Evaluation of Delay Models for Motor Vehicles at Light Rail Crossings

RICHARD A. BERRY AND JAMES C. WILLIAMS

The application of theoretically and empirically based delay equations, developed for isolated traffic signal operation, to the problem of isolated at-grade light rail crossings was evaluated. Data were collected at 24 crossings of five transit authorities to validate a model for average stopped delay from among 21 candidates. The successively validated model was the pretimed delay equation from the 1985 *Highway Capacity Manual*. When a background cycle is imposed by traffic signals controlling the crossing, the delay model developed by Allsop was found to be suitable when applied with its lower limit parameters.

Because light rail transit (LRT) has minimal right-of-way requirements and, for the most part, crosses streets and highways at-grade, it is much less costly to build than conventional heavy rail transit, with its grade-separated operation. However, when streets are crossed at-grade, the quality of traffic service provided within the street network decreases because of an increase in stops and delay to motor vehicle traffic.

For most recently constructed LRT systems, the decision to operate at-grade has been a policy decision due, in part, to economics. In addition, there is a widespread perception among transit planners that at-grade crossing capacity should be based on the potential person-capacity of the crossing, thus inherently favoring unobstructed right-of-way for the light rail vehicle. Because there are currently no accepted methods to measure the traffic impact of at-grade light rail crossings, each transit property is required to develop its own approach, often without the benefit of the experience of other transit properties.

Little research has been done to objectively measure the magnitude of the traffic impacts. Research has been concentrated on the application of techniques developed for the evaluation of signalized street intersections (*1–3*), but has generally been unable to support the validity of their use at LRT crossings with field data, simulation, or statistical analysis (*1*,*3*,*4*).

In this work, theoretically and empirically based delay equations developed for isolated traffic signals are evaluated and compared with field data collected from five LRT systems currently operating in the United States.

OPERATIONAL OVERVIEW OF LIGHT RAIL TRANSIT

The degree of interaction between the light rail vehicles and motor vehicle traffic (and, thus, their mutual impacts) is briefly discussed in this section. Four general operational aspects of light rail transit affecting this interaction are

- Operating philosophy of the transit authority,
- Location of the at-grade crossing with respect to nearby signalized intersections,
- LRT scheduling, and
- Traffic control devices at the crossing.

Operating Philosophy

The operating philosophy of transit authorities for at-grade crossings can vary from requiring the trains to follow the rules of the road and wait for gaps in the traffic to cross streets, to having unconditional preemptive authority to cross streets on demand. The particular operating philosophy of a transit authority also affects the other aspects.

Crossing Location

At-grade LRT crossings are either isolated from signalized intersections or are adjacent to or within signalized intersections. In the latter cases, the timing of the traffic signal must consider the light rail movement through the intersection, either by allowing the LRT vehicle to preempt the traffic signal or by allowing the LRT vehicle to move through the intersection on a particular phase. This work, however, is principally concerned with isolated crossings. If a crossing is sufficiently removed from a signalized intersection, traffic impacts are limited only to those associated with crossing capacity and delay. It must be recognized, however, that where LRT crossings are close to traffic signals, queues generated by the crossing may interfere with the intersection and vice versa. Simulation experiments by Cline et al. (*4*) found that traffic signals within about 400 ft of LRT crossings may experience this problem.

Schedule

In measuring delay at LRT crossings, the effect of the operating schedule must be considered. LRT schedules are typically constructed in 30-sec to 1-min increments and LRT vehicle operators are often considered "on time" if their arrival at a time point is within the schedule increment. This scheduling ensures a variance in the arrival time of light rail vehicles at crossings. Sources of the variance include

R. A. Berry, DeShazo, Starek and Tang, Inc., 2317 Colonial, Mesquite, Tex. 75150. J. C. Williams, Department of Civil Engineering, University of Texas, Arlington, Tex. 76019-0308.

- Variations in station dwell time caused by surges in boarding and alighting, and the occasional boarding and alighting of handicapped passengers;
- Traffic congestion when operations are in mixed traffic;
- Variable time headways (scheduled) caused by changing ridership throughout the day; and
- Individual train operator and vehicle characteristics.

Traffic Control Devices

Two types of traffic control devices are used at LRT crossings: passive (such as crossbucks or stop signs) and active (flashing lights, with or without gates, and standard traffic signals). Active traffic control devices allow moderate disruption to motor vehicle traffic while providing a higher level of protection to the alert driver.

LITERATURE REVIEW

Much of the previous work on the impact of LRT crossings on public streets has concentrated on whether at-grade crossing capacity is adequate. This work has concentrated on two major measures of effectiveness: the volume-to-capacity (v/c) ratio and delay.

When dealing with isolated LRT crossings (i.e., crossings not directly associated with a signalized intersection), the v/c ratio is seldom of much concern. An at-grade LRT crossing with active traffic control devices is essentially an intersection with two-phase traffic signal control. Even on LRT lines with relatively frequent service, the fraction of time the cross street is blocked is typically low when compared with the fraction of time the street is "blocked" at nearby signalized intersections by the red interval. (For the purposes of this work, the time when the motor vehicle traffic may not cross the LRT guideway is considered to be the blockage time.) Multiphase signalized intersections are much more likely than at-grade LRT crossings to control the overall capacity of a street (*5*). Therefore, most techniques using the v/c ratio have been developed for LRT crossings adjacent to, or within, intersections, where one or more of the timed phases may be controlled by the LRT vehicle. Often an adjustment of the v/c ratio to account for the LRT crossing has been used as a first step in estimating the resulting delay on the cross street.

Stone and Wild (*1*), using capacity techniques from the 1965 *Highway Capacity Manual* (HCM) (*6*) and the work of May and Pratt (*7*) and Crommelin (*8*), developed a regression equation relating the intersection utilization factor (v/c ratio) to individual vehicle delay.

The San Diego Association of Governments (*9*) also used the 1965 HCM (*6*) to assign levels of service to LRT crossings. Load factors were approximated using field-measured stopped delay at LRT crossings, and were then compared to the cycle length of adjacent traffic signals.

Gibson et al. (*10*) assumed that nonconflicting traffic movements would be allowed to move with the LRT vehicles during preemption, and that all intersection traffic must be stopped only during the clearance time immediately prior to preemption. This assumption typically resulted in increases to the v/c ratio of less than 5 percent.

A method developed for the San Diego Trolley Bayside Line (*11*) estimated the percentage of the traffic signal cycle used by light rail operations for each movement, then increased the affected movements appropriately. Level of service was then calculated, as described in Circular 212 (*12*), using critical movements.

Grote (*2*) also used a critical movement analysis (*12*) to estimate the v/c ratio of the crossing (assuming one of the critical movements to be LRT). Individual vehicle delay was estimated using Stone and Wild's regression equation (*1*).

Radwan and Hwang (*3*) estimated the gain of passenger-seconds through intersections containing LRT using a modified Webster's delay equation and a probabilistic procedure for estimating train arrivals at the LRT crossing.

The NETSIM traffic simulation model was used by Cline et al. (*4*) to evaluate delay and queuing for isolated LRT crossings and those within intersections. Regression equations predicting motor vehicle delays were formulated from the results of the simulation effort.

EXPERIMENTAL DESIGN

Field studies were performed to collect empirical data from a range of transit properties in order to validate the measures of effectiveness proposed for traffic impact evaluation. Although this step is crucial to evaluating alternative methods of estimating the traffic impacts of at-grade LRT crossings, it has been omitted in most previous studies. Only Cline et al. (*4*) attempted validation, and then by using simulation methods, which, although using typical values for signalized intersections, were not validated for light rail applications.

