivers would experience different delays depending on whether they are on the on-ramp or the freeway mainline section. This difference would affect the estimated delay for individual drivers but is not expected to significantly affect the estimate of average speed for all drivers on that freeway segment.

Notice that the length of freeway section operating at capacity (downstream of the artificial queue) would be reduced from its true real-world length. The mainline queue, however, would be increased in the idealized case (since it also includes the ramp vehicles queueing) over its true length in the real-world case. The net effect of this assumption is probably to underestimate congestion—but note that the processor is unable to reduce the demand on downstream links due to queueing, so when it analyzes flow conditions on downstream links, it will tend to overestimate congestion and thus underestimate average travel speeds over the entire freeway facility.

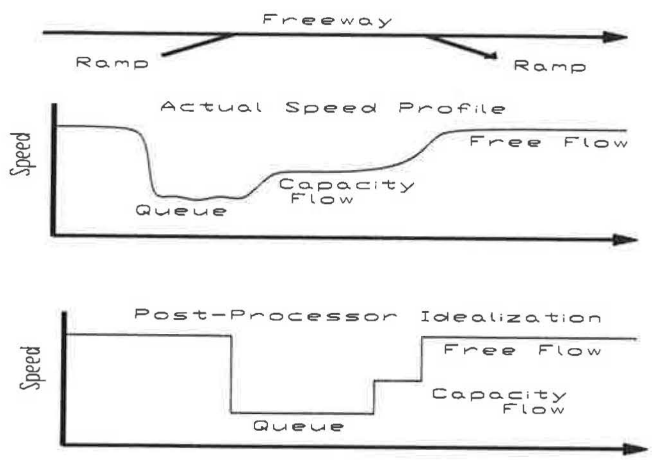

FIGURE 5 Actual versus idealized speed profiles for freeways.

For a typical arterial street the queueing situation is different. The arterial has a relatively high through capacity until it encounters a major cross street. At this location, the through capacity is cut roughly in half by a traffic signal and queues form upstream of the traffic signal (see Figure 6). The signal also reduces the volume of traffic proceeding downstream from the intersection; however, this effect is partially counteracted by the addition of traffic turning from the cross street onto the arterial at the intersection.

The postprocessor's assumption of queueing occurring on the same link on which demand exceeds capacity works quite well for nonfreeway links (see Figure 7). Note in Figure 7 that queues due to midblock signals are not captured in the queue calculation if these midblock bottlenecks have a higher capacity than the controlling bottleneck for the link.

For freeways and arterial streets, the postprocessor is unable to deal with queues extending upstream beyond the link causing the queue. The queue in excess of the length of the individual link must be assumed to stack vertically on the same link. The total delay due to the queue is preserved with the exception that impacts on upstream links (blocking of intersections for example) are neglected. The effect of this limitation is to underestimate the congestion resulting from queues.

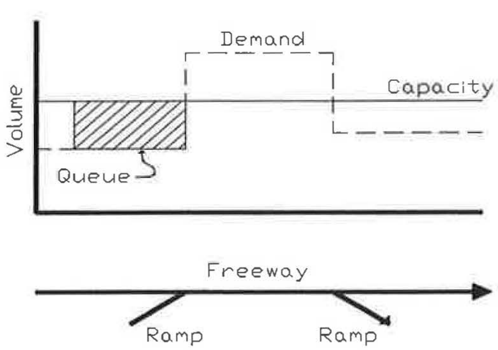

FIGURE 4 Freeway queueing pattern.

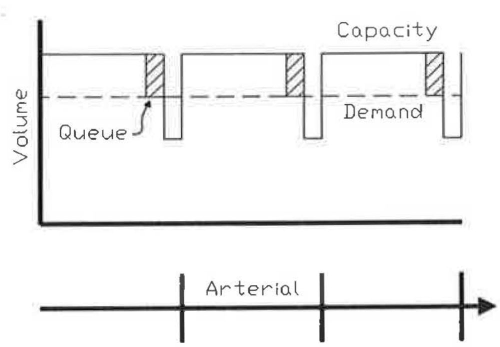

FIGURE 6 Arterial street queueing patterns.

