Reconciling Estimated and Measured Travel Times on Urban Arterial Streets

KENNETH G. COURAGE, RANDALL H. SHOWERS, AND DOUGLAS S. MCLEOD

Although the *Highway Capacity Manual* (HCM) modeling process is widely accepted, there is evidence of significant disparity between estimated arterial travel speeds and speeds measured in the field. The HCM suggests that speed and travel time values measured in the field are preferable to the computed estimates. Often the validity of the model estimates may be challenged by competing interests. The primary objective was to reconcile the differences between estimated and measured travel times on arterial streets. The principal product is a set of recommendations for modifying the estimation and measurement procedures to reduce the disparity between them. Four tasks were involved: (*a*) examine the arterial speed computational methodology to identify sources of disagreement with field measures performed with moving vehicles, (*b*) compare a large sample of measured travel speeds with travel speed estimates carried out using the HCM methodology, (*c*) develop candidate adjustment factors that can be applied in practice to improve the agreement between estimated and measured speeds, and (*d*) test the candidate adjustment factors against the field data and recommend specific modifications to the travel time procedures and the HCM model. Although the HCM models were intended for analyses on the planning and operational level, the focus here is on planning applications. The main difference between planning and operational analyses is the levels of detail of the input data and in the required level of accuracy of the results. It is important in either case that the travel time estimates be unbiased, that is, the procedures should not consistently underestimate or overestimate the travel times. The results offer a reasonable explanation for the apparent discrepancies between estimated and measured travel times and delays on arterial streets. They also provide a practical means of adjusting the estimated delay values to produce a very close agreement with the corresponding measurements.

Florida's efforts at growth management have gained national attention and respect for their policy content as well as their technical methodology. One of the main features of the system is a mandate for periodic assessment of the level of service (LOS) for public facilities—more specifically, roads. For arterial streets the current LOS evaluation criterion is average travel speed. The *Highway Capacity Manual* (HCM) (*1*) provides a technique for estimating the average traffic speed based on known values of traffic volumes and signalized intersection capacities. The Florida methodology relies heavily on the HCM technique.

Florida has its own LOS manual (*2*) that assists local agencies in applying the HCM model at a planning level. The Florida LOS manual includes software for performing the computations, tables for deriving approximate estimates, guidelines for preparing input data, and limitations on the acceptable values for assumed parameters.

K. G. Courage and R. H. Showers, University of Florida, Transportation Research Center, 512 Weil Hall, Gainesville, Fla. 32611. D. S. McLeod, Florida Department of Transportation, 605 Suwannee Street, Tallahassee, Fla. 32399.

STATEMENT OF PROBLEM

The HCM modeling process is widely accepted, but there is evidence of significant disparity between estimated arterial travel speeds and speeds that are measured in the field (*3*). The HCM suggests in Chapter 11 that speed and travel time values measured in the field are preferable to the computed estimates. Often the validity of the model estimates may be challenged by competing interests, especially in growth management applications.

In a properly timed arterial traffic control system, it is usually possible to travel progressively through several signals without stopping. Field studies often show no delay at all for specific travel time runs. The HCM model always predicts some delay at each intersection. It must be understood that the HCM model is a deterministic approximation of a stochastic process. It is not expected to produce an accurate prediction of the travel time for each run; however, it should be able to produce an unbiased estimate of the average travel time over several runs. Recent evidence indicates that in some cases the HCM method tends to overestimate average travel times to a degree that cannot be overlooked (*4*).

A more reliable method is required for estimating vehicular delay and travel times on arterial streets without the need for moving-vehicle studies. The accuracy of such a method must be adequate at least for planning purposes. Moving-vehicle studies are very labor-intensive and cannot be applied to the hypothetical situations that generally are involved in planning applications.

STUDY OBJECTIVES AND TASKS

The primary objective of the study was to reconcile the differences between estimated and measured travel times on arterial streets. The principal product of the study is a set of recommendations for modifying both the estimation and measurement procedures to reduce the disparity between them. Four tasks were involved:

1. Examine the arterial speed computational methodology to identify sources of disagreement with field measures performed with moving vehicles.
2. Compare a large sample of measured travel speeds with travel speed estimates carried out using the HCM methodology.
3. Develop candidate adjustment factors that could be applied in practice to improve the agreement between estimated and measured speeds.

4. Test the candidate adjustment factors against the field data and recommend specific modifications to the travel time procedures and the HCM model for use in the Florida's LOS manual.

The HCM models were intended for analyses at the planning and operational levels, but this study focused on planning applications. The main difference between planning and operational analyses is the levels of detail of the input data and in the required level of accuracy of the results. It is important in either case that the travel time estimates be unbiased: the procedures should not consistently underestimate or overestimate the travel times. This is because they are treated as deterministic models for decision-making purposes.

