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Contents

The Bandwidth-Constrained TRANSYT Signal-Optimization Program

STEPHEN L. COHEN and C. C. LIU

ABSTRACT

A discussion is presented of previous attempts to combine bandwidths and delay and stop considerations as criteria for computing signal-timing plans for arterial signal systems. In particular, deficiencies in these previous attempts are pointed out. A new approach that involves constraining the TRANSYT-7F model to preserve the two-way band computed by a bandwidth program is described. This new approach was tested on 10 widely varying arterial data sets by using the MAXBAND program to develop the green bands, and the NETSIM model to evaluate the effectiveness of the resultant signal-timing plans with a weighted combination of delay and stops as the measure of performance. It is shown that no statistically significant improvement in arterial performance is obtained by adjusting offsets only, even in the case of short block spacing. However, if both offsets and green times are adjusted, statistically significant improvements in arterial performance are obtained.

The TRANSYT model is the most widely used computer program for developing signal-timing plans for urban signal systems (1). An Americanized version of the program, TRANSYT-7F was developed for use in the United States and has been successful (2). The TRANSYT program is based on delay and stops in that a macroscopic traffic model is used to estimate delay and stops, based on volumes, capacity, and signal timing. A weighted combination of delay and stops, called the performance index (PI), is the criterion used. Offset and green-phase times are adjusted to make the estimate of PI as small as possible (e.g., optimize PI).

However, some reluctance has been evident in the traffic engineering community about applying the TRANSYT program to signalized arterials because the program often does not produce good progression bands. As an example, consider the space-time diagram in Figure 1. The signal offsets for the eight-intersection network were developed by using the TRANSYT program. This can be compared with the space-time diagram for the same network and traffic conditions shown in Figure 2. Here, the offsets were obtained by using a bandwidth-based program, MAXBAND (3). A substantial number of practicing traffic engineers prefer a timing plan such as the one shown in Figure 2 over a timing plan such as that shown in Figure 1. Engineers often take the signal-timing plans produced by TRANSYT and make offset and even green-time adjustments to improve the progression bands. These adjustments, which are made in an ad hoc manner, will degrade the performance of the signal-timing plans relative to delay and stops. They also do not produce the widest green band that could be achieved. For instance, it is unlikely that one could make adjustments by hand on the timing plan shown in Figure 1 and arrive at progression bands as wide as those shown in Figure 2.

Because many traffic engineers want arterial signal-timing plans to have good progression bands, they prefer to use maximal bandwidth-based programs such as MAXBAND and PASSER II (4), which optimize the total two-way green band on signalized arterials, but give delay and stops consideration only insofar as the computation of green time is concerned.

Based on these comments, it is evident that it would be useful to combine the delay-stop optimization approach and the bandwidth optimization approach in some way to be able to produce signal-timing plans that combine the best features of both. The purpose of this paper is to report on an approach that does this.

DISCUSSION OF PREVIOUS RESEARCH

A number of research studies have been done in the field of combining delay-stops and bandwidth considerations. These studies may be divided into two categories:

1. Those that modify a delay-based program to give consideration to bandwidth (or more generally, progression).
2. Those that adjust bandwidth-based signal-timing plans to reduce further delay.

In the first category, an obvious approach is to use a bandwidth solution as a starting point for a TRANSYT optimization. One researcher (5) has previously performed a study in which this approach was taken. Based on the somewhat limited sample of two test arterials, some indication was evident that using a bandwidth solution as a starting point for TRANSYT had some potential for giving signal-time plans that were an improvement over those obtained using the default zero offsets. However, this approach in no way guarantees that there will be progression bands, much less maximal progression bands. For example, consider Figure 3, which is a space-time diagram for the same arterial with volume and capacity conditions the same as in Figures 1 and 2. The timing plan shown in Figure 3 was developed by using the TRANSYT program with the timing plan shown in Figure 2 as the starting solution. It can be ob-

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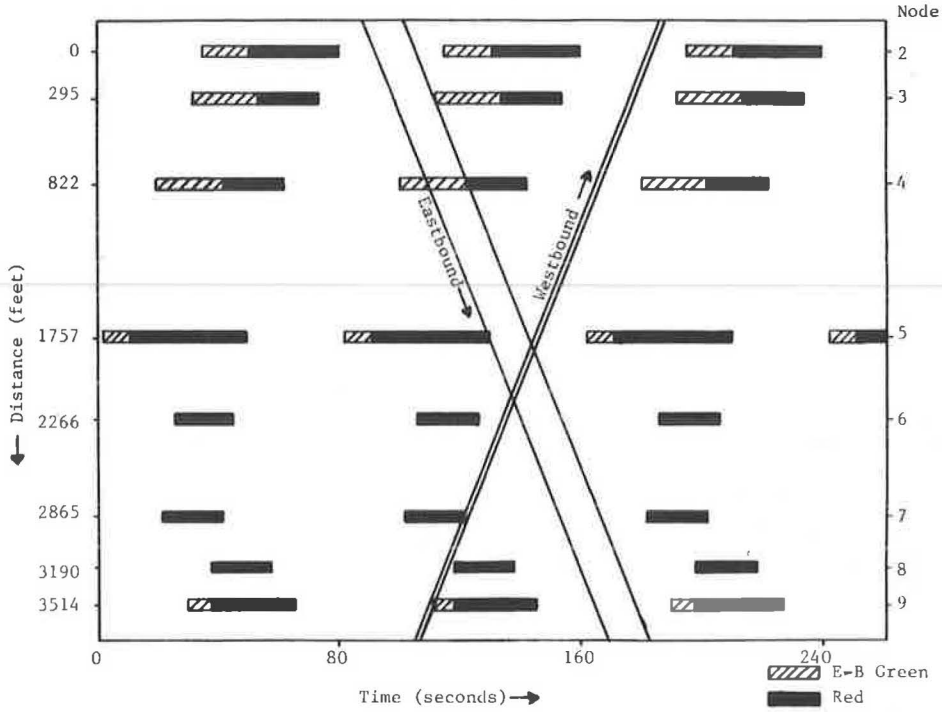

FIGURE 1 Time-space diagram of TRANSYT-7F-generated signal policy.

served that the progression bands are better than in Figure 1, but not as good as those in Figure 2.

A better approach has been taken by Wallace and his associates (6), who replaced the minimization of PI in TRANSYT with the maximization of PROS/PI. The quantity PROS, which stands for PRogression Opportunities, is defined as follows: extend the concept of progression to include progression opportunity, which is defined as "the opportunity presented at a given traffic signal and a given point in time to travel through a downstream signal without stopping." The quality PROS is defined as the sum of all such progression opportunities.

In maximizing PROS/PI, the TRANSYT program will try to achieve a value of PI that is as small as possible, while at the same time trying to achieve a

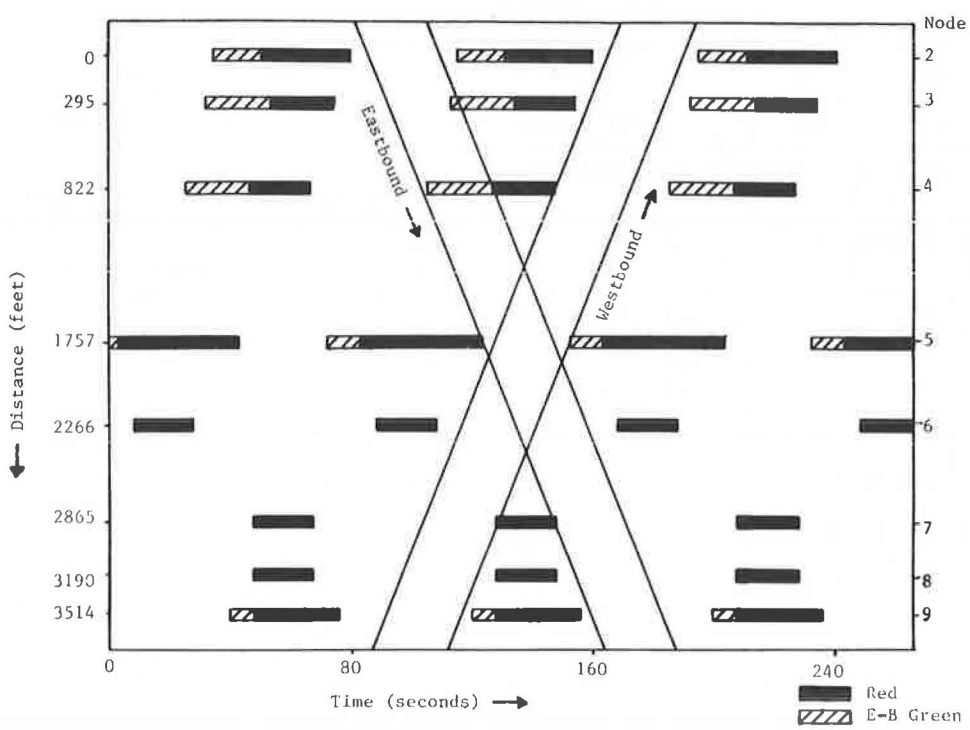

FIGURE 2 Time-space diagram of MAXBAND-generated signal policy.

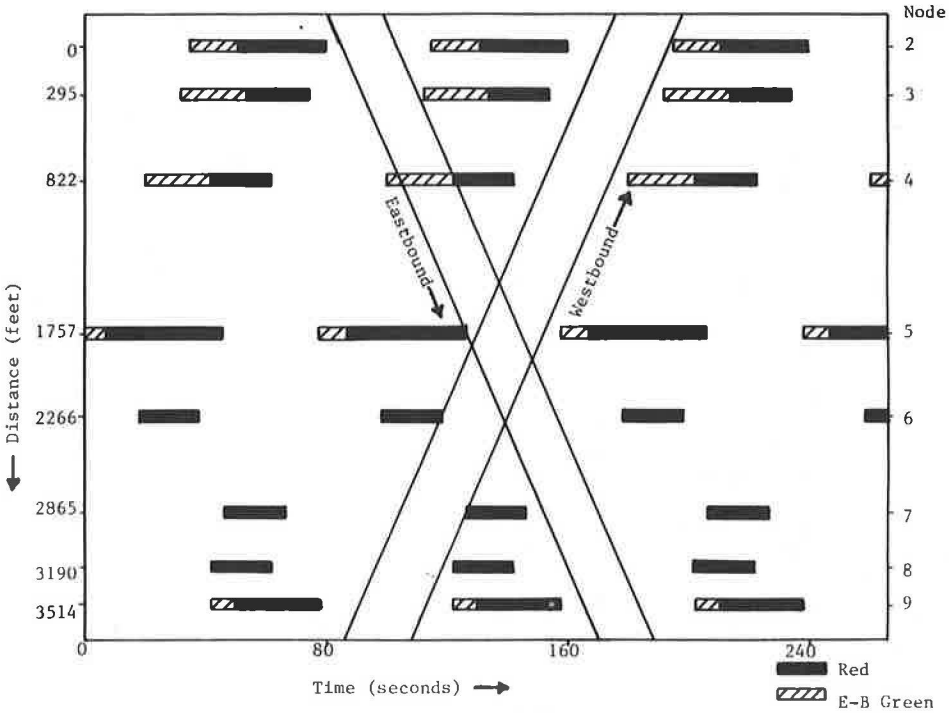

FIGURE 3 Time-space diagram of TRANSYT-7F-generated signal policy using MAXBAND results as the initial setting.

value of PROS that is as large as possible. Obviously, bandwidth alone could have been used in place of PROS in this approach. The results, based on 5 arterial networks that had a range of from 5 to 20 intersections, were promising. Delay measures of effectiveness, as measured by TRANSYT, were improved over the PASSER II bandwidth solutions. Furthermore, bandwidths were at least equal to those produced by PASSER II. However, putting PROS (or bandwidth) directly into the TRANSYT objective function has the disadvantage that, when green times are optimized, the side streets tend to be discriminated against in that they will get less green time. The reason for this is as follows: the quantity (1/PI) as a function of side-street green time increases to some optimum value and then decreases as side-street green time is increased; PROS (or bandwidth) is a monotonically decreasing function of side-street green; thus, the product [PROS * (1/PI)] will have an optimum value shifted toward smaller values of side-street green.

The second category of approaches depends on the observation that bandwidth solutions do not lead to a complete specification of the offsets. This is because many, if not most, of the intersections on an arterial will have slack green time available. Slack green time is defined as "green time available at an intersection that is outside the band." This is shown in Figure 2, which depicts an eight-intersection arterial. There are 26.3-sec bands in both directions, and the cycle length is 80 sec. It can be observed that the two-way band touches the left and right edges of the arterial green at Intersections 4, 5, and 7, but that the other intersections have slack green time. The slack green times range from 6.4 sec at Intersection 2 to 15.4 sec at Intersection 8. Thus, the offsets at Intersections 2, 3, 6, and 9 may be adjusted within the slack green time allowances without affecting the through band.

One approach to utilizing the slack green time was taken by Wallace (7) in the development of the PROS model that was described previously in this section. In this work, PROS was compared with the results of PASSER II on five arterials. The result was that optimizing PROS gave through bands that were equivalent to the through bands given by PASSER II. However, at the same time, when the TRANSYT model was used to compare the delay statistics, it was found that PROS provided only small improvements over PASSER II-74, as measured by reductions in delay. The results of the study may be summarized as follows: maximizing PROS, in most cases, has the effect of giving through bands comparable with PASSER II, with slack green times adjusted in order to provide the maximum amount of secondary progression.

Consideration of secondary progression was added to PASSER II in 1980, and to the MAXBAND program in 1982. In this approach, the band is centered so that an equal amount of slack green time is available on both sides of the two-way band. This is shown in the time-space diagram given in Figure 2. The amount of improvement to be expected over a random assignment such as was found in the original MAXBAND program is given in Table 1. In this table, two arterial networks [described by Cohen (5)] were used to determine the improvement to be expected relative to reduced delay and stops (or, more generally, PI) in

TABLE 1 Comparison of MAXBAND and MAXBAND Centered Signal-Timing Plans

Network	Program	Delay (sec/veh)	Stops (stops/veh)	PI (= delay + 4 * stops) (vehicle-hr/hr)
Hawthorne Boulevard	MAXBAND	68.3	1.77	138.3
Hawthorne Boulevard	MAXBAND-C	62.8	1.63	127.0
University Avenue	MAXBAND	41.7	1.44	87.3
University Avenue	MAXBAND-C	39.4	1.38	82.6

centering the bands produced by MAXBAND. The measures of effectiveness were obtained using the NETSIM model. Based on the admittedly limited sample, it can be seen that the improvements in PI were about 7 to 8 percent for both arterials, which is a fairly substantial improvement.

The major problem with these approaches to adjusting slack green time is that the effect of actual traffic volumes and traffic patterns, especially those turning onto the arterial from side streets (called secondary flow), are not taken into account. The PROS and centering approaches provide secondary progression opportunities, of which secondary flow may take advantage, but they do not directly respond to traffic patterns.

Another approach to adjusting slack green time has been suggested by Chang (8,9). In this approach, a simple delay model based on the delay-offset relation has been added to PASSER II. It explicitly models secondary flow from the intersections immediately upstream of an intersection and adjusts the offset at that intersection within the slack green time allowance to minimize the delay so calculated. Thus, the two-way band is explicitly preserved and the intersections with slack green time have their offsets adjusted to further reduce delay. However, results cited in the paper were based on a limited sample of one four-intersection arterial.

Nevertheless, Chang's approach has a number of advantages over the other approaches described previously in this paper. Unlike the bandwidth starting solution for TRANSYT described by Cohen (5), the two-way band is preserved. Unlike the PROS/PI approach described by Wallace and Courage (6), the side streets are not discriminated against because the green times are held fixed. Unlike the secondary progression approaches--PROS and centering--traffic effects, particularly secondary flow effects, are explicitly considered.

However, Chang's approach has some defects, namely

1. In adjusting offsets at a given intersection, only the effects on the intersections immediately upstream are included.
2. In the delay-offset model, the platoon structure is not modeled.
3. No capability exists for making adjustments to green time while at the same time preserving the bands.

The purpose of this paper is to propose an alternative approach to Chang's, an alternative that addresses the problems just cited.

CONSTRAINED MODEL LOGIC

The authors start with the approach taken earlier by Cohen (5), namely, using a bandwidth solution generated by the MAXBAND program as a starting solution for TRANSYT-7F. To facilitate further discussion, TRANSYT's optimization submodel will first be briefly described.

The optimization process in TRANSYT-7F is controlled by a subroutine, HILLCL, in which the offset and/or green times are changed iteratively by specified amounts. The new or revised timing values are simulated by TRANSYT's traffic simulation model located in subroutine SUBPT. The resulting PI is compared with the previous value to determine whether the last change was an improvement. This process is repeated for all hill climb step sizes that were input from Card Type 4. Figure 4 shows a flow chart of the program logic of subroutine HILLCL relevant to the current study (10).

The incorporation of a two-way progression band

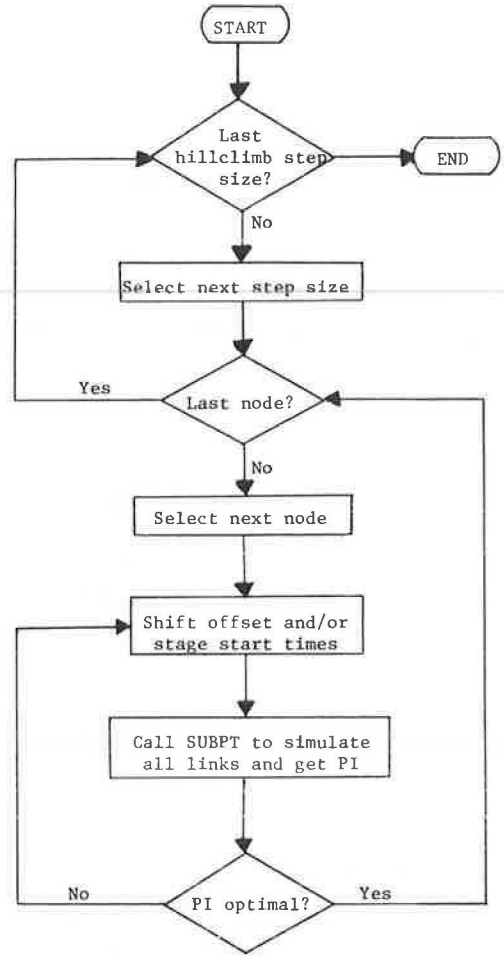

FIGURE 4 Existing subroutine HILLCL.

into the optimization structure just described involves, first as required input to TRANSYT, the desired bandwidths of both directions of travel and the progression speeds. This is actually accomplished by specifying the starting and ending time points (there is a maximum of four) at each intersection of the two-way band in an additional input stream. This information is readily available from MAXBAND or any other progression-band-based arterial model.

The constrained model, in essence, is to simply provide a check after each shift of timing values. As long as the shift does not cause any red time (including dual left-turn green time) to encroach on the through band, arterial progression is preserved. Care should be taken here that in situations with phase overlap, left-turn green time for one direction would be considered red for the other and vice versa. One can easily visualize the constrained optimization process as to first plot or "nail down" the MAXBAND's two-way band on an empty time-space diagram and then enable TRANSYT to generate timing plans within the band constraints. It should be noted that this approach allows the possibility of unequal bands in the two directions in order to accommodate unbalanced flows.

A total of three additional routines were added to TRANSYT-7F to perform the constraining functions. Modifications were also made to subroutine HILLCL, including some error checking pertaining to progression band input. The modified HILLCL logic is shown in Figure 5.

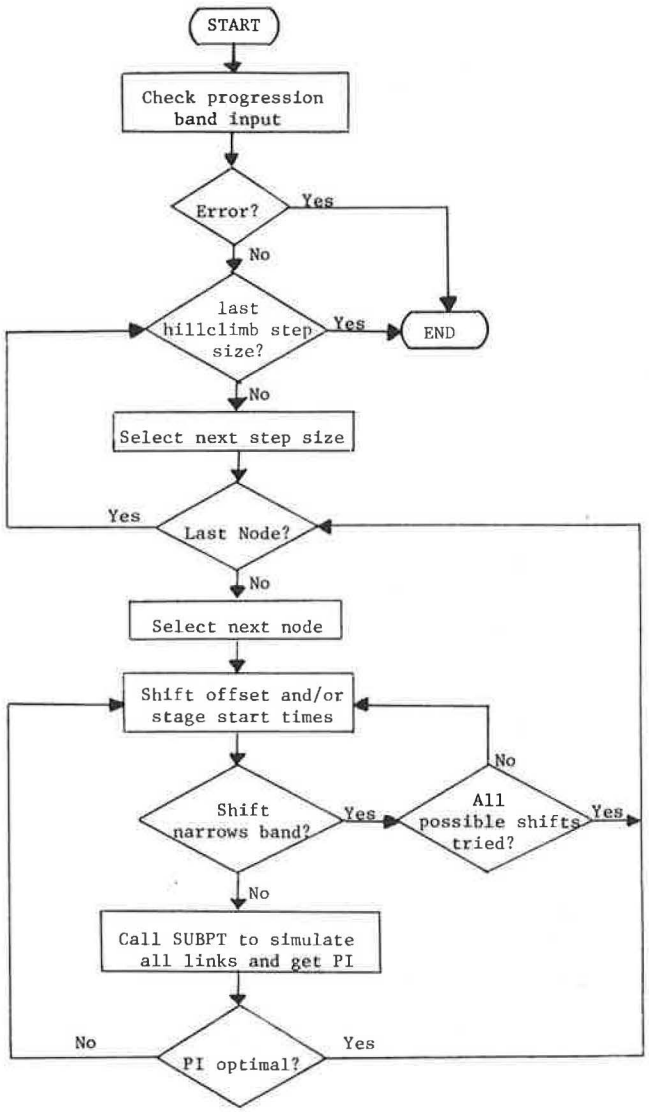

FIGURE 5 Constrained subroutine HILLCL.

CONSTRAINED TRANSYT RESULTS WITH OFFSET ADJUSTMENT

The constrained TRANSYT model, hereafter referred to as TRANSYT-7F(C), was first run with all positive hill climb step sizes. This would prohibit TRANSYT from changing MAXBAND's green split, enabling it to perform offset adjustments only.

Ten arterial data sets were available, including seven that have been described previously (11). The additional three arterials were added because they are characterized by short block spacing (average spacing 500 ft or less). One of the findings by Chang et al. (8) was that slack green time adjustments show the greatest potential for improvement in short block situations. Table 2 gives a brief description of each arterial.

The NETSIM model was run for the MAXBAND and TRANSYT-7F(C) timing plans for each arterial data set. A stop weighting factor of four was used in TRANSYT; the results are given in Table 3. It can be seen that the percentage improvement, as estimated by TRANSYT, is quite small, averaging approximately +3 percent over all networks. The average over all NETSIM runs was only +0.9 percent. To test whether the change due to slack green time adjustment is significant, a Wilcoxon matched-pairs test was done comparing MAXBAND and TRANSYT-7F(C) as estimated by NETSIM. The result was that no significant difference was found at the 5 percent level of significance. Therefore, it may be concluded that no evidence exists that slack green time adjustment alone will significantly reduce delay or stops over centering the band. Further, the changes in PI for North Michigan Avenue (which had average block spacing of 300 ft) were smaller than the average change over all networks, which contradicts the assertion by Chang et al. (8) that the largest improvements due to slack green adjustment may be expected on arterials with the shortest block length.

CONSTRAINED TRANSYT WITH OFFSET AND GREEN TIME ADJUSTMENT

When negative hill climb step sizes are intermixed with positive ones on Card Type 4, both offset and green time are optimized by TRANSYT. The 10 arterials described in the previous section were again run with the TRANSYT-7F(C) under this scenario. It is noted that in this case, it could happen that the bandwidth actually increases, if it is advantageous for TRANSYT to shift green time to the arterial through movements. However, it should also be noted that any increase in arterial through green will occur only if it is advantageous from the point of view of reductions in PI. It could happen that green time may be shifted to the side streets, especially in cases in which a large slack green time is available. This is to be contrasted with the PROS/PI approach vehicle, which will always shift green time to the main street to improve PROS, regardless of its effect on PI. The constrained model is in no way bounded to produce exactly the same input progression bandwidth, although it may be the case. Rather, as mentioned earlier, the notion is to prevent red time encroachment on the band. Table 4 gives a summary of the results of the run.

TABLE 2 Arterial Descriptions

Arterial	Location	Signalized Intersections	Lanes	Progression Speed	Cycle Length	Signal Spacing Range (ft)	Average Signal Spacing (ft)
Hawthorne Boulevard	Los Angeles, California	13	8	45	90	560-2,600	1,189
University Avenue	Provo, Utah	10	4	30	80	480-1,440	820
Nicholasville Road	Lexington, Kentucky	12	4	35	80	520-2,160	1,177
North 33rd Street	Salt Lake City, Utah	9	4/6	35	75	353-1,605	1,131
Frederica Road	Owensboro, Kentucky	12	4	45	80	582-2,310	1,167
Fannin Boulevard	Houston, Texas	15	6	35	80	300-1,900	711
San Felipe Road	Houston, Texas	12	4	35	80	250-1,400	741
K Street	Washington, D.C.	11	6/4	28	80	323-679	525
M Street/Key Bridge	Washington, D.C.	8	4	30	80	285-935	502
North Michigan Avenue	Chicago, Illinois	13	8/6	30	90	280-325	304

TABLE 3 MAXBAND versus TRANSYT-7F(C): Offsets Only Optimized

Arterial	TRANSYT PI		Difference (%)	NETSIM PI		Difference (%)
	MAXBAND	TRANSYT(c)		MAXBAND	TRANSYT(c)	
Hawthorne Boulevard	233.6	224.3	4.0	263.4	255.1	+3.2
University Boulevard	83.2	81.7	1.8	98.0	97.6	+ .4
North 37 Street	235.7	234.9	.3	242.3	241.9	+ .2
Nicholasville Road	184.7	175.0	5.2	209.0	212.7	-1.8
Fredrica Road	119.9	115.8	3.4	109.9	105.2	+4.3
Fannin Boulevard	181.2	173.8	4.0	176.0	172.9	+1.8
San Felipe Road	229.6	220.8	3.8	170.3	171.5	- .7
M Street/Key Bridge	54.8	54.0	1.4	55.1	55.9	-1.5
K Street	219.1	210.4	4.0	230.9	224.5	+2.8
North Michigan Avenue	174.5	169.9	2.6	171.0	169.9	+ .6
Average			+3.0			+0.9

Note: PI = performance index.

TABLE 4 MAXBAND versus TRANSYT-7F(C): Offsets and Greens Optimized

Arterial	TRANSYT PI		Difference (%)	NETSIM PI		Difference (%)
	MAXBAND	TRANSYT(c)		MAXBAND	TRANSYT(c)	
Hawthorne Boulevard	233.6	208.0	11.0	263.4	242.7	+ 7.8
University Boulevard	83.2	78.5	5.6	98.0	92.3	+ 5.8
North 23 Street	235.7	227.4	3.5	242.3	248.4	- 2.6
Nicholasville Road	184.7	167.1	9.5	209.0	204.5	- 2.2
Fredrica Road	119.9	108.5	9.5	109.9	109.6	+ .3
Fannin Boulevard	181.2	151.9	16.1	176.0	158.4	+10.0
San Felipe Road	229.6	207.4	9.7	170.3	167.2	+ 1.8
M Street/Key Bridge	54.8	51.0	6.9	55.1	53.8	+ 2.3
K Street	219.1	198.7	9.3	230.9	221.8	+ 3.9
North Michigan Avenue	174.5	151.6	12.1	171.0	152.0	+11.1
Average			+9.4			+ 4.2

Note: PI = performance index.

It can be observed that the changes (as measured by TRANSYT PI) produced by both offset and green time adjustments are three times as large as those produced by offset adjustment alone. Further, a Wilcoxon test was performed comparing the NETSIM estimates of MAXBAND versus TRANSYT-7F(C) with green times and offsets optimized. It was found that the results for the 10 arterials were significantly different at the 5 percent significance level. Therefore, it may be concluded that adjusting both offsets and green times while preserving the two-way band has the capability of significantly improving the performance, relative to PI, of centered bandwidth timing plans.