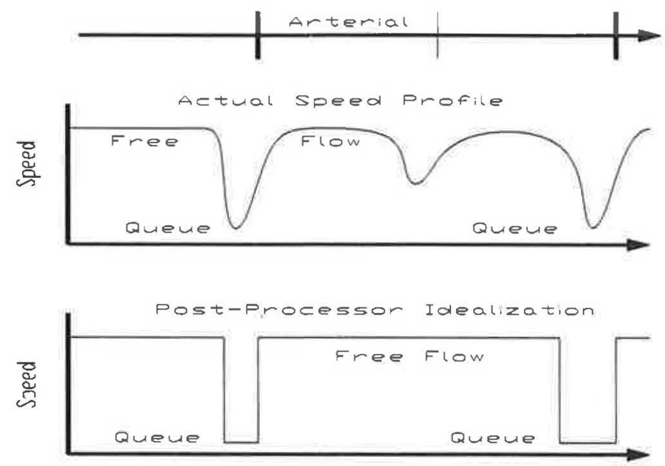

FIGURE 7 Actual versus idealized speed profiles for arterials.

The postprocessor is also unable to incorporate the reduction in downstream flows that occurs with queueing. Thus higher demands are carried through to all of the queueing calculations for the downstream links. The effect of this limitation is to overestimate the congestion effects of queueing, thus counteracting the underestimate cited earlier.

Temporal Treatment of Queueing

The temporal variation in demand can be dealt with by dividing the peak period into a sequence of time slices. For example a 5-hr peak period might be divided into five hour-long time slices with a percentage of the total demand estimated to occur during each hour. The hourly volumes can be estimated using peak-hour factors derived from traffic counts or home interview surveys.

The temporal variation of queues is handled in the postprocessor by splitting each peak period into hour-long time slices. Within each time slice, the demand and the capacity are assumed to be constant. This assumption underestimates the effects on queueing of peaking within the hour; however, the underestimate is considered to be minor. The computation requirements for the hourly time slices were approaching the capacity of the MINUTP software package, so it was not deemed feasible to go to 15-min time slices.

For each 1-hr time slice, the following two computations are made:

$$\text{average link speed} = \text{average queue speed} * \left(\frac{\text{average queue length}}{\text{link length}}\right) + \text{uncongested speed} * \left[1 - \left(\frac{\text{average queue length}}{\text{link length}}\right)\right] \quad (6)$$

$$\text{VHT} = \text{demand} * \text{link length/average link speed} \quad (7)$$

where

VHT = vehicle hours traveled;
average speed in queue = capacity/lane * 25 ft/vehicle;
average queue length = average queue * 25 ft/vehicle;
average queue = $(Q1 + Q2)/2$;
$Q1$ = queue at start of time slice;
$Q2$ = queue at end of time slice = $(Q1)$ + (demand rate) * (1 hr) − (capacity rate) * (1 hr); and
uncongested speed = free-flow speed/$[1 + (v/c)^{10}]$.

If the average queue length exceeds the link length, then the average link speed is set equal to the average queue speed, and the link length is set equal to the queue length.

At the end of the computations for all time slices, the following computations are made:

$$\text{VMT} = \text{total demand} * \text{link length} \quad (8)$$

$$\text{delay} = \text{VMT} * [(1/\text{average link speed}) - (1/\text{free-flow speed})] \quad (9)$$

where VMT equals vehicle miles traveled.

The density of 25 ft/vehicle (used in calculating queue length and queue speed) is typical for vehicles queueing on an arterial street at a traffic signal but not for vehicles on a freeway. To determine the density for vehicles queueing on a freeway, it is necessary to know the v/c ratios for the upstream links at which the queue occurs; however, this information is not available to the postprocessor. Consequently, the 25 ft/vehicle assumption has been applied to freeway queues as well. Other researchers may wish to improve on this assumption.