BACKGROUND DISCUSSION

HCM Chapter 11 Model

The structure of the HCM Chapter 11 model is illustrated in Figure 1. Note that the intersection delay, as determined in HCM Chapter 9, is an important element of this model. The average arterial speed is determined by dividing the distance between intersections (Points 1 and 2) by the total travel time between the points. The total travel time is determined as the sum of the running time and the total intersection delay. The running time is obtained from HCM Table 11-4 as a function of arterial classification, signal density, and free speed. The intersection delay is obtained from the HCM Chapter 9 analysis.

Figure 1 also shows a typical time-space trajectory for a moving vehicle between Points 1 and 2. Each vehicle traveling on this segment of roadway will have a different time-space trajectory. The essence of the Chapter 11 model is a representation of the two travel time elements shown in Figure 1 as a deterministic approximation of the "average" vehicle's trajectory.

Moving-Vehicle Studies

Moving-vehicle studies may be carried out in several ways with different levels of instrumentation. Since the LOS basic measure of effectiveness as specified by the HCM is average travel speed, it is theoretically necessary to measure only the total time required to travel a given roadway segment or section for comparison purposes. This can be done easily with nothing more than a stopwatch. However, since the HCM model computes the total travel time as the sum of two components—running time and stopped delay time—the candidate adjustment factors needed to reconcile the two techniques must be applied separately to each travel time element. This requires that both travel time elements be available from both techniques. For this reason, detailed time-space trajectories were obtained for each moving-vehicle run.

Sources of Bias

There are three general reasons for discrepancy between estimated and measured travel times on an arterial roadway: errors in the data, deficiencies in computational procedures, and conceptual differences between procedures.

Errors in Data Used by Computational Procedures

Data errors can be caused by field errors; however, it is more likely that they will result from the use of assumed values for data items that are very difficult or costly to measure accurately in the field—examples are saturation flow rates, turning movement volumes, and traffic signal timing. The study procedures for each of these data items are simple and straightforward, but it is very difficult to cover an entire roadway section simultaneously with moving-vehicle studies. Furthermore, it is not possible to guarantee that each data item is applied at the exact moment of passage of the moving vehicle through the system. Thus, particularly for planning studies, it is necessary to rely on assumptions and approximations in developing the input data for travel time estimates.

It could be argued that errors in the input data could affect the results either way (i.e., underestimate or overestimate), but most of the data items have a direct effect on the volume/capacity (v/c) ratio. The v/c ratio, in turn, has a nonlinear influence on delay. An overestimation in the v/c ratio will produce a much larger error in the delay estimate than a corresponding degree of underestimation. This introduces a bias toward overestimation of travel times, an

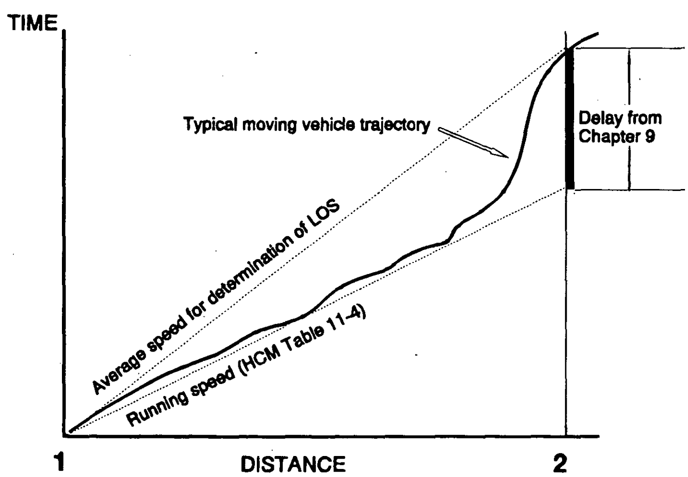

FIGURE 1 Existing Chapter 11 model structure.

effect that can be worse during the peak periods when the operation is fully saturated (i.e., v/c approaching 1.0).

Deficiencies in Computational Procedures

Some apparent deficiencies in the HCM methodology could produce large discrepancies between estimated and measured travel times. For example, the running time estimates obtained from HCM Table 11-4 are based only on free speed and signal density for a given arterial classification. They are assumed to be independent of traffic volume, number of lanes, and other parameters that could be expected to influence running speeds. The running speed estimates are particularly vulnerable to situations in which speeding occurs because most agencies are reluctant to recognize an operating speed that exceeds the speed limit.

The direct application of the Chapter 9 delay model to estimate the stopped delay at signalized intersections is also questionable. The main problem is that the delay is computed in Chapter 9 as the sum of two components. The first component estimates the delay that would occur if all vehicles arrived at the intersection with uniform spacing. The second component is a correction factor that accounts for randomness in the arrival pattern, including the "cycle failures" that result when the arrivals exceed the approach capacity for one or more cycles. This component is called the incremental delay term.

This model is entirely appropriate when applied at isolated signals; however, the direct extension to arterial routes with coordinated signals is somewhat difficult to rationalize. In coordinated arterial systems the "metering" effect of the upstream signal can be expected to reduce the randomness of the arrivals at the downstream signal. In particular, it is reasonable to anticipate a much lower occurrence of cycle failures at the downstream signal because temporary overcapacity situations are cushioned by the upstream signal. Therefore, the application of the incremental delay term equally at all intersections can be expected to overestimate the total delay, and therefore the average travel time.