Data Identification

Two sources were used to identify what traffic, roadway, and light rail data should be collected. The 1985 HCM (*13*) contains a detailed list of traffic and roadway factors that affect roadway capacity and traffic delay estimates. Gerlough and Huber (*14*) review a number of well-known models for estimating the motor vehicle delay at signalized intersections. The components of these models provided additional guidance on the selection of appropriate traffic and light rail data. Specific factors targeted for collection included

- Area type,
- Number of lanes,
- Lane widths,
- Grades,
- Volumes,
- Arrival type,
- Saturation flow rates,
- Cycle length,
- Effective green times, and
- Effective red times.

Site Selection

Principal site selection factors included

- The ability to collect traffic impact data at isolated LRT crossings (i.e., crossing without parallel movements of motor vehicle traffic); and
- The need to obtain data from a range of roadway, traffic, and light rail conditions.

Five transit authorities with isolated LRT crossings were selected. Two other transit authorities with only median or adjacent running alignments were visited.

The 14 crossings studied included a range of traffic control strategies, traffic volumes, approach lanes, light rail headways, train lengths, and light rail control criteria. The transit properties themselves included older properties with substantial operating experience and newer properties which had only recently begun rail operations.

Candidate Transit Properties

Seven transit properties were visited during the course of the two field studies. The seven transit properties were

1. Port Authority Transit (PAT) of Allegheny County, serving the Pittsburgh, Pennsylvania, metropolitan area;
2. Niagara Frontier Transportation Authority (NFTA), serving Buffalo, New York;
3. Greater Cleveland Regional Transportation Authority (GCRTA), serving the Cleveland, Ohio, metropolitan area;
4. San Francisco Municipal Railway (Muni), serving the San Francisco, California, metropolitan area;
5. Santa Clara County Transportation Agency (SCCTA), serving the San Jose, California, metropolitan area;
6. Sacramento Regional Transportation District (SRTD), serving Sacramento, California; and
7. San Diego Trolley, Inc. (SDTI), serving the San Diego, California, metropolitan area.

Data on the traffic impacts of isolated LRT crossings were successfully collected for every authority except GCRTA and SCCTA. These two authorities did not have any isolated LRT crossings.

Data Collection Procedures

A simple data collection procedure was used at each transit property visited. First, each at-grade LRT crossing was inspected to determine which crossings were suitable for the study. Criteria included traffic volumes, number of lanes, location and types of nearby traffic control devices, sight lines for data collection, and crossing warning devices. Second, a meeting was held with personnel from the transit property to select crossings and discuss light rail operations. Next, a street intersection in the vicinity of the crossing was selected for collecting saturation flow rate data typical of the area. Finally, the actual collection of the field data was performed both at the LRT crossings and at the street intersections. Upon completing collection of the field data, each site was measured and check lists were reviewed to ensure that all necessary data had been collected.

Actual data collection of at-grade LRT crossing operations in the field was performed via videotape of the selected LRT crossings. In total, approximately 53 hr of videotape were made of LRT crossing operations.

Saturation flow rate data were collected at signalized intersections near each crossing to serve as control data for typical traffic operations in the area. Intersections were chosen where the traffic characteristics were judged to be similar based on the cross-section of the intersection approach, the traffic volume, the traffic speed, and the intersection environment.

Data Analysis

Initial data analysis was performed by viewing the videotapes and manually removing information from them. The video camera's clock superimposed on each tape provided a time base for the data. Data collected from the videotape included

1. Traffic volumes in 15-sec and 1-min increments,
2. Train volumes,
3. Train headways,
4. Crossing protection equipment operating times,
5. Number of motor vehicle stops,
6. Individual motor vehicle stopped delay,
7. First car lost times,
8. Saturation flow rates, and
9. Queue lengths.

Further analysis of the data obtained from the videotapes was performed with the goal of estimating

1. Arrival distribution of the motor vehicle traffic,
2. Distribution of train headways,
3. Effective blockage time, or red time, incurred at the crossing,
4. Crossing's effective green time to cycle length ratio,
5. Crossing's capacity,
6. Crossing's *v/c* ratio,
7. Percent stops for the motor vehicle traffic,
8. Average individual stopped delay, and
9. Mean, 85th percentile, and 95th percentile queue lengths.

Many of the findings of the field studies have been reported by Berry and Williams (*15*).

During the planning of the field studies, three methods for estimating delay at signalized intersections were investigated. The first might be described as an input-output method. For this method, the elapsed travel time over a length of roadway is measured for each vehicle. The approach delay is the difference between the "normal" travel time and the travel time where delay due to LRT vehicle crossings is incurred. This method was rejected because of the difficulty in videotaping the necessary length of roadway and the need to define a "normal" travel time.

The second method investigated is widely used for intersection delay studies and results in an estimate of stopped delay. Described by Reilly et al. (*16*) and the 1985 HCM (*13*), this method used counts of stopped vehicles taken at regular intervals, such as 10 or 15 sec. The sum of the number of stopped vehicles is then multiplied by the interval between the stopped vehicle counts and divided by the total volume during the study period. The one drawback is that the time interval should not be an integer factor of the traffic signal cycle length. This method was rejected because that constraint could not be ensured.

The third method investigated was used in this project. It consists of tracking the trajectories of individual vehicles on their approach to the crossing and noting the total time, if any, that they are stopped. Although this method of estimating stopped delay is more labor intensive than the other two, it should provide the most accurate and precise estimate of crossing delay to motor vehicles.

SUMMARY OF FINDINGS

Estimates of the average individual stopped delay for the different LRT crossing studies are shown in Tables 1 through 5. These estimates were calculated by summing the total amount of stopped delay observed over each discrete time period (one signal cycle or 5 min, depending on the method of crossing control; 15 min; or 1 hr) and dividing by the traffic volume observed during that time period.

The study sites have been classified by the type and operation of control at the crossing, and are shown in separate tables. Crossings with no control or flashing light units are listed in Table 1; those with flashing light units and gates are listed in Table 2. Two crossings were controlled by standard traffic signals which alternated right-of-way between the cross street and the LRT; there was no street parallel with the LRT. These signals operated on a background cycle and would periodically stop vehicles when no train was crossing. Furthermore, the LRT vehicle preempted signal operation upon its approach to the crossing. Therefore, delay to motor vehicles at the crossings can be separately tabulated: delay when stopped for LRT crossings (Table 3) and delay when stopped due to the normal cycling of the traffic signal (Table 4). Total delay (summing delay under both conditions) at these two crossings is shown in Table 5.