The resulting travel speeds in queues are shown as follows:

Facility	*Capacity (vph)*	*Speed (mph)*
Freeway	2,000	9.5
Expressway	1,200	5.7
Arterial	900	4.3
Collector	600	2.8
Ramp	1,700	8.0

Queues remaining at the end of the last time slice for a peak period are not added to the computations. This will tend to underestimate peak-period delay when it extends beyond the peak period, thus partially compensating for the overestimates in delay resulting from the postprocessor's inability to reduce demands downstream of the queue.

EXPERIMENTAL APPLICATION OF POSTPROCESSOR METHODOLOGY

The improved speed-flow curve and rudimentary queueing analysis was coded into a network processing utility (Netmerge) in the MINUTP software package. This routine was run on a 185-zone, 1,100-link loaded highway network that had been developed to forecast year 2010 traffic for the city of Hayward. The resulting VMT, VHT, average speed, and delay estimates were then isolated for two test facilities in this network: the proposed Route 238 freeway, and the existing Mission Boulevard (see Figure 8).

1. The proposed Route 238 freeway is a four-lane freeway that would extend 5.03 mi from Interstate 580 to Mission Boulevard at Industrial Parkway.
2. The Mission Boulevard test section is 3.23 mi long and has five traffic signals spaced an average of 0.8 mi apart. The

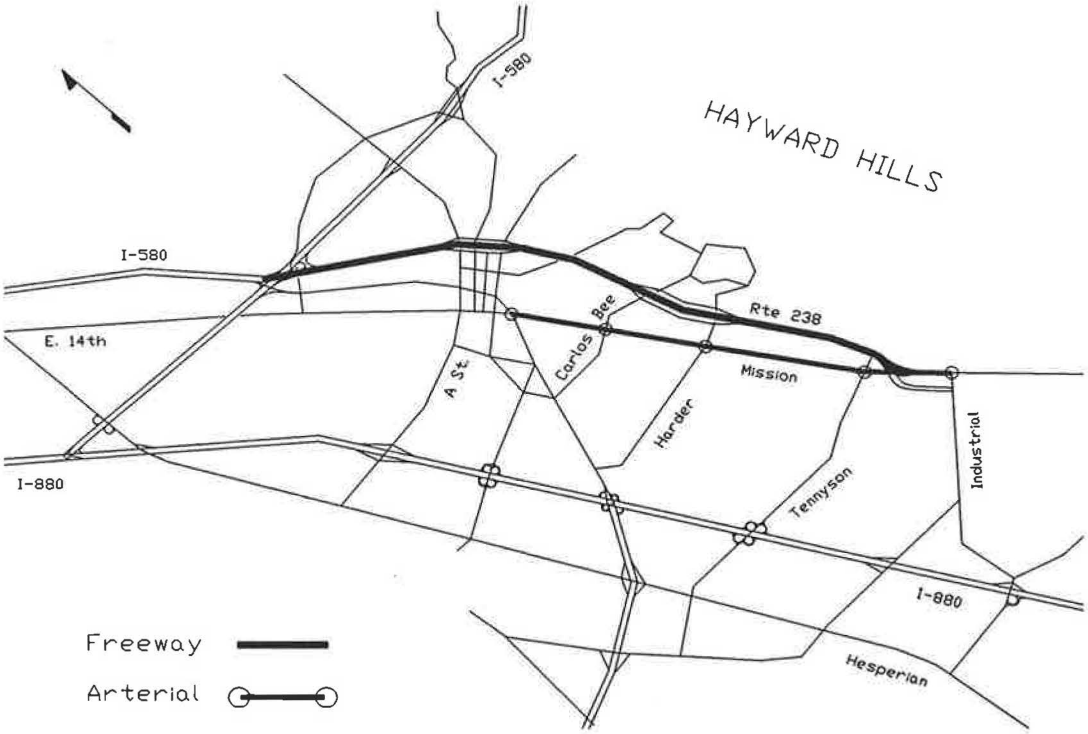

FIGURE 8 Hayward model study area: Route 238 and Mission Boulevard test sections.

signals are uncoordinated. Mission Boulevard varies between four and six lanes with left-turn pockets. The speed limit is 35 mph.