Another important factor is the effect of progression quality on delay. This is incorporated in the HCM model as the progression factor (PF). It is common practice to assume Arrival Type 4 on coordinated arterial streets. This implies a progression correction factor ranging from 0.7 to 0.9. The limits and default values for the computation of the progression factor are explained in HCM Table 9-13, uniform delay (d_1) adjustment factor, DF.

Figure 2 shows the distribution of progression factors obtained by running TRANSYT-7F (*5*) on several available data sets to generate a sample of approximately 100 links. The progression factor was established by running TRANSYT-7F twice for each data set: once with coordination and once without. Note that a large proportion of the observations fell outside of what is generally accepted as the range for Arrival Type 4.

Conceptual Differences Between Estimation and Measurement Procedures

In the discussion of the first two sources of bias, discrepancies between the estimated and measured values generally would be resolved in favor of the measured values. In other words, both procedures are addressing the same phenomena and the differences would be attributed to shortcomings in the input data or the estimation procedure. In this case, the two procedures are addressing different phenomena by the same name.

The average travel time computed by the HCM model applies to all vehicles on the approach, regardless of arrival time. On the other hand, the measured travel time applies primarily to the subset of vehicles within the progression band. Clearly, the vehicles receiving the benefits of progression may be expected to have higher overall speeds than the rest of the vehicles. Therefore, it should not be surprising that the results of moving-vehicle studies are more optimistic than the HCM estimates.

This raises an interesting philosophical question: which of the two speeds is more appropriate as an LOS criterion? Since the HCM defines LOS, it could be argued that the estimated values are the only ones compatible with the HCM. On the other hand, the LOS criterion is intended to be based on motorist perception of disutility, and it is reasonable to propose that it is best applied to the coordinated arterial traffic flow. Theoretically, the two definitions will converge as traffic volumes approach their capacity.

STUDY DESCRIPTION

The main objective of this paper is to identify sources of bias and recommend adjustments that will eliminate the bias between the field data and estimation models. To make comparisons, moving-vehicle data and arterial data are needed. The overall study procedure is illustrated in Figure 3. Field data on travel time trajectories and arterial characteristics were furnished by consultants under contract to the Florida Department of Transportation (FDOT). Five counties in the southeast part of Florida containing both urban and rural roadways were represented; Miami and Fort Lauderdale urban areas were predominant. The moving-vehicle study locations and general characteristics of the sample are described in general in Table 1 and in more detail in Table 2. A total of 656 runs were included, covering 161 km (100 mi) of arterial routes containing 316 signalized intersections. A sample summary of the data and graphics for each route is presented in Figure 4. The moving-vehicle equipment and study methodology are described elsewhere (*6*).

The level of detail in the arterial characteristics varied from location to location. In general, the normal planning level requirements for field data were greatly exceeded, but default values were used where necessary to perform the estimates of travel time and delay. In some cases g/C ratios were observed in the field; in other cases they were determined from signal timing records. Some observations of progression quality were made by observing the proportion of vehicles arriving on the green phase. Where field observations were not carried out, a default arrival type was assigned on the basis of peak direction. Concurrent mid-block traffic counts were carried out to obtain representative 15-min volumes on all routes. Recent peak-period turning movement counts were used where available to estimate the proportion of turns leaving the arterial approaches at each intersection. Where no turning movement counts were available, a default value (usually 12 percent) was applied. Saturation flow rates were observed in some cases, and representative default values were applied in others.

The HCM estimates of travel time and delay were performed using the ART-PLAN spreadsheet program (*7*). ART-PLAN performs a straightforward implementation of the HCM Chapter 11 methodology. A sample ART-PLAN analysis summary is presented in Figure 5. Versions of ART-PLAN have been developed for both the 1985 and 1994 methods. The modified estimates of travel time

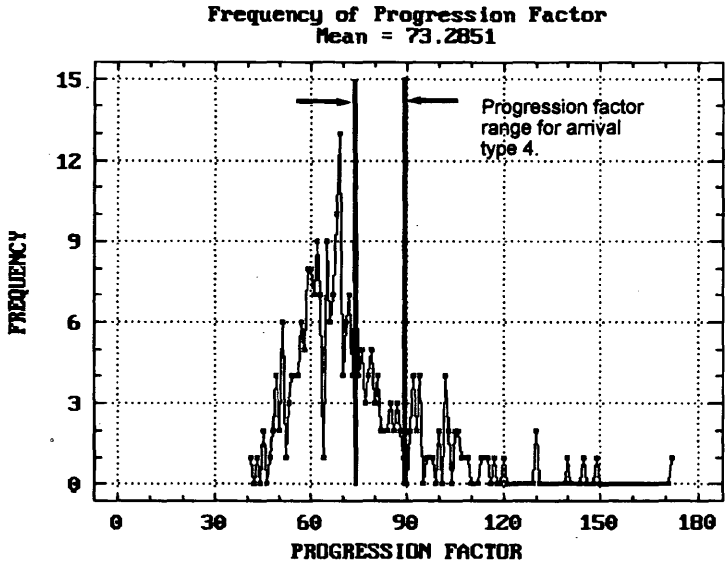

FIGURE 2 Progression adjustment factor variation.

and delay resulting from adjustment factors were performed using a combination of standard programming methods for data analysis.