TABLE 1 LIGHT RAIL-RELATED STOPPED DELAY—NO CONTROL AND FLASHING LIGHT UNIT CONTROL

Site	Cross Street	Peak Hour Volume (vph)	No. of Peak Hour Trains (tph)	Crossing v/c Ratio	Stops on Approach (%)	Individual Stopped Delay: Range for 5 min period (sec/veh)	Individual Stopped Delay: Range for 15 min period (sec/veh)	Individual Stopped Delay: Hourly Average (sec/veh)
	Ocean Avenue							
1	AM Obs	261	16	0.22	4.8	0.0-1.4	0.0-0.9	0.3
2	PM Obs	262	12	0.29	8.0	0.0-2.3	0.0-1.4	0.6
	Potomac Avenue							
3	AM Obs	417	28	0.47	51.5	0.0-19.9	1.6-14.2	11.8
4	PM Obs	270	26	0.25	32.3	0.0-27.6	2.1-13.8	8.1
	Mt. Lebanon Blvd							
5	AM Obs	561	22	0.38	11.2	0.0-3.5	0.4-2.0	1.0
6	PM Obs	614	41	0.45	22.6	0.0-14.4	0.4-5.6	2.6

TABLE 2 LIGHT RAIL-RELATED STOPPED DELAY—FLASHING LIGHT UNITS WITH GATES

Site	Cross Street	Peak Hour Volume (vph)	No. of Peak Hour Trains (tph)	Crossing v/c Ratio	Stops on Approach (%)	Individual Stopped Delay: Range for 5 min period (sec/veh)	Individual Stopped Delay: Range for 15 min period (sec/veh)	Individual Stopped Delay: Hourly Average (sec/veh)
	65th Street							
7	AM Obs	1115	8	0.42	9.7	0.0-13.7	1.4-5.4	2.6
8	PM Obs	1054	8	0.38	9.2	0.0-19.8	0.3-12.5	4.1
	Alhambra Blvd							
9	AM Obs	379	8	0.28	9.1	0.0-5.1	0.1-3.0	1.3
10	PM Obs	456	8	0.36	12.3	0.0-11.2	0.0-4.7	2.7
	Alhambra Blvd							
11	AM Obs	290	8	0.25	8.7	0.0-7.4	0.1-3.2	1.5
	H Street							
12	AM Obs	616	8	0.25	13.0	0.0-23.1	1.9-8.7	4.6
	Dairy Mart Road							
13	AM Obs	320	8	0.07	8.1	0.0-6.7	0.7-2.0	1.6
14	PM Obs	425	8	0.09	10.9	0.0-11.3	0.5-6.4	2.8

TABLE 3 LIGHT RAIL-RELATED STOPPED DELAY—TRAFFIC SIGNAL CONTROL

Site	Cross Street	Peak Hour Volume (vph)	No. of Peak Hour Trains (tph)*	Crossing v/c Ratio	Stops on Approach (%)	Individual Stopped Delay		
						Range for one cycle (sec/veh)	Range for 15 min period (sec/veh)	Hourly Average (sec/veh)
	Church Street							
15	AM Obs	381	11/9	0.12	34.9	6.7-43.9	12.5-19.0	14.0
16	PM Obs	672	10/9	0.14	31.3	8.0-37.4	18.8-28.8	24.6
	Chippewa Street							
17	AM Obs	793	10/9	0.33	33.7	0.9-39.5	10.6-25.3	15.4
18	PM Obs	504	9/10	0.22	25.6	2.0-32.5	6.5-13.8	9.0
	Church Street							
19	AM Obs	776	11/9	0.17	22.8	0.0-24.8	6.0-9.3	8.0
20	PM Obs	721	9/9	0.22	28.4	2.8-24.7	7.2-12.1	8.3

* Inbound/Outbound

TABLE 4 BACKGROUND CYCLE STOPPED DELAY—TRAFFIC SIGNAL CONTROL

Site	Cross Street	Peak Hour Volume (vph)	No. of Peak Hour Trains (tph)	Crossing v/c Ratio	Stops on Approach (%)	Individual Stopped Delay		
						Range for one cycle (sec/veh)	Range for 15 min period (sec/veh)	Hourly Average (sec/veh)
	Church Street							
15	AM Obs	208	29	0.12	60.6	4.0-25.8	13.2-17.6	14.4
16	PM Obs	378	27	0.24	73.5	3.1-29.1	13.9-19.0	16.2
	Chippewa Street							
17	AM Obs	448	38	0.30	70.8	0.0-29.0	7.6-13.8	10.1
18	PM Obs	326	41	0.23	52.1	0.0-23.0	6.2-7.4	6.8
	Church Street							
19	AM Obs	478	43	0.16	56.3	0.0-22.0	6.3-9.7	7.4
20	PM Obs	432	40	0.16	48.6	1.1-22.7	5.4-8.4	6.7

TABLE 5 COMPOSITE STOPPED DELAY—TRAFFIC SIGNAL CONTROL

Site	Cross Street	Peak Hour Volume (vph)	No. of Peak Hour Trains (tph)	Crossing v/c Ratio	Stops on Approach (%)	Individual Stopped Delay		
						Range for one cycle (sec/veh)	Range for 15 min period (sec/veh)	Hourly Average (sec/veh)
	Church Street							
15	AM Obs	381	20	0.56	68.0	4.0-46.4	18.6-22.5	20.9
16	PM Obs	672	19	0.28	72.6	5.5-37.4	16.5-22.4	19.7
	Chippewa Street							
17	AM Obs	793	19	0.54	73.6	0.0-50.8	14.6-27.5	20.4
18	PM Obs	504	19	0.38	59.3	0.1-32.5	10.9-16.2	13.0
	Church Street							
19	AM Obs	776	20	0.26		0.2-54.3	10.1-14.7	12.7
20	PM Obs	721	18	0.40	57.6	N.A.	N.A.	N.A.

Crossings that are listed twice (Alhambra Boulevard in Table 2 and Church Street in Tables 3–5) indicate observations made over 2 days, and are shown separately for each day.

PERFORMANCE OF DELAY MODELS

Part of the focus of this project was to determine if any of the many existing delay models developed for the analysis of street intersections was suitable for estimating the delays to motor vehicle traffic caused by isolated at-grade LRT crossings. This project did not have as a goal the development of a new delay model of either theoretical or empirical form. Eleven different delay models were applied during the validation process. These models reflect a broad range of the theoretical and empirical aspects of intersection delay. Each modeler's original work contains a detailed discussion of its construction.

Because the analysis of the field data resulted in calculated average individual vehicular stopped delay, the values of models that estimate approach delay also have been adjusted to reflect average stopped delay. As stated above, direct measurement of approach delay was rejected because of the difficulties of videotaping the necessary length of roadway and defining normal, undelayed travel times. We recognize that the relationship between approach delay and stopped delay is not a static factor. However, little guidance can be found in the literature on the dynamic relationship between approach and stopped delay. The magnitude of the adjustment of approach delay is provided by Reilly et al. (*16*) and the 1985 HCM (*13*). They recommend multiplying values for individual average stopped delay by a static factor of 1.3 in order to estimate the individual average approach delay. Sadegh and Radwan (*17*) support this factor in their study of the 1985 HCM delay model. In comparing it with Webster's model (*18*), they note that the first term of each, the uniform delay term, is similar in most respects. Webster's model estimates average individual approach delay, and the 1985 HCM model estimates average individual stopped delay. The primary difference between them is found in the coefficients. The coefficient of the 1985 HCM model is 76 percent of the coefficient of Webster's model. This translates into a factor of approximately 1.32. The adjustment factor used in this effort is

$$\text{Avg. ind. stopped delay} = 0.76 * (\text{avg. ind. approach delay})$$

The delay models included in the validation process are listed below. Except as noted, the delay models included in the validation are taken from Gerlough and Huber (*14*).