The question might be raised as to what price in terms of delay and stops is being paid by constraining TRANSYT. To investigate this question, TRANSYT was run unconstrained on the 10 networks, optimizing both green times and offsets. The resultant timing plans were compared with the constrained TRANSYT timing plans, as shown in Table 5. The average difference in NETSIM PI was -2.4. A Wilcoxon test showed no significant difference in PI at the 5 percent level over the 10 networks, although, in individual instances (e.g., Fannin Boulevard), some difference may have existed.

TABLE 5 TRANSYT-7F versus TRANSYT-7F(C): Offsets and Greens

Arterial	NETSIM PI		Difference (%)
	TRANSYT	TRANSYT(c)	
Hawthorne Boulevard	234.1	242.7	-3.7
University Boulevard	87.2	92.3	-5.8
North 33 Street	246.7	248.4	-0.7
Nicholasville Road	213.2	204.5	+4.1
Fredrica Road	102.5	109.6	-6.9
Fannin Boulevard	145.3	158.4	-9.0
San Felipe Road	168.5	167.2	+0.8
M Street/Key Bridge	53.7	53.8	-0.2
K Street	219.6	221.8	-1.0
North Michigan Avenue	150.4	152.0	-1.1
Average			-2.4

Note: PI = performance index.

CONCLUSION

From the research conducted in this work, it can be concluded that the constrained TRANSYT approach to combining bandwidth and delay considerations in developing arterial signal-timing plans has a number of advantages over other approaches that have been examined:

1. Unlike the bandwidth starting approach, this approach guarantees that the progression band is preserved.
2. Unlike the PROS/PI approach, this approach does not discriminate against the side streets because side-street green time may increase at intersections with slack green time if such adjustments improve PI.
3. Unlike the PROS or centering approaches, this approach explicitly adjusts for traffic patterns.
4. Unlike the delay-offset approach, this approach explicitly considers platoon structure and effects on intersections beyond the nearest ones, and allows adjustment of green times in addition to offsets.

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Discussion

Edmond Chin-Ping Chang*

The paper by Cohen and Liu was a review of several alternatives for combining bandwidths, delay, and stop as criteria to optimize arterial signal-timing plans. Two major approaches were made to combine the minimum-delay and maximum-progression considerations by modifying the delay-based program to maximize bandwidth or adjusting the bandwidth-based signal-timing plans to minimize delay. A method was evaluated in their paper by constraining TRANSYT-7F to minimize system delay, while preserving the two-way progression solution optimized by MAXBAND.

In the first approach, TRANSYT was used with PASSER II, MAXBAND, and the PROS/PI function to provide the maximum progression. The PROS/PI method maximizes the sum of all the progression opportunity (PROS) at all traffic signals in a given time for traveling to downstream signals without stopping. However, putting PROS/PI or bandwidth directly into the TRANSYT objective function may result in less green time for the cross street because the PROS/PI or bandwidth is automatically increased if the arterial green time is increased.

The second approach fine tuned the green times available at noncritical intersections to reduce system delay by using

1. The PROS/PI model in TRANSYT,
2. The arbitrary bandwidth centering in MAXBAND, and
3. The system delay-offset optimization model in PASSER II-84.

In general, these methods can provide good through bandwidths with slack green times adjusted to maximize the secondary arterial progression. The PROS and progression bandwidth centering approaches allow the maximum secondary flow to utilize the secondary progression opportunities, but they do not directly put traffic signal patterns or the side-street traffic in the optimization process. The other approach suggested by Chang explicitly preserved the two-way progression bands and adjusted offsets to reduce the total system delay based on the PASSER II calculations. Unlike the PROS and arbitrary bandwidth centering approaches, traffic effects from the secondary flow are explicitly considered without discriminating against the side-street traffic demand.

To enhance Chang's approach, an alternative method was studied in their paper to provide a constrained offset optimization with TRANSYT-7F's optimization submodel by using the MAXBAND green split and offsets as a starting solution. The two-way progression band coordinate was input into the TRANSYT-7F optimization by specifying the starting and ending time points at each intersection. The constrained model checks the bandwidth coordinates after each shift of timing values. This constrained TRANSYT optimization process could be described as first plotting or "nailing down" the MAXBAND's two-way band on an empty time-space diagram, and then allowing TRANSYT to optimize offsets or the combinations of offsets and green times within the band constraints. The arterial progression is preserved so that the shift does not cause any red time to interfere with the through band and the dual left turn. The constrained TRANSYT offset or offset-green optimization was made to optimize only the offsets or to optimize both offset and green splits without affecting the MAXBAND progression solution.

It was indicated in this study that no statistically significant improvement is obtained by adjusting only the offsets. However, significant improvements in arterial performance are obtained if both offsets and green times are adjusted. Overall, this study shows a feasible approach for incorporating the maximum bandwidth and minimum delay analysis.

However, additional discussion of Chang's approach (1) is needed:

1. Unlike the usual delay-offset analysis, a system offset optimization method using the sectioning method was formulated in PASSER II-84.
2. The platoon arrival was considered by a simplified platoon projection model in PASSER II-84. The platoon propagation on intersections beyond the neighboring intersections is considered by PASSER II.
3. The adjustment to green time splits to account for the progression effect was considered in

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the PASSER II initial progression calculations. The 1984 study made by Skabardonis and May at the University of California at Berkeley demonstrated the superiority of green split calculations and less delay of PASSER II to MAXBAND (2).

In the system offset optimization process, PASSER II-84 first identifies the offset slack-time allowance ranges for each intersection in the arterial. The algorithm then optimizes the offsets within the slack time for each intersection from the lowest possible optimum offset while keeping all the other offsets constant. When the solution of the minimum arterial system average delay is found for a particular signal within the slack-time allowance, the search continues on to the next intersection until no further reduction of the total arterial system delay can be found. The major benefit of this system offset optimization process is that the search always continues for minimizing the arterial system delay within the slack-time constraint in PASSER II-84. The optimization algorithm is constantly focused on the system optimization objective instead of on the local optimization objective. An interconnected signal system may result in nonuniform but controlled platooned traffic during different signal cycles.

Many field studies indicated that delay can be controlled by compacting random flow into a platoon along arterial streets with good progression. At first, it is desired to achieve one coherent platoon of traffic per cycle, preferably a length not exceeding the through green for the maximum progression flow. It is also desirable to obtain the repeated arrival of these platoons in green and not in red through proper signalization. For pretimed signal systems, implementation of an optimized set of cycle, green times, phase sequence, and offsets is desired. For coordinated actuated systems, either prescheduled time-space solutions or platoon-identification techniques applied in real-time computerized traffic signal control are required.

PASSER II considers the effect or delay of arterial progression by estimating the progressive movements arriving at the through green onto the downstream intersections. Three principal factors are included in the platoon projection model of PASSER II-84:

1. Proportion of the total traffic in the progression platoon,
2. Platoon size and rate of platoon dispersion, and
3. Progression quality between each consecutive intersection.

The percentage volume that progressed is calculated by percentage of total through traffic in the arterial progression band, length of the platoon leaving the upstream intersection, and time period for the arterial through saturation flow to clear the upstream intersection. The platoon's length at the downstream intersection depends on the original platoon length leaving the upstream intersection, average travel time, and number of vehicles in the platoon. The platoon dispersion rate increases with increasing travel time and with smaller platoon size in the arterial progression bandwidth.

The progression quality between two intersections could best be described by the amount of through-green time being used for progression. The time period used by the progressed platoon depends on the platoon length arriving at the upstream intersection, length of the through-green time at the downstream intersection, and progression quality between the two intersections. The optimal time-space diagram can be used to examine the quality of progression. Good progression would result in a larger progression bandwidth and bad progression might result in a smaller band or no progression bands.

Based on the NETSIM evaluations in the PASSER II-84 study, it appeared that the effects of slack-green adjustment would depend on how the original green-time splits were first calculated by PASSER II or MAXBAND. It should also be noted that the NETSIM evaluation of average delay and stops on the whole system and total arterial direction might indicate different results. That is, when the total arterial study delay is fine tuned based on the PASSER II-84 or MAXBAND progression solution, the delay measurement may decrease on some links but may increase on other links. When fine tuning intersection slack green time or offsets to optimize secondary progression on the arterial directions, the method and objective function used played a decisive role in reducing the total system delay or initially the arterial delay.

A consistent and satisfactory trend of delay estimation was noted between PASSER II-84 and NETSIM in the PASSER II-84 enhancement study. However, PASSER II-84 predictions of delay reduction were somewhat higher than those predicted by NETSIM. From Chang's limited NETSIM evaluations, the greatest improvement was found in the arterial system performance instead of in the total system delay reduction. The arterial system delay was found to be reduced from 0 to 23 percent. As has been noted, Chang's study results may be different when the performance of various signal system operations are evaluated only on the total system basis. Therefore, this constrained TRANSYT-7F study might indicate different results if NETSIM evaluations were also made separately for both arterial directions and total arterial system. Results might also be different if this study began with the PASSER II-84 progression solution instead of with the MAXBAND solution.

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Authors' Closure

Chang's discussion mainly concerns the authors' discussion of the model used in PASSER II-84 to fine tune the offsets to provide further reductions of delay over that achieved by the centered bandwidth approach used in MAXBAND and PASSER II-80. In the authors' paper, three points were made concerning this model, all of which are disputed by Chang. Each of these three points will be discussed here.

1. "In adjusting offsets at a given intersection, only the effects on the intersections immediately upstream are included." This statement is essentially correct. Chang's model considers upstream intersec-

tions only insofar as the assumption that there is a green band that passes through them. His model does not take into account the possible changes in delay at up-stream intersections caused by fine tuning at a given intersection, provided that such fine tuning does not encroach the band. TRANSYT does take into account such changes.

2. "In the delay-offset model, the platoon structure is not modeled." This statement is essentially correct. Chang's model assumes that the platoon is rectangular in shape and that platoon dispersion can be modeled by assuming that the rectangular platoon increases in length and decreases in height uniformly as it travels down the arterial. Such a model is only a crude estimate of actual platoon shapes that are much more irregular and that disperse in nonuniformity and in irregular patterns as they traverse sections of roadway. The histogram-based platoon structure and exponential smoothing platoon dispersion model in TRANSYT give a substantially better description of actual platoon behavior than that found in PASSER II-84.

3. "No capability exists for making adjustments to green time while at the same time preserving the bands." In Chang's discussion, he argues that green time is adjusted during the bandwidth optimization procedure and that the green phase times computed by PASSER are better than those computed by MAXBAND. This statement is true, but irrelevant. One of the authors' basic conclusions was that, given the maximum, centered, two-way green band on an arterial, fine tuning of offsets alone does not on the average produce a statistically significant improvement in system delay. Chang argues that a different result might have been achieved had PASSER-II been used instead of MAXBAND. This is possible because the heuristic optimization technique used in PASSER does not produce the widest possible green bands, unlike MAXBAND, which guarantees a global optimum. Therefore, PASSER-II solutions will, in general, have larger amounts of slack green time available for fine-tuning adjustments, and thus more opportunities for delay improvements. This appears counterproductive to the intent of both the authors' approach and Chang's approach, which was to attain the lowest possible delay consistent with the widest possible green band.

Directional Weighting for Maximal Bandwidth Arterial Signal Optimization Programs

EDMOND CHIN-PING CHANG, CARROLL J. MESSER, and STEPHEN L. COHEN

ABSTRACT

The concept of maximizing two-way progression to compute signal-timing plans for signalized arterials has been used for 60 years. One of the unknown questions that exists is how the available two-way band should be apportioned between the two directions of traffic flow. Until now, the two directions have been weighted in proportion to the ratio of the average volume in each direction. However, preliminary studies have indicated that it would be better to apportion the two-way progression bandwidths than to use the volume-ratio criterion alone. Described is a bandwidth weighting algorithm that is based on delay. A simple delay model developed for the PASSER II program was used to estimate delay. Through extensive testing, using the NETSIM model on nine real-world arterial data sets, it was found that three different expressions for the bandwidth ratio should be used; which expression was to be used depended on whether the directional volume ratio was less than 0.45, between 0.45 and 0.55, or more than 0.55. All three expressions involve the ratio of delay in the two directions. A blind test was performed by using six scenarios based on two real-world arterials that were not included in the nine test arterials used for preliminary testing. Based on comparisons using the NETSIM model, the result of this blind test indicated that the weighting algorithm developed in this research generally performed better than both the arbitrary equal-weighting and the MAXBAND average volume-ratio criteria, which have been used up to now.

The concept of maximizing progression bandwidth as the criterion for calculating optimal offsets in arterial signal systems has been used for approximately 60 years. At first, graphic manual methods were used. With the introduction of the digital computer, the bandwidth optimization problem was computerized, and a number of programs were developed (1,2). Two of them, MAXBAND and PASSER II, also optimize the left-turn phase sequence (3,4). Both programs can weight the bands to provide a wider progression band in one of the two directions. Neither of them, however, provides any guidelines for adjusting the weighting factor other than to suggest setting it equal to the ratio of the average volumes in the two directions.

Recent feasibility studies conducted by the FHWA, U.S. Department of Transportation, have indicated that proportioning the total two-way bandwidth in the ratio of volume distribution does not provide the lowest systemwide delay. The FHWA feasibility study also indicated that the fundamental causal factors and general relationships existing between bandwidth ratio and delay could not be accurately predicted, based on current technology in arterial traffic signal-timing optimization.

STUDY OBJECTIVES

In this study, the factors for determining the best directional weighting for arterial bandwidth optimization were reviewed, performance of the factors that influence the directional weighting factors were compared, and an algorithm for future development was recommended.

Specifically, three objectives of the study were to

1. Determine the factors influencing the directional weighting factor;
2. Develop a single-pass algorithm to estimate the optimal band split before the original maximum-bandwidth calculations by either MAXBAND or PASSER II and provide proper directional bandwidth weighting in the bandwidth optimization; and
3. Apply and test the algorithm developed against the equal-directional weighting and the ratio of the sum of directional volume methods; independent testing of the algorithm was conducted by the FHWA for performance evaluation.

STUDY SCOPE

The following were performed during the study: a literature review, analytical analysis, algorithm development, algorithm demonstration, and computer runs of bandwidth optimization programs to determine the effectiveness of directional bandwidth weighting. Pretimed, common cycle, and coordinated traffic signals with multiphase control for arterial streets were emphasized during the research.

LITERATURE REVIEW

Traffic demand and traffic congestion along arterial corridors require effective traffic management to

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improve traffic flow. Computer programs for optimizing signal timing along arterial street systems came during the early 1960s with the coordinated offsets for maximum throughput (1-8).

During the 1964-to-1966 period, Little first developed the maximum progression bandwidth calculations along an arterial street (by computing offsets) for given cycle times, distances, and travel speeds (4,7). In 1966, Brook developed an algorithm that improved Little's program by developing a progression scheme that maximizes the total of the two-direction through bands over the cycle length for a set of offsets, cycle lengths, and link speeds (6). In 1967, Bleyl extended Brook's algorithm by selecting the offsets that minimize the total interference to the progression band (1). Messer developed the PASSER II program by expanding Bleyl's development. In 1975, Little further extended the maximum bandwidth optimization by formulating the signal synchronization problem as a mixed-integer linear program (7,9-12). Despite different methods currently available, which are based on delay, many traffic engineers still prefer maximum bandwidth settings because of the easily understood, time-space diagrams and the apparent favoring of progressive movement along major arterial street systems (11-13). In addition, the results of several studies [for example, Wagner (1969), Wallace (1979), Rogness (1981), Cohen (1983), and Chang (1984)] demonstrated that the bandwidth method does yield consistently good results on arterial signal systems (11-13).

Bandwidth Weighting Problem

Several computer programs that maximize bandwidth have been developed, including SIGART, SIGPROG, NO-STOP-1, PASSER II, and MAXBAND (2,4,7,9,14). Both PASSER II and MAXBAND allow the users to adjust the directional bandwidth split. They do not supply the best directional split, other than to suggest the use of the ratio of total through-traffic volume in each direction. However, it is not clear whether the simple proportionality of bandwidth ratio to volume ratio gives the signal settings with the lowest delay. Furthermore, factors such as capacity, green time, and available bandwidth in each direction are ignored. To demonstrate the directional bandwidth weighting problem, two examples provided by FHWA indicated that

1. The use of directional volume ratio to split the progression bandwidth may not give the solution with the lowest delay. For example, on Hawthorne Boulevard, the bandwidth was optimized using MAXBAND for east-west traffic having a volume ratio of 2 to 1. The resulting signal offsets were input into the NETSIM model. From the results tabulated (Table 1), it may be seen that use of the volume-ratio criterion would suggest a bandwidth ratio of 2 to 1, which would give 11 to 12 percent more delay than the ratios of 6 to 1 or greater.

2. The amount of bandwidth available in each direction on an arterial is limited by the duration of the shortest green interval in each direction. Thus, it could happen that it would be appropriate to favor the direction that has more green time available to progressive movements, regardless of the volumes. To demonstrate this directional bandwidth weighting concept, assume a two-directional arterial with one direction arbitrarily defined as the outbound or A direction and the other as the inbound or B direction. For example, assume a situation in which the shortest through-green time in the inbound direction is larger than the shortest through-green time in the outbound direction. If equal weighting is given for both directions, bandwidth available in each direction is limited to the shortest green time in the outbound direction. On the other hand, giving more weight to the inbound direction may result in a situation in which the inbound band equals the shortest inbound green time with the outband band being equal to the shortest outbound green time because both MAXBAND and PASSER-II optimize the weighted sums of the inbound and outbound bandwidth.

3. In the preceding example, if the inbound bandwidth becomes equal to the shortest inbound green time, any additional bandwidth available is given to the arterial outbound direction. For example, it can happen that the user uses a 4-to-1 ratio of inbound/outbound bandwidth, but the actual final ratio is less than 4 to 1 because of the inbound band filling the shortest inbound green. This effect is shown in Figures 1 and 2. The symbols "GREENIN" and "GREENOUT" represent the inbound and outbound green times. In Figure 1, inbound/outbound weighting was equal to 1; in Figure 2, inbound/outbound weighting was equal to

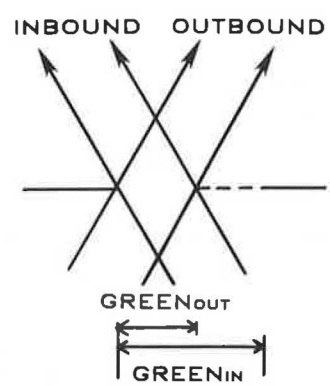

FIGURE 1 Equal bandwidth weighting.

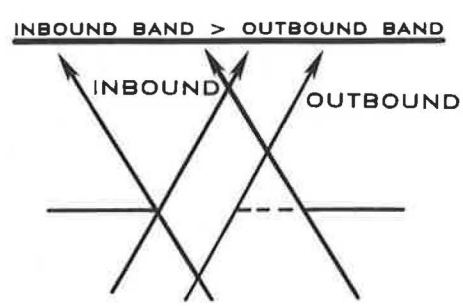

FIGURE 2 Inbound bandwidth weighting greater than outbound bandwidth weighting.

TABLE 1 Hawthorne Boulevard NETSIM Test Results

East-West Volume Ratio	East-West Band Ratios	Delay (sec/vehicle)	Optimal Deviation (%)
	1/1	80.73	26.9
2/1	2/1	71.44	12.3
	3/1	68.09	7.0
	4/1	67.07	5.4
	5/1	66.39	4.4
	6/1	63.61 (minimum)	0.0
	7/1	64.11	0.8
	8/1	64.06	0.7
	10/1	63.85	0.4

2. Figures 1 and 2 represent a special case in which the shortest green times occurred at the same intersection. However, they may occur at any intersection. This point can be further demonstrated by using the example of Foothill Drive, which is an eight-intersection arterial with left-turn lanes and left-turn phases at all intersections. The shortest inbound through-green time was 43 percent of the cycle, and the shortest outbound through-green time was 27 percent of the cycle. Optimization of offsets and left-turn phase sequence with equal directional weighting gave a bandwidth of 27 percent of the cycle in both directions; using an inbound/outbound band ratio of 3 to 1 gave an inbound bandwidth of 43 percent and an outbound bandwidth of 27 percent. The effect of Foothill Drive on delay is shown in Table 2.

TABLE 2 Foothill Drive NETSIM Test Results

In/Out Volume Ratio	Target In/Out Band Ratio	In Band (%)	Out Band (%)	Total Band (%)	Delay (sec/vehicle)
1.75/1	1/1	27	27	54	95.33
1.75/1	3/1	43	27	70	91.44

4. It is possible that the settings of shortest green time may have nothing to do with the volume ratios. For example, the volume ratio used for the Foothill Drive scenario was inbound/outbound = 1.75 to 1. One intersection may have an unusual requirement for a longer left-turn phase in the light-volume direction. This can further complicate the problem because of the arbitrary nature of and interactions between setting minimum green time and optimizing arterial bandwidth.

Traffic Signal Optimization and Simulation Programs

Computer techniques for off-line, fixed-time signal-timing plan optimization have received widespread interest. Two primary approaches for coordinating traffic signals along arterial streets are (a) the bandwidth-maximization procedure and (b) minimization of a disutility function such as delay, stops, fuel consumption, and air pollution. The former includes PASSER II and MAXBAND; the latter includes TRANSYT-7F as an example. Research by Huddert (1969), Wallace (1979), Rogness (1981), Cohen (1981), and Chang (1984) indicates the possibility of arriving at a compromise between the method of maximizing bandwidth and minimizing delay (using a stop penalty) in computing traffic signal progression (7,11-13,15,16).

PASSER II

PASSER (Progression Analysis and Signal System Evaluation Routine) is an acronym for a series of practical computer programs developed by the Texas Transportation Institute (TTI), Texas A&M University System. The PASSER II computer model was first developed by Messer and others and modified to an off-line computer program by Messer et al. (3,6,14). It was developed primarily for high-type arterial streets with modern eight-phase protected left-turn lanes and phases. The PASSER II maximum bandwidth solution has been well accepted and implemented throughout the United States. The theory, model structure, methodology, and logic in the PASSER II computer program have been evaluated and documented.

A recently concluded Highway Planning and Research study entitled "Reduced-Delay Optimization and Other Enhancements of PASSER II-80" was conducted by Chang at TTI to develop, compare, and evaluate the effectiveness of the enhanced PASSER II-84 program for an arterial street system by considering both the maximum bandwidth procedure and minimum delay signal-timing optimization algorithm (12,15,16).

MAXBAND

The original bandwidth formulations introduced by Little for setting traffic signals to achieve maximal bandwidth were developed into a portable, off-line, Fortran 77 computer program called MAXBAND. The program produces cycle time, offsets, speeds, and left-turn phase sequences to maximize bandwidth by applying Land and Powell's MPCODE branch-and-bound optimization algorithm (3,4,7,13).

In addition to arterials, the program can also handle a three-arterial triangular loop with arbitrary weighting of each arterial bandwidth. MAXBAND is currently being expanded by TTI to optimize small network problems.

TRANSYT

The TRANSYT computer program developed by Robertson (1969) can determine a set of phase splits and offsets that minimize a performance index given by a linear combination of stops and delays (11-13). The optimization procedure used by TRANSYT is a sequential flow analysis with a gradient search technique to minimize delay from subsequent simulation runs (13,15,16).

Regardless of the inability to analyze alternative phase sequences, TRANSYT has been widely accepted and is the common optimization computer program for analyzing arterial networks. The platoon dispersion model of TRANSYT has proven to be a good descriptor and predictor of platoon behavior. The optimized signal-timing plans determined by it have been found to give consistently better results than other existing optimization programs (11-13,15,16).

NETSIM

All of the signal-timing optimization programs incorporate evaluations for selecting an optimum solution, but most of them are limited to approximate measures of effectiveness (MOEs). The NETSIM simulation program developed by FHWA has been applied to relatively sophisticated network traffic signal control strategies and validated against field data; it has provided successful quantifiable comparisons in most applications (11-13,15,16).

Because of the complexity of performing field experiments, the NETSIM program was selected. The following assumptions common to arterial signal timing were made:

1. Volumes for each movement are constant over study period.
2. Platoon structure retains a coherent length.
3. Link speeds are uniform and known.
4. Queues are deterministic and of known length.

DIRECTIONAL WEIGHTING PROGRESSION

The directional weighting progression can be stated as a constrained offset optimization problem. This

offset optimization was made by developing a single-pass method to estimate the optimal directional progression bandwidth split before the original-progression calculation of either MAXBAND or PASSER II. This single-pass method will be executed to provide directional weighting factors for later progression calculations. This calculation before progression requires green split, traffic volume, and intersection spacings.

The main factors that can influence the single-pass directional weighting problem for MAXBAND and PASSER II may include

- Physical layout of signal system
- Traffic volume and travel speed
- Signal timing factors

Maximum progression solutions are based on green times and intersection spacings. The purpose of directional bandwidth weighting is to determine how much offset or extra green time should be added or deleted from the progression bandwidth in either the A or B direction. This problem differs from the slack green adjustment problem in which the location of the two-way band was adjusted to reduce overall delay without changing the amount of bandwidths in either direction. The slack green time is defined as the green time available for through movements but not used in the progression bandwidth solution. The offsets obtained by using the current slack green allocation algorithm in MAXBAND were used without modifications in the later examination.

Therefore, the directional bandwidth weighting problem can be reformulated into a constrained offset optimization problem, summarized as follows:

- Objective function:
  Maximize progression
  Minimize system delay
- Given:
  Cycle, green time, travel time
- Constraint:
  Total bandwidth
  Desirable progression speed
  Minimum green

The algorithm was a noniterative, one-shot precalculated method to predict the tradeoffs of adjusting directional bandwidth or the resultant offset changes. To minimize delay and stops, factors in addition to directional volume splits and minimum green times for progression movements are considered in this algorithm. The internally sensitive relationships of the estimated progression system delay and tradeoff of incremental delay changes as a function of bandwidth weighting factors are analyzed in the algorithm. These performance measures are derived from saturation flow ratio (relationship of volume and saturation flow rate) and travel time (relationship of distance and travel speed). In summary, the major study objective is to develop relationships between the progression bandwidths in the A and B directions as a function of saturation ratio and travel time. These relationships include the following:

1. Saturation ratio = (volume/capacity) * (cycle/green)
   a. Volume levels,
   b. Critical movement combinations, and
   c. Minimum green time combination.
2. Travel time = distance/travel speed
   a. Distances between intersections
   b. Existence of protected left-turn lane, and
   c. Desirable travel speed.

The other question concerned the effect of minimum greens, which was mentioned earlier. It was found that the MAXBAND and PASSER programs are formulated as follows: maximize the weighted sum of the A and B directional bands (the objective function) subject to a constraint on the ratio of the A and B bands. This formulation has the following consequence. For example, if the shortest green of the A direction is smaller than the shortest green of the B direction and the band of the A direction equals the shortest green of the A direction, this constraint may prevent the objective function from obtaining its optimum value. The answer is, in this case, to relax the constraint, which can be done most easily by giving (in this case) more weight to the B direction.

The factors that are important are determined in the best directional bandwidth weighting algorithm. Those factors were identified for determining the best directional bandwidth weighting. The factors include

- Intersection-specific demand volumes,
- Intersection-specific saturation flows,
- Green-time restrictions,
- Left-turn phase requirements,
- Saturation flow ratio, and
- Travel time between intersections.

An experimental plan for testing the effect of the directional factors by using NETSIM and MAXBAND was designed. The factors were selected for their relative sensitivity to the link weighting performance function, as defined earlier.

TESTING OF FACTORS

A detailed experimental plan was developed for testing the important factors in computing the best directional bandwidth weighting by using the MAXBAND and NETSIM programs.

The MAXBAND program was used to develop the timing parameters with directional weighting ratios varying between 1/10 and 10/1. It was also used to enumerate the timing parameters for all the possible directional weightings given by any reduced-delay algorithms. MAXBAND's capability of varying the directional bandwidth with two directional weighting factors was particularly useful in this investigation.