The reduced data for all of the results for each run (i.e., measured travel time estimated by various methods) were combined into a unified data base. The analysis of the reduced data was performed by a combination of dBase and SAS procedures.

INITIAL COMPARISON OF ESTIMATED AND MEASURED TRAVEL TIMES

The comparison of unadjusted travel time estimates with the field data produced very discouraging results. Considering the complete sample, the estimated travel time estimates averaged 3,000 percent higher than the corresponding measured values. Substituting the 1994 HCM Chapter 11 method, the degree of overestimation was reduced to 974 percent. Clearly, the unadjusted data require further attention.

Inspection of the data indicates that the discrepancy is concentrated in a relatively small proportion of the cases, each of which has an unreasonably high v/c ratio. The v/c ratio is very sensitive to the values used for traffic volume, saturation flow rate, and *g/C* ratio. Accurate modeling requires very precise data for all of these items. The level of required accuracy generally exceeds the accuracy normally associated with planning level data, which rely heavily on assumptions and approximations. Planning estimates of these

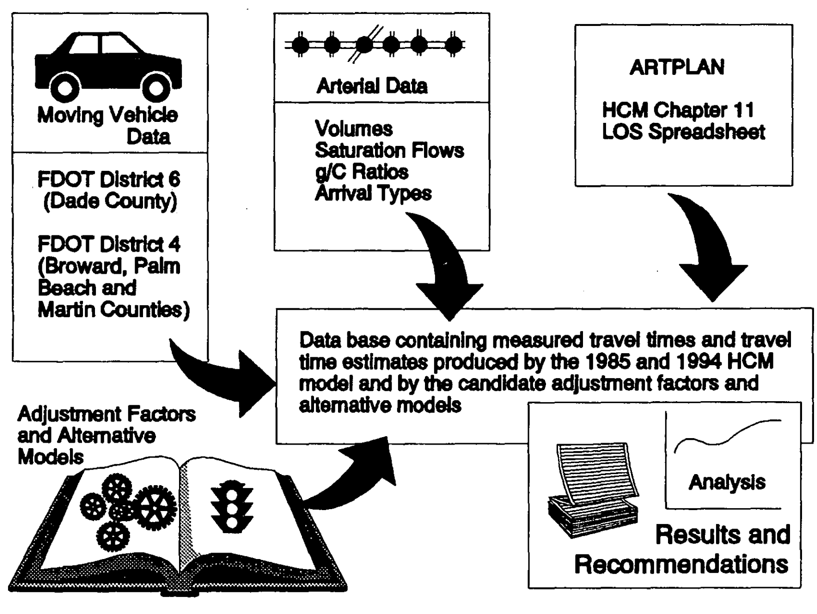

FIGURE 3 Study procedure and data flow.

TABLE 1 Summary of Travel Time and Delay Study Characteristics

	Total	Breakdown by FDOT District		Breakdown by HCM Arterial Class	
		4	6	I	II
Route kilometers[1]	161	95	66	102	60
Total runs	656	512	144	405	251
Run kilometers[1]	2223	1589	685	1542	732
Number of signals	316	182	134	178	138
Signals per route	6.9	6.2	9.4	7.0	6.7
Signals per km[1]	2.0	2.0	2.0	1.9	2.3

[1] 1 km = 0.6 mi.

data items generally will not support accurate modeling. This is very difficult to address in an adjustment factor.

The HCM model is defined to be valid for v/c ratios of less than 1.2. When v/c ratios exceed this threshold in the field, the result is extensive and prolonged congestion. Several of the routes represented in the field data had computed v/c ratios much greater than 1.2. A logical alternative would be to discard all cases in this category; however, the proposed model must be able to deal with such cases. A more practical candidate adjustment would involve placing an upper limit of 1.2 on the computed v/c ratio.

In a report describing the travel time data collection and capacity analysis performed in District 4 (*8*), the consultant indicated that the calculated speeds were reasonably close to the measured speeds, except when the *g*/*C* ratio was less than 0.4. The consultant recommended that future projects of this nature devote more effort to collecting turning movement counts to reduce the dependence