- May;
- Allsop, both lower and upper limits;
- Wardrop;
- Webster;
- Allsop's approximation of Webster's model;
- Miller;
- Hutchinson;
- Texas Transportation Institute (*4*), with and without the modified intercept term;
- Stone and Wild (*1*);
- 1985 HCM (*13*), using both the pretimed (A) and actuated (B) progression factors; and
- NCHRP Project 3-28(2) (*19*), both uniform and overflow delay equations.

The May, Allsop, Wardrop, and NCHRP uniform delay models all assume uniform vehicle arrivals in the intersection. Webster, Allsop's approximation of Webster, Miller, Hutchinson, and the HCM model all contain terms to estimate the delay caused by random vehicle arrivals in addition to uniform arrivals. The two Texas Transportation Institute models and the Stone and Wild model are the result of regression analyses correlating the relationship between the v/c ratio of an intersection approach and the delay expected on that approach.

In addition to the foregoing models, the following six model fragments were also included:

- Uniform arrival component of the Webster model,
- Random arrival component of the Webster model,
- Uniform arrival component of the HCM model,
- Random arrival component of the HCM model,
- Uniform arrival component of the Hutchinson model, and
- Random arrival component of the Hutchinson model.

These variants comprise the individual components of 3 of the 11 primary models. They were included without regard to the assumptions and boundary conditions associated with their parent models. Two factors influenced the decision to include these variants. First, except for the two Texas Transportation Institute models, the application of the at-grade LRT crossing problem to each of the other nine delay models selected is outside the bounds used when they were validated. Second, it was not known if any of the selected models would be successfully correlated with the field data. Hence, it was felt that the examination of the delay models should include as many as possible. It must be recognized, however, that the random components of the three delay models may be inappropriate because they estimate the delay over and above the uniform delay. Consequently, they should be applicable only in conjunction with their uniform delay components and where the v/c ratio approaches one. This is an unlikely case, however, because a LRT crossing, with only two phases, will not, in most cases, have a high v/c ratio. For equal approach conditions, the crossing capacity will normally exceed the capacity of up- and downstream signalized intersections. The data reflect this; hence, the models were tested at low v/c ratio ranges.

All of the models except the 1985 HCM and NCHRP models estimate the average individual vehicular delay. The 1985 HCM and NCHRP models estimate average individual vehicular stopped delay. The estimates of the other models have been reduced by a factor of 0.76 to account for the difference between approach delay and stopped delay.

MODEL EVALUATION

Using 15-min analysis periods, the data collection effort resulted in four to eight data points per site. Because of the small number of data points, rigorous statistical testing has not been performed. However, it is still possible to draw general conclusions about the appropriateness of applying each model from the trends it exhibits.

Three indicators have been used to rank the performance of the delay models:

1. The coefficient of determination (R^2) resulting from the observed and predicted values;
2. The mean difference between the observed and predicted values; and
3. The variance of the difference between the observed and predicted values.

Figures 1, 2, and 3 list the five best models for each site as determined by indicators one, two, and three, respectively. The site numbers are defined in Tables 1–5, and the delay model symbols are defined in Table 6. The information in these figures for the two sites controlled by standard traffic signals (Church and Chippewa streets) included delay accrued only when traffic is stopped for light rail vehicle crossings. As mentioned, these signals operated on a background cycle and would stop motor vehicle traffic periodically when no light rail vehicle was crossing, thus creating additional delay. The results using total, or composite, delay are shown in Figures 4, 5, and 6.

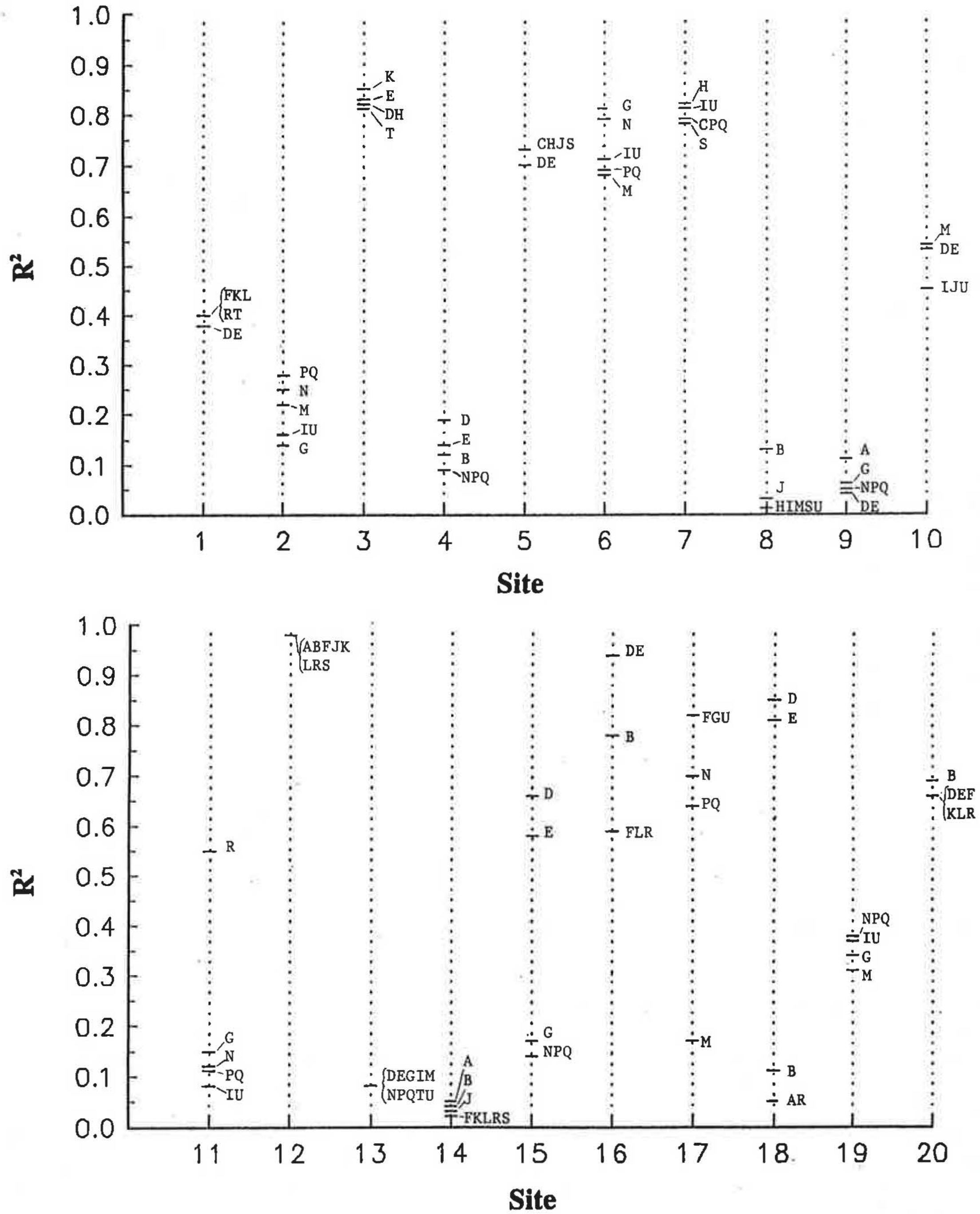

FIGURE 1 Regression analysis.