The average speeds and delay estimated by the described method (using the MINUTP software package) were compared for the Route 238 freeway facility against comparable speed estimates generated by FREQ freeway model simulation runs for the same freeway. The same link lengths, free-flow speeds, and capacities were coded in the FREQ model as were used in the MINUTP model run. The weaving analysis capability in FREQ was turned off to ensure that both models were using the same capacities. Auxiliary lanes (under 1,000 ft long) were coded in the FREQ model at major on-ramps to ensure that the idealized capacities assumed in the MINUTP model run were matched by the operational analysis at ramp merge points contained in the FREQ model.

Table 1 shows the results of this comparison. Note that FREQ calculates ramp delay but not ramp VMT or speeds. Note also that in this example there is relatively little congestion predicted for the freeway mainline. Some queueing is predicted for the on-ramps, thus limiting the amount of congestion predicted to occur on the freeway mainline.

The results show four tests of the postprocessor speed-flow curves and queueing analysis against the results of the FREQ run. The first two entries show the results of using standard planning model runs without a postprocessor (MINUTP). The next two columns show the results of applying the postprocessor to calculate queues and the improvements that result from using a speed-flow curve of $1 + v/c^{10}$.

The first column shows the results using the standard planning model run using the standard BPR curve. The planning model significantly underestimates mainline and ramp delay and overestimates average travel speed on the freeway mainline.

The second column shows the improvement in delay estimates that result if the $1 + v/c^4$ curve is used in a standard planning model instead of the standard BPR curve. The ramp delay estimate is significantly improved, but unfortunately the model now appears to overestimate delay on the freeway mainline and underestimates the average travel speed on the freeway mainline.

The postprocessor is applied in the third column, using the same curve that was used in the second column. Because there is little congestion on the freeway, the addition of the postprocessor's queueing analysis has little effect on the mainline freeway speed estimates (there is a slight improvement due to splitting the peak-period demand into five hourly periods and averaging the speed over the entire peak period rather than for the peak hour as used in the standard model runs). The postprocessor, however, does a significantly better job of identifying the delay that FREQ predicts would occur at the on-ramps.

TABLE 1 Straight Planning Model Versus Postprocessor Versus FREQ for Route 238 Freeway

Freeway	Standard MINUTP Runs		MINUTP Runs with Post-Processor		FREQ No Weave
Speed Eqn.	$1+0.15(v/c)^4$	$1+(v/c)^4$	$1+(v/c)^4$	$1+(v/c)^{10}$	HCM Curve
Queue Calc.	No Queue	No Queue	Queueing	Queueing	Queueing
VMT (Mainline)	115,979	115,979	115,979	115,979	116,255
VHT (Mainline)	2,066	2,820	2,790	2,484	2,328
Ave.Speed (Mainline)	56.1	41.1	41.6	46.6	49.9
Delay (Mainline)	134	884	856	552	390
Delay (Ramp)	15	100	510	480	483
Total Delay	149	984	1,366	1,032	873

The estimates of ramp delay and average freeway mainline speeds are even further improved when the curve of $1 + v/c^{10}$ is included in the postprocessor (Column 4).

TRANSYT-7F was run for various combinations of cycle lengths and signal coordination conditions to determine the likely range of average travel speeds to be expected on Mission Boulevard. Unfortunately, field data were not available to verify these results independently. Given the many years that TRANSYT-7F has been successfully used in California and the United States to simulate arterial signal operations and optimize signal timing, this was not considered to be a serious deficiency.

The peak hour was simulated with TRANSYT-7F, and the results expanded to the total peak period by multiplying VHT, VMT, and delay by 4 (since the peak-hour volumes are 25 percent of the total peak-period volumes). Average travel speed was obtained directly from the peak-hour results output by TRANSYT-7F. A test was also made running TRANSYT-7F five times (once for each hour of the peak period) and summing the results; however, this was found to significantly underestimate total delay because queues from one run of TRANSYT-7F could not be carried over to the following period.

Table 2 shows the results of the TRANSYT-7F runs. It illustrates the volatility of arterial street operations for different cycle lengths and signal coordination conditions.