NETSIM was used to evaluate the relative importance of the respective factors that influence directional weighting. As indicated in Figure 3, MAXBAND was first executed to provide a MAXBAND-optimized solution for different directional weighting factors. Finally, NETSIM runs were made to vary the

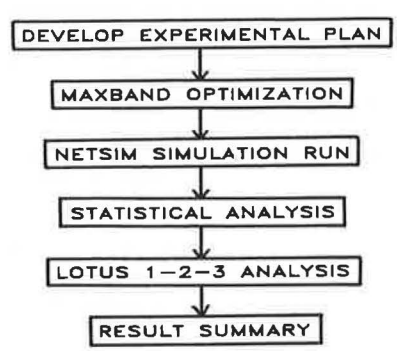

FIGURE 3 Experimental simulation plan.

TABLE 3 Summary of Experimental Simulation Study

Case	Street	No.	Cycle	Progression Band (sec) Out	Progression Band (sec) In	Efficiency	Attainability	Progression Speed (mph) Out	Progression Speed (mph) In	Total System (sec/vehicle) Delay	Total System (sec/vehicle) Stops	Arterial System (sec/vehicle) Delay	Arterial System (sec/vehicle) Stops
1	M Street	8	80	28.7	28.7	0.36	0.83	30	28	59.45	1.86	11.2	0.37
2	University Drive	10	80	10.7	10.7	0.13	0.52	29	30	43.44	1.58	28.7	0.27
3	33rd Street	11	75	7.1	7.1	0.10	0.37	35	35	80.52	1.66	190.7	0.55
4	Broadway Avenue	11	60	21.4	21.4	0.36	0.69	37	37	39.93	1.48	41.6	0.31
5	Nicholasville Road	12	80	18.5	18.5	0.23	0.54	33	34	82.30	2.32	18.1	0.56
6	Frederica Street	12	80	23.1	23.1	0.29	0.59	40	41	77.01	2.48	84.5	0.45
7	South Halsted Street	12	65	16.3	16.3	0.25	1.00	29	30	88.91	1.52	129.2	0.47
8	Fannin Boulevard	15	80	24.1	9.8	0.21	0.61	36	34	66.67	2.38	92.0	0.38
9	Wisconsin Avenue	17	70	22.4	22.4	0.32	0.72	31	33	77.38	2.75	60.2	0.45

Note: Combined MAXBAND and NETSIM runs with variable speed (base case).

relative offsets of the different MAXBAND-optimized solution, thus evaluating the system performance to the solution alternatives.

Nine arterial networks, which were previously coded for MAXBAND and NETSIM, were selected to test the factors identified. Concentrated efforts were made to check that all of the possible data requirements were satisfied by the test arterials selected. Summarized in Table 3 are the basic characteristics of the nine test arterials. Because MAXBAND can provide variable link speed, both the variable speed and fixed speed options were used to provide flexibility in generating the maximum bandwidth solution. The interlink and intralink speeds were constrained to be within 10 percent of the original link design speed for the given cycle length and phase sequence.

The NETSIM analysis was simulated for a half-hour study period. The optimized MAXBAND offsets were generated from the inbound-versus-outbound bandwidth weighting factors varying from 1 versus 10 to 10 versus 1 at increments of 1. Thus, a total of 19 cases were made for each test arterial to represent the variations of bandwidth weighting factors. Two replicated runs with the same MAXBAND timing plan were made to reduce the statistical variability that may be produced in the NETSIM microscopic simulation environment. In other words, MOEs, delay, and so forth were averaged over both replications for each test case.

Experimental Simulation Plan

Simulation and statistical analyses were conducted to determine which factors are best suited for the evaluation of the directional bandwidth weighting problem. Major activities in this study task were to (a) modify the test case data and (b) test the factors affecting directional weighting. At first, the data coding was transformed to the combined link-node coding scheme for MAXBAND and NETSIM runs. Efforts were made such that adequate and compatible data inputs were available with the computer models used. After the data were modified, pilot simulation runs were made to establish a base for later comparisons. Then the modified NETSIM data sets were used to determine the effects of the various factors identified. The Statistical Analysis System (SAS) (17) was used to provide the basic descriptive statistics for the following questions:

1. What are the factors that have significant influence on the directional weighting factor?
2. How sensitive are the factors that influence the directional weighting factor?
3. What are the basic factors that are required for a single-pass preprocessor algorithm for determining the directional bandwidth split that will lower systemwide total delay?

The data collected in the simulation were then evaluated by the SAS to determine the relative importance of the factors to be put into the algorithm. Also examined by the SAS was whether the enhanced directional weighting factors could provide better combinations of reduced-delay offsets and directional bandwidth. Because the initial green split, phase sequence, and offsets between intersections were given, the evaluation focused on MAXBAND offset optimization capability using different directional bandwidth weighting ratios. The major independent variables considered were the weighting ratios, the relative offsets between the consecutive intersections, and the resultant arterial delay.

The NETSIM evaluation can provide microscopic link-to-link statistical simulation and analysis, but the output is difficult to compare except on a total systemwide basis. To study in detail the effects of various directional weighting factors on the NETSIM system performance from the nine test cases, the arterial portions of the MOEs were also separated from the side streets. Because the MOEs on the side streets are unaffected by the offsets, this detailed analysis reduced the variability of the study results. A detailed SAS analysis was summarized from the NETSIM evaluation and then downloaded and displayed through the popularly used LOTUS 1-2-3 microcomputer program. This process is shown in the flowchart in Figure 4.

Test Results

Evaluations of delay and stops were performed for all nine test cases for both the whole system and separated arterial travel directions. These evaluations were analyzed by average NETSIM system delay and stops, average NETSIM arterial delay and stops, and average NETSIM arterial inbound-versus-outbound directional delay ratio versus directional bandwidth weighting ratios.

The typical NETSIM simulation results of the average volume ratio, delay, and stops MOEs are shown in Figures 5 and 6 by the test arterial of Broadway in Lexington, Kentucky (Case No. 4). These two figures show the NETSIM average system delay and stops (y-axis) versus the different directional weighting values (x-axis) for the inbound, outbound, and average outbound and inbound travel directions. Table 4 gives the different directional weighting ratios used in the analysis.

The results of this testing-of-factors analysis indicate the following:

1. Directional weighting can substantially affect the arterial system delay and stops, according to the analysis using NETSIM simulation.
2. The variable speed options in MAXBAND are

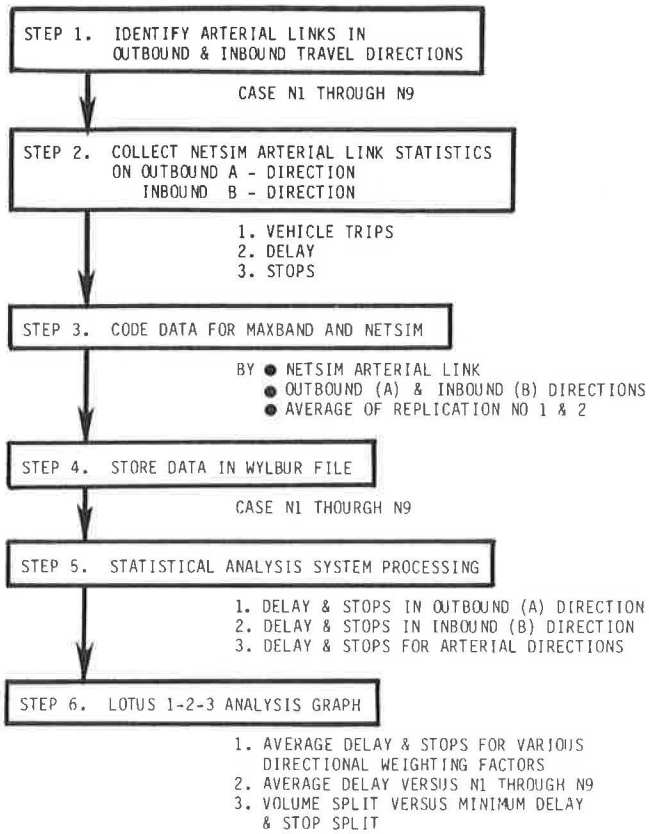

FIGURE 4 Detailed arterial analysis plan.

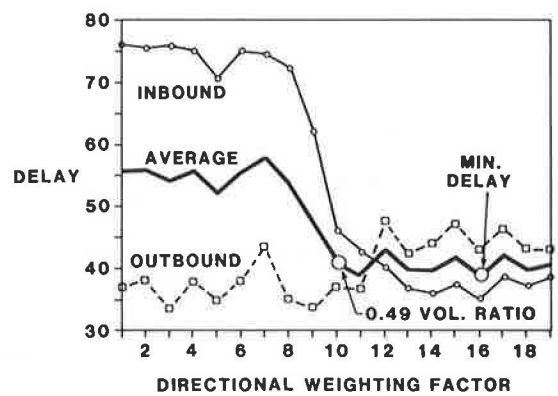

FIGURE 5 NETSIM average arterial delay study.

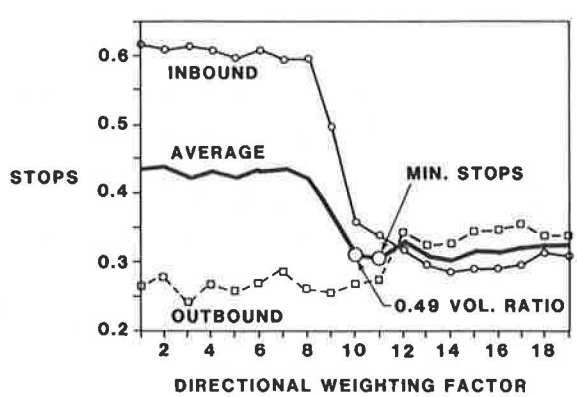

FIGURE 6 NETSIM average arterial stops study.

TABLE 4 Relationships Among MAXBAND Inbound and Outbound Weighting, Inbound-Versus-Outbound Ratio, and Normalized Directional Bandwidth Ratio

No.	Inbound Weight	Outbound Weight	Ratio of In-Out	Normalized Ratio [in/(in + out)]	
1	1	10	1/10	0.091	(weight heavy in outbound direction)
2	1	9	1/9	0.10	
3	1	8	1/8	0.111	
4	1	7	1/7	0.125	
5	1	6	1/6	0.143	
6	1	5	1/5	0.167	
7	1	4	1/4	0.20	
8	1	3	1/3	0.25	
9	1	2	1/2	0.333	
10	1	1	1/1	0.50	(equal weight)
11	2	1	2/1	0.667	(weight heavy in inbound direction)
12	3	1	3/1	0.75	
13	4	1	4/1	0.80	
14	5	1	5/1	0.833	
15	6	1	6/1	0.857	
16	7	1	7/1	0.875	
17	8	1	8/1	0.889	
18	9	1	9/1	0.90	
19	10	1	10/1	0.909	

powerful in producing various directional weighted bandwidths for given allowable through-green times and minimum green times.

3. The directional volume ratio alone cannot provide an accurate estimation of the minimum NETSIM delay directional weighting ratio. This is particularly true when

a. Inbound and outbound arterial volumes are nearly equal,
b. Link-volume ratios between intersections are inconsistent, and
c. Amounts of available through-green times are different due to the constraints of intersection-specific green splits.

In the example in Figure 5, the directional volume ratio [INBOUND/(INBOUND + OUTBOUND)] is 0.49, which indicates the lower volume level in the inbound direction. However, the NETSIM evaluation both in the system and arterial directions indicated that a higher inbound weighting could provide less delay and fewer stops. This suggested that a separate indicator, such as the directional delay ratio, should be used in estimating the likely lower-delay directional bandwidth ratio.

4. The possible reductions in delay and stops may range between 1 and 10 percent for various directional weighting ratios used according to the nine test cases studied. The absolute magnitude of improvements, that is, the reductions of delay and stops, may sometimes have less practical value because of the small amount of average delay reductions. Evidently, cases also exist that are insensitive to the arterial progression bandwidth ratio.

ALGORITHM DEVELOPMENT

A single-pass algorithm was developed to determine the best bandwidth weighting. It is compatible with the existing input data required in the current MAXBAND and PASSER II programs. Basically, the directional bandwidth weighting algorithm developed by TTI is a simplified aggregated platoon projection model that is similar to the platoon dispersion model used in the TRANSYT-7F program. This simplified model predicts the aggregated platoon travel behavior on a link-to-link basis for the given volume levels, saturation flow rates, MAXBAND-calculated green

splits, and estimated travel times between intersections.

The system inputs of the directional bandwidth weighting algorithm include the following (18,19):

- Intersection-specific demand volumes
- Intersection-specific saturation flows
- Green-time restrictions
- Left-turn phase
- Travel time between intersections, computed from speeds

The system outputs include the following:

- Directional INBOUND/(INBOUND + OUTBOUND) volume ratio
- Directional INBOUND/(INBOUND + OUTBOUND) delay ratio
- Directional inbound-versus-outbound bandwidth weighting ratio or the target bandwidth ratio for use in the MAXBAND optimization runs

Theoretical Background

An interconnected signal system can result in nonuniform flow rates during cycles. If progression between signals is good, most of the traffic will arrive at the downstream intersection during the green phase of the signal. This phenomenon results in an average arrival rate during the green phase of the cycle that is greater than the average arrival rate during the red phase. On the other hand, poor progression could result in a greater arrival rate during the red phase than during the green phase. To estimate the arterial signal system performance under interconnected operations, the tentative NCHRP delay equation was modified and used in this study (6,15,16,20). The primary interest is to consider both the interactions between percent of the approach's through volume arriving on the through green and the percent of available through-green time during the whole cycle length.

The percent of an approach's through traffic coming from the through-traffic movement of an adjacent upstream intersection and arriving during the through green at the downstream intersection depends on several factors. Three principal factors considered are

1. The percent of the total through traffic in the progression platoon,
2. The size of the platoon and the rate of platoon dispersion, and
3. The quality of arterial progression between the intersections.

The optimal arterial progression time-space diagram can be used to determine the quality of progression between the intersections. A good progression system would result in better usage of the green time than would a bad progression system. The through green available for progression was used to predict the estimated arterial directional delay due to the combined effects of given MAXBAND green splits and link travel times. It was also applied with the percent through traffic and platoon dispersion factor to estimate the minimum-delay directional bandwidth weighting ratio.

This simplified platoon projection model was applied in the PASSER II model to estimate arterial delays for evaluating arterial signal system operations. The previous detailed NETSIM analysis made by Chang (1984) indicated a consistent trend between the NETSIM simulated delay and the delay predicted by this simplified platoon model (16). The NETSIM analysis in this study also indicated that a reduced-delay directional weighting algorithm could be developed from this algorithm.

Program Structure

Figure 7 shows the overall structure of this single-pass directional bandwidth weighting algorithm. The system consists of five major modules: an input module, a platoon projection module for outbound and inbound travel directions, a cumulative delay and stops estimation module, a directional delay and stops calculation module, and a directional bandwidth weighting estimation module.

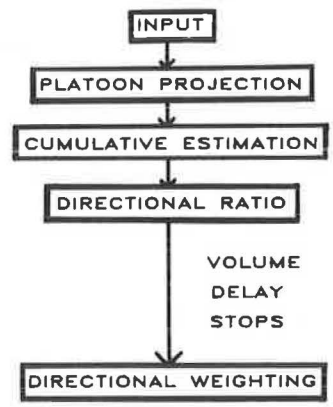

FIGURE 7 Program structure of the directional bandwidth weighting algorithm.

Input Module

The input module reads the input data from the data stored in the MAXBAND array or from a temporary card image file on FORTRAN file 5. The only conversion required from the ordinary MAXBAND input deck is to assume that the same upstream arterial volume levels and saturation flow rates exist for the downstream intersection, if a SPECIFY card was used rather than VOLUME and CAPACITY cards. This additional input provides consistent information for (a) estimating the saturation flow ratio and (b) calculating the aggregated directional delay and stops for the given inbound-versus-outbound directional bandwidth weighting ratio.

Platoon Projection Module

Essentially, the platoon projection module predicts the link-to-link progression platoons for the given volume, saturation flow rate, green time, and travel time between intersections. First, the module calculates the green time needed to clear the standing queue and transforms these values into an equivalent progression platoon size for estimating the progression through bandwidth leaving that particular intersection. Then a platoon dispersion factor is calculated based on results of a previous TTI field study to predict the downstream platoon size (6).

The single-pass method was used before the MAXBAND optimization process. A ratio of the estimated travel time and cycle length was used in place of the offset for time-based signal coordination. Two separate sets of analyses are made to estimate the platoon projection adjustment factor for the through-arterial

movements at each traffic signal with respect to both outbound and inbound arterial travel directions.

Cumulative Delay and Stops Estimation Module

This module calculates and accumulates the estimated delay and stops information by using a modified tentative NCHRP delay equation. An enhanced Akcelik stops estimation equation (20) similar to those used in the PASSER II-84 model was applied. This analysis is performed on an intersection-by-intersection basis. The cumulative delay and stops information is stored separately for outbound and inbound travel directions.

Directional Volume, Delay, and Stops Calculation Module

After accumulating the volume, delay, and stops information for the arterial through movements at every intersection, this module calculates the [INBOUND ÷ (INBOUND + OUTBOUND)] volume, delay, and stops ratios for the arterial inbound and outbound travel directions.

Directional Bandwidth Weighting Estimation Module

The last module of the directional bandwidth weighting algorithm applies the normalized directional bandwidth ratio, as shown in Table 4. This ratio transfers the [INBOUND/(INBOUND + OUTBOUND)] ratio into the MAXBAND-type inbound-versus-outbound bandwidth weighting ratio or the target bandwidth ratio (K). Because the algorithm was developed mainly based on the NETSIM analysis of the nine test cases at integer weighting, the same discrete weighting analysis was made for additional verification.

Examination of the characteristics of the nine test cases indicated that for three ranges of values of the average volume ratio, three distinct expressions should exist for the bandwidth ratio as a function of the directional delay ratio. These three ranges were

1. $0.0 \leq$ volume ratio < 0.45
2. $0.45 \leq$ volume ratio ≤ 0.55
3. $0.55 <$ volume ratio ≤ 1.0

The resulting value for the bandwidth ratio [INBOUND/(INBOUND + OUTBOUND)] is then converted into the ratio INBOUND/OUTBOUND, which is then rounded to a fractional integer ratio between 1/10 and 10/1 for direct use in the MAXBAND program. Figure 8 shows a description of this process in pseudocode.

ALGORITHM DEMONSTRATION

At the completion of the algorithm, the FHWA supplied a total of six scenarios for an independent evaluation of the directional bandwidth weighting algorithm. These test scenarios consist of three traffic patterns for each of two networks not selected for

```
{ SELECT INBOUND VERSUS OUTBOUND WEIGHTING RATIO FOR MAXBAND OPTIMIZATION RUN }
{ DIR.VOL.RATIO = (INBOUND / (INBOUND + OUTBOUND) )  VOLUME RATIO }
{ DIR.DLY.RATIO = (INBOUND / (INBOUND + OUTBOUND) )  DELAY  RATIO }

  IF DIR.VOL.RATIO > 0.55 THEN              { IF INBOUND VOLUME IS HEAVY }
    BEGIN
      INBOUND=ROUND(0.35+SQRT(DIR.DLY.RATIO)/(1-SQRT(DIR.DLY.RATIO)));
      OUTBOUND=1;
    END
    ELSE
    BEGIN
      INBOUND=1;
      IF DIR.VOL.RATIO < 0.45 THEN          { IF OUTBOUND VOLUME IS HEAVY }
        BEGIN
         OUTBOUND=ROUND(0.35+(1-SQRT(DIR.DLY.RATIO))/SQRT(DIR.DLY.RATIO));
        END
      ELSE
        IF DIR.DLY.RATIO > 0.50 THEN        { IF VOLUME IS ABOUT EQUAL }
          BEGIN
            INBOUND=ROUND(0.35+SQRT(DIR.DLY.RATIO)/(1-SQRT(DIR.DLY.RATIO)));
            OUTBOUND=1;
          END
        ELSE
          BEGIN
             INBOUND=1;
             OUTBOUND=ROUND(0.35+(1-SQRT(DIR.DLY.RATIO))/SQRT(DIR.DLY.RATIO));
          END
      END;

      { OUTPUT INBOUND AND OUTBOUND BANDWIDTH WEIGHTING FACTORS }

      WRITELN(OUTFIL, 'INBOUND BANDWIDTH WEIGHT = ',INBOUND,
                      'OUTBOUND BANDWIDTH WEIGHT = ',OUTBOUND);
```

FIGURE 8 Identification of the structure of the directional bandwidth weighting algorithm.

testing. The algorithm developed was used to determine the best bandwidth weighting factor for each test scenario. The FHWA then used the NETSIM model to compare the bandwidth weighting ratio, computed from the algorithm developed, with both the equal weighting ratio and MAXBAND volume weighting ratio.

Test Cases

The two test cases were the high-type arterial Hawthorne Boulevard and the low-type arterial North Michigan Avenue. Both arterials have 13 intersections and variable spacings. The six test scenarios included the existing traffic pattern and two modified traffic patterns. All scenarios were undersaturated. The FHWA provided TTI with the MAXBAND data listings and the MAXBAND outputs with fixed cycle length and the MAXBAND-computed green splits. This base case was made with equal directional weighting in MAXBAND. The FHWA then performed the subsequent NETSIM evaluations by using the output calculated from the MAXBAND volume weighting and the directional weighting factors supplied by TTI.

These test cases were performed with four replications and 30-min study periods in each case. The results of the NETSIM delay, stops, and combined delay and stops NETSIM performance index (PI) were compared statistically by the FHWA. The NETSIM PI used is the same weighted sum of delay and stops as is used in TRANSYT. A weighting of 4 for stops was used. A subsequent analysis was made to examine the differences in using equal weighting, MAXBAND volume weighting, and the TTI calculated weighting. The SAS analysis of variance (ANOVA) was used with the DUNCAN and Student-Newman-Keuls options to evaluate the statistical differences (17).

Test Results

Table 5 shows the average PI during the four replications for each case and each bandwidth weighting method, that is, the equal bandwidth weighting (E or EQ), MAXBAND volume weighting (M or MX), and the TTI-calculated directional bandwidth weighting method (T or TTI). The SAS evaluation results of this algorithm demonstration for the six test cases are given in Table 6. Results of this study indicated the following:

1. Directional weighting can effectively improve the arterial street performance as indicated by the NETSIM-simulated delay, the stops, and the combined delay and stops (PI) evaluations.
2. The directional weighting algorithm developed by TTI provided better weighting than did equal weighting, and most of the time did better than the MAXBAND volume weighting methods.

TABLE 5 Summary of NETSIM Evaluation of Six Test Cases

Test Arterial	No.	NETSIM Performance Index		
		EQ(E)	MAX(M)	TTI(T)
Hawthorne Boulevard	H1	253.9	252.6	239.6
	H2	249.6	245.1	244.7
	H3	272.4	268.5	271.8
North Michigan Avenue	M1	180.4	163.0	160.9
	M2	152.1	153.3	146.2
	M3	166.0	161.1	159.8

Note: The number in each cell identifies the NETSIM performance index calculated by combining the NETSIM-simulated delay and stops (PI = delay + 4 * stops).

TABLE 6 NETSIM Evaluation of Six Test Cases

Test Arterial	No.	Delay	Stops	NETSIM Performance Index
Hawthorne Boulevard	H1	E,M,T	E,M,T	E,M,T
	H2	E,M.T	E,M.T	E,M.T
	H3	E,T,M	E,T.M	E,T.M
North Michigan Avenue	M1	E,M.T	E,M,T	E,M.T
	M2	M.E,T	E.M,T	M.E,T
	M3	E,M.T	E.M,T	E,M.T

Note: Results are from SAS ANOVA test using the Duncan option at the 5 percent significance level. The number in each cell identifies the grouping calculated by the SAS ANOVA Duncan test, with the comma separating each group level with significant statistical difference. Groups separated by a period were not significantly different. The ranking of the mean values is arranged from the highest value to the lowest value, reading from left to right. For example, "E,M,T" means that equal weighting and MAXBAND weighting were not significantly different but both were significantly different from TTI weighting. Both the Duncan and Student-Newman-Keuls tests were performed with the same results for all six test cases.

CONCLUSIONS AND RECOMMENDATIONS

Based on the results of this study, the following conclusions can be drawn:

1. Directional weighting can effectively improve the arterial street performance as indicated by the NETSIM simulated delay, the stops, and the combined measure of delay and stops (PI).
2. The directional weighting algorithm developed by TTI provided better weighting than did equal weighting. It also indicated that it often performed better than the MAXBAND volume weighting methods.
3. Because of the inherent NETSIM simulation variations and the complexity of different variables involved, the difference between various directional weighting methods indicated that practical improvements existed quantitatively but sometimes not statistically.
4. As suggested by the algorithm, weight heavily the progression bandwidth for the high-volume direction if the directional volume difference is higher than 20 percent. If the difference of directional volume is within 20 percent, the ratio suggested by the estimated delay ratio from the algorithm should be used.

It is recommended that

1. The algorithm should be programmed and implemented into MAXBAND or PASSER II programs between the initial green-split module and the progression optimization module.
2. Future modification of the progression project module should be made to include the upstream side-street left-turn traffic impacts into the progression effects on downstream intersections.

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Investigation of Optimal Time to Change Arterial Traffic Signal-Timing Plan

BASIM K. JREW and PETER S. PARSONSON

ABSTRACT

The objective of this study was to use several off-line computer programs to provide a technique for determining the optimal afternoon time during which to change the off-peak timing plan to the peak-period timing plan for a particular Atlanta arterial. The study made use of PASSER II-80 (an arterial-optimization model), TRANSYT-7F (a model for optimizing arterials or grids), and SOAP/M (an intersection-optimization model). An attempt to establish different optimal timing plans for the off-peak hour (1:00 to 2:00 p.m.) and the peak hour (5:00 to 6:00 p.m.) using PASSER was unsuccessful because it was found that both hours required the same cycle length. The TRANSYT optimization program produced different cycle lengths for the two hours. The authors adjusted these cycle lengths to 85 sec for the off-peak hour and 115 sec for the peak hour so that there would be a clear superiority of one over the other at each of the two times of day. Twenty TRANSYT simulation runs were then performed forward in time, from 1:00 to 6:00 p.m., by using the off-peak optimal timing plan and the volumes for each 15-min period. Another 20 TRANSYT simulation runs were performed backward in time, from 6:00 to 1:00 p.m., by using the peak-hour optimal timing plan and the volumes for each 15-min period. The two plots of performance index versus time of day intersected at 4:15 p.m., the optimal time to change plans. The TRANSYT-oriented procedure involved considerable effort and computer time. It was theorized that the TRANSYT procedure might be replaced by a relatively simple SOAP/M analysis of only the critical intersection. However, it was found that at all times during the afternoon the off-peak cycle length had a lower traffic performance index; therefore, the SOAP/M analysis failed to produce an optimal time to change the plan.

The nature of the study was to use several off-line computer programs to determine the optimal afternoon time during which to change from the off-peak timing plan to the peak-period timing plan.

The City of Atlanta is expanding the city's existing computerized traffic control system to include signals on Piedmont Road. The seven intersections from Lakeshore Drive to E. Wesley Road were selected for the study. This study was undertaken to assist the City of Atlanta in implementing optimal signal-timing plans for both off-peak and peak periods, and was intended to develop a technique for changing the timing plan from one to another during various periods of the day.

SUMMARY OF PROCEDURE

The off-line computer programs PASSER II-80, TRANSYT-7F, and SOAP/M were applied to the Piedmont Road study route.

PASSER II-80 is used to determine optimum progression along an arterial street (1). This program can optimize the cycle length. PASSER can also optimize phasing, but the city preferred that no attempt be made to change the phasing from that now existing on Piedmont Road. It does not change with time of day.

TRANSYT-7F is used to optimize a coordinated signal system to reduce stops, delay, and fuel consumption (2). The program optimizes phase lengths and offsets of the coordinated traffic signals in order to minimize a traffic performance index (PI), which is a linear combination of stops and delay. The program consists of the following two traffic models: first, a simulation model takes preliminary signal timings and determines the before PI; then an optimization model makes changes to the signal timings until the PI is minimized.

SOAP/M is a microcomputer version of the Signal Operation and Analysis Program (3). The program is widely used to evaluate and optimize intersection performance in terms of stops, delay, and fuel consumption.