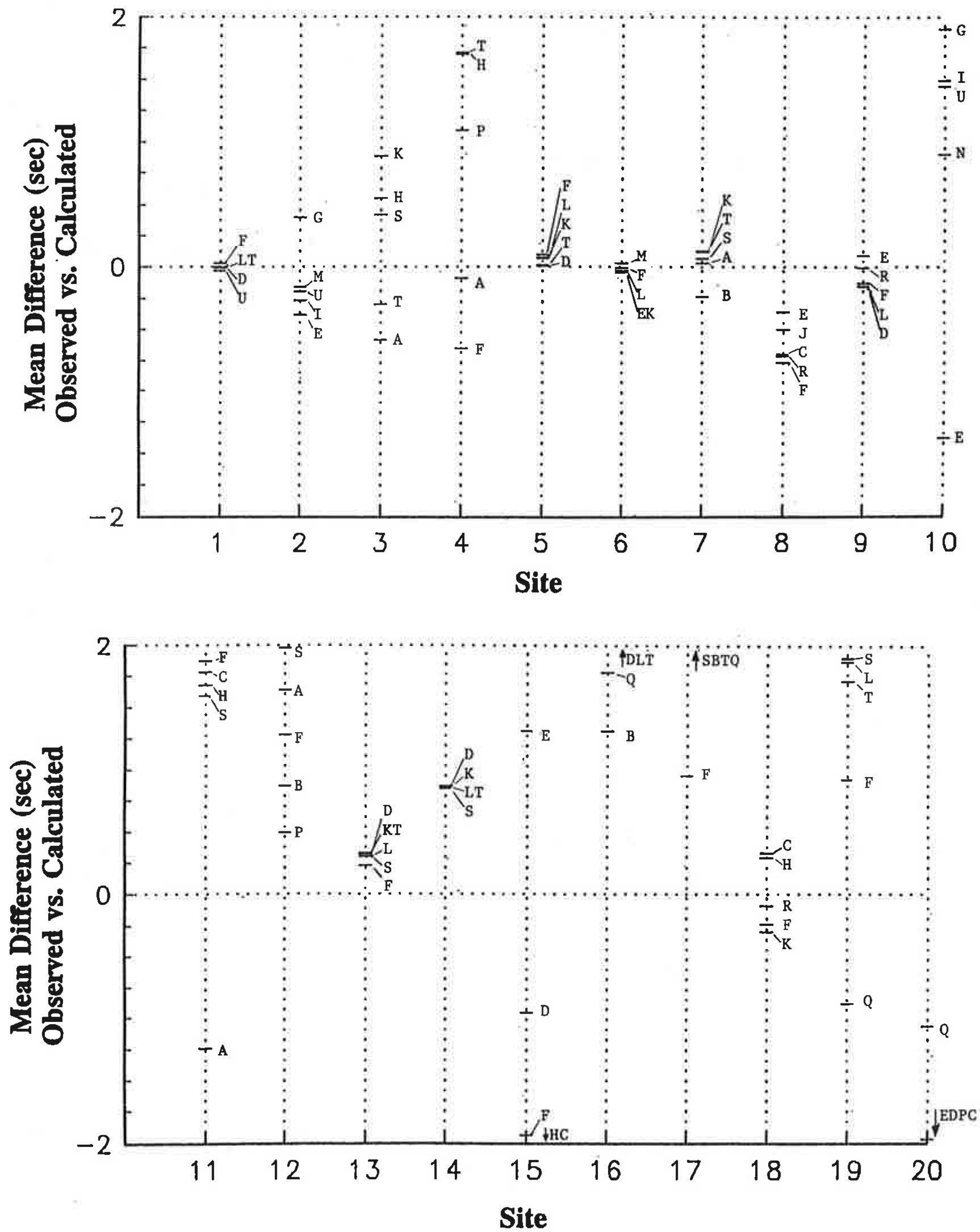

FIGURE 2 Mean difference between observed and calculated delay times.

For the purpose of overall model selection, a ranking procedure was used for each indicator. The best equation was given a score of 22, and the worst equation was given a score of one. Limits were placed on the rankings, and values outside the limits were given a score of zero. For the regression analysis, a value of zero was assigned if the R^2 value was found to be less than 0.50. For the analysis of the mean and standard deviation of the differences, a value of zero was assigned if the mean or standard deviation exceeded 5.0 sec. Five seconds represents the difference between Levels of Service (LOS) A and B and one half of the difference between LOS B and C in the 1985 HCM. It is an appropriate range in light of the low delays and resultant levels of service observed during the field studies.

As evident in Table 6, a wide range of R^2 values results from the regression analysis. The morning observation at H Street on the SDTI South Line has the best series of R^2 values; the evening observation at 65th Street on the SRTD Butter-

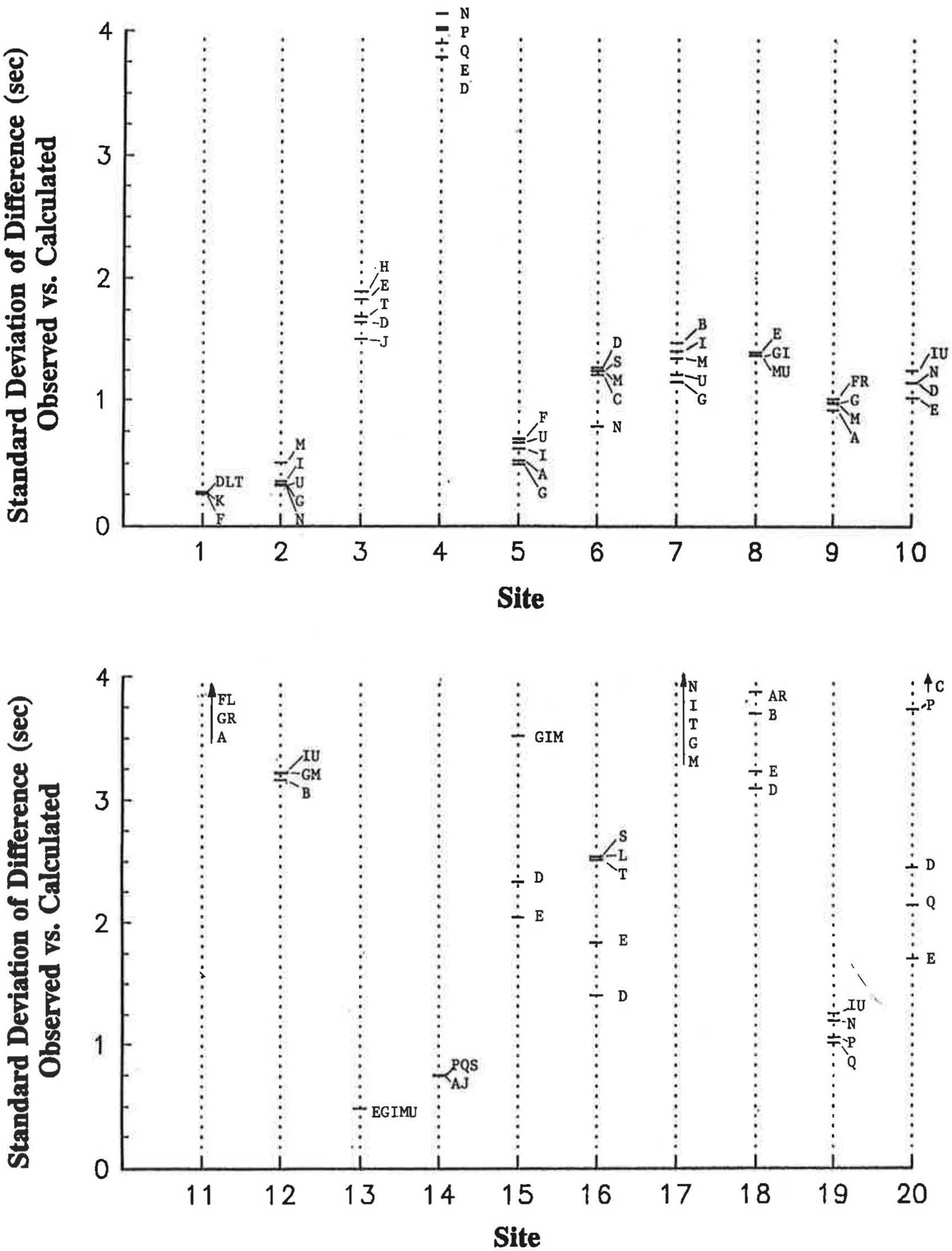

FIGURE 3 Standard deviation of differences between observed and calculated delay.