The TRANSYT runs include various intersection design refinements to optimize the operation of Mission Boulevard (such as left- and right-turn pockets) that could not be specified in the MINUTP planning model run. TRANSYT was allowed to find an optimal equi-sat (equal degrees of saturation) signal timing solution for the splits for the noncoordinated condition. The MINUTP planning model run used in the postprocessor, however, coded only the number of through lanes at each intersection and assumed a 50:50 *g/c* split for each street at each major intersection for the purposes of determining link capacity.

Two TRANSYT runs were selected from Table 2 for comparison with the postprocessor. The 190-sec cycle runs were selected to show the results for an optimal cycle length under uncoordinated conditions and the impacts of signal coordination at the same cycle length.

Table 3 shows the results of various tests of the postprocessor against the two selected TRANSYT-7F simulations of Mission Boulevard. The reader can also compare these results with the additional TRANSYT-7F results shown in Table 2. The postprocessor comes quite close to predicting the correct average speed when the cycle length on the arterial is 100 sec.

TABLE 2 Summary of TRANSYT-7F Runs: Estimates of Delay and Average Speeds on Mission Boulevard

Arterial Street	190 Second Cycle		120 Second Cycle		100 Second Cycle	
	No Coordination	Coordination	No Coord	Coordination	No Coord	Coordination
VMT	18164	18164	18164	18164	18164	18164
VHT	1308	981	1333	903	1679	1315
Delay	699	469	820	390	1167	803
Ave.Speed	13.9	18.5	13.6	20.1	10.8	13.8

TABLE 3 Straight Planning Model Versus Postprocessor Versus TRANSYT-7F: Estimates of Delay and Average Speeds on Mission Boulevard

Arterial Street	Standard MINUTP Runs		Standard MINUTP Runs with Post-Processor		TRANSYT7-F 190 Second Cycle	
Speed Eqn.	$1+0.15(v/c)^4$	$1+(v/c)^4$	$1+(v/c)^4$	$1+(v/c)^{10}$	simulation	
	No Queueing Analysis		Queueing Analysis		No Coord	Coordination
VMT	17,258	17,258	17,258	17,258	18,164	18,164
VHT	533	755	1,665	1,648	1,308	981
Delay	38	260	1,162	1,145	699	469
Ave.Speed	32.4	22.9	10.4	10.5	13.9	18.5

The standard MINUTP model runs both significantly underestimate delay and overestimate average speed. The postprocessor runs overestimate delay and underestimate speed, but they are generally much closer at predicting the speed for noncoordinated conditions. The postprocessor appears to better estimate speeds that would occur under nonoptimal signal timing and turn pocket situations. A less conservative link capacity assumption (using an assumed *g/c* higher than 50 percent) would probably result in higher speed estimates from the postprocessor that more closely approximate the average travel times that could be achieved under optimized signal timing and coordination.

CONCLUSIONS

The proposed postprocessor methodology results in significantly improved estimates of delay and travel speeds compared with the raw planning model output. For the specific speed-flow curves and queueing density assumptions used in this example, the methodology appears to conservatively underestimate travel speeds. But this underestimate is still much closer to the speeds that would be predicted by traffic operations models than are the raw estimates coming from the planning model. Further improvements in the speed-flow curves for arterial streets and in the queue density assumptions used for freeways would probably improve these results.

REFERENCES

1. T. Imada and A. D. May. *FREQ8PE: A Freeway Corridor Simulation and Ramp Metering Optimization Model.* UCB-ITS-RR-85-10. Institute of Transportation Studies, University of California, Berkeley, June 1985.
2. *TRANSYT-7F User's Manual.* Transportation Research Center, University of Florida, Gainesville, Oct. 1988.
3. Comsis Corp. *Traffic Assignment.* FHWA, U.S. Department of Transportation, Aug. 1973.
4. *Special Report 209: Highway Capacity Manual.* TRB, National Research Council, Washington, D.C., 1985, Figure 3-4.
5. *MINUTP Technical User Manual.* Comsis Corp., Mountain View, Calif., Feb. 1988.
6. Cambridge Systematics. *Analysis of Temporal Demand Shifts to Improve Highway Speed Modeling.* Arizona Department of Transportation, Phoenix, April 1988.

Publication of this paper sponsored by Committee on Transportation and Air Quality.