TABLE 2 Summary of Routes for Travel Time and Delay Studies

Area/Route Name	Number of Signals	Signal Density sig/km[1]	Route Length km[1]	Class & Speed kph[1]	Number of Studies
DISTRICT 6 (Dade County, Florida)					
NW 42 St/LeJune Rd A	4	2.6	1.5	II/65	12
NW 42 St/LeJune Rd B	4	2.4	3.4	II/65	12
SW 152 St	5	1.3	4.0	I/73	12
NW 125 St (1)	6	2.4	2.4	II/56	16
NW 125 St (2)	11	2.6	4.2	I/56	16
NW 125 St (3)	9	3.9	2.3	II/56	16
NW 79 St East	15	2.1	7.0	I/65	16
SR 860 MGD	16	2.1	7.7	II/65	12
Bird Rd (1)	5	2.7	1.9	II/65	15
Bird Rd (2)	11	2.2	5.0	II/65	12
SW 87 Ave (1)	6	1.3	4.7	I/65	16
SW 87 Ave (2)	14	1.6	9.0	I/65	12
Red Road/SW 57 Ave	7	1.1	6.2	I/65	16
NW 72 Ave	2	0.6	3.4	I/56	14
NW 107 Ave South	5	2.1	2.3	I/65	13
DISTRICT 4 (Broward, Palm Beach and Martin Counties, Florida)					
SR 5/Federal Hwy	4	0.9	4.5	I/73	42
SR 5/US 1	6	1.0	6.3	I/65	18
SR 870/Commercial Blvd	4	1.2	3.2	I/73	51
SR 7/US 441	6	1.5	3.2	I/73	62
SR 845/Powerline Rd	8	2.0	4.0	II/73	39
SR 858/Hallandale	4	1.7	2.4	II65	46
SR 808/Glades Blvd	4	1.9	2.1	I/73	48
SR 870/Commercial Blvd	6	1.8	3.2	I/56	41
ST 805/S Dixie Hwy	3	1.8	1.6	II/56	40
SR 824/Pembroke Rd	4	1.9	2.1	II/56	49
SR 816/Oakland Pk Blvd	12	3.0	4.0	I/65	50
SR 7/US 441	9	2.8	3.2	I/65	52
SR 814/Atlantic Blvd	8	3.2	2.4	II/56	60
SR 824/Broward Blvd	7	3.6	1.9	II/56	47
SR 858/Hallandale	12	3.6	3.2	I/65	66

[1] 1 km = 0.6 mi.