field Line has the worst series of R^2 values. Overall the actuated HCM equation (HCM B) has the best ranking among the delay models, with the pretimed HCM equation (HCM A) a close second. Note also that the regression based models did not appear to perform better than the theoretical models.

Similar results occurred for the regression analysis of the composite traffic signal operations. The primary difference between the general application and the composite application is that the best model, the actuated HCM equation (HCM B), was not a clear leader. The NCHRP Overflow delay model was within one point of the HCM equation. The remaining seven models with scores are all grouped together without clear preference among them.

Examination of the mean difference between the observed and calculated delay values shows that among the five best models, there is little variation, usually less than 1 sec. The

TABLE 6 MODEL CODES USED IN FIGURES 1–6

Model Code	Model	
A	Allsop	lower limit
B		upper limit
C		approx. of Webster's model
D	HCM	pretimed coefficients
E		actuated coefficients
F		uniform delay component
G		random delay component
H	Hutchinson	
I		random delay component
J	Miller	A (v/c < 0.5)
K		B (v/c > 0.5)
L	NCHRP	uniform delay
M		overflow delay
N	Stone and Wild	
P	TTI	modified coefficient
		unmodified coefficient
Q	Wardrop	
R	Webster	
S		uniform delay component
T		random delay component

Note: May's model and the uniform arrival component of Hutchinson's model are identical to the uniform arrival component of the Highway Capacity Manual delay model.

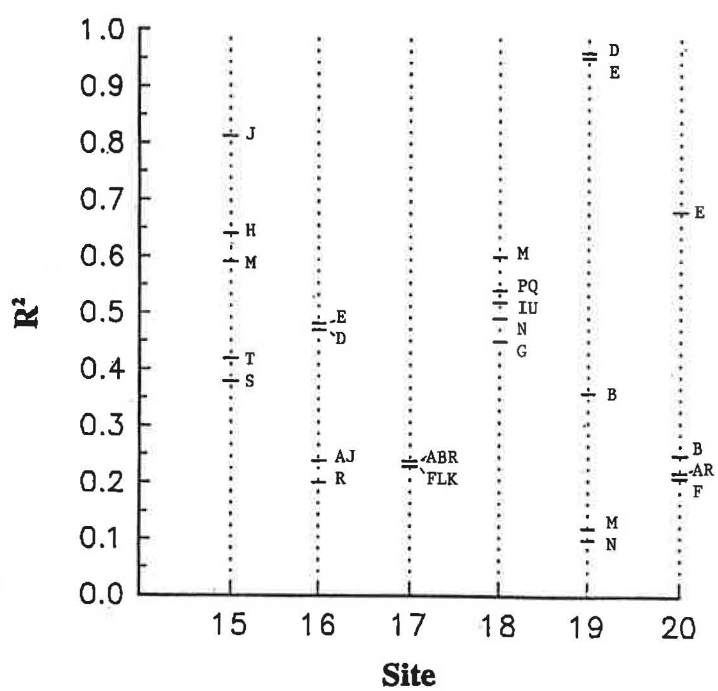

FIGURE 4 Regression analysis of sites with traffic signal control—composite analysis.

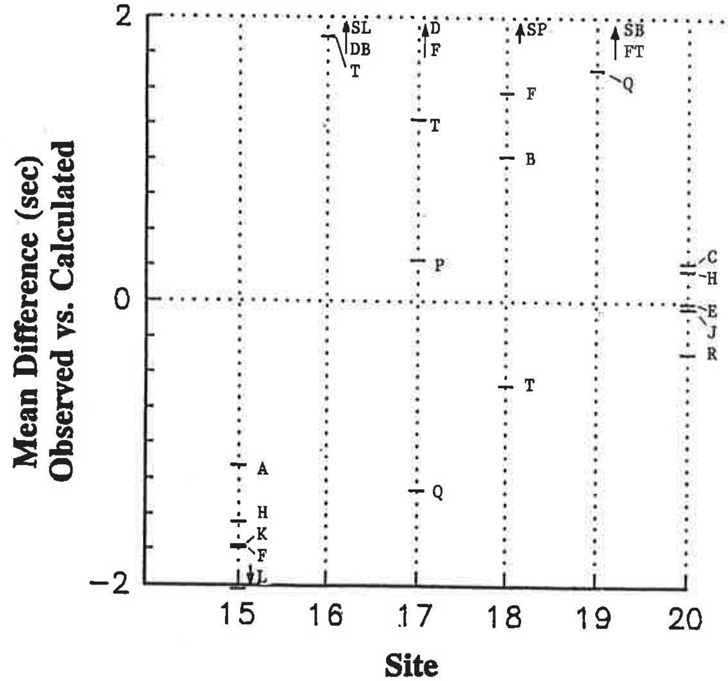

FIGURE 5 Mean difference between observed and calculated delay times for sites with traffic signal control—composite analysis.

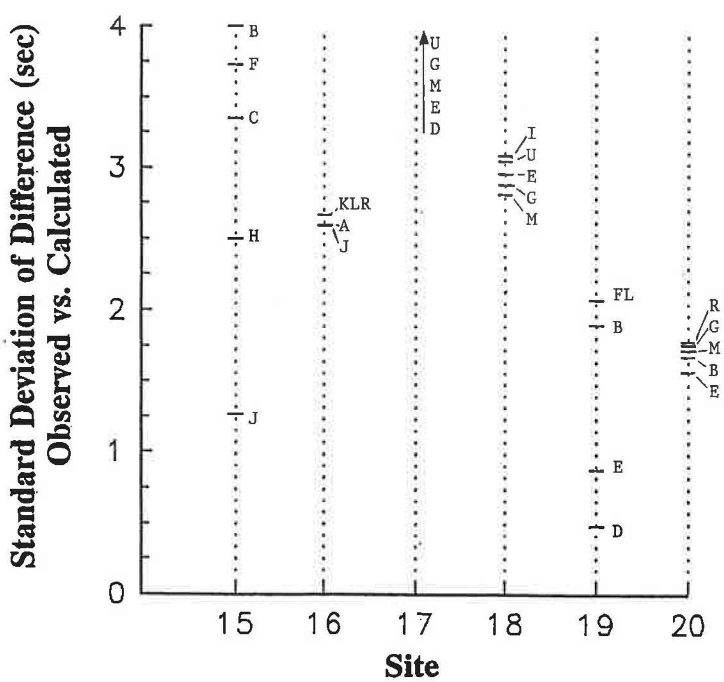

FIGURE 6 Standard deviation of differences between observed and calculated delay for sites with traffic signal control—composite analysis.

difference between the best model and the worst model is greatest at Church Street (No. 2) on the Transit Mall in Buffalo, New York, during the evening peak period, at 5.79 sec. Overall, the best model was the uniform delay component of Webster, with the pretimed HCM equation (HCM A) a close second. Again, the regression-based models did not perform as well as the theoretical models in the rankings, and, again, the results of the composite analysis were similar.