The research plan was first to use PASSER to determine the optimal timing plan for off-peak traffic from 1:00 to 2:00 p.m. and the optimal plan for peak-hour traffic from 5:00 to 6:00 p.m. It was expected that the two plans would be different, especially in cycle length, because of the heavier traffic volumes during the peak hour.

Next, it was intended that the off-peak plan be used as input to the TRANSYT-7F program and that a simulation run using the traffic volumes for each 15-min interval be performed. The optimal off-peak plan (1:00 to 2:00 p.m.) would be run forward in time, that is, using the increasing volumes from 1:00 to 6:00 p.m. A plot would be prepared for PI versus time of day. Then, the optimal peak-hour plan (5:00 to 6:00 p.m.) would be run backward in time, that is, using the decreasing volumes from 6:00 to 1:00 p.m., and a second curve would be plotted. The intersection of the two curves was to be the optimal time during which to change from one plan to another during the afternoon.

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Finally, SOAP/M was to be run forward and backward in a similar fashion, for only the critical intersection; it was hoped that this simpler procedure would point to approximately the same time of day to change plans as was indicated by TRANSYT.

The results did not completely meet expectations. First, the PASSER runs pointed to a single optimal cycle length for both the off-peak hour and the peak hour. Because the purpose of the project was to investigate the change from one plan to a different one, the first step was repeated using the TRANSYT program. Two different optimal cycle lengths resulted, but the PI of the shorter was not significantly superior to that of the longer during the off-peak hour. It was decided to increase the longer cycle length; thus shorter cycle length was significantly better than the longer one during the off-peak hour and the longer was definitely better than the shorter during the peak hour. This set of timing plans appeared to furnish a base for further steps to determine the optimal time to change from one timing plan to another.

As planned, 20 TRANSYT simulation runs were then performed forward in time from 1:00 to 6:00 p.m. by using the off-peak plan. The volumes for each run were the 15-min values for that time of day multiplied by 4 to give equivalent hourly volumes. Another 20 runs were performed backward in time from 6:00 to 1:00 p.m. by using the peak-hour plan. The two curves of PI versus time of day crossed at 4:15 p.m., a reasonable outcome.

SOAP/M was used to determine stops and delay at the critical intersection of the arterial system. Ten optimization runs were performed using the off-peak cycle length at 30-min intervals from 1:00 to 6:00 p.m. The volume for each run was the 30-min volume for that time of day multiplied by 2 to give an equivalent hourly volume. Another 10 optimization runs were performed by using the peak-period cycle length at 30-min intervals from 6:00 to 1:00 p.m. Again, the volumes used were for each 30-min time of day. In an attempt to show the optimal time to change plans, a plot of PI versus the time of day was prepared. It was hoped that this approach could eliminate the need to run TRANSYT-7F, thereby reducing the analysis effort. It turned out that a single plan was optimal for the entire period from 1:00 to 6:00 p.m.; thus, in this study the critical intersection could not successfully represent the arterial for this purpose.

DATA COLLECTION

The study procedure included data collection, which is common to all of the programs, data preparation, and network coding of the input data for each individual program. The traffic data were collected in the afternoon during off-peak and peak periods (1:00 to 6:00 p.m.) at all of the seven intersections.

Five major types of data were collected for use in the three programs. Each of these types will be described.

Network Data

The field measurements and the node-link identification scheme are shown in Figure 1. The sketch shows the geometrics of each intersection, number of approach lanes, lane width, node number, link number, and the link distances.

Traffic Volume Data

Two types of traffic volume data were needed: (a) the city-furnished 24-hr machine-count volume data used to determine the time period during which a given timing plan should be in operation; and (b) the turning movement volumes for the approaches to each signalized intersection, obtained every 15 min from 1:00 to 6:00 p.m.

Saturation Flow Data

Headway samples were collected to calculate saturation flow rate for each major link, following standard TRANSYT-7F procedures (2). When a signal turned green, the headway was sampled beginning with the third vehicle of the queue as the platoon discharged.

Speed Data

Both the floating-car method and a radar gun were used to collect speed data at the major links in the network system. Free-flowing traffic was sampled; speeds were not affected by the downstream signal.

Signal Phasing

The existing phase sequences were inventoried and used as input to the computer programs. On Piedmont Road, the phasing does not change with time of day. The phasing was held constant in all computer runs; PASSER was not allowed to optimize the phasing.

Much of the input data prepared for the PASSER II-80 program is similar to that prepared for TRANSYT-7F. Intersection distances, progression speeds, allowable cycle lengths, turning movement volumes, saturation flow rate, and minimum phase duration are used in both programs. The link input volume, lost time, and green extension data were prepared specifically for the TRANSYT-7F program.

FINDINGS

The findings of the study were based on the output of the three programs mentioned earlier.

Passer II-80 Output Results

The output of the PASSER II-80 program consists of three printed reports and printer plots. The first report is simply a listing of the input data as submitted to the computer. The second report includes guidelines for minimum and maximum cycle lengths for each intersection. The third report presents the best solution for signal timing at each intersection in the coordinated system. The printer plot of the time-space diagram shows the uniform bandwidth and the speed of the progression for both directions.

An evaluation of the cycle length was performed to determine the best solution optimum progressions for the off-peak (1:00 to 2:00 p.m.) and peak periods (5:00 to 6:00 p.m.) for Piedmont Road. The cycle length ranged from 60 to 120 sec for off-peak-period evaluation and from 100 to 120 sec for peak-period evaluation. The smallest permissible cycle length was selected as 85 percent of the largest individual cycle length (1). The maximum cycle length was taken as 120 sec (1).

Seven PASSER runs were performed to select the off-peak cycle length. The results are given in Table 1. The best solution for the off-peak cycle length was found to be 110 sec with a 39 sec uniform bandwidth for both directions and an average arterial delay of 15.35 sec per vehicle. The progression speeds were found to be 41 mph for Direction A (northbound) and 42 mph for Direction B (south-

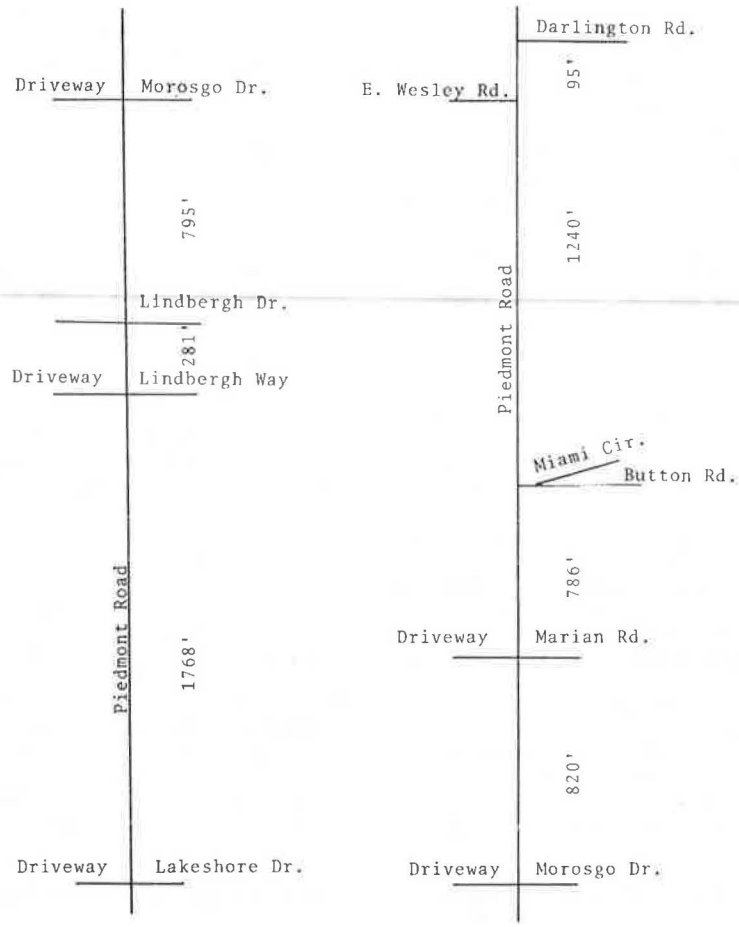

FIGURE 1 Field measurements and the node-link identification scheme.

bound). The results indicated an increase in the average delay and the percentage of efficiency as the cycle length increased.

Another two PASSER runs were performed to permit evaluation of the peak-period cycle length. The resulting outputs are given in Table 2 as Runs 1 and 2. The best solution for the peak period was found to be 110 sec with a 44 sec uniform bandwidth in both directions. The average arterial delay was 16.66 sec per vehicle. The progression speeds were 40 mph for the northbound direction and 41 mph for the southbound direction. (Run No. 3 in Table 2 is explained later in the paper.) The two tables show that the same cycle length was selected for both periods of time, but with different signal-timing plans and different speeds of progression. The off-peak signal-timing plan provided a higher speed, a lower percentage of efficiency, and lower average delay than did the peak signal timing plan.

That PASSER did not select different cycle lengths for the two traffic-analysis periods presented an obstacle to further work to determine the optimal time to change timing plans. Therefore, cycle lengths were investigated further using TRANSYT-7F.

TRANSYT-7F Output Results

Five basic outputs are available from the TRANSYT-7F program:

- Input data report
- Traffic performance table
- Flow profile plots
- Signal-timing table
- Time-space diagram

TABLE 1 PASSER II-80 Output Results for Off-Peak Hour

			Band A		Band B				
Run No.	Cycle Range (sec)	Best Solution (sec)	Time (sec)	Speed (mph)	Time (sec)	Speed (mph)	Efficiency (%)	Total Delay (sec)	Average Delay (sec/veh)
1	60-75	75	17	44	17	45	24	298,163.93	12.51
2	60-80	80	21	44	21	45	27	301,086.61	12.63
3	60-85	85	25	44	25	45	29	306,706.53	12.87
4	60-90	90	28	44	28	45	32	319,358.76	13.40
5	60-95	95	31	44	31	45	34	333,242.40	13.98
6	60-110	110	39	41	39	42	36	365,822.66	15.35
7	60-120	110	39	41	39	42	36	365,822.66	15.35

TABLE 2 PASSER II-80 Output Results for Peak Hour

Run No.	Cycle Range (sec)	Best Solution (sec)	Band A Time (sec)	Band A Speed (mph)	Band B Time (sec)	Band B Speed (mph)	Efficiency (%)	Total Delay (sec)	Average Delay (sec/veh)
1	100-110	110	44	40	44	41	40	518,716.33	16.66
2	100-120	110	44	40	44	41	40	518,716.33	16.66
3	115-120[a]	115	46	40	46	41	40	564,147.03	18.12

[a]An additional run to provide a timing plan for C = 115 sec was needed for TRANSYT-7F.

The TRANSYT-7F program was used in two stages. During the first stage, the optimal cycle lengths for the off-peak (1:00 to 2:00 p.m.) timing plan and the peak (5:00 to 6:00 p.m.) timing plan were determined. During the second stage, the optimal time to change from one timing plan to another was determined, considering the entire period from 1:00 to 6:00 p.m.

First-Stage Evaluation

The optimization process of the TRANSYT-7F program was performed for cycle lengths of from 75 to 120 sec. Ten runs were performed for the off-peak hourly volume (1:00 to 2:00 p.m.) and another 10 runs for the peak hourly volume (5:00 to 6:00 p.m.). The output timing data of PASSER II were used as input timing for all of the TRANSYT-7F runs.

Tables 3 and 4 give summaries of the TRANSYT-7F output. The optimal cycle length for each hour was selected to be the one with the lowest PI.

Tables 3 and 4 show that the cycle length that produced the lowest PI was 85 sec for the off-peak hour and 100 sec for the peak hour. The 85-sec cycle produced a PI of 115.94 during the off-peak hour and a higher value, 195.10, during the peak hour. The 100-sec cycle produced a PI of 170.48 during the peak hour. The PI of this cycle was only 117.44 during the off-peak hour.

A plot of PI versus time of day for the 85-sec and 100-sec cycles is shown in Figure 2. This figure shows an insignificant (1.3 percent) difference in PIs during the off-peak hour and a 13 percent difference in PIs during the peak hour. The 100-sec cycle length probably would perform better than the 85-sec cycle length from 2:00 to 5:30 p.m. Therefore, Figure 2 does not indicate that there would be any point in additional research to determine an optimal time to change from one timing plan to another.

This difficulty was sidestepped by replacing the 100-sec solution for the peak hour with the 115-sec solution. Looking at Table 4, it can be seen that the 115-sec cycle length produces the second lowest PI (172.23) during the peak hour; therefore, it was reasonable to select it.

The 115-sec cycle length was plotted along with the 85-sec off-peak cycle length in Figure 3. This figure shows significant differences in PIs during both the off-peak hour and the peak hour. The differences are seen to be 7 and 12 percent, respectively. Figure 3 indicates a basis for this project

TABLE 3 TRANSYT-7F Output Results for Off-Peak Period Cycle-Length Evaluation

Run No.	Cycle Length (sec)	Total Arterial Delay (veh-hr/hr)	Average Arterial Delay[a] (sec/veh)	Total Uniform Stops (veh/hr)	Fuel Consumption (gal/hr)	Performance Index
1	75	55,205	8.61	9,165.0	259.63	118.85
2	80	56.431	8.80	8,643.6	252.11	116.46
3	85	56.583	8.82	8,547.5	251.2	115.94
4	90	60.851	9.49	8,442.3	250.7	119.48
5	95	62.707	9.78	7,897.5	243.96	117.55
6	100	63.999	9.98	7,695.4	241.61	117.44
7	105	66.832	10.42	7,661.1	242.69	120.03
8	110	72.661	11.33	7,530.0	243.06	124.95
9	115	72.759	11.35	7,465.1	242.48	124.60
10	120	72.009	11.85	7,200.0	239.17	126.01

[a]Average arterial delay = [total arterial delay (veh-hr/hr) x 3,600 sec] ÷ [total arterial flow (vph)]. Total arterial flow = 23,086 vph = sum of link flows at the seven nodes.

TABLE 4 TRANSYT-7F Output Results for Peak-Period Cycle-Length Evaluation

Run No.	Cycle Length (sec)	Total Arterial Delay (veh-hr/hr)	Average Arterial Delay[a] (sec/veh)	Total Uniform Stops (veh/hr)	Fuel Consumption (gal/hr)	Performance Index
1	75	117.462	13.91	14,672.5	410.25	219.36
2	80	104.401	12.36	14,183.4	397.32	202.90
3	85	97.577	11.55	14,043.5	392.07	195.10
4	90	95.236	11.28	12,442.7	367.05	181.64
5	95	92.488	10.95	11,869.3	357.84	174.91
6	100	90.310	10.69	11,544.2	352.56	170.48
7	105	92.220	10.92	11,537.8	353.32	172.34
8	110	93.646	11.09	11,388.6	352.76	172.73
9	115	96.482	11.43	10,907.4	347.18	172.23
10	120	99.779	11.82	10,892.5	348.06	175.42

[a]Average arterial delay = [total arterial delay (veh-hr/hr) x 3,600 sec] ÷ [total arterial flow (vph)]. Total arterial flow = 23,086 vph; 30,400 vph = sum of link flows at the seven nodes.

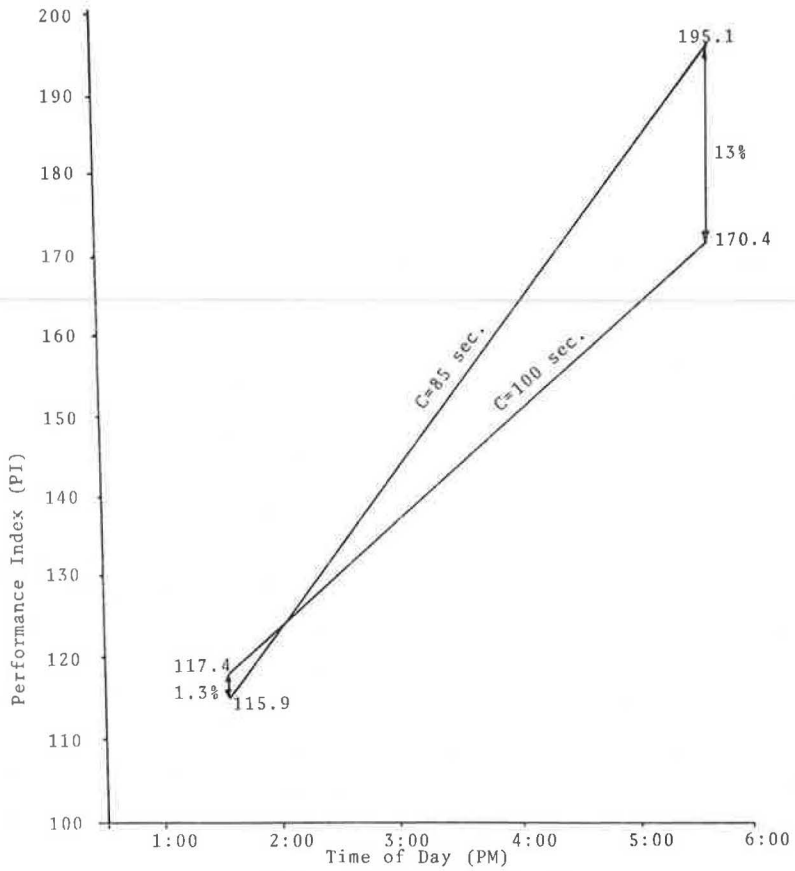

FIGURE 2 Comparison of 85-sec and 100-sec cycle lengths.

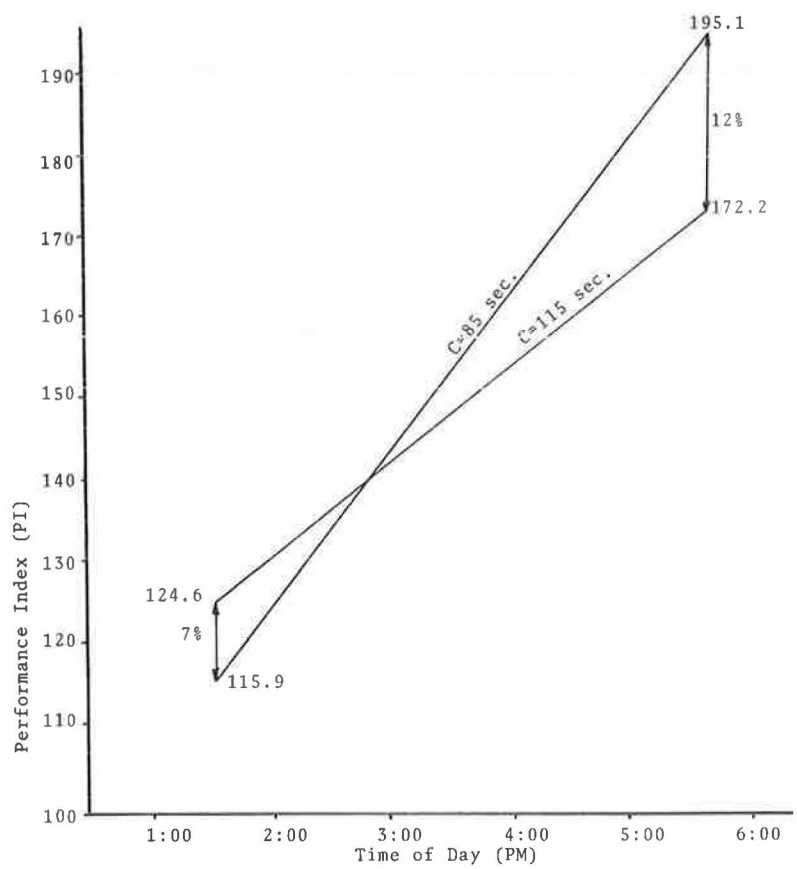

FIGURE 3 Comparison of 85-sec and 115-sec cycle lengths.

to continue its research to determine an optimal time to change from one timing plan to another. Therefore, the final cycle lengths selected were 85 sec for the off-peak hour and 115 sec for the peak hour.

Figures 2 and 3 can easily be misunderstood. The plotted points pertain to conditions from 1:00 to 2:00 p.m. and from 5:00 to 6:00 p.m., not from 2:00 to 5:00 p.m. as well. The straight lines connecting the points do not represent an assertion that the PI varies linearly as the afternoon progresses. (That subject is taken up later in the paper.) Neither does Figure 3 mean that, for example, the timing plan should be changed at 2:45 p.m.; that determination is made later, in a different way. The meaning of Figure 3 is only that the shorter cycle length was significantly better than the longer one during the off-peak hour and that the longer cycle was definitely better than the shorter one during the peak hour. Therefore, the researchers were in a position to take additional steps to determine the optimal time to change from one timing plan to the other.

The PASSER-80-derived timing plan for an 85-sec cycle length was used as the input to the TRANSYT-7F simulation program as the initial run to develop the off-peak signal-timing plan. This simulation program produces measures of effectiveness (MOEs) that are not shown in a PASSER-II-80 output, such as total network delay, stops, and fuel consumption. The TRANSYT-7F optimization program was used to produce the final off-peak signal-timing plan. The results for both runs are given in Table 5. This table shows that TRANSYT optimization significantly reduced stops, delay, fuel consumption, and PI. The average delay was reduced by 9 percent, the total uniform stops were reduced by 13 percent, and the total fuel consumption was reduced by 7 percent; the PI was reduced by 11 percent.

TABLE 5 TRANSYT-7F Simulation and Optimization Runs for Off-Peak Timing Plan

	Average Delay (sec/veh)	Total Delay (veh-hr/hr)	Total Uniform Stops (veh/hr)	Total Fuel Consumption (gal/hr)	Performance Index
Initial run	9.73	62.368	9,808.2	270.71	130.48
Final run	8.83	56.583	8,547.5	251.2	115.94
Saving (%)	9	9	13	7	11

Note: C = 85 sec.

The same procedure was followed for the optimum cycle length of the peak hour. The input data were obtained by running PASSER for peak-hour volumes and C = 115 sec, as shown in Table 2, Run 3. Table 6 gives the results of the TRANSYT-7F initial run and the TRANSYT-7F final run for the peak hour. The results indicate a greater reduction in stops, delay, fuel consumption, and PI than was obtained for the off-peak hour. The TRANSYT optimization of signal settings during the peak hour produced a savings of 15 percent in average delay, 33 percent in total uniform stops, and 19 percent in fuel consumption; the PI was reduced by 24 percent.

TABLE 6 TRANSYT-7F Simulation and Optimization Runs for Peak Hour Timing Plan

	Average Delay (sec/veh)	Total Delay (veh-hr/hr)	Total Uniform Stops (veh/hr)	Total Fuel Consumption (gal/hr)	Performance Index
Initial run	13.42	113.293	16,175.4	430.07	225.62
Final run	11.43	96.482	10,907.4	347.18	172.23
Saving (%)	15	15	33	19	24

C = 115 sec.

Second-Stage Evaluation

The second stage of applying the TRANSYT-7F program was to use the optimum cycle lengths developed for the off-peak hour and for the peak hour to determine the optimal time to change plans.

Twenty simulation runs were performed for the off-peak optimum cycle length (C = 85 sec) at 15-min intervals from 1:00 to 6:00 p.m. Another 20 simulation runs were performed for the peak-hour optimum cycle length (C = 115 sec) at 15-min intervals from 6:00 to 1:00 p.m.

Table 7 gives the PI of each run for the off-peak and peak cycle lengths. For each 15-min period, the PIs resulting from the two cycle lengths were compared. The results indicated that the 85-sec cycle performed better during the off-peak period and that the 115-sec cycle performed better during the peak period. The results also indicated that during the off-peak period, the off-peak cycle length produced a maximum of 8 percent better PI than the peak cycle length; during the peak period, the peak cycle length produced a maximum of 11 percent better PI than the off-peak cycle length.

TABLE 7 TRANSYT-7F Performance Index Comparison

Time Period (p.m.)	C = 85 Sec		C = 115 Sec		Difference (%)
	Run No.	P.I.	Run No.	P.I.	
1:00-1:15	1	129.54	21	140.53	+8
1:15-1:30	2	129.65	22	139.97	+7
1:30-1:45	3	130.85	23	142.62	+8
1:45-2:00	4	130.14	24	141.05	+8
2:00-2:15	5	133.30	25	143.42	+7
2:15-2:30	6	135.89	26	144.62	+6
2:30-2:45	7	135.88	27	145.99	+7
2:45-3:00	8	139.85	28	149.25	+6
3:00-3:15	9	138.77	29	147.78	+6
3:15-3:30	10	143.28	30	147.01	+3
3:30-3:45	11	146.42	31	148.87	+2
3:45-4:00	12	150.95	32	153.64	+2
4:00-4:15	13	153.82	33	155.41	+1
4:15-4:30	14	176.40	34	169.10	-4
4:30-4:45	15	180.06	35	174.92	-3
4:45-5:00	16	192.21	36	179.62	-7
5:00-5:15	17	220.26	37	199.18	-11
5:15-5:30	18	204.66	38	190.33	-8
5:30-5:45	19	187.51	39	184.87	-1
5:45-6:00	20	171.34	40	169.64	-1

Note: PI = performance index.

The PIs in Table 7 were plotted versus time of day in Figure 4. The solid curve represents the off-peak cycle length, and the dashed curve represents the peak cycle length. The two plots show that for both cycle lengths, the PI increased as the afternoon progressed and volumes increased. Compared with the 115-sec cycle length, the 85-sec cycle length produced lower PIs during the off-peak period and higher PIs during the peak period. The 115-sec cycle length produced higher PIs during the off-peak period and lower PIs during the peak period. The optimal time to change the timing plan from off-peak period to peak period was determined to be 4:15 p.m., where the two curves cross.

SOAP/M Output Results

The SOAP/M program was applied to the critical intersection, Piedmont and Marian roads (Node 5). It

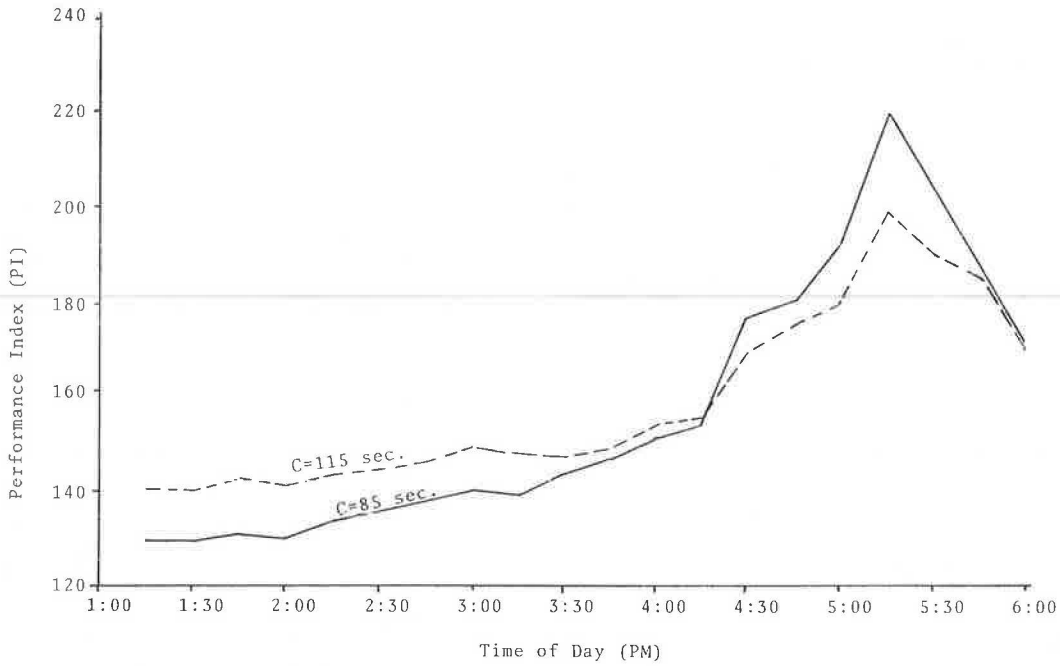

FIGURE 4 Optimal time of changing timing plans from off-peak to peak period.

was hoped that this simpler analysis might produce an optimal time to change timing plans similar to the time indicated by the TRANSYT-7F procedure.