END POINT	LINK LENGTH (ft)	TRAVEL TIME (sec)	DELAY (sec)	STOPS	ACCELE-RATION NOISE	SPEED (MPH) AVG.	RUN.
NW 6TH AVE	1438	34.4	1.7	0.2	1.99	28.5	30.0
NW 4TH AVE	656	18.0	2.5	0.2	1.86	24.7	28.7
DIXIE HWY	1245	122.7	85.1	1.0	1.98	6.9	22.6
NE 1ST AVE	194	7.3	0.0	0.0	2.40	18.1	18.2
US 1 SR 5	1752	73.6	31.4	0.6	2.16	16.2	28.3
NE 8TH AVE	715	22.6	2.4	0.2	2.39	21.5	24.1
NE 10TH AVE	593	24.0	7.6	0.1	1.92	16.8	24.6
NE 14TH AVE	1364	50.6	17.6	0.6	1.98	18.3	28.2
LAYNE BLVD	1366	40.6	9.0	0.4	2.40	22.9	29.5
GOLDEN ISLES	305	21.5	11.3	0.4	2.49	9.6	20.3
DIPLOMAT PKWY	490	15.7	2.2	0.0	1.89	21.2	24.6
THREE ISLES BLV	472	23.4	9.3	0.3	2.43	13.7	22.8
TOTAL	10597	455.0	180.7	4.4	2.26	15.8	26.3

```
CROSS                              SPEED(MPH)
STREET           ..5...10...15...20...25...30...35...40...45...50...55...60...65
NW 8TH AVE       |-+----+----+----+----+----+----+----+----+----+----+----+----|
                 |                   ..*..
                 |                         .*.
                 |                           .*.
                 |                             .*.
                 |                              .*.
                 |                              .*.
                 |                             .*.
                 |                            ..*..
                 |                            ..*..
                 |                           ..*..
                 |                        ...*...
                 |                     ....*....
                 |                       ....*....
                 |                       ....*....
NW 6TH AVE       |-+----+----+----+----+----+----+----+----+----+----+----+----|
                 |                       ...*...
                 |                         ...*...
                 |                           ..*..
                 |                           ..*...
                 |                         ...*...
                 |                        ...*...
NW 4TH AVE       |-+----+----+----+----+----+----+----+----+----+----+----+----|
                 |                   .....*.....
                 |                        ...*...
                 |                       ....*....
                 |                         ...*...
                 |                         ...*...

                        Part of the output intentionally omitted.
                 |
                 |               ....*....
GOLDEN ISLES     |-+----+----+----+----+----+----+----+----+----+----+----+----|
                 |                  ...*...
                 |                    ...*...
                 |                    ...*...
                 |                     ...*...
                 |                   ....*....
DIPLOMAT PKWY    |-+----+----+----+----+----+----+----+----+----+----+----+----|
                 |                     ...*...
                 |                    ...*...
                 |                  ....*....
                 |                 ....*....
THREE ISLES BLVD |-+----+----+----+----+----+----+----+----+----+----+----+----|
                                 * : Average speed      .... : 95% confidence interval
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FIGURE 4 Sample MVRAP output summary of travel time study sample.

on assumed values for turning movements. It was also suggested that deficiencies in the computational methodology could be at fault.

As a first-level screening technique, the proposed v/c limit of 1.2 was applied to the data. This reduced the travel time discrepancy to 43 percent for the 1985 HCM model and 37 percent for the 1994 model. Although neither of these results could be considered satisfactory, they do set the stage for the development of adjustment factors that could reconcile the discrepancy. The difference between the 1985 and 1994 computational methods is relatively small. Since the 1994 method has been approved for use in the HCM, it is a logical choice over the now obsolete 1985 method.

In the rest of the analyses described in this paper, the 1994 HCM Chapter 11 model will be used and an upper limit of 1.2 will be imposed on all v/c ratios. The results associated with these conditions will be referred to as the base values.

DEVELOPMENT OF CANDIDATE ADJUSTMENT FACTORS

Each of the sources of bias described previously must be addressed independently in the development of adjustments. The goal of this exercise is to improve the travel time estimation procedures within the existing structure of the HCM model. Departures will be proposed only when they can be shown to produce worthwhile improvements in accuracy and when they can be reconciled with the existing model.

EB PEAK DIRECTION RESULTS

EB LINK	THRU FLOW RATE	V/C RATIO	THRU DELAY	THRU APPROACH LOS	SPEED (MPH)	ARTERIAL LINK LOS	
1-2	1749	0.39	2.7	A	30.7	B	
2-3	1749	0.34	0.7	A	29.4	B	
3-4	1749	1.07	102.9	F	5.9	F	*Warning
4-5	1749	1.04	56.1	E	11.4	F	*Warning
5-6	1749	0.42	3.5	A	23.9	C	
6-7	1749	0.44	4.6	A	22.0	D	
7-8	1749	0.44	6.5	B	25.1	C	
8-9	1749	0.39	3.4	A	28.5	B	
9-10	1749	0.46	8.5	B	12.0	F	
10-11	1749	0.41	5.3	B	19.4	D	
11-12	1749	0.39	1.8	A	25.7	C	

EB Arterial Speed = *** mph
LOS = F
NOTE: Intersection Capacity Exceeded

WB OFF-PEAK DIRECTION RESULTS

WB LINK	THRU FLOW RATE	V/C RATIO	THRU DELAY	THRU APPROACH LOS	SPEED (MPH)	ARTERIAL LINK LOS