The importance of the mean difference is illustrated when estimating the level of service of the crossing. If the mean difference for the delay equation being used is great, the resulting level of service estimate may be in error. For example, if the mean difference for the HCM pretimed equation is 10 sec and the value guiding the estimate is 15 sec of stopped delay, then the true level of service is between LOS A (15 sec − 10 sec = 5 sec) and LOS C (15 sec + 10 sec = 25 sec).

The standard deviation of the differences is important in estimating possible variation and its influence on the resultant level of service estimate. Examination of the standard deviations of the differences between the observed and calculated delay values shows that among the five best models, there is, again, little variation. The difference between the best model and the worst model is greatest at Church Street (No. 2) during the evening peak period, at 2.44 sec. Overall, the actuated HCM equation (HCM B) again has the best ranking among the delay models, with the pretimed HCM equation (HCM A) again a close second, and, once again, the regression-based models did not perform better than the theoretical models. The results of the composite analysis were similar to the general application.

CONCLUSIONS

This study evaluated the application of theoretically and empirically based delay equations, developed for isolated traffic signal operation, to the problem of isolated at-grade LRT crossings. The key finding of the study is that the pretimed HCM equation is a suitable model for isolated LRT crossing evaluation.

At crossings where control is provided by traffic signals operating with background cycles, Allsop's equation was found to be a suitable model when applied with its lower limit parameters.

In addition to these findings, it was found that

- The results from the exclusive use of the random delay component of multiple component delay models were not better than those from use of the complete delay model, or only the uniform component of the model, and
- The delay models based on regression analysis did not perform better than the theoretical delay models.

REFERENCES

1. T. J. Stone and W. A. Wild. Design Considerations for LRT in Existing Medians: Developing Warrants for Priority Treatments. In *Special Report 195: Light Rail Transit: Planning, Design, and Implementation*. TRB, National Research Council, Washington, D.C., 1982.
2. W. Grote. *Impacts of Light Rail Transit Operations on Urban Intersection Delay and Capacity*. Master's thesis. University of Utah, 1984.
3. A. E. Radwan and K. P. Hwang. Preferential Control Warrants of Light Rail Transit Movements. In *Transportation Research Record 1010*, TRB, National Research Council, Washington, D.C., 1985.
4. J. C. Cline, T. Urbanik, and B. Rymer. Delay at Light Rail Transit Grade Crossings. In *Texas Transportation Research Report 339-10*, Texas State Department of Highways and Public Transportation, 1987.
5. R. A. Berry. Assessing the Operation of At-Grade Light Rail Transit Crossings. Presented at Traffic Congestion 88: Issues and Answers, ITE, 1988.
6. *Special Report 87: Highway Capacity Manual*. HRB, National Research Council, Washington, D.C., 1965.
7. A. D. May, Jr. and D. Pratt. A Simulation Study of Load Factor at Signalized Intersections. *Traffic Engineering*, 1968.
8. R. W. Crommelin. Employing Intersection Capacity Utilization Values to Estimate Overall Level of Service. *Traffic Engineering*, 1974.
9. San Diego Association of Governments. *San Diego Trolley: The First Three Years*. Office of Planning Assistance, UMTA, U.S. Department of Transportation, 1984.
10. P. A. Gibson, B. B. Lin, and R. Robenhymer. Traffic Impacts of Light Rail Transit. In *Special Report 195: Light Rail Transit: Planning, Design, and Implementation*. TRB, National Research Council, Washington, D.C., 1982.
11. Gannett-Fleming/Schimpeler Corradino. Traffic and Circulation Technical Report. *Bayside LRT Line Preliminary Engineering and Environmental Impact Report Study*, Metropolitan Transit Development Board, San Diego, Calif., 1987.
12. *Transportation Research Circular 212: Interim Materials on Highway Capacity*. TRB, National Research Council, Washington, D.C., 1980.
13. *Special Report 209: Highway Capacity Manual*. TRB, National Research Council, Washington, D.C., 1985.
14. D. L. Gerlough and M. J. Huber. *Special Report 165: Traffic Flow Theory—A Monograph*. TRB, National Research Council, Washington, D.C., 1975.
15. R. A. Berry and J. C. Williams. Traffic Characteristics of At-Grade Light Rail Crossings. *Compendium of Technical Papers*, 59th Annual Meeting of the ITE, 1989.
16. W. R. Reilly, C. C. Gardner, and J. H. Kell. *A Technique for Measurement of Delay at Intersections*. Report RD-76-135/137, FHWA, U.S. Department of Transportation, 1976.
17. A. Sadegh and A. E. Radwan. Comparative Assessment of 1985 HCM Delay Model. *Journal of Transportation Engineering*, ASCE, Vol. 114, No. 2, 1988.
18. F. V. Webster. *Traffic Signal Settings*. Road Research Technical Paper 39, Great Britain Road Research Laboratory, 1958.
19. JHK & Associates. NCHRP Signalized Intersection Capacity Method. NCHRP Project 3-28(2), TRB, National Research Council, Washington, D.C., 1982.

Publication of this paper sponsored by Committee on Traffic Control Devices.

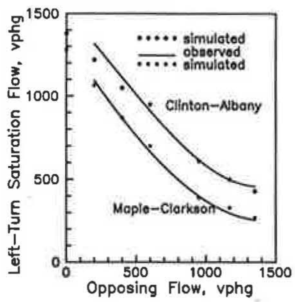

FIGURE 2 Observed and simulated saturation flows of opposed left turns at two intersections.

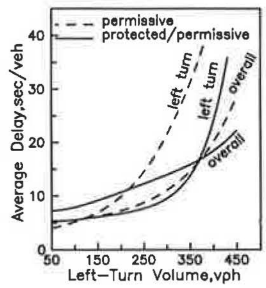

FIGURE 3 Example characteristics of full-actuated signal operations involving permissive phasing and protected/permissive phasing.

which concern opposed left turns from a shared lane, further show that the model is a reasonable tool for analysis.

The saturation flows shown in Figure 2 are similar to those reported in an earlier study (*8*). Nevertheless, they reveal that the saturation flows of opposed left turns can vary significantly from one intersection to another. The differences between the saturation flows as shown in Figure 2 are primarily attributable to the fact that the Clinton-Albany intersection has a much larger storage area than the Maple-Clarkson intersection. The Clinton-Albany intersection allows an average of 2.7 vehicles to complete the turns after a signal change interval begins, compared with an average of only 1.3 vehicles at the Maple-Clarkson intersection (*9*).

The simulation analysis performed in this study is based on the following conditions:

- Left-turn drivers have a critical gap of 5 sec.
- There is a 15 percent chance that the first left-turn vehicle in a queue will turn in front of the first opposing vehicle immediately after the green light is turned on.
- Vehicles approach the intersection randomly at an average speed of 30 mph. All vehicles are passenger cars.
- The saturation flows of straight-through, unopposed left-turn, and right-turn movements are 1,700, 1,500, and 1,350 vphg, respectively.