The SOAP/M program was used to evaluate the off-peak cycle length and peak-hour cycle length that were selected by the TRANSYT-7F program for the coordinated arterial system. The TRANSYT-7F input data of Node 5 for every 30-min interval were provided for the SOAP/M program. Ten runs were made for the 85-sec off-peak cycle length at 30-min intervals from 1:00 to 6:00 p.m. (Table 8). Another 10 runs were made for the 115-sec peak cycle length at the same time intervals (Table 9).

It is indicated in Tables 8 and 9 that the off-peak cycle length performed better in terms of total delay, but that the peak cycle length performed better in terms of the percentage of stops. Fuel consumption for any one time of day was essentially the same for both cycle lengths. The off-peak cycle length had a better PI than the peak cycle length at all times; therefore, the SOAP/M analysis failed to produce an optimal time to change the timing plan.

Figure 5 shows a plot of the PIs versus the period of time from 1:00 to 6:00 p.m. The solid-line plot represents the off-peak cycle length and the dashed-line plot represents the peak cycle length.

TABLE 8 SOAP/M Results for Node 5 for Off-Peak Cycle

Time Period (p.m.)	Run No.	Total Stop (%)	Total Delay (veh-hr/hr)	Fuel Consumption (gal/hr)	Performance Index
1:00–1:30	1	69	17	36	34.87
1:30–2:00	3	70	17	37	35.21
2:00–2:30	5	69	17	37	35.22
2:30–3:00	7	71	18	39	37.08
3:00–3:30	9	72	19	40	38.67
3:30–4:00	11	72	19	41	39.02
4:00–4:30	13	76	23	47	46.20
4:30–5:00	15	77	24	50	48.76
5:00–5:30	17	78	24	53	50.46
5:30–6:00	19	76	23	49	47.15

Note: C = 85 sec.

TABLE 9 SOAP/M Results for Node 5 for Peak Cycle

Time Period (p.m.)	Run No.	Total Stop (%)	Total Delay (veh-hr/hr)	Fuel Consumption (gal/hr)	Performance Index
1:00–1:30	2	63	22	37	38.32
1:30–2:00	4	63	22	37	38.39
2:00–2:30	6	63	21	37	37.63
2:30–3:00	8	64	22	38	39.20
3:00–3:30	10	66	24	40	42.03
3:30–4:00	12	66	24	41	42.35
4:00–4:30	14	71	29	49	50.67
4:30–5:00	16	72	30	52	53.16
5:00–5:30	18	71	30	53	54.18
5:30–6:00	20	71	29	51	51.56

Note: C = 115 sec.

The figure shows that the off-peak cycle length performed better than the peak cycle length at all times.

SUMMARY AND CONCLUSIONS

Optimal off-peak and peak-hour timing plans for the afternoon hours for this particular arterial were produced in this study. A technique was demonstrated for determining the optimal time to change the off-peak timing plan to the peak-period plan. The PASSER results appear to have been improved by allowing TRANSYT to perform its optimization procedure on them. A SOAP/M analysis of the critical intersection failed to indicate an optimal time to change plans; there was no indication that SOAP/M could replace the more involved TRANSYT analysis of the entire route.

ACKNOWLEDGMENT

The authors are indebted to Peter J. Yauch, formerly with JHK & Associates in Atlanta, for suggesting this research topic and for providing helpful comments during the work.

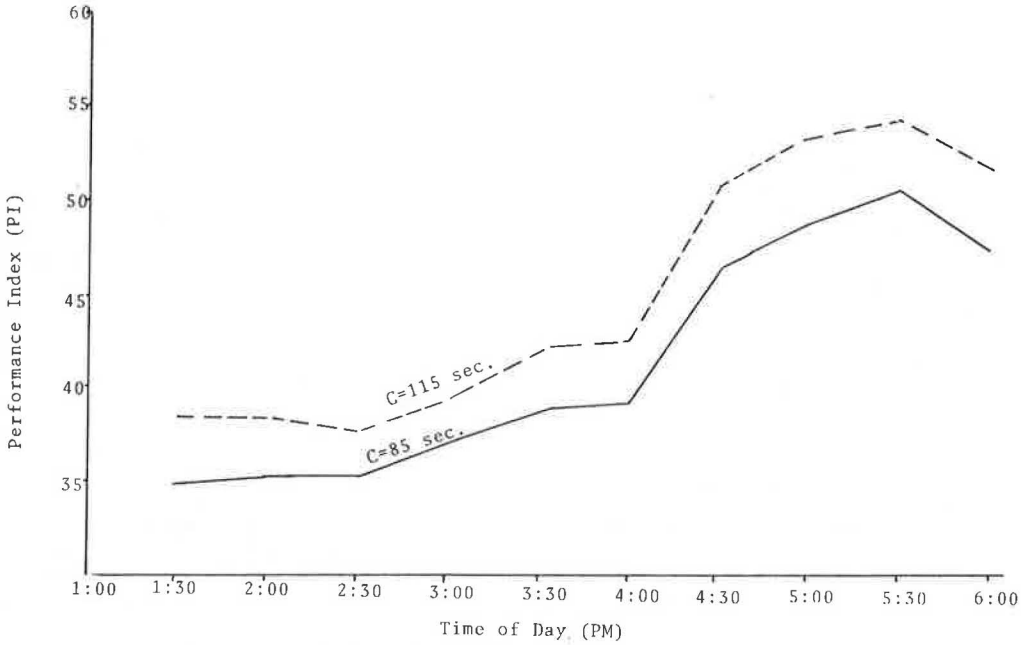

FIGURE 5 SOAP/M runs on Node 5 as an isolated intersection.

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Discussion

Edmond Chin-Ping Chang*

In the paper by Jrew and Parsonson, the uses of three off-line computer programs to determine the optimal time to change the arterial signal-timing plans for a particular Atlanta arterial were studied. Attempts were made to develop guidelines to select the optimal signal-timing plans between the off-peak and peak-hour periods by using the PASSER II-80, TRANSYT-7F, and SOAP/M computer programs.

Mainly, two experiments were performed to evaluate the alternatives of

1. Using either PASSER II-80 or TRANSYT-7F to indicate the optimal time to change the arterial traffic signal-timing plan; and
2. Avoiding the time-consuming TRANSYT-7F computerized procedure by evaluating only the critical intersection with the SOAP/M analysis.

The optimal timing plans were developed for both the off-peak and peak-hour traffic using PASSER II and TRANSYT-7F. It was expected that two different cycle lengths might be obtained for the off-peak and peak-hour traffic. The optimal timing plan was then run forward in time, with the volumes increasing from the off-peak to peak hour. A plot was prepared for performance index (PI) versus time of day. The optimal peak-hour plan was then simulated backward in time by using the traffic volumes tracking back from peak-hour to off-peak period; a second performance curve was obtained. The intersection of the two curves could indicate the optimal time to change timing plan.

However, the study results did not completely meet the expectation that an indication of the optimal time to change the arterial signal-timing plans for this particular arterial would be provided. First, the PASSER II-80 run did not indicate two separate optimal cycle lengths for the off-peak hour and peak hour. Therefore, the same process was repeated using TRANSYT-7F. The TRANSYT analysis results indicated that the shorter cycle did not perform significantly worse than did the longer cycle during the off-peak period. It also indicated that the longer cycle length performed better than the shorter cycle length during the peak hour. The optimal time to recommend signal-timing changes was determined. Finally, the SOAP/M program was run forward and backward in time by using the same volume level for the critical intersection. It was hoped that this simplified approach would indicate approximately the same time of day for changing timing plans as was indicated by TRANSYT-7F. However, the SOAP/M analysis resulted in only one plan for the entire off-peak and peak period, as was obtained using PASSER II-80. It was therefore concluded in the Jrew and Parsonson paper that the critical intersection approach could not successfully indicate an optimal time to change the timing plan and the approach should not be used for this purpose.

Several points should be noted from this investigation:

1. It is very difficult to provide an optimal solution if only a two-phase signal phase control option is used in the PASSER II analysis.

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2. Good initial solution is important for TRANSYT-7F because of the possibility of reaching a local optimization solution in a TRANSYT-7F analysis (1).

3. PASSER II does provide variations in measurements of effectiveness (MOEs) evaluated on the approach, intersection, and arterial system level.

4. Most existing arterial signal-timing optimization methods were developed for fixed-time traffic signal equipment. This requires that coordinated timing plans for an actuated signal system be converted to its equivalent pretimed settings, which sometimes means loss of some advantages of actuated signal control (2).

5. The latest evolution of the PASSER II-84 model provides additional advantages by offering the performance evaluations closely related to the 1985 Highway Capacity Manual (3).

One major advantage of PASSER II is its ability to provide the best combination of cycle and phase sequences for maximizing the total two-way progression. In this study, it was decided not to optimize the phasing other than to use the existing two-phase operation. Because PASSER II was not allowed to optimize the phasing, the cycle was basically controlled by the critical lane volume of the conflicting movement pairs at the critical intersection. It was also noted that the levels of traffic volume remained the same regardless of time of day, as indicated in the SOAP/M analysis. Therefore, it is obvious that neither PASSER II-80 nor SOAP/M would show any differences between the optimized cycle lengths in the evaluation.

There are increasing concerns that the data preparation and analysis efforts required for using TRANSYT-7F be reduced. It is also desired that a minimum-delay arterial timing plan that has good progressive operation be provided. A proven successful approach is to use MAXBAND or PASSER II to generate the maximum bandwidth solutions as starting initial solutions for the subsequent TRANSYT-7F analysis. It can also guarantee the initial green times, offsets, and progression bandwidth to provide a base point for arterial signal-timing optimization. The detailed data collection procedures documented in the TRANSYT-7F manual provide excellent guidelines for the general users.

PASSER II MOEs can provide good and consistent evaluations of the optimal arterial signal-timing plans for design analysis and operation evaluation on the arterial progression operations. To demonstrate the variability in PASSER II performance evaluations, a series of PASSER II-84 runs were made using the Skillman Avenue example. The PASSER II-84 performance evaluations of this particular data set are shown in Figure 6, which shows arterial systemwide measurements versus background cycle length. The progression efficiency and average system delay are plotted against cycle length ranging from 65 sec to 130 sec, as shown separately on the curves in Figure 6. As indicated, the arterywide measurements depend on the different cycle lengths selected by PASSER II under quad-left, multiple-phase operations for this particular coordinated arterial.

Level-of-service criteria were updated in the PASSER II-84 version 2.3B package to conform to the new technology developed in the 1985 Highway Capacity Manual. PASSER II-84 provides several operational MOEs separately for each traffic movement, intersection, and arterial system. These measures can be used to evaluate the existing operations or estimate the proposed signal-timing plan. Generally, the accepted performance evaluation criteria for describing level of service for individual movements used in PASSER II-84 are as shown in Table 10.

TABLE 10 Accepted Performance Evaluation Criteria for Describing Level of Service for Individual Movements Used in PASSER II-84

Level of Service	Volume-to-Signal Capacity Ratio	Movement Delay (sec/veh)	Probability of Clearing Queue
A	≤0.60	≤ 6.5	≥0.995
B	≤0.70	≤19.5	≥0.90
C	≤0.80	≤32.5	≥0.75
D	≤0.85	≤52.0	≥0.50
E	≤1.00	≤78.0	<0.50
F	>1.00	>78.0	<0.50

The delay criteria used in PASSER II-84 are equivalent to the average delay criteria established by the Highway Capacity and Level of Service Committee of the Transportation Research Board in the 1985 Highway Capacity Manual for stop delay of (5, 15, 25, 40, 60) where average delay equals 1.30 times the stop delay. The ratios used in PASSER II-84 for evaluating volume-to-signal capacity ratio

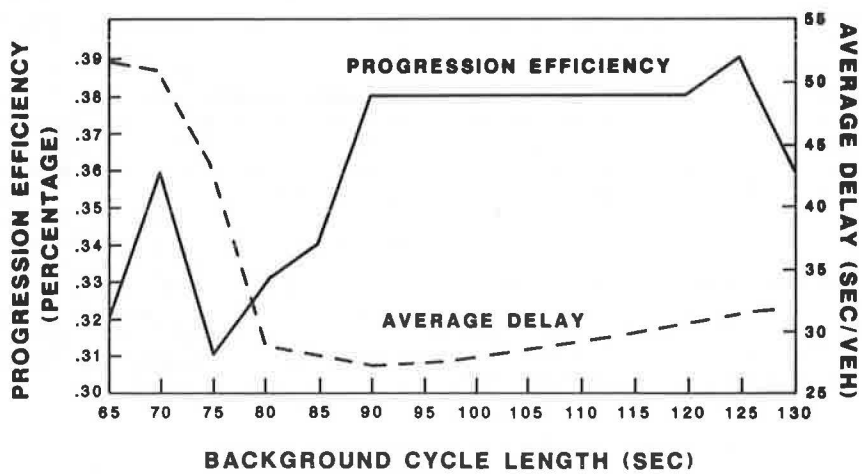

FIGURE 6 PASSER II-84 performance evaluations: arterial systemwide measurements versus background cycle length.

(v/c, saturation or the x ratio) are reasonable criteria. There are no generally accepted criteria for v/c ratio or probability of queue clearance for intersections. It should be noted, however, that traffic delays usually become excessive at volume-to-capacity ratios exceeding 0.85.

Long cycle lengths tend to increase arterial progression efficiency and reduce the volume-to-signal capacity ratio, but at the same time increase arterial system delay. Excessively long cycles may create lane blockage and driver confusion on the cross street, and account for reductions in saturation flow rate. Research in Canada also suggests that green-light durations of longer than 50 sec may also become inefficient for traffic operation. Therefore, it is common practice in PASSER II analysis to provide the widest range in the first run to optimize the green splits, cycle length, and phase sequences for the entire arterial. Then, a desirable cycle range of plus and minus 5-sec range according to the MAXMIN DELAY CYCLE LENGTH of the most critical intersection is coded for the maximum and the minimum allowable cycle length ranges. Several arterial optimization runs could then be made to provide better evaluation of the alternative timing plan by specifying different geometric design options, traffic volume, traffic flow characteristics, and signal-timing control parameters.

On the other hand, the modeling of actuated arterial signal operations requires detailed time-series analysis to analyze the effects of variations in traffic volumes. A need exists for developing macroscopic arterial traffic signal models to study and optimize timing plans for arterial signal systems with actuated signal controllers. Approaches similar to that used in the FREQ-model can provide a tool for evaluating different arterial traffic congestion management strategies with respect to different times of day.

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Authors' Closure

The authors thank the discussant for taking the time to prepare a discussion of their paper.

The first four paragraphs of his discussion are essentially a summary of the paper. A sentence in his fourth paragraph states "Finally, the SOAP/M program was run forward and backward in time by using the same volume level for the critical intersection." Our paper states that "the volumes used were for each 30-min time of day," so the discussant probably intends that his sentence state ". . . using the same volume levels. . ."

The fifth paragraph of the discussion begins "Several points should be noted from this investigation." The discussant's point is that "It is very difficult to provide an optimal solution if only a two-phase signal phase control option is used in the PASSER II analysis." Later, in the sixth paragraph, he states that "In this study, it was decided not to optimize the phasing other than to use the existing two-phase operation." There appears to be a misunderstanding here. Nowhere in the paper is it stated that the signals had only two phases, nor did we say that during our oral presentation at the Transportation Research Board's 65th Annual Meeting. Actually, several signals had more than two phases.

The discussant's second point is that TRANSYT-7F needs to begin with a good initial solution. Possibly, he suspects that our TRANSYT results were local, not global, optima because of poor initial settings. It is stated in our paper that "The output timing data of PASSER II were used as input timing for all the TRANSYT-7F runs." Further, the step sizes of the TRANSYT optimization iterations were selected by using the standard pattern known to be successful in breaking out of a local optimum.

The discussant's third point is that PASSER II provides a number of MOEs. The authors agree, but are not sure what his point is with respect to this paper.

The discussant goes on to mention (in the fourth item on his list of points to note from the investigation) that timing plans (developed for fixed-time controllers) sometimes are implemented by using actuated controllers. This is true, but the authors do not understand how this is a comment on our paper; we did not mention the type of controllers on our arterial nor did we present any data that was linked to the controller type.

The discussant's fifth point is that the latest version of PASSER, called PASSER II-84, provides certain advantages. Perhaps he is hinting that our results would have been different in some way had this version been available at the time of our research.

In the next paragraph, the discussant states that in our research "the levels of traffic volume remained the same regardless of time of day, as indicated in the SOAP/M analysis." The authors interpret this to mean that he believes that our SOAP/M analysis used constant volumes, the same for all times of day. We do not understand this comment because it is stated in our paper that the volumes used were for each 30-min time of day.

In his next paragraph, the discussant recommends that MAXBAND or PASSER II be used to generate initial settings for subsequent TRANSYT-7F analysis. The authors agree; it is stated in our paper that we used PASSER.

The remainder of the discussant's comments deal primarily with PASSER II-84's level-of-service criteria and with selection of cycle length. Neither of these points appears to be directed at our paper.

Abridgment

Potential Performance Characteristics of Adaptive Control at Individual Intersections

FENG-BOR LIN and DONALD J. COOKE

ABSTRACT

Adaptive signal control of individual intersections relies on advance information obtained primarily by detectors for real-time optimization. The development of this mode of control involves a number of decisions about detector utilization, prediction of flows, selection of optimization procedures, data processing requirements, and so forth. In an effort to search for an effective adaptive control, computer simulation is used in this study to analyze the potential characteristics of an adaptive control strategy. The analysis concerns (a) the effectiveness of signal optimization, (b) the impact of information availability and utilization, and (c) the sensitivity of control efficiency to information errors.

Adaptive signal control, as referred to in this paper, is a mode of control that relies primarily on advance information provided by detectors to search for and to implement optimal signal-switching sequences. The technology for adaptive signal control of individual intersections is already available. Limited field experiments (1,2) have also been conducted to assess several adaptive control strategies. Currently, however, it is not clear what would constitute an adaptive control strategy that is far superior to the various traffic-actuated controls being used today.

As a first step toward resolving this issue, computer simulation is used in this study to analyze three aspects of the performance characteristics of an adaptive control strategy. This strategy is similar to the Optimization Policies for Adaptive Control (OPAC) as proposed by Gartner (3). The objectives are to (a) examine the effectiveness of signal optimization in improving signal operations; (b) determine the amount of advance information needed, and how such information can be effectively utilized; and (c) assess the sensitivity of the control efficiency to certain types of information errors.

CONTROL STRATEGY

The control strategy examined in this study utilizes a series of overlapping optimization stages (Figure 1). A stage is a time interval for which an optimal signal-switching sequence is to be identified. An optimal switching sequence, in turn, is a combination of green, signal-change, and red intervals that minimize the total delay at an intersection.

To facilitate the optimization, each stage is divided into subintervals. In each subinterval, the right-of-way is allocated to a particular phase or to several phases that have no conflicting movements. An algorithm is used to generate all alternative ways of allocating the right-of-way and to identify the corresponding feasible switching sequences. A feasible sequence is one that satisfies a set of specified requirements for minimum green, maximum green, and signal-change intervals.

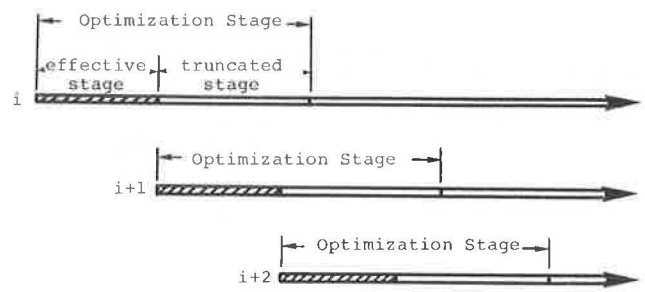

FIGURE 1 Effective stage and truncated portions of overlapping optimization stages.

Each feasible sequence is evaluated by a traffic model in terms of the total delay. Such a traffic model is part of the control strategy. The evaluation is based on the flow conditions at the beginning of the current stage and on new information provided by a detector in each lane. The optimization process involves the identification of the sequence that minimizes the total delay.

A microscopic simulation model is used to provide simulated settings for adaptive control operations. It also serves as a tool for evaluating the optimal switching sequences generated by the control strategy. Delays determined from this model for pretimed control agree well with delays estimated from Webster's delay formula (4) when the delays are not time dependent.

EFFECTIVENESS OF SIGNAL OPTIMIZATION

Because of the need for real-time operation, adaptive control has to optimize the signal operation for an expected flow condition rather than for an actual condition. The discrepancies between the expected and the actual conditions may partially offset the benefits of optimization. Even if the impact of such discrepancies is negligible, there is still a limit to what optimization can do for signal control. This point is demonstrated by Figure 2, which shows vari-

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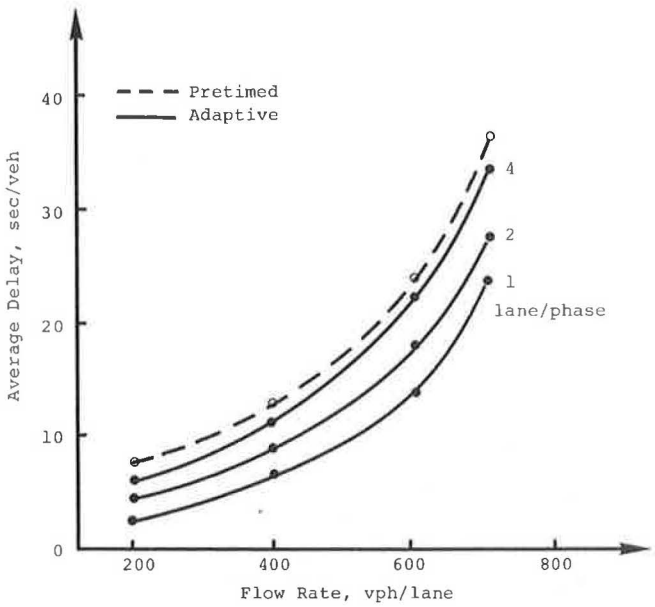

FIGURE 2 Variations in efficiency of adaptive control with traffic volume.

ations in efficiency of adaptive control with traffic volume.

Figure 2 shows that, with only one lane per phase for a two-phase operation, adaptive control is much more efficient than pretimed control. When the number of lanes per phase increases, however, the efficiency of adaptive control decreases. The obvious reason for this phenomenon is that the optimal switching sequence for multiple lanes is not the optimal sequence for each individual lane. Therefore, a careful experimental design is needed for the potential of an adaptive control strategy to be assessed.

IMPACT OF STAGE SUBDIVISION, STAGE TRUNCATION, AND STAGE SIZE

Stage Subdivision

Dividing a stage into smaller subintervals results in a larger number of feasible switching sequences. In contrast, long subintervals could result in a sluggish transfer of the right-of-way under light flow conditions. Therefore, in the absence of information errors, smaller subintervals can produce better control efficiencies. This point is demonstrated in Figure 3, which shows variations in control efficiency with stage subdivision.

A statistical analysis shows that the use of 3-sec subintervals can produce statistically significant improvements (at a 5 percent level of significance) over the use of 5-sec subintervals. Such improvements, however, are less than 0.9 sec/veh in delays for two-phase operations with flow rates of up to 700 vph/lane.

The advantage of using smaller subintervals can be fully realized only if expected events in a given subinterval take place within that interval. Furthermore, it should be noted that the use of smaller subintervals could increase data processing requirements.

Stage Truncation

Generally, it is preferable to truncate the tail portion of an identified optimal switching sequence

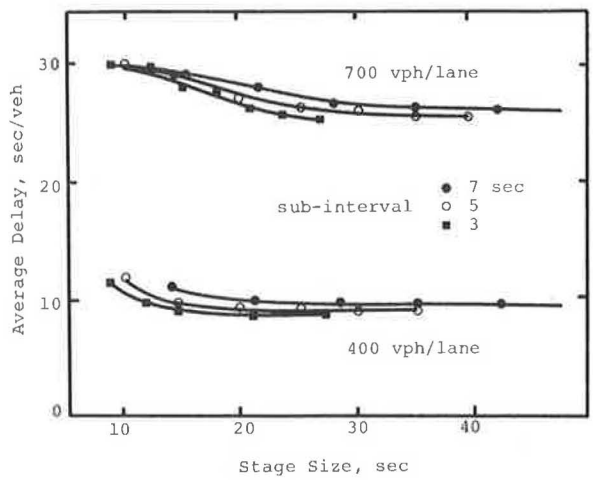

FIGURE 3 Variations in control efficiency with stage subdivision.

and to implement only the remaining portion of the sequence (see Figure 1). The reason for this is that, when additional arrival information becomes available and is used for optimization, a better switching sequence may be found for the tail portion of a stage. The issue is how large a truncation is needed.

Figure 4 shows that stage truncation can exert a noticeable influence on control efficiency. The control efficiencies produced without truncation are always poorer than those produced with truncation. The importance of truncation, however, diminishes when the traffic volume decreases. Simulation results based on a large sample of arrival patterns indicate that there is no significant difference (at the 5 percent level) between control efficiencies produced respectively by 15-sec truncations and 20-sec truncations.

Stage Size

Stage size determines the amount of advance information needed for optimization. For example, a 20-sec stage would require about 20 seconds of advance in-

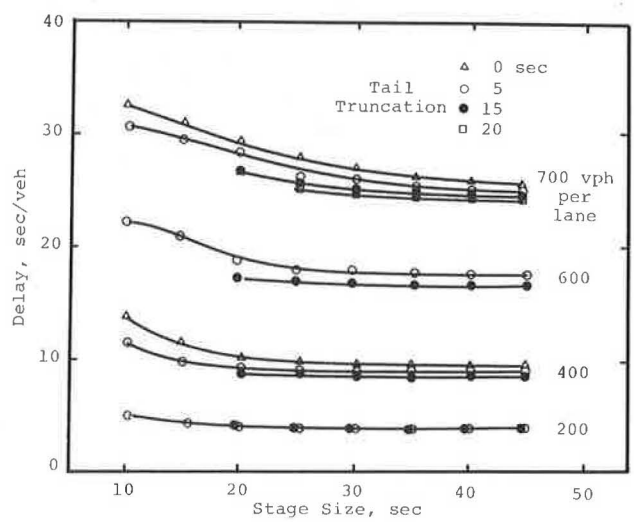

FIGURE 4 Variations in control efficiency with stage size and stage truncation.

TABLE 1 Delays Based on Correct and Incorrect Queue Discharge Times (sec/veh)

No. of Lanes per Phase	Flow (vph per lane)	Case 1		Case 2		Case 3		Average Increase (%)
		Correct	Incorrect	Correct	Incorrect	Correct	Incorrect	
2	200	4.8	5.0	5.0	5.4	4.7	4.9	5.5
4	200	5.7	6.1	5.8	6.1	5.6	6.2	7.6
2	400	8.9	9.7	8.8	9.5	8.6	9.0	7.2
4	400	10.4	11.1	10.6	10.9	10.0	10.7	5.5
2	600	16.7	18.5	16.9	18.0	18.0	18.0	5.8
4	600	20.2	20.6	19.5	20.9	20.0	21.0	4.7
2	700	28.3	31.9	28.0	33.0	27.1	30.2	14.0
4	700	32.2	33.8	31.8	34.4	32.6	34.0	5.8

Note: Data are derived from 50-min, two-phase operations.

formation. If such advance information can be obtained and used without errors, longer stages can result in more efficient signal operations. This point is shown in Figure 4, the implication of which is that an increase in the available amount of accurate advance information can improve control efficiency. It should be noted, however, that lengthening the stage size has a diminishing return. Furthermore, the use of long stages is more important when traffic volume is heavy than when it is light. Little can be gained in control efficiency when more than 25 seconds of advance information is used for optimization.

EFFECTS OF INFORMATION ERRORS

Two types of information errors are addressed in this study: those in estimated queue discharge characteristics and those in expected arrival sequences.

Errors in Estimated Queue Discharge Characteristics

The actual discharge headway or discharge time of a queueing vehicle cannot be known in advance. Under this circumstance, the worst control logic is one that randomly generates a discharge headway or a discharge time from a known distribution to represent the actual discharge characteristic of queueing vehicle. Table 1 shows that the resulting information errors have a small detrimental impact on control efficiency if this logic is adopted. Such an impact can be further reduced if randomly generated discharge times are replaced by the mean observed discharge time for each queueing position.