12-11	1,614	0.38	7.0	B	16.9	E
11-10	1,614	0.43	11.0	B	13.9	E
10-9	1,614	0.36	4.5	A	17.0	E
9-8	1,614	0.40	8.5	B	23.7	C
8-7	1,614	0.40	8.4	B	23.5	C
7-6	1,614	0.39	6.4	B	19.8	D
6-5	1,614	0.96	52.8	E	5.6	F
5-4	1,614	0.99	90.5	F	8.0	F
4-3	1,614	0.32	1.0	A	32.1	B
3-2	1,614	0.36	3.5	A	24.1	C
2-1	1,614	0.60	28.4	D	15.3	E

WB Arterial Speed = 14.4 mph
LOS = E

FIGURE 5 Sample ART-PLAN output summary.

Incremental Delay Adjustment

It has already been pointed out that when one intersection effectively controls the arrival of vehicles at the next intersection downstream, it is not appropriate to apply the full incremental delay term to the second intersection. Consider, for example, the hypothetical case in which the second intersection is located only a few meters downstream of the first. In this case, each vehicle leaving the first intersection would arrive more or less immediately at the second, and there would be no random component. Oversaturation of the second intersection would also be impossible, because all the excess demand would be absorbed at the first intersection. In this extreme case, no incremental term should be applied.

Now, as the distance between the intersections is increased, the random element in the arrival pattern will reappear. At some point the influence of the first intersection will be eliminated and the full incremental delay will apply. For a given classification of arterial the proportion of the incremental delay that should be applied is clearly dependent on the signal spacing.

Lacking any theoretical basis to describe this effect, a very simplistic model was proposed and tested as a candidate adjustment factor. The proportion of the incremental delay term to be applied at an intersection on a coordinated arterial route was assumed to be directly proportional to the distance between signals. The full value of the incremental term was applied when the distance reached a specified threshold. The threshold values, based somewhat on judgment, were established at 0.81 km (0.5 mi) for Classification I routes and 0.4 km (0.25 mi) for Classification II routes. So the incremental term was multiplied by a factor of

$$\text{Min}\left(\frac{\text{segment length}}{\text{reference length}}, 1.0\right) \qquad (1)$$

This reduced the estimated delay at signals with short spacing. The overall effect on the data collected for this study was a reduction of the overestimate of travel time from 37 to 27 percent.

Floating Car Advantage

Another suggested source of bias was the advantage given to the travel time study vehicle as compared with the "average" vehicle because of its position in the progression band. In a properly timed arterial system, the test vehicle, or floating car, usually arrives on the green signal. This does not mean that there will be no delay to the test vehicle, because often it will be impeded by a queue of vehicles that are still waiting to be serviced. However, the ratio of uniform delay to vehicles arriving on the green as compared with all vehicles arriving during the cycle should be a good indicator of the extent of the floating car advantage. This ratio is therefore proposed as a candidate adjustment factor.

Consider an approach to a signalized intersection. Let

g = length of green phase (sec)
C = cycle length (sec)
L = green ratio (g/C)
q = average arrival rate over whole cycle (veh/sec)
s = average steady-state departure rate on green phase (veh/sec)
q_g = average arrival rate on green phase (veh/sec)
q_r = average arrival rate on red phase (veh/sec)
P = proportion of arrivals on green phase
R_p = platoon ratio = P/L
A_g = total arrivals on green (veh/cycle)
A_r = total arrivals on red (veh/cycle)
X = degree of saturation = $q/(Ls)$
$d1$ = average uniform delay to all vehicles on approach = $0.38C(1 - L)^2/(1 - LX)$ by HCM delay equation
$d1_g$ = average uniform delay to all vehicles arriving on green phase
F_{FC} = floating car advantage factor = $d1_g/d1$

Then,

$$A_g = qCP = qCLR_p \tag{2}$$

$$q_g = A_g/LC = qR_p \tag{3}$$

$$A_r = qC(1 - P) = qC(1 - LR_p) \tag{4}$$

$$q_r = qC(1 - LR_p)/C(1 - L)$$
$$= q(1 - LR_p)/(1 - L) \tag{5}$$

Referring to Figure 6, the area of the triangle ABC represents the total delay to all vehicles arriving over the entire cycle. The smaller area, A′B′C represents the total delay to vehicles arriving on the green phase only, as a subset of the total delay.

To obtain average delay values, the total delay values given by the areas of the respective triangles must be divided by the number of vehicles represented per cycle, that is,

$$d1 = \text{Area ABC}/qC \tag{6}$$

and

$$d1_g = \text{Area A'B'C}/qCLR_p \tag{7}$$

Since the concern is the ratio of $d1{:}d1_g$, the factor of 0.38 will be dropped from the HCM equation. This factor is used to convert total delay to stopped delay and will be the same for both $d1$ and $d1_g$.

Referring again to Figure 6, the queue at the end of the red phase (QEOR) may be computed as

$$\text{QEOR} = qR_pC(1 - L) \tag{8}$$

$$\text{QEOR} = qrC(1 - L) = \frac{qC(1 - LR_p)(1 - L)}{(1 - L)}$$
$$= qC(1 - LR_p) \text{ veh} \tag{9}$$

The time required to clear the arrivals on red (GQR) will be

$$\text{GQR} = \frac{\text{QEOR}}{S} = \frac{qC(1 - LR_p)}{\frac{q}{LX}} = CLX(1 - LR_p) \text{ sec} \tag{10}$$