When the opposing volume is very heavy, left turns can be made only in the first few seconds after the green onset or after the change interval begins. The simulated number of such turns varies from one cycle to another; the average is approximately two vehicles per cycle.

PHASING PLAN SELECTION

Figure 3 gives an insight into the relative performance characteristics of a control with permissive left turns and another with protected/permissive left turns. This comparison is based on an opposing flow of 500 vph and a cross-traffic pattern that has a critical lane flow of 500 vph. The figure shows that permissive left turns can bring about shorter left-turn delays and overall delays when the left-turn volume is small. As the left-turn volume increases, protected/permissive left-turn phasing can easily provide a better service to the left-turn vehicles, although permissive left-turn phasing may still yield shorter overall delays. When the left-turn volume is sufficiently high, it becomes possible for protected/permissive left-turn phasing to reduce not only the left-turn delays but also the overall delays. It is also apparent from the figure that delays are more sensitive to the left-turn volume when permissive left-turn phasing is used. In contrast, protected/permissive left-turn phasing can provide more equitable services to the vehicles in every lane. As a result, the delays resulting from such a phasing arrangement are less sensitive to the left-turn volume. In terms of overall delays, the control efficiencies may deteriorate by more than 30 percent when an improper phasing arrangement is implemented.

The choice between permissive and protected/permissive phasings naturally requires a trade-off between maximizing overall control efficiency and avoiding excessive delays and queue lengths in any traffic lane. What constitutes unacceptable delays and queue lengths can vary from one signal installation to another. Field observations made at the Market Street–Sandstone intersection in Potsdam, New York, indicate that an opposed left-turn flow of 230 vph suffering an average stopped delay of more than 40 sec per vehicle often incurs a queue length in excess of 400 ft. In principle, the queue length in any lane should not be allowed to extend to the upstream intersection. For most intersections, this perhaps implies a maximum allowable queue length of about 500 to 700 ft. Whenever possible, stopped delays should also be kept under 40 sec per vehicle to prevent the level of service from degenerating into a Level E or, possibly, a Level F as defined in the 1985 *Highway Capacity Manual* (*10*).

In this study, a phasing plan is considered superior to a competing plan if the following conditions are satisfied:

1. The stopped delay in every lane is less than 40 sec per vehicle;
2. The queue length in every lane is less than 500 ft; and
3. The overall delay is smaller than that produced by the competing plan.

Under very heavy flow conditions, neither permissive phasing nor protected/permissive phasing may be able to maintain acceptable delays and queue lengths. In such a case, the phasing arrangement that can provide more equitable services by eliminating extremely long delays and queue lengths is considered to be better.

Based on these criteria, the two options of left-turn phasing are compared under a variety of conditions. In this comparison, the signal timing settings for a given condition are adjusted to achieve near-optimal signal operations for both permissive phasing and protected/permissive phasing. The delays and queue lengths derived from computer simulation for such operations are then used to determine the preferred phasing strategy. The stopped delay of each simulated vehicle is the total stopped delay accumulated from the moment the vehicle is generated at a location 600 ft upstream of the stop line until it clears the intersection.

The results of this simulation analysis are shown in Figure 4 for cases involving one opposing lane and in Figure 5 for those involving two opposing lanes. In these figures, the combinations of left-turn volume and opposing volume that allow permissive phasing and protected/permissive phasing to achieve comparable signal operations are represented by one curve for a specified level of cross traffic. The combinations of left-turn volume and opposing volume above such a curve represent conditions favorable to the implementation of protected/permissive phasing; those combinations below the curve should preferably be treated with permissive left-turn phasing.

In Figures 4 and 5, low, moderate, and heavy cross-traffic levels are represented by critical movement volumes, denoted as Q_c, of 100, 600, and 900 vph, respectively. In fact, such volumes alone really cannot give an accurate picture of the impact of the cross traffic. From the viewpoint of left-turn phasing, the most important element related to the cross traffic is the amount of green time consumed by the cross traffic, not the critical movement volume. It should be noted that

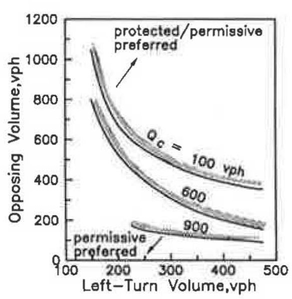

FIGURE 4 Preferred left-turn phasings (one opposing lane).

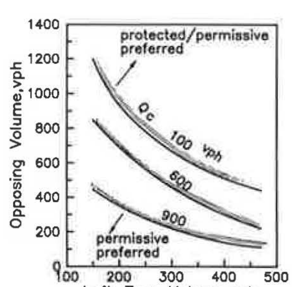

FIGURE 5 Preferred left-turn phasings (two opposing lanes).

the average cross-street green times related to the above critical volumes fall into the following ranges when permissive phasing is in place: 6 to 12 sec for Q_c = 100 vph; 18 to 25 sec for Q_c = 600 vph; and 30 to 45 sec for Q_c = 900 vph.

Figures 4 and 5 show that the choice between permissive phasing and protected/permissive phasing can be significantly affected by left-turn volume, opposing volume, number of opposing lanes, and the level of cross traffic. The effective length of the left-turn bay has less obvious impact on such a choice, probably because both permissive left turns and protected/permissive left turns can be adversely affected by a left-turn bay of insufficient length. Nevertheless, as shown in the next section, providing sufficiently long left-turn bays can greatly enhance the ability of protected/permissive phasing to improve signal operations.

Within the levels of the cross traffic considered in this study, Figures 4 and 5 show that the provision of a separate left-turn phase is difficult to justify when the left-turn volume is less than 140 vph (an equivalent of not more than 15 sec of green time for the cross street). As the left-turn volume increases, it becomes easier for protected/permissive phasing to produce better signal operations. It is also obvious from these figures that the use of the product of left-turn volume and opposing volume (*2*) to guide phasing decisions is not suitable for full-actuated control. Direct application of Figures 4 and 5 would allow more intelligent decisions. It should be noted, however, that these figures are derived from comparing signal operations that have good signal timing settings. How improper timing settings may affect the relationships presented in the figures has not been investigated.

A hidden feature of Figures 4 and 5 should also be pointed out. Generally, protected/permissive phasing would be most effective in improving signal operations when the capacity of an intersection cannot adequately accommodate permissive left turns but is sufficient to accommodate protected/permissive left turns. The capacity of an intersection is considered to be inadequate if near-optimal timing settings cannot prevent stopped delays and queue length in every lane from exceeding 40 sec per vehicle and 500 ft, respectively. Under very heavy flow conditions both permissive phasing and protected/permissive phasing may be unable to maintain acceptable signal operations. In such a case, protected/permissive phasing may only be able to mitigate the severity of congestions by redistributing overcongestions among several lanes. Under this circumstance, protected/permissive phasing may not have a clear-cut advantage over permissive phasing. Consequently, the phasing selection should be made more cautiously. To assist in the selection of phasing plans, the capacity constraints imposed on permissive phasing and protected/permissive phasing are shown in Figures 6 and 7. These figures are applicable to intersections where full-length exclusive left-turn lanes are available; they are also applicable to intersections where left-turn bays are not blocked.

LENGTH OF LEFT-TURN BAY

If the length of a left-turn bay is not long enough, the vehicles using the bay and its adjacent lane may block each other. When such an operating condition exists, the effectiveness of protected/permissive phasing in improving signal operations