This reduction is possible because the use of the mean discharge times can result in rather small estimation errors. Based on observed queue discharge times, it can be shown that the use of the mean discharge times would produce a mean absolute error of estimate of only about 0.8 sec for the first queueing position and 1.5 sec for the tenth queueing position. Therefore, the estimation of queue discharge times is not a critical concern in the development of an adaptive control strategy.

Errors in Expected Arrival Sequences

For a given flow rate, vehicles can arrive at an intersection in numerous sequences of headway. An expected arrival sequence that is used for optimization will differ from the actual sequence. Such differences may result from detector malfunctions, speed variations, lane changes, the use of predicted flows, and so forth. Under the most unfavorable conditions, the expected and the actual arrival sequences may be independent of each other. It is indicated in Table 2 that, in such a case, the use of erroneous arrival sequences has a profound detrimental impact on control efficiency.

The extent of the impact, as shown in Table 2, can be reduced by relying heavily on detectors to collect data. Nevertheless, determining how to provide reliable arrival information for optimization is a real challenge in the development of an adaptive control strategy.

CONCLUSIONS

The efficiency of adaptive control can vary significantly from one flow pattern to another. Field assessments of an adaptive control strategy should have sound experimental designs to avoid generating biased information. Twenty-five seconds of advance information appears to be sufficient when the strategy described in this paper is implemented. Stage subintervals of 3 to 5 sec are adequate and stage truncation does not have to exceed 15 sec. The control efficiency is not sensitive to errors in estimated queue discharge times; however, it is very sensitive to errors in vehicle arrival sequences.

TABLE 2 Delays Based on Correct and Incorrect Arrival Sequences (sec/veh)

No. of Lanes per Phase	Flow (vph per lane)	Case 1		Case 2		Case 3		Average Increase (%)
		Correct	Incorrect	Correct	Incorrect	Correct	Incorrect	
2	200	4.8	11.2	5.1	12.7	4.9	11.4	138
4	200	6.3	10.4	5.9	10.1	6.1	9.5	64
2	400	11.3	18.2	10.2	14.9	10.6	14.5	48
4	400	10.3	14.7	10.8	14.2	11.0	14.3	35
2	600	17.5	28.8	18.2	34.2	16.5	32.7	84
4	600	20.5	26.6	19.6	24.5	20.7	27.8	30
2	700	31.6	48.6	28.8	41.2	29.7	44.8	49
4	700	35.7	47.8	32.5	42.1	34.3	48.0	34

Note: Data are derived from 50-min, two-phase operations.

ACKNOWLEDGMENT

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Developing Interconnection Guidelines for Isolated Traffic Signals

EDMOND CHIN-PING CHANG

ABSTRACT

The increase in levels of traffic congestion along major urban signalized arterials makes efficient traffic management and utilization of arterial facilities important considerations. Significant improvements in traffic flow and reductions in vehicular delay may be realized by interconnecting individual, isolated intersections into a coordinated signal system, or by adding an adjacent signal to an existing progression system. Current analytical methods and computer programs offer capabilities of optimizing traffic signal coordination for a series of signalized intersections. Recently, transportation research has been directed toward the alternative development of short-range, low-capital improvements for the safe and efficient movement of people and goods. The criteria measuring these alternative improvements include the following: traffic volume distribution, travel time, delay, and quality of traffic flow. However, the proper procedures and methods for analyzing the effects of coordinating isolated signalized intersections are insufficient. Because interconnection can be significant to the total signalized operation, guidelines are needed to identify where to implement signal interconnections. Summarized in this paper is research sponsored cooperatively by the Texas Department of Highways and Public Transportation and the FHWA, U.S. Department of Transportation. In the study, guidelines and procedures are developed to identify where interconnection of signalized intersections should be implemented. Efforts were made to evaluate interconnection of isolated traffic signals into a progression system to provide coordinated operations. Described are the study approach and development of good coordination of isolated traffic signals by using both simulation analysis and field validation. In this way, better signal interconnection, efficient utilization of the street system, and smooth traffic operations can be provided.

The increase in levels of traffic congestion along major urban signalized arterials makes efficient traffic management and utilization of arterial facilities important considerations. Significant improvements in traffic flow and reductions in vehicular delay may be realized by interconnecting individual isolated intersections into a coordinated signal system, or by adding an adjacent signal into an existing progression system.

Current analytical methods and computer programs offer capabilities of optimizing traffic signal coordination for a series of signalized intersections. Recently, transportation research has been directed toward the alternative development of short-range, low-capital improvements for the safe

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and efficient movement of people and goods. The criteria measuring these alternative improvements include the following: traffic volume distribution, travel time, delay, and quality of traffic flow. However, the proper procedures and methods needed to analyze the effects of coordinating isolated signalized intersections are insufficient. Because interconnection decisions can be significant within the total signalized operation, guidelines are needed to identify where to implement signal interconnection.

Summarized in this paper is research sponsored cooperatively by the Texas State Department of Highways and Public Transportation (TSDHPT) and the FHWA, U.S. Department of Transportation. In the study, guidelines and procedures are developed to identify where interconnections of signalized intersections should be implemented. Described are the study approach and the development of good coordination of isolated traffic signals by using both simulation analysis and field validation. A simple procedure for analyzing whether interconnection of isolated signalized intersections will be beneficial with respect to the increasing traffic volume is applied. In this way, better signal interconnection, efficient utilization of the street system, and smooth traffic operations can be provided.

Efforts were made to evaluate interconnection of isolated traffic signals into a progression system to provide coordinated operations. Specific study objectives were to

1. Identify factors that influence interconnection feasibility of isolated signalized intersections;
2. Evaluate effectiveness of isolated versus interconnection control, and isolated control versus interconnection with progression phasing;
3. Develop guidelines to identify where interconnection of a series of signalized intersections into a progression system should be implemented; and
4. Develop a simple, easy-to-use evaluation procedure for examining the need for signal interconnection.

STUDY BACKGROUND

Modern traffic control strategies utilize optimization of signal-timing plans, installation of control equipment, and provision of interconnection to reduce vehicular delay and fuel consumption (1). Wagner (2) found that "it is fuel efficient if traffic can be kept moving (without stopping). Lost fuel by stopped vehicles may be reduced with more efficient traffic control systems, especially during the off-peak periods when the number of stops and the overall delay may be improved through traffic control improvements." Suhbier and Byrne determined that one-half of vehicular fuel usage was caused by traffic delay at arterial intersections (3). Because arterial traffic occupies the major portion of area-wide travel, improvements in arterial traffic control can effectively reduce fuel consumption throughout the day.

The coordination of adjacent signals can (a) reduce overall travel time and stops and delays, and (b) decrease fuel consumption and air pollution. Even though fewer publications exist addressing when to interconnect a series of isolated signalized intersections, interconnection has been recognized as a viable alternative in traffic control improvement. Wagner studied the four possible traffic control system improvements (4) and found that the typical improvement in average travel time was as follows:

Traffic Control Improvement	Travel Time Savings (%)
Interconnection and optimization of signals	25
Signal timing optimization	17
Advanced master control system improvements	15
Freeway surveillance and control	20

Wagner found that "the most dramatic improvements in traffic performance on signalized arterials and networks are those resulting from the combined action of interconnecting previously uncoordinated pretimed signals with a master controller, together with the introduction of new optimized timing plans." He indicated that "simply retiming signals that were already interconnected without any hardware changes averaged a 12 percent improvement in speed or travel time" (4). In addition, Wagner also found that signal-timing reoptimization was the most-cost-effective enhancement action. However, possible improvement by signal-timing optimization depends on the quality of the signal-timing plan, geometric constraints of the arterial street, traffic characteristics, and quality of the existing arterial progression system.

Several attempts were made to relate factors for coordination. In several studies conducted by Yagoda, Whitson, White, Messer, and others, a coupling index (I) was developed, which was the simple ratio of link volume and link length (5-7):

$$I = V/L \quad (1)$$

where

I = coupling index,
V = approach link volume [vehicles per hour (vph)], and
L = link length to next signal (ft).

By computing this index for each link in the potential coordinated system, a measure of the signal interconnection needs could be determined.

The Traffic Control System Handbook indicated that "any two or more signals which are less than one-half mile apart or within a cycle length of travel time should be coordinated" (6). Pinnell identified various factors that affect arterial signal control strategies (6):

- Distance between signalized intersections,
- One-way versus two-way street operations,
- Signal phasings,
- Arrival characteristics, and
- Traffic fluctuations with time.

Other researchers found that a number of factors are important in determining the need for interconnection (7-14), including

- Geographic relationship--Distance between intersections. Intersections to be interconnected should be adjacent to each other without being affected by the natural and artificial boundaries, such as rivers and controlled-access facilities.
- Volume levels--Presence of larger link volumes usually implies a greater need for coordination between adjacent traffic signals.
- Traffic flow characteristics--If traffic arrivals are uniform throughout the cycle, the red phase of the cycle would produce the same delays and stops as would the green phase. On the other hand,

controlled flow in platoons enhances the coordination benefits with the extra consideration of platoon dispersion.

Presented in this paper is a model for arterial traffic signal design; the study developed to evaluate the feasibility for interconnection of isolated traffic signals is described (7-9).

Model Development

Intersections should be interconnected only if the arrival flow rates downstream can be guided into compact platoons through effective traffic signal timing. Fluctuation in arrival rates is influenced primarily by two factors to bring flow rates away from uniformity over time: (a) degree of volume variation at the upstream intersection, and (b) amount of platoon dispersion occurring between intersections.

Volume Considerations

It is not necessary to interconnect a system of intersections if the volume level is uniform and balanced during most operational periods. However, because of the different green time used during each signal of the progression system, the amount of delay and stops could be affected by the coordinated offsets under normally fluctuated arrival conditions. Several factors may contribute to the uniform arrival of vehicles at an intersection:

- An intersection isolated by distance relative to the other upstream arterial signalized intersection,
- Consequential traffic volumes entering at midblock, and
- Significant truck movement between intersections.

Thus, the desirable condition for interconnection is the imbalance in level of volume entering at the upstream intersection. In addition, significant traffic volume entering at midblock or a large truck traffic volume between intersections will force arriving flows to slow down such that interconnection can not eliminate the traffic congestion problems.

Consider the typical link flow pattern between two adjacent intersections, as shown in Figure 1. The entry volume for the downstream intersection (Link 3) consists of the right-turn volume (Link 2), through volume (Link 1), and left-turn volume (Link 4) from the upstream intersection. The degree of traffic flow imbalance at the upstream intersection is represented by the ratio between the maximum link traffic volume feeding from the upstream intersection and the sum of all the link traffic volume arriving at the upstream intersection. It can be stated as shown in Equation 2:

$$\text{Imbalance} = q_{max} \div \bar{q} \qquad (2)$$

where

I = imbalance index;
q_{max} = maximum flow, usually the through-movement flow (vph); and
$\bar{q}$ = average flow rate entering a link (vph).

The flow entering on the downstream intersection is influenced by the arriving flow over time. The imbalance index, as calculated from the maximum link flow divided by the average upstream link flow, is an index representing the fluctuation of traffic volume along a downstream link. It varies as

$$1 \leq (q_{max} \div \bar{q}) \leq X \qquad (3)$$

When this factor is 1, uniform flow exists. That is, cross-street, midblock, and turning traffic at the upstream intersection are approximately equal to the major entering flow. Interconnection of upstream and downstream signalized intersections in this case is not desirable. However, when the imbalance factor approaches X, or the total number of approach lanes, the effect of flow rate is at its maximum on the downstream intersection. This condition of heavy imbalance will create the most desirable situation for progression. The existence of imbalance can describe the relationships between flow rates and vehicle platoon formation. However, the additional effects of platoon dispersion have not yet been considered by this equation.

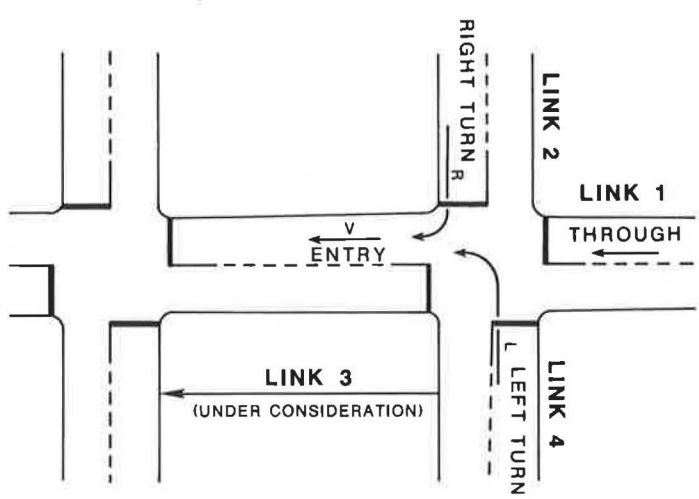


FIGURE 1 Entry flow for a typical link.

Platoon Dispersion

Platoon dispersion results from the drivers' adjusting the relative distance between their vehicle and adjacent leading and trailing vehicles. The dispersion of a platoon of vehicles leaving a signalized intersection has been described in previous research by Nemeth and Vecellio and in the North Dallas Corridor study. It approximated dispersion rate in terms of percent of platoon length by the following equation (7,8):

$$\text{Rate of dispersion, } D = (L + \Delta L)/(L * 1 + t) \quad (4)$$

where

L = length of the standing platoon (sec),
ΔL = change in length over distance and time (sec), and
t = average travel time (sec).

The change in platoon length related to the time and distance traveled can be further expressed by simplifying Equation 4 into Equation 5:

$$D = 1/(1 + t) \quad (5)$$

Interconnection Model

By combining the previous volume and platoon dispersion concepts, an interconnection desirability index (I) can be used to describe both the characteristics of platoon dispersion and traffic signal system as follows:

$$I = [(X * q_{max})/(q1 + q2 + q3 + \ldots qx)] - \{(N - 2) * [1/(1 + t)]\} \quad (6)$$

Equation 6 may have a value ranging from 0 to 2. By normalizing for a range of 0 to 1 and rearranging Equation 6, it can be simplified as shown in Equation 7:

$$I = [1/(1 + t)] * [(X * q_{max})/(q1 + q2 + q3)] - (N - 2) \quad (7)$$

where

t = link travel time, link length divided by average speed (min);
X = number of departure lanes from the upstream intersection;
q_{max} = straight-through flow from the upstream intersection (vph);
$q1, q2, \ldots, qx$ = traffic flow arriving at the down-stream approach from the right-turn, and through movements of up-stream traffic signals (vph); and
N = number of arrival lanes feeding into the entering link of the down-stream intersection.

In other words, the coordination requirements of each one-way link are measured in this approach by incorporating the platoon dispersion effect through use of I. In Equation 7, a value of 1 indicates the most desirable condition for interconnection and 0 indicates the least desirable condition. The scale shown in Figure 2 is suggested as a possible tool for applying the signal interconnection in the traffic control strategy. As indicated, when I has a value of 0.25 or less, isolated operation is recommended. On the other hand, when I has a value of 0.50 or greater, interconnected system operation is recommended. Other evaluation indicators are needed to assist in the interconnection decision if the calculated value of I is between 0.25 and 0.50. The interconnection of traffic signals at a study intersection is warranted when the I equals or exceeds 0.35. The relative need for traffic signal interconnections at possible locations could be indicated by the relative interconnection desirability index on both sides of the study intersection.

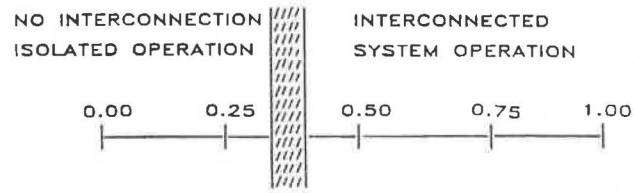


FIGURE 2 Interconnection desirability index.

It should be noted that this approach considers the potential benefits resulting from the interconnection of an isolated intersection or intersections by measuring the combined effects of geographic relationships, traffic volume levels, and traffic flow characteristics. However, this formula does not hold for the case when straight-through flow from the upstream intersection (q_{max}) is 0, yet turning flows are relatively high and the intersections are closely spaced, in which case interconnection may be desirable. Treating the heavy turning flows as through movements in the equation could solve the problem in this extreme case. By using this approach, an interconnection desirability index of 1 would indicate the most desirable condition for interconnection, and 0 the least desirable. It could be suggested that the scale shown at the bottom of Figure 2 be used as a tool for the delineation of the traffic signal control strategies.

STUDY PROCEDURE

In this study, the experimental simulation and field study were designed to develop guidelines for traffic signal interconnection. They were developed based on geographic relationships, volume levels, and traffic flow characteristics. Simulation models were used as theoretical test bases to investigate conditions that cannot easily be reproduced in the field. Field data were then collected on selected arterials to validate the simulation results.

Simulation Study

It is suggested by current technology that intersection spacings, percentages of turning traffic, and general volume levels are candidate elements. A review of existing traffic models suggests that PASSER II, TRANSYT-7F, and NETSIM can be used to determine interconnected traffic signal operations. Basically, PASSER II and TRANSYT-7F were used to optimize phase sequence and offsets for pretimed traffic signals under isolated versus interconnected operations. However, the simulation of existing isolated traffic control conditions could not be thoroughly evaluated by the first two models. The NETSIM model was also used to evaluate the coordinated operations of a series of isolated actuated traffic signals. It was further used as a base to analyze isolated versus interconnected actuated traffic control.

Alternative traffic control strategies under dif-

ferent geometric and traffic levels were devised to test the effectiveness of interconnection. The experimental simulation plan, as shown in Figure 3, was used to collect simulation data and establish numerical guidelines under different intersection spacings and left-turn percentages. The PASSER II runs were made to provide the optimal settings of cycle length and proper phase sequence. The TRANSYT-7F runs primarily examined the detailed effects of intersection spacings and the percentages of traffic turning left onto the arterial.

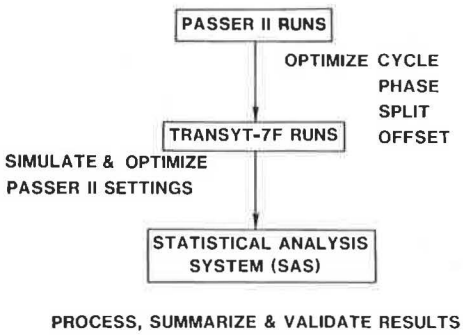


FIGURE 3 Experimental simulation design plan.

The major variables studied include

- Numbers of signal phases,
- Preferred phase sequences,
- Allowable cycle length ranges based on volume levels,
- Volume levels,
- Speed variations,
- Left-turn movement percentages, and
- Intersection spacings.

This means that a large number of simulation cases would be required if all the combinations of variables were to be used. Scenario runs of the computer program were made for the range of factors identified in order to determine the practical accuracy, sensitivity, and applicability of the simulation model.

It was assumed in this simulation study that

1. Approach volumes are constant over the study time period;
2. Platoon structure remains coherent along the arterial;
3. The link speed remains uniform;
4. Origin-destination turning traffic volumes are consistent:
   a. All side-street left-turn traffic flows into through movement;
   b. All main-street left-turn traffic is originally from the through movement on the main street;
   c. Downstream through traffic on the main street is equal to the arterial through traffic plus side-street left-turn and right-turn traffic, and minus the downstream left-turn traffic; and
5. Directional link volumes are balanced.

A synthetic four-node arterial street, as shown in Figure 4, was used to obtain separate but compatible simulation results by using both the PASSER II and TRANSYT-7F models. Sets of PASSER II runs were first made to choose appropriate signal phase sequence and phase length for both two-phase and four-phase operations with different intersection spacings. The TRANSYT-7F was then used to simulate and optimize PASSER's best settings. These TRANSYT-7F results were further compared with the results from PASSER II studies. Because of the amount of data reduction required, a simplified version of the PASSER II program was developed for direct data processing by the statistical analysis system (SAS). Performance measures of effectiveness (MOEs)--such as delay, stops, and queue clearance--were analyzed under regular PASSER II runs, TRANSYT-7F-simulated PASSER II best-setting runs, and TRANSYT-7F optimization runs.

Figure 5 shows an example of the performance measurement of average delay on one approach compared with the spacing variations given that all other variables remain constant. The simulation result also indicated the wide variation of operational performance with respect to the spacings of progression systems. It also showed the results from different platoon dispersion models applied in these two models. They both confirmed that the rule-of-thumb or ideal spacing for good arterial progression is between the distance of 1/4 mile (1,320 ft or 440 m) and 1/3 mile (1,760 ft or 580 m).

Traffic control scenarios were devised to test the effectiveness of signal interconnection under different geometric and traffic levels. Guidelines under conditions of different intersection spacings and left-turn percentages were established. The TRANSYT-7F was primarily used to examine the effects of intersection spacings and the percentages of traffic turning left both off and onto the arterial. The computer program evaluated needs for interconnection. Selected NETSIM runs, similar to the TRANSYT-7F runs, were conducted for investigating actuated arterial control on a four-intersection

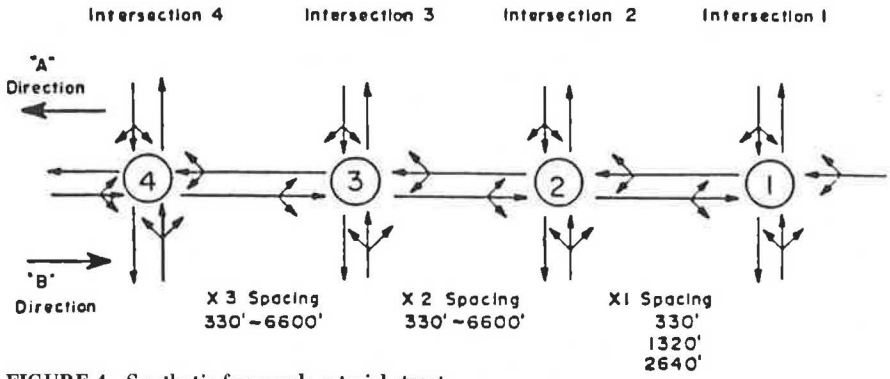


FIGURE 4 Synthetic four-node arterial street.

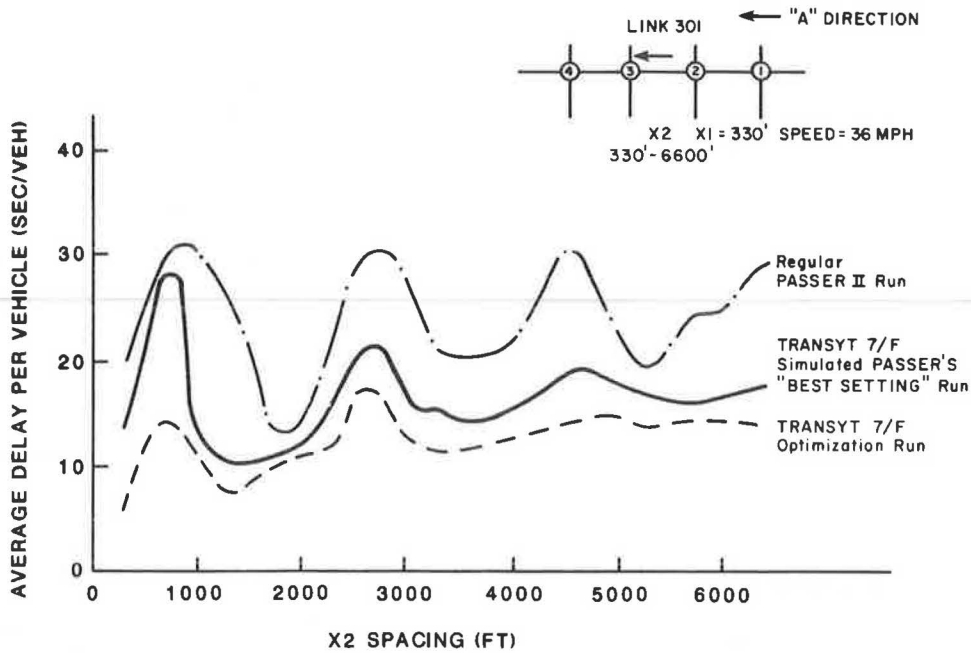


FIGURE 5 Selected link performance MOE versus intersection spacing under PASSER II run, TRANSYT-7F-simulated PASSER II run, and TRANSYT-7F optimization run.

system. It evaluated whether intersection spacing and the percentage of turning traffic would affect actuated control similar to how they affected pretimed control.

Field Study

Two field studies were performed in the travel time and delay study. The first field study was made on Lamar Boulevard and U.S. 183 in Austin, Texas. Both are high-volume high-type facilities; Lamar Boulevard has low-to-medium speed operations and U.S. 183 has medium-to-high-speed operations. The study showed that good progression was available throughout the two systems regardless of the variable spacing and the saturated operations along the two arterials. However, the field study did not provide enough validation of the simulation analysis because the percentage of left-turn traffic volume and the corresponding traffic volume were not properly identified.

Nevertheless, it was indicated in the travel time and delay study that a positive relationship existed between the travel time delay and the travel time versus background cycle length used. As indicated in Figure 6, the travel time delay was plotted against the travel-time-to-cycle-length ratio for both signal systems. It is suggested by Figure 6 that travel time delay within the interconnected signal system decreases gradually from 0.4 to 0.6 of travel-time-to-cycle-length ratio and then increases as travel time increases. It also indicated that the combined travel time and background cycle length ratio can provide a better indicator than can distance alone to represent relationships among distance, travel speed, progression design speed, and level of traffic volume for a coordinated system.

The second detailed field data collection was performed on TSDHPT's NASA 1 FACTS system to collect data on signal timing, travel time, delay, and queue stops. The NASA 1 computerized traffic signal control system, south of Houston, was selected to calibrate the computer models. The cross streets were Kings Row, El Camino, Space Park, Nassau Bay, Point

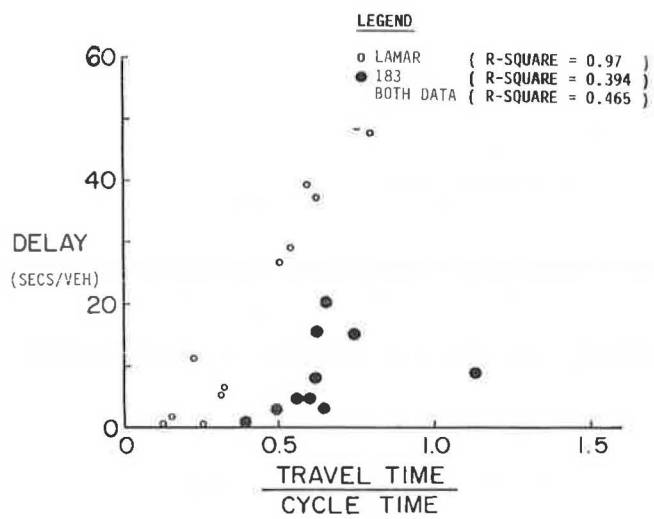
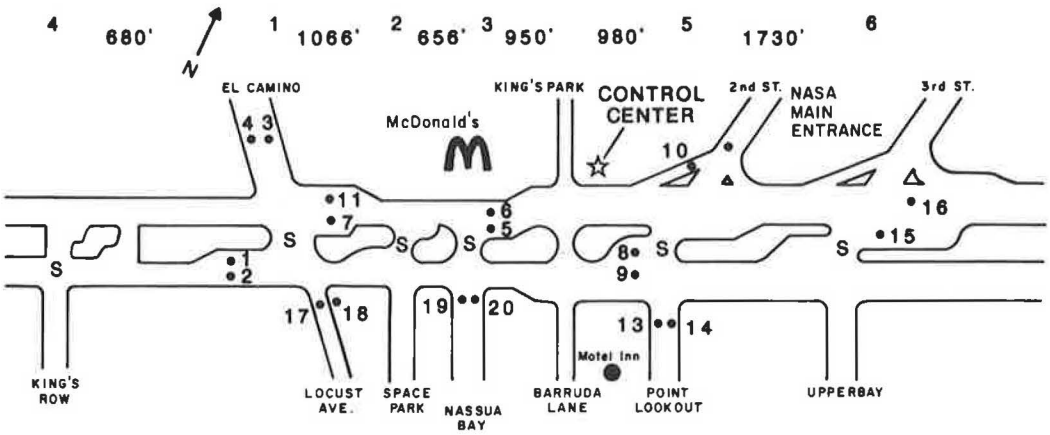


FIGURE 6 Summary of field study results in Austin, Texas: travel time delay versus travel-time-to-cycle-length ratio.