Now, during the period GQR, the arrivals on green will accumulate at a rate of q_g veh/sec. The time GQA required to clear all of the vehicles, including those that arrived on the red and the queued portion of the green, will be

$$\text{GQA} = \frac{\text{QEOR}}{s - q_g} = \frac{qC(1 - LR_p)}{\frac{q}{LX} - qR_p} = \frac{qC(1 - LR_p)}{\frac{q(1 - LR_pX)}{LX}}$$
$$= \frac{CLX(1 - LR_p)}{1 - LR_pX} \text{ sec} \tag{11}$$

Now the area of the triangle A′B′C may be computed as

$$0.5\, q_g\, \text{GQR GQA} = \frac{0.5\, q_g\, CLX(1 - LR_p) \cdot CLX(1 - LR_p)}{1 - LR_pX} \quad \text{veh-sec/cycle} \tag{12}$$

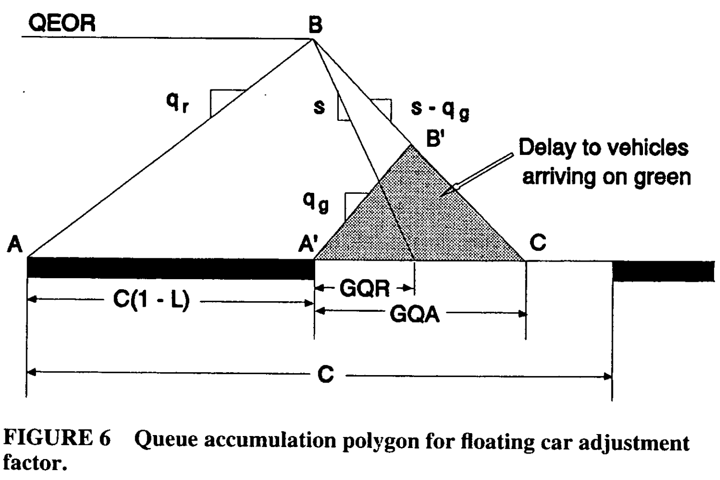

FIGURE 6 Queue accumulation polygon for floating car adjustment factor.

To determine a unit delay (sec/veh) the number of vehicles per cycle arriving on the green must be computed by CLq_g, and the uniform delay per vehicle arriving on the green becomes

$$dl_g = \frac{0.5q_g\cancel{CL}X(1 - LR_p)CLX(1 - LR_p)}{\cancel{CL}q_g(1 - LR_pX)}$$

$$dl_g = \frac{0.5CLX^2(1 - LR_p)^2}{1 - LR_pX} \tag{13}$$

The floating car advantage factor, F_{FC}, may be computed as

$$\frac{dl_g}{dl} = \frac{\dfrac{0.5CLX^2(1 - LR_p)^2}{1 - LR_pX}}{\dfrac{0.5C(1 - L)^2}{1 - LX} \cdot \dfrac{(1 - LR_p)}{(1 - L)}}$$

$$= \frac{LX^2(1 - LR_p)(1 - LX)}{(1 - LR_pX)(1 - L)} \tag{14}$$

for the special case of random arrivals (i.e., $R_p = 1$), this equation simplifies to

$$\frac{dl_g}{dl} = \frac{LX^2(1 - L)(1 - LX)}{(1 - LX)(1 - L)} = LX^2 \tag{15}$$

It should be appropriate to apply this factor to the uniform delay as long as there is some discernable progression. By definition, there is no discernable progression with Arrival Type 1. So, as a matter of judgment, no floating car adjustment was applied to those cases in the study for which Type 1 was indicated. The adjustment was applied for all other arrival types, which reduced the discrepancy between the estimated and measured travel times to a negligible level.

STUDY RESULTS

To summarize the preceding discussion,

1. The measured and estimated (HCM Table 11-4) *running times* agreed surprisingly well without further adjustment. Virtually all of the discrepancy in *travel times* was in the intersection delay values.
2. The imposition of an upper limit of 1.2 on the v/c ratio was necessary as a first-level filter to eliminate gross discrepancies between the measured and estimated delays. This created a base condition with a 37 percent overestimation of travel time.
3. The application of the incremental delay reduction factor for closely spaced intersections reduced the travel time overestimate to 27 percent.
4. The additional application of the floating car adjustment factor to the uniform delay term effectively eliminated the discrepancy between measured and estimated travel times.

These results are presented graphically in Figure 7 and Table 3. Figure 7 also shows the degree of overestimation of the intersection delay in addition to the travel times. A breakdown of the estimation error for the fully adjusted results is also provided by FDOT district (Districts 4 and 6) and by HCM arterial classification (Classes I and II). Both of these breakdowns indicate minimal errors for any of the categories. This lends additional credibility to the validity of the results.

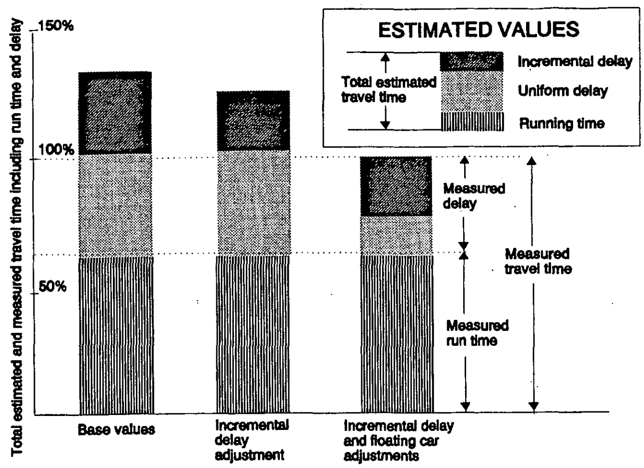

FIGURE 7 Comparison of measured and estimated delays.

CONCLUSIONS AND RECOMMENDATIONS

The results offer a reasonable explanation of the apparent discrepancies between estimated and measured travel times and delays on arterial streets. They also provide a practical means of adjusting the estimated delay values to produce a very close agreement with the corresponding measurements.

These results will be most useful for planning level analyses. The adjustment factors appear to eliminate the bias from the travel time and delay estimation models, but there is a substantial error and variability in the comparison of several of the individual runs. More accurate values would be required for the field data to be used in the computational models if the results were intended for operational analysis purposes. The findings of this study should be implemented as follows:

1. The travel time data collection program should be modified to compute the running speed in a manner compatible with the HCM. The stopped delay should be multiplied by the HCM factor of 1.3 before being subtracted from the total travel time.
2. An incremental delay reduction factor should be considered for the HCM Chapter 9 and 11 methodology. This modification should not, however, be used to justify the operation of an arterial route beyond its capacity.

TABLE 3 Summary of Results

	Error (%)	
	Total Travel Time	Intersection Delay
Base data	37/O	101/O
Incremental delay adjustment	28/O	77/O
Uniform and incremental delay adjustments	0	1/O
District 4	4.1/O	—
District 6	1.4/U	—
Classification I	2.2/O	—
Classification II	3.2/U	—

NOTE: O = overestimate, U = underestimate.

3. The uniform delay reduction factor should be incorporated as a supplementary ART_PLAN output, labeled specifically as the estimated speed or travel time for a floating car study.

The data should be analyzed further to explore alternative models that could reduce the variability of the estimates and produce better agreement between estimated and measured travel time and delay for individual runs.

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