Lookout, and Upperbay, as shown in Figure 7. Field data were collected during the noon rush and off-peak periods. Calibration of the combined PASSER II and TRANSYT-7F runs and validation of operational measures were then completed. Interconnected studies were conducted on Tuesday, Wednesday, and Thursday of one week and isolated intersection studies were made during the following week. PASSER II optimized phasing was used at all intersections during both simulation and field studies. Data collected for the test arterial were used to calibrate the operational scenarios and factor levels in the PASSER II and TRANSYT-7F runs. The basic data include the following measurements: arterial street, arterial link, cross street, intersection, and arterial performance.

Field data were used to calibrate computer models and provide real-world data in evaluating the computer models. Essentially, this field study was used



*-Modified 'T' Intersection
Westbound Traffic has Protected Green

FIGURE 7 Texas SDHPT NASA 1 FACTS system.

to investigate the effects of intersection spacings, offsets, and average delay measurements in response to the different signal-timing settings in the field. These data were then applied to establish guidelines for interconnection of isolated traffic signals. The data were collected from the stop-delay study, travel time and delay study, and platoon dispersion study. The volume count was collected with assistance from TSDHPT's D-19 personnel by using the NASA 1 FACTS system sampling detectors. Selected queue counts and stop delay measurements were collected manually at each signalized intersection location.

STUDY RESULTS

Both the simulation and field data were summarized to study the conditions for effective arterial traffic signal control. In this study, realistic and quantitative relationships were established among the study factors important to the interconnection decisions. One factor used for predicting the potential interconnection benefits is the interconnection desirability index. Another measure is the estimated arterial delay experienced by the motorists.

Simulation models were used as a theoretical testbed to enumerate study conditions that cannot be easily reproduced or controlled in the field. Emphasis was placed on investigating the generalized relationships among the study factors and their sensitivities with respect to systemwide performance. This simulation mainly establishes linkage between the estimated arterial link delay and the proposed interconnection conditions. Test scenarios were examined for accurate and reliable representation of the candidate application sites.

In the study, the operational performance for various conditions was investigated, assuming that the potential interconnection became operational. Two separate analyses were investigated: the interconnection index analysis, and the combined PASSER II and TRANSYT-7F analysis. In the first analysis, the basic variation of the interconnection desirability index was studied as a function of intersection spacings, progression design speed, intersection volume levels, and left-turn percentages. In the second analysis, estimated arterial street performance statistics were combined by applying the approaches of both the PASSER II and TRANSYT-7F programs.

Because of the inherent variability of the various field conditions, this study evaluated whether the interconnection can provide effective operation without undue delay to the arterial signal system for given already-installed traffic signals. The combined PASSER II and TRANSYT-7F runs evaluated the effectiveness of interconnection versus interconnection with progression phasing under different volume levels. In this approach, the detailed simulation capability of TRANSYT-7F was applied to predict platoon travel behavior, and the optimized cycle length and phase sequence optimization in PASSER II were used.

The field and simulation data were used to determine where interconnection of a series of isolated signals is desirable in improving traffic operations. It was found that the factors influencing the arterial link delay the most are the following: traffic volume level (the resultant Webster's minimum delay cycle length), intersection spacings, travel speeds, and left-turn movement percentages. Three types of sensitivity analyses were made to investigate the variability of an average link delay per vehicle against the major study variables. The results are shown in Figures 8-10. Figure 8 shows the effects of intersection spacing on average link delay; spacing ranges from 330 ft to 2,640 ft. Figure 9 shows the average link delay versus low, medium, and high volume levels; it has a saturation flow ratio of from 0.50 to 0.83. Figure 10 shows the average link delay according to three different progression design speeds of 27 mph, 36 mph, and 45 mph. It was also found that the dispersion effects of the progression platoon and the background cycle used can influence the total progression system operations.

The results indicated that the lower arterial link delay occurs at a distance of 1/4 to 1/3 mile or 0.4 to 0.5 cycle length of travel time during a two-phase operation, or 0.35 to 0.55 cycle length during a four-phase operation. This finding confirmed that the ideal progression spacing is approximately the travel time of one-third to one-half of the cycle length multiplied by the design speed for any generalized arterial.

The results demonstrated that highly fluctuated relationships existed between the potential arterial link delay and the intersection spacings. It also showed that the circular pattern of average vehicular delay due to the possible progression platoon

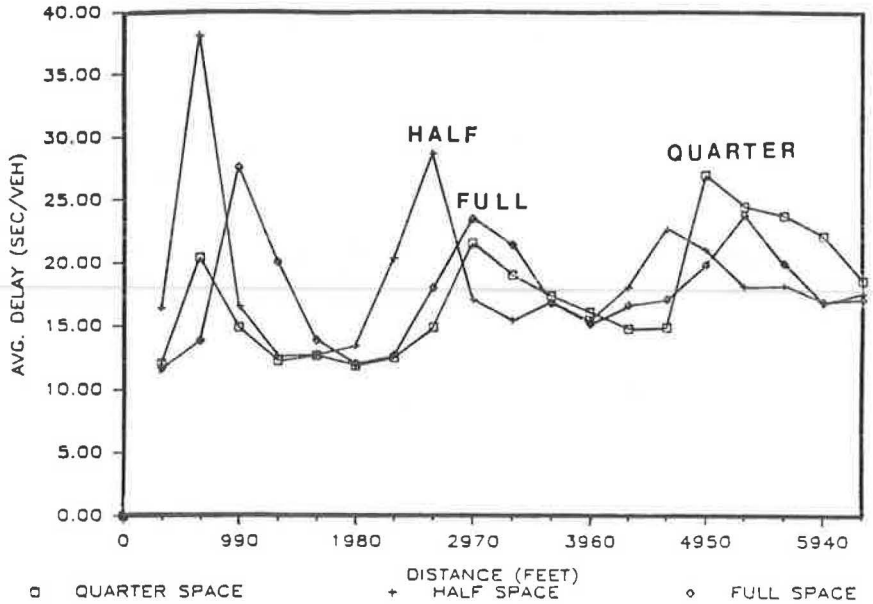


FIGURE 8 Summary of simulation study results: effects of intersection spacing.

propagated downstream from the upstream traffic signal even under good arterial progression operation. As indicated, the effectiveness of the traffic signal interconnection relies heavily on the following study factors:

- Location of ideal spacing,
- Intersection traffic volume,
- Left-turn percentage,
- Intersection spacing,
- Progression design speed, and
- Arterial platoon travel speed.

CONCLUSIONS AND RECOMMENDATIONS

Improving urban mobility requires the efficient utilization of existing urban arterial facilities. Signalized arterial design requires concurrent functioning of existing traffic control devices and proper signal-timing settings as one unit in the field. The ability of signalized intersections to move traffic depends on the physical intersection layouts as well as the signalization used. An efficient procedure was developed with which traffic engineers can make decisions about interconnections of isolated arterial traffic signals to optimize arterial traffic operations. Described is the development of interconnection guidelines for minimizing arterial systemwide delay and maximizing progression in multiphase traffic signal systems.

Traffic signal optimization depends heavily on the relationships of the intersection spacings, travel speed, cycle length, roadway capacity, and side friction along the arterial. Effective traffic signal operations will provide a safe crossing gap

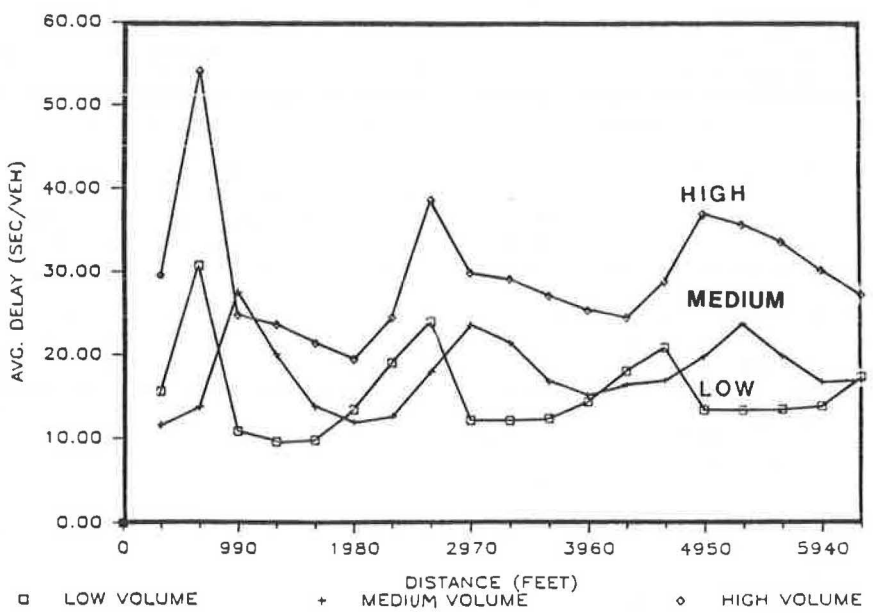


FIGURE 9 Summary of simulation study results: effects of traffic volume level.

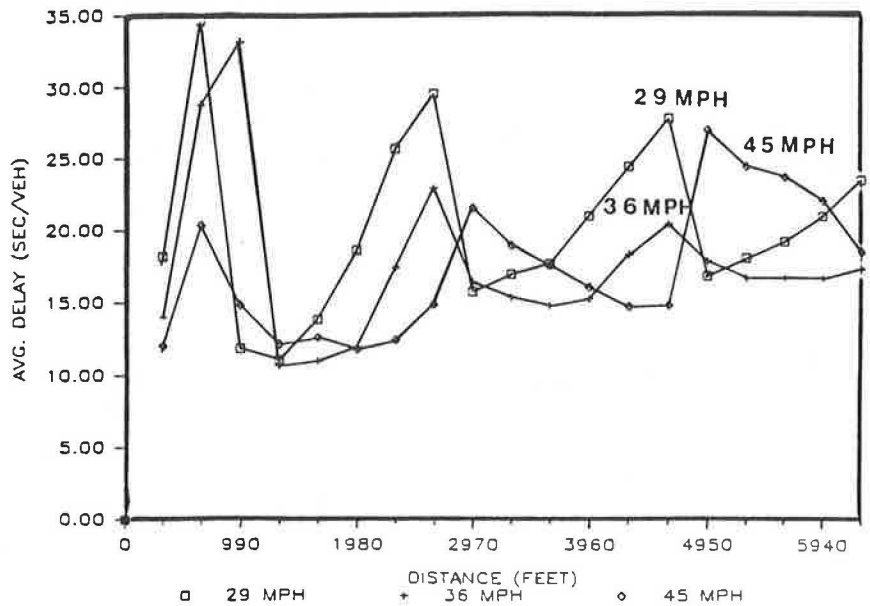


FIGURE 10 Summary of simulation study: effects of travel speed.

for the cross-street turning traffic and guide the randomly arriving traffic into compact platoons. Guidelines can assist traffic engineers in making decisions about proper arterial traffic signal interconnection. Although the interconnection index system provides a realistic analysis of the effect of interconnection of a traffic signal at a specific location, it is recognized that it can serve only as a tool to aid the judgment of the traffic engineer, and is not an absolutely final answer that would overcome the need for experienced and objective analysis.

Faced with highly fluctuated patterns of traffic arrival, good signal patterns can tailor the arterial traffic signal control to suit sensitive traffic demand. It was found that a proper interconnection operation could alleviate total system traffic loading to a certain extent without sacrificing good progression operation. However, care should be taken to monitor travel speeds against design speeds to suit traffic demand for successful arterial progression. It was also found that close monitoring of the traffic flow in the field is necessary to assure successful signal-timing implementation and minimize delay from the maximum progression calculation.

Additional research is recommended on (a) calibration of platoon dispersion models under various traffic signal control strategies, (b) field validation of the interconnection warrants, (c) evaluation of the impact of traffic progression in arterial signal optimization, and (d) development of better measurements for describing the quality of arterial progression of traffic signal systems. Studies are needed to extend this research to permit on-line traffic control network configuration of traffic signal control systems in order to control open networks rather than closed networks.

Revision of the internal simulation mechanism should be made inside the NETSIM program to reduce the step size. Thus, the simulation cost for coordination of fixed-time and actuated signals under isolated or interconnected operations is decreased. Special study is recommended that will compare the progression platoon size and the progression bandwidth in order to use green time efficiently without sacrificing the progression solution to further minimize total arterial system delay measurement.

ACKNOWLEDGMENTS

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Control of Congestion at Highly Congested Junctions

A. D. MAY and F. O. MONTGOMERY

ABSTRACT

The results are reported of experiments in Bangkok designed to test fixed-time signal control as a replacement for manual police control at highly saturated junctions with flows of up to 15,000 vehicles per hour. Data collection methods, use of video techniques, analysis procedures, and resulting traffic-junction performance parameters are described. The results demonstrate that it is possible to replace police control at such junctions, and that at linked junctions substantial savings in travel time can be obtained. However, existing computer-based techniques do not readily produce optimum control strategies, and additional work is needed to enable them to do so. The problems caused by substantial fluctuations in demand and in operating conditions are discussed, and proposals for further work are outlined.

The results to date are reported of a study of the appropriateness of using procedures of the United Kingdom for traffic signal control in a highly congested developing city, Bangkok, and the feasibility of using fixed-time signal settings to release traffic policemen for more important duties.

The design of individual signalized junctions follows long-established procedures (1); although some recent studies have suggested modifications to the values used for saturation flow and passenger car unit (pcu) equivalents (2), others suggest that there has been little change in parameters during at least 20 years (3). Similar techniques are applied elsewhere in the developed world (4,5). Procedures for optimizing the fixed-time control of networks of signal-controlled junctions are also long established, and the Transport and Road Research Laboratory's TRANSYT program (6) is generally accepted as the most successful optimization method. Recent developments in vehicle-actuated control of signal networks offer the potential for significant improvements in the TRANSYT method (7).

However, it is suggested by recent experience in Bangkok that neither the procedures for individual junction design nor those for network control may be appropriate for the much more heavily congested conditions experienced in cities in the developing world. Bangkok has a widely dispersed land-use pattern; only 9 percent of its land is allocated to

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roads, and those roads are often cul-de-sacs connected by a coarse network of major roads (8). As a result, the major junctions have very high flows sustained for more than 12 hr per day, with no pronounced directional peaks and often with 50 percent or more traffic turning (9). Although vehicle types are not dissimilar to those in the developed world, the vehicle mix contains a much higher proportion of motorcycles and driving standards are markedly different.

Even though all signalized junctions have fixed-time controllers, and approximately 50 junctions are coordinated in an urban traffic control system that initially proved successful (10), it is now common practice for traffic police to switch the controllers to manual for several hours per day. This practice involves a substantial number of traffic police during each peak period, provides no certainty of reducing congestion at individual junctions, and removes the ability to achieve additional reductions in congestion by signal coordination.

The study was designed to test the possibility of introducing improved fixed-time signal settings at individual junctions and groups of coordinated junctions. Such settings would need to cope with the different traffic patterns and vehicle types and the high levels of congestion and variability experienced. They would, if successful, increase system capacity; reduce congestion, journey times, and variability; and release a substantial number of traffic police to perform other tasks that cannot be automated.

STUDY PROGRAM AND METHOD

The study involved the following:

1. Identification of one isolated junction at which to test signal settings unaffected by conditions at adjacent junctions, and a group of junctions sufficiently closely spaced to require linkage of signal settings;
2. Determination of appropriate parameters for describing traffic performance at junctions; in particular, through car units (tcus) (11) for different types of vehicle and turning movement (including the frequent U-turn maneuver) and saturation flows on individual junction approaches;
3. Measurement of demand for individual movements by different classes of vehicle at individual junctions, and the variation in that demand during the 2.5-hr evening peak and from day to day;
4. Use of currently available programs to determine signal settings for the isolated junction, and comparison of the performance of those settings with those used during manual operation by police;
5. Use of currently available programs and procedures (modified as necessary in the light of Point 4) to determine signal settings at the group of linked junctions, and comparison of the performance of those settings with those used during manual operation.

Site selection was substantially disrupted by the introduction by police, against engineering advice, of a major one-way system one month after the beginning of the project. Because the future of the one-way system was (and still is) uncertain, no junctions directly affected could be selected. Other factors considered were the need for high levels of saturation, frequent police intervention, and suitable vantage points for surveys. The following sites were finally selected:

- An isolated four-way junction in northeast Bangkok, 2 km from the next signalized junction, with a peak throughput of 8,600 veh/hr, and
- A set of four four-way junctions on a major east-west radial to the east of the city center and immediately south of the one-way system; it is generally accepted that this site is one of the most congested parts of the city. Interjunction spacings are 600, 350, and 900 m, and the busiest junction has a peak throughput of 15,000 veh/hr.

Data requirements were of three kinds: junction performance, traffic demands, and system performance. All data on junction performance were collected by using video film taken from high buildings. Video was selected because it could provide a permanent record and because of the impracticability of roadside recording of such large flows. Initial surveys were conducted for 2 or 3 days at each junction for 2 hr during the afternoon peak period.

Subsequently, each junction was filmed on a number of days during which experimental settings and police performance were being monitored. All data were stored on floppy disks by using the VISTA system developed by Wootton Jeffreys PLC and an Apple IIe microcomputer. Transfer required between 4 and 14 person-hours per hour of data, depending on the detail of information to be retained, and therefore compared well with the labor costs of roadside surveys. Because junctions were regularly saturated, throughput flows observed on film did not reliably represent demand. Instead, input flows were recorded manually, per minute, unclassified, at vantage points (usually footbridges) upstream of the longest queues.

The main measures of system performance that were of concern to the police were queue lengths; traffic engineers were mainly concerned with travel times on individual movements. Because both groups needed to be convinced of the success of the experiment, both types of data were collected. At the isolated junction, one set of eight observers recorded travel times from the input flow observation points until vehicles crossed the stop line. Staff availability limited this survey to using number plate matching for white automobiles only for the seven most important movements. Another four observers recorded queue length at the start of a green signal on each arm and also noted any disruptions of traffic. In the four-junction complex, travel times on 14 of the 16 links were recorded by observers on tall buildings and on the other two by pairs of observers using number plate matching. Input flows were recorded by another 14 observers, and a group of up to 12 local and project engineers observed queue lengths and sources of disruption.

RESULTS

Junction Characteristics

Junction characteristics were determined from the initial survey films, primarily by using the VISTA-based data records. In the initial analysis, attempts were made to derive tcu values and saturation flows together for each turning movement by using asynchronous methods (12). It has since been shown that this procedure introduces bias into the analysis (13); the procedure was abandoned in favor of sequential analysis. Values of tcu were determined by analyzing headways on six approaches to three junctions on a lane-by-lane basis. The headway of a specified type of vehicle was defined as the time from the back of the preceding vehicle's crossing the stop line to the back of the specified vehicle's doing so. Using this definition, the tcu value of

the specified vehicle type is approximately the ratio of its mean headway to that of passenger cars straight ahead. The values obtained are given in Table 1.

Motorcycles were treated separately because of their behavior. During the red period, they percolate to the front of the queue, so that at the start of the green signal a large bunch of motorcycles is in front of the other vehicles. When the signals change, this bunch moves off quickly, the lead motorcycles reaching speeds of up to 100 km/hr. These motorcycles all leave within the first 6 sec and have a tcu value approaching 0. The remaining motorcycles have a tcu value that is affected by the type of vehicle near them and hence is determined by the lane that they use:

TABLE 1 Values of tcu for Specified Vehicle Types

Type of Vehicle	Value of tcu
Samlor (three-wheeled taxi)	0.89
Light-medium commercial vehicle[a]	1.54
Bus	1.84
Automobile making right turn (unopposed)[b]	1.00
Automobile making acute left turn	1.26
Automobile making U-turn	1.26

[a]Heavy commercial vehicles are banned from Bangkok streets during the working day.
[b]Traffic drives on the left in Thailand.

Position of Motorcycle	Value of tcu
Motorcycles in curb lane (or lane adjacent to bus lane)	0.65
Motorcycles in right-turn lane	0.62
Other motorcycles	0.53

The ratio of motorcycles reaching the head of the queue to those crossing within the main traffic stream was determined by the effective red/green (R/G) ratio for the approach. For the isolated junction, this ratio was found to be 0.48 R/G, and for the linked junctions, 0.31 R/G. The differences appear to be explained in part by number of lanes, lane width, and amount of weaving traffic.

Saturation flows were calculated for each movement rather than for each lane because lane discipline was not always reliable. The method used was based on cumulative discharge graphs plotted for each cycle, from which the first 6 sec and any periods of undersaturation were excluded. The mean saturation flow per lane of 2,060 pcu/hr for the four-junction complex compared well with the 1,980 ±210 pcu/hr found in an earlier study of some of the approaches (14). However, several of the average plots showed a marked falloff in saturation flow with stage length; this implies that the longer cycles used by the police are less likely to be efficient.

Isolated Junction Experiments

After appropriate parameters for the isolated junction were calculated, signal settings were determined by using the programs SIGSET and SIGCAP, which are microcomputer-based (15,16). Initial experiments with these settings in November 1984 were unsuccessful and ran the risk of losing police support for additional experiments. The cause was traced to errors in estimates of input flow and saturation flow, both of which appeared to fluctuate substantially from day to day.

Signal settings were recalculated by using improved estimates of input flow and saturation flow, and with an adjustment for early motorcycles, which involved for each state omitting the first 6 sec of input flow and transferring 6 sec to the inter-green (all-red interval). After recalculation, an additional set of experiments was carried out in March 1985. Experimental days were alternated with police days, and signal settings were adjusted on four occasions in light of experience. The experimental settings all used a 180-sec cycle, and only varied by up to 8 sec in individual stage lengths.

In contrast, the police timings were extremely variable, with cycle lengths ranging from 180 to 380 sec, and with a variety of stage sequences. It appeared that the police were basing their timings on observation from the controller of the congested eastern approach, which they attempted to keep clear, and on occasional reports from police motorcyclists. This operation involved up to four officers at any one time. From time to time, the police reverted to the preexisting fixed-time settings, which (a) were markedly different from the experimental ones, (b) imposed considerable delay, and (c) almost certainly contributed to police mistrust of fixed-time control. On Wednesday, the Tuesday experimental timings were inadvertently left in the controller, and the police elected to use these for a substantial period.

In Table 2, data are presented on the input flows, saturation flows, queue lengths, and average delay recorded for the critical eastern and western approaches on the 6 days. Although the input flows were reasonably stable, they varied by up to ±10 percent on an hourly basis, with marked changes in peak hour and peak direction. Saturation flows exhibited coefficients of variation between cycles of up to 29 percent; as a result, even day-to-day differences of up to 385 tcu/hr/lane were not significant.

Comparison of travel times indicated that results were extremely sensitive to the signal settings imposed, stressing the need for careful adjustment of the times initially recommended. Two problems in particular appear to have contributed to the error in the initial signal settings. The first problem was the treatment of the motorcycles during the first 6 sec; as a result, the distribution of delays on the different approaches was almost certainly underestimated. The second problem was a particular case of reduced saturation flow on the western arm caused by long queues of right-turning vehicles; this was difficult to detect statistically, but was readily apparent to observers.

The final signal settings, on Day 5, represented a substantial improvement over those on Days 1 and 3. Queues and delays under police control were consistently better, although those on Day 5 were slightly worse than those on Day 6. In Table 3, which shows total vehicle delay for the eight major movements per day, additional demonstration of this is provided. It can be seen that differences in signal operation resulted primarily in a trade-off between the east-west and west-east movements, with lesser trade-offs between east-north, north-south, and north-east movements and west-south, south-east, and south-north movements. The difference in delay between the two days represents only about 3 percent of the total delay incurred, confirming that fixed timings were performing virtually as well as police control, even if the distribution of delays was different.

The police were sufficiently impressed by the timings to retain them for a four-week trial in

TABLE 2 Isolated Junction Experiments and Results

Day	1	2	3	4	5	6
Day of week	Thursday	Monday	Tuesday	Wednesday	Thursday	Friday
Date	7/3/85	11/3/85	12/3/85	13/3/85	14/3/85	15/3/85
Timings[a]	E	P	E	P[b]	E	P
Input flows[c]						
4:00 p.m.						
West	1,942	1,931	1,799	1,814	1,836	1,816
5:00 p.m.						
East	1,663	1,625	1,806	1,563	1,568	1,664
West	1,798	1,752	1,768	1,895	2,000	1,973
6:00 p.m.						
East	1,700	1,612	1,532	1,603	1,460	1,645
Saturation flows[d]						
West-east						
Mean	4,364	4,089	4,205	4,400	4,087	4,225
Standard deviation	393	465	567	294	718	440
West-south and west-west						
Mean	1,574	1,557	1,629	1,412	1,695	1,332
Standard deviation	277	401	472	389	452	408
East-west						
Mean	2,701	2,669	2,496	2,464	2,502	2,710
Standard deviation	313	299	389	529	501	326
East-north						
Mean	1,543	1,527	1,536	1,387	1,310	1,695
Standard deviation	145	186	204	274	263	410
Queue lengths[e]						
4:00 p.m.						
West	900	200	250	110	210	700
5:00 p.m.						
East	480	270	770	130	320	180
West	NA	370	130	130	130	300
6:00 p.m.						
East	350	370	630	270	380	320
Average delay[f]						
4:00 p.m.						
West-east	262	117	165	91	124	293
East-west	269	114	244	103	198	98
5:00 p.m.						
East-north	294	181	317	125	316	104
West-east	420	125	107	110	77	180
East-west	221	118	445	153	290	111
6:00 p.m.						
East-north	205	200	619	271	347	84

[a] E = using experimental settings; P = timing set by police.
[b] However, police inadvertently used experimental timings for much of the period.
[c] In veh/hr, excluding motorcycles.
[d] In tcu/hr for movement shown, mean and standard deviation.
[e] Average at start of green (m).
[f] Average (sec) from survey point to entering junction; delays for west-south and west-west not measured.

TABLE 3 Isolated Junction Experiments: Comparison of Recorded Delays on Days 5 and 6

	Day 5		Day 6		Day 5 and Day 6
Movement	Throughput[a]	Average Delay[b]	Throughput[a]	Average Delay[b]	Total Delay[c]
East-west	2,633	247	2,221	102	423
East-north	744	361	855	102	181
North-south	1,338	137	1,045	120	58
North-east	1,455	140	1,458	106	50
West-east	3,715	95	3,709	213	−437
West-south[d]	413	128	412	246	−49
South-east	1,565	131	2,162	162	−145
South-north	1,346	94	1,362	110	−22
Total	13,209	155	13,224	150	
Total delay[c]	2,043		1,985		58

[a] Vehicles except motorcycles during 2.5-hr period.
[b] Sec/veh.
[c] Veh-sec x 10^3.
[d] Estimated.

which the officer on site only intervened when he considered it necessary. Unfortunately, the trial had to be terminated after two weeks when major road works severely disrupted the eastern arm. However, during those two weeks there was no need for manual intervention.

LINKED JUNCTION EXPERIMENTS

By using the experience of the isolated junction experiment, improved estimates were made of saturation flows at each of the four linked junctions. Detailed movements were estimated from input flows and turning movement counts; however, because of the time taken to analyze these data, input flows were collected in April 1985, some 3 months before the experiment, and turning movements were taken from the March 1984 video record. These were initially input into SIGSET and SIGCAP, which suggested a 240-sec cycle, and later into TRANSYT 8 (17) for which the omission of the first 6 sec of flow was replaced by a modification in the start lag to allow for the zero-tcu motorcycles.

The input management procedure in TRANSYT 8 was used in an attempt to avoid queues on the critical 350-m link blocking the upstream junction. Because of the known variation in flows, a range of tests was conducted with TRANSYT using flows in the range of ±15 percent of the April average and cycle lengths of 150 to 270 sec. These indicated an optimum of 180 sec at flows over 1.05 of the April flows.

During the week before the experiments began, input flows were checked and found to be substantially different from the April ones; it is indicated in Table 4 that the differences in mean flows on the 10 input arms varied by between -23 percent and +41 percent. It was known that the April flows had been recorded during a school holiday period, but the scale and distribution of the differences was wholly unexpected. It appears that they represent at least in part route changing by drivers in response to prevailing conditions and patterns of police control. Even within the individual survey periods, flows varied substantially, with coefficients of variation of up to 15 percent. TRANSYT times had to be recalculated for the new flows, but a 180-sec cycle was still the optimum.

The splits and offsets recommended by TRANSYT were implemented on the first experimental day, Tuesday, July 2, 1985. Over a total of 5 experimental days, 18 adjustments were made in light of experience. Most of these were minor changes in stage length, which typically ranged from 1 to 6 sec. One change in sequence was introduced in order to avoid a delayed right turn reducing saturation flow for the through movement.

The largest changes were made in the offsets, which in one case was changed by 57 sec. It appeared, in particular, that the input management procedure of TRANSYT 8 was imposing undue delays on the approach arms. It was also clear that the offset calculations were inappropriate in highly saturated conditions where progression could not be achieved, and that offsets would have been better based on queue management requirements (18). Again, the timings contrasted markedly with those used by police. Detailed results are not yet available, but cycle lengths of up to 8 min have been recorded, and at one junction a nine-stage cycle was frequently used.

In Table 5, summary statistics are presented for the five experimental days and the five monitored police days. On the two Mondays, a substantially lower level of total travel occurred (in veh-km/hr) and the two days were excluded from further comparison because complete travel time data were not available for Day 1. For the remainder, little difference in total travel occurred, although the distribution by approach and by time of day almost certainly varied substantially.

For the four experimental days, total travel time was 180 veh-hr/hr or 6 percent lower, and average system speed 6 percent higher than on the four police days. The four experimental days included Day 2, on which experimental signal settings were clearly in need of adjustment; Day 3, on which a level crossing closure blocked the main output from the system for up to 7 min at the height of the peak; and Day 5, on which there was a half-hour tropical rainstorm and a subsequent half-hour reduction in output on one arm, apparently caused by parents picking up children from school.

No serious incidents occurred on police Days 7 to 10. Day 4, also on which no serious incidents occurred, and for which total travel was 3 percent higher than average, achieved a saving of 640 veh-hr/hr, or 21 percent, and a 29 percent increase in system speed. This suggests that in incident-free conditions substantial savings in travel time are possible.

In practice, no day was entirely free of incidents. In addition to the major incidents on Days 3 and 5, several vehicle breakdowns and minor accidents occurred, including one that reduced capacity by 20 percent for three cycles at one stop line, and two periods of manual operation to permit distinguished persons free movement through the network. In all cases, these incidents led to longer queues, which on occasion blocked the junctions. It was noticeable that, when this happened, drivers con-

TABLE 4 Linked Junction Experiments: Variation in Input Flows

Arm[a]	A	B	C	D	E	F	G	H	I	J	Total
5 April Days											
Mean	2,305	2,063	1,406	1,189	2,028	3,024	2,358	1,765	802	704	17,642
Standard deviation	83	36	77	138	54	72	122	59	41	34	292
6 June and July Days											
Mean	1,765	2,538	1,801	1,264	1,835	2,533	3,270	1,691	897	994	18,589
Standard deviation	254	34	85	184	130	239	271	117	70	49	554
Change (%)	−23[b]	+23[b]	+28[b]	+6	−9	−16[b]	+39[b]	−4	+12	+41[b]	+5

Note: Flow is vehicles excluding motorcycles, 4:00 to 5:00 p.m.

[a] B C D E
A—+—+—+—F
J I H G

[b]Significant at 95 percent level.

TABLE 5 Linked Junction Experiments: Summary of Results

Day	Day of Week	Date	Timings[a]	Total Travel[b]	Total Travel Time[c]	System Speed[d]
1	Monday	1/7/85	P	20,508	NA	NA
2	Tuesday	2/7/85	E	21,398	2,986	7.17
3	Wednesday	3/7/85	E	21,563	3,103	6.95
4	Thursday	4/7/85	E	22,169	2,336	9.49
5	Friday	5/7/85	E	21,325	2,744	7.77
6	Monday	8/7/85	E	20,056	2,847	7.04
7	Tuesday	9/7/85	P	21,815	2,791	7.82
8	Wednesday	10/7/85	P	21,772	2,589	8.41
9	Thursday	11/7/85	P	21,776	3,189	6.83
10	Friday	12/7/85	P	21,322	3,323	6.42
2-5			E	21,616	2,792	7.74
7-10			P	21,671	2,973	7.29

[a]E = using experimental settings; P = timings set by police.
[b]In veh-km/hr averaged over 2.5 hr.
[c]In veh-hr/hr averaged over 2.5 hr.
[d]Ratio of b to c in km/hr.

tinued entering the junction, often violating the red signal to do so. The police were clearly reluctant to control such behavior, and on one occasion the local traffic engineer elected to revert to manual control for a 12-min period to ease the situation. However, the results indicate that even under such conditions, the fixed-time settings can perform at least as well as police operation.

ADDITIONAL ANALYSIS

The experiments have demonstrated that automatic control is capable of controlling traffic even with saturation levels of virtually 100 percent, variable flows, and frequent incidents. It was noticeable, however, that several adjustments had to be made to the timings calculated by using standard analysis tools, and that occasional incidents might encourage local staff to introduce manual operation. Additional work is proposed on these issues in an attempt to produce more appropriate analysis tools and guidance on the best procedures for manual intervention. The main issues on which additional analysis is planned are described in the following four subsections.

Recalculation of Signal Timings

The timings calculated initially were sufficiently different from the final ones to increase delay substantially. Although it will always be necessary to make some adjustments to initial timings, excessive delays may lead the police to cancel the fixed times, and it may be difficult anyway to make the most appropriate adjustments, either because of lack of resources or the complexity of the network. A clear case therefore exists for improving the initial calculations. For individual junctions, the most obvious sources of error are in tcu values, saturation flows, and input flows generally, and more specifically in the treatment of lead motorcycles and of reductions in saturation flow. Timings are to be recalculated by using SIGSET and SIGCAP, using parameters measured for the day in question, in an attempt to replicate the timings that were found to be successful.

Sensitivity to Varying Conditions

It appears likely that the range of input flows observed, particularly for the linked junctions, would in practice merit a range of signal settings. In a fixed time system, such variations are not practicable, and the settings selected could be determined on a number of different bases. One possibility is to use average flows; however, this could lead to excessive delays on high flow days. Another possibility is to use maximum flows; however, it may be that different distributions of flow arise even for a given high total flow. A third possibility is to develop timings that minimize the delay experienced over the range of conditions. It is on the last of these three possibilities that additional analysis is being concentrated.

A related issue is the question of when to introduce plan changes, if flows vary sufficiently to merit this. Because changes of even a few seconds per stage can add considerably to queue lengths, choice of both time and size of plan change can be critical. This issue is also to be examined further.

Determination of Offsets

It was clear from the linked junctions experiment that TRANSYT's procedures for determining offsets were inappropriate; it also appeared that the input management facility imposed unnecessarily severe restrictions. When junctions are saturated, progression in the normal sense is impossible because each vehicle will be delayed for at least one cycle at each junction. The key requirement, instead, is to avoid the disruption of upstream junctions by queues and, if possible, to reduce the number of standing waves in a queue.

Detailed observation indicated that problems did not occur at junctions, provided that the tail of the queue was moving by the time that the stage for its main feed ended. This suggests a somewhat less conservative basis for queue management than that proposed elsewhere (18). If stationary vehicles remained in the junction, other drivers were encouraged to enter the junction illegally, and other movements were disrupted.

Attempts were made to increase the offset on the shortest link to avoid this, but such attempts were not completely successful because queue lengths fluctuated considerably from cycle to cycle. The whole of this length was filmed, and the film is being analyzed further to identify a basis for offset calculation. On longer links, queue lengths were rarely sufficient to block junctions, but it was noticeable that queue formation was impaired when a standing wave occurred. This suggests the need to determine offsets so that the new platoon joins the queue just as it starts to move.

As part of these offset recalculations, it is intended that the need for management of input flows be assessed, as well as the most appropriate basis for doing so. In particular, the need to impose control on free left turns, which have been identified by others as a source of queue management problems (18), will be further investigated.

Incident Management

Experience suggests that capacity-reducing incidents are likely to be frequent occurrences, and that at high levels of saturation they will have potentially serious effects on congestion. If police can be diverted from regular manual control, it is clearly merited that they be used to avoid the occurrence of incidents and to assist them in determining when to intervene and how to do so. As an input to this process it is intended to use the video record to calculate the congestion costs of recorded incidents

and to demonstrate the disruptive effect of drivers' actions after the initial incident. As a basis for developing guidelines for manual intervention, a comparison will also be made between the performance of manual and fixed-time control during periods with long queues.

CONCLUSIONS

The major conclusions from the study to date are as follows.

1. It is possible to use automatic control of traffic signals in the conditions observed, which are among the most congested in the world.
2. Although the travel time savings at an isolated junction may be small or nonexistent, the benefits of achieving good coordination at a complex set of junctions are likely to be substantial; analysis suggests that a saving of up to 20 percent may be achievable in incident-free conditions and that fixed-time control, even during major incidents, performs at least as well as police control during incident-free conditions.
3. An additional major benefit is in the release of police manpower for other duties; it appears in particular that it is merited that their attention be diverted to the enforcement necessary to avoid congestion-inducing incidents or to mitigate their effects.
4. Although standard analysis tools provide a suitable starting point for determining timings, considerable additional adjustment is likely to be required; means of improving the initial predictions are being investigated.
5. The major emphasis in highly congested conditions must be on queue regulation, and existing analysis procedures need to be modified to deal more adequately with queue prediction and limitation.
6. At levels approaching 100 percent saturation, performance is very dependent on fluctuations in traffic conditions and on the occurrence of incidents; additional work is needed to develop methods for determining delay-minimizing settings for such conditions.
7. The substantial variations in flows from day to day and month to month, coupled with the sensitivity of performance to such variations, place major demands on survey design.
8. The use of video-based survey techniques was of considerable benefit in simplifying data collection, permitting flexibility in analysis, and providing illustrative material for training purposes.

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Congestion-Based Control Scheme for Closely Spaced, High Traffic Density Networks

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ABSTRACT

The development and field testing of a traffic control policy designed for congested conditions in the high-density sectors of the Manhattan central business district (CBD) are described. Rather than providing progressive movement in the conventional sense, the primary objective of this control policy is to minimize the frequency and extent of intersection spillback. In the Manhattan CBD, queues develop along the cross streets; these queues often spill back into the upstream intersections, physically blocking the movement of traffic along the north-south arterials. The traffic control policy described yields signal timing for the one-way cross streets that exhibit a backward progression and flared green times that increase in the direction of traffic flow. The arterial traffic is serviced by a signal-timing pattern that exhibits zero relative offsets. The NETSIM traffic simulation model was used to test different concepts during the development phase of the effort. The new policy was then compared with the existing timing plan, by using NETSIM, and the results indicated that the number and duration of spillback blockage were markedly decreased, with a concomitant reduction in vehicle travel time and number of stops, coupled with an increase in vehicle trips serviced. A before-and-after field study yielded similar results, with the new policy providing a 20 percent reduction in overall travel time.

A study designed to identify high traffic density sectors (HTDSs) in mid-Manhattan and to develop methods for alleviating congested traffic conditions was performed. During the course of this study a new traffic control policy was developed, which expressly addresses the problem of overflow queues causing intersection spillback. This approach was adopted in response to simulation studies and field observations that indicated that recurring spillback was the primary factor responsible for traffic congestion.

Described are the sequence of activities and the rationale of the traffic signal control policy. Representative field results are presented.

PREVIOUS RESEARCH

Congestion on and saturation of street networks are familiar occurrences in central business districts (CBDs) and other high-activity centers of many urban areas. Under congested conditions, both capacity and operational efficiency are severely degraded, resulting in suboptimum utilization of the available facilities.

The treatments designed to reduce congestion and oversaturation in urban grids can be classified into the following three categories (1):

- Signal--minimal response signal control policies
- Signal--highly responsive signal control policies
- Nonsignal--other treatments in a signalized environment

The minimal-response signal control policies affect cycle lengths, splits, and offsets, and are applicable in an environment in which little day-to-day variation in the traffic pattern exists.

The well-accepted criteria for optimum cycle length at an intersection are minimization of delay and of congestion (2). Gazis and Potts (3) studied the problem of minimizing delay at oversaturated intersections. For fixed-time settings, they demonstrated that when saturation flows in the two critical directions were equal, the minimum delay was given by the settings that cause competing queues to disappear at the same time.

Optimum signal split is a function of relative demand on the competing approaches, platoon coherence, and constraints on minimum green time to satisfy pedestrian demands. The definition of congested or oversaturated traffic operations at an intersection implies that green time for a given approach is terminated before all demand is satisfied (3). Traffic can therefore be serviced during the entire green interval. It is possible, however, that lack of platoon coherence near the end of the green interval may cause traffic demand to be less than service capacity rates.

The highly responsive signal control policies involve detector-based, computer-controlled systems in which cycle length, split, and offset are varied, from cycle to cycle, in response to the existing traffic conditions. These policies are appropriate for environments with no discernible traffic pattern. Queue-actuated control (1), minimum delay via split switching (4), and queue proportionality in real time (5) are some of the highly responsive signal control strategies applicable in a congested environment.

Although the traffic-responsive signal control

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policies offer the greatest potential for combatting congestion, the costs associated with detectorization and computer-controlled systems are beyond the resources of most jurisdictions. Furthermore, it must be realized that as traffic demand approaches saturation and oversaturation, the responsive systems often act as fixed-time settings.

Nonsignalized treatments such as parking control, regulation of turns, and lane arrangement are designed to increase capacity. Two of the most frequent violations that aggravate the congestion problems are intersection blockage and violations of parking regulations. In NCHRP Report 194 (1), a detailed discussion is provided on various nonsignalized treatments and their effectiveness in reducing congestion in urban street networks.

SELECTED HIGH TRAFFIC DENSITY SECTORS

A review (6) of previous studies supplemented by traffic density data, collected on videotape using aerial surveillance, identified several HTDSs in Manhattan's CBD. These sectors were first screened to identify and delineate candidate sectors for additional study. The selection criteria used included feasibility criteria and infeasibility criteria.

Criteria for the identification of HTDSs that are candidates for metering (feasibility criteria) are

1. Effective red time (when the signal indication is green, but the existence of spillback precludes vehicle movement through the intersection) identified at multiple intersections;
2. Congestion (i.e., queuing), which involves several contiguous streets, often resulting in queues extending through intersections (spillback), which impedes cross-flow throughput; and
3. Opportunity for traffic diversion, storage upstream of the HTDS, or both.

Criteria that will prevent the application of metering to candidate HTDSs (infeasibility criteria) are

1. Potential blockage or impedance of major facilities by stored traffic (e.g., hospitals, fire stations, transit terminals, major interchanges);
2. Political considerations that preclude congested conditions in certain areas (e.g., United Nations building, selected embassies and public buildings);
3. Possibility that stored traffic will interfere with, or unduly delay, transit or emergency vehicles; and
4. Metering that could store traffic in tunnels or other locations where increases in vehicular emission or noise cannot be tolerated.

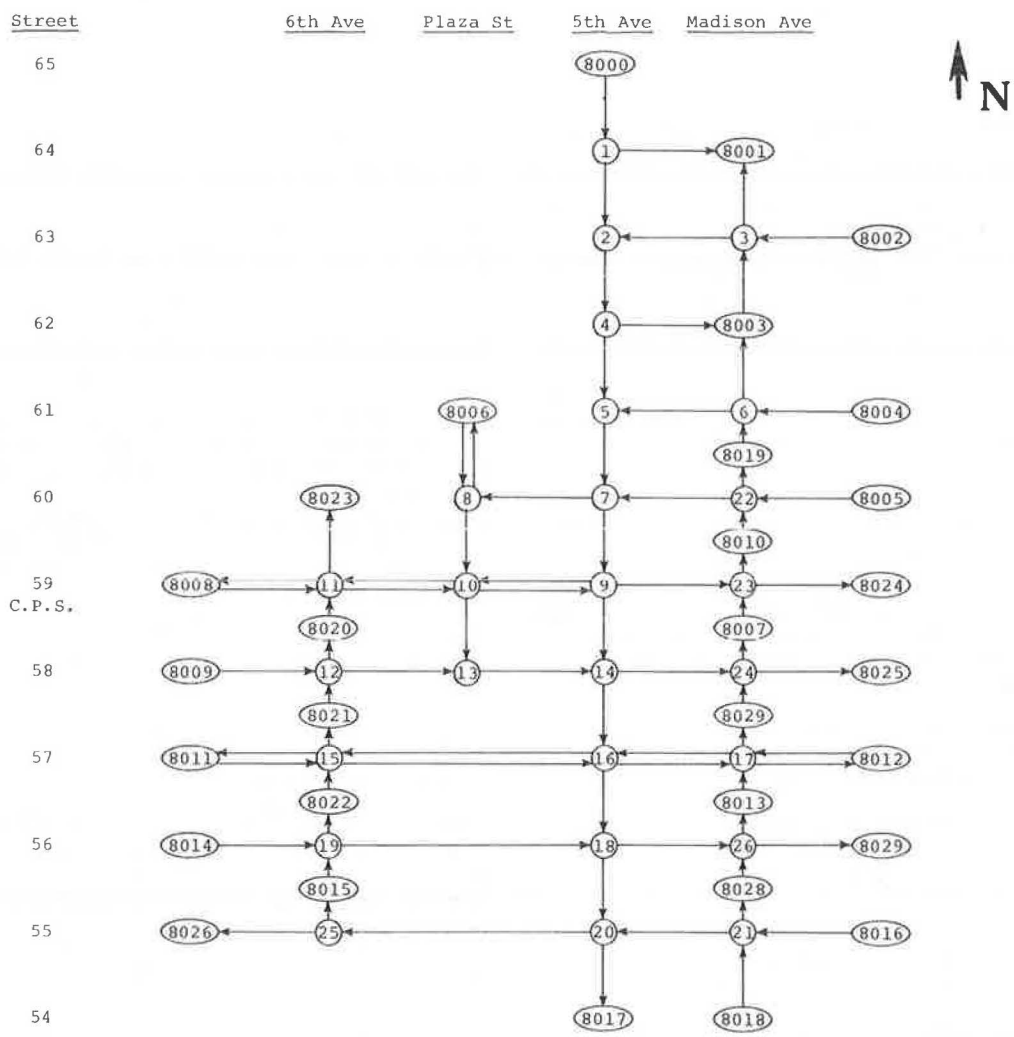

FIGURE 1 Fifth Avenue arterial network.

After detailed analysis (7) and extensive discussions with New York City Department of Transportation personnel, two HTDSs were selected for the application of a new control policy:

- Arterial system: Fifth Avenue between 63rd and 54th streets, including Grand Army Plaza and all entrance and exit links (Figure 1).
- Grid system: the corridor defined by Avenue of the Americas (6th Avenue) on the east, 8th Avenue on the west, from 45th Street on the north to 32nd Street on the south, including all entrance and exit links (Figure 2).

DESCRIPTION OF TRAFFIC ENVIRONMENT

The Manhattan CBD is characterized by a closely spaced signalized Cartesian grid system of primarily one-way streets, as is indicated in Figures 1 and 2. Because of the extremely heavy concentration of activities in a relatively small area, substantial traffic demand exists during both peak and off-peak hours. The high traffic demand in a signalized environment characterized by short streets and extremely high pedestrian volumes leads to significant levels of traffic congestion and associated adverse environmental impacts.

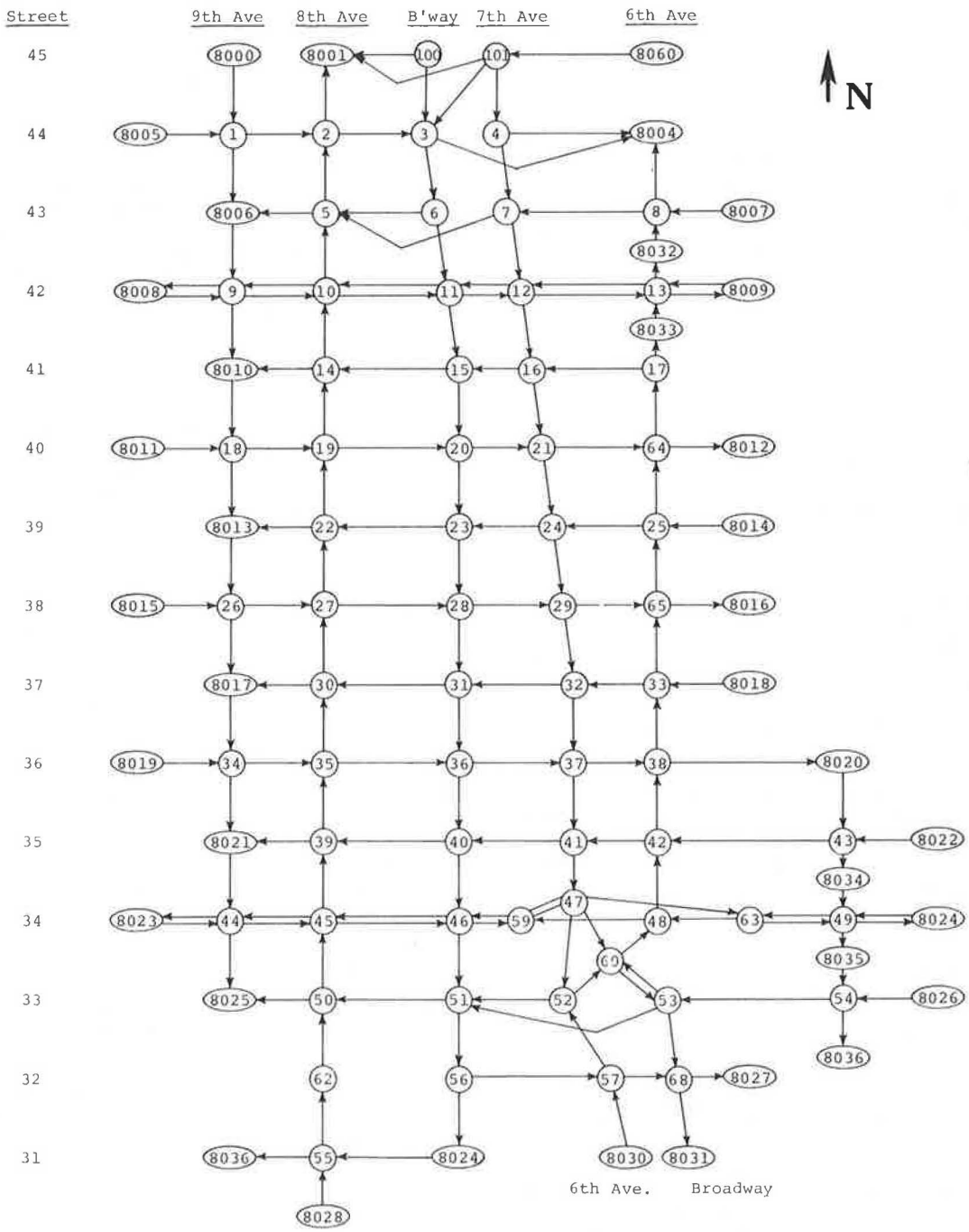

FIGURE 2 Grid network.

Congestion occurs throughout most of the workday. Intersection spillback arising from overflow queues is frequent, particularly in the vicinity of business activity centers. The problem is exacerbated by frequent illegal curb parking and double parking, violation of the WALK and DON'T WALK signs by pedestrians, and truck loading and unloading operations.

Congestion is particularly intense inbound in the morning on the north-south avenues, during the day on east-west crosstown streets, and on outbound avenues in the evening. Traffic often backs up into the surrounding street system during these peak periods, spreading delays and the problem of air pollution over a wide area.

CAUSES OF CONGESTION

To develop a control strategy to reduce congestion, the initial step was to determine the root causes of congestion in the selected HTDSs. Field observations of traffic conditions and simulation of traffic operations by using NETSIM (8) indicated that the dominant factor affecting traffic flow along the north-south arterials within the selected HTDSs was the recurring spillback of cross-street queues. This spillback intermittently blocks traffic flow on the north-south arterials, thus effectively reducing the available capacity there. As a result, the throughput along these arterials is reduced to a fraction of the theoretical arterial capacity based on the number of available lanes and green time.

The intersection spillback due to overflowing cross-street (east-west) queues is caused by the following factors:

- Inadequate green time for cross streets.
- Extensive parking activity along cross streets, which restricts vehicle movement.
- Very long discharge headways by traffic on cross streets. These low discharge rates reflect the one lane available combined with the impedance experienced by turning vehicles encountering heavy pedestrian traffic.
- Poor pedestrian signal discipline.
- Limited storage capacity (i.e., short streets) on some cross streets.
- Circulation patterns:
  a. The north-south arterials in these HTDSs act as distributors, wherein more vehicles turn onto the cross streets from the arterials than turn onto the arterials from the cross streets. Thus, the volume of traffic departing the arterial along the cross streets is often greater than that on the cross-street approach to the intersection. Consequently, vehicles turning from the arterial onto the cross street often encounter queues that block their progress and effectively remove a lane from service.
  b. Because most cross streets service one-way flow and alternate in direction, the process described in the preceding pattern leads to one arterial link losing its right-most lane, the next losing its left-most lane, and so on. Thus, the through-arterial traffic exhibits a weaving pattern, a high incidence of lane changing, and poor lane utilization.
- Through vehicles on cross streets that discharge during their green phase often encounter a long queue on the receiving cross street. All too frequently, this traffic cannot clear the intersection within the green phase because of excess demand relative to the available storage capacity on the receiving cross street. The result is an intersection spillback condition that blocks the arterial flow.

EXISTING TRAFFIC SIGNAL CONTROL POLICY

Existing traffic control consists of a single-dial, pretimed system. A computer-based signal control system is in the design stage. With few exceptions, all intersections operate on a two-phase, 90-sec cycle. Signal offsets are designed to provide progressive movement along all north-south arterials, which generally service one-way traffic. The ratios of green time to cycle length for these arterials are generally in the range of 0.55 to 0.6; the cross streets, most of which service one-way traffic flow, receive the remainder of the green time.

In this grid network of one-way traffic flow, the resulting control of the east-west cross streets exhibits signal offsets that vary from one street to the next. This characteristic is the outcome of the signal closure condition, which states that the sum of offsets around every closed loop is a multiple of the signal cycle. Thus, because adjacent arterials are servicing traffic in opposite directions with progressive offsets, the cross streets have signal offsets that cannot provide a high level of continuous movement.

Progressive offsets along the arterials are appropriate when (a) queue lengths are small relative to street length (approximately 230 ft for the networks selected) and (b) no blockage has occurred for several cycles. For situations that produce relatively long queues (100 ft or longer) for many cycles in sequence, offsets designed to maintain progressive movements, without consideration of standing queues, are not optimal and can actually amplify congested conditions. This offset design, in the absence of any blockage due to spillback, is as follows:

1. The queue on the upstream feeder link along the arterial receives the green indication before the approach with the long queue.
2. The through vehicles discharging the feeder approach reach the tail of the queue on the receiving link well before this queue starts to move. Thus, these discharging vehicles must stop after moving less than 150 ft, thereby generating a new shock wave.
3. The motorists on the feeder approach perceive a red indication controlling the traffic on the downstream approach at the time they are provided with a green signal. Because the motorists anticipate that they will have to stop again, their rate of discharge from the feeder link is often sluggish (i.e., long headways).
4. This sluggish rate of discharge reduces the throughput of the feeder link and of all vehicles entering the feeder link from farther upstream.

In the presence of intersection blockage, the whole issue of providing optimal offsets along the arterial becomes moot.

REVISED CONTROL POLICY

A new control policy for the selected HTDSs operating under saturated flow conditions was developed with the following considerations:

- Because the intermittent, recurring spillback of cross-street queues is the dominant factor influencing traffic operations, the control policy must explicitly address this spillback phenomenon and act to reduce its frequency, as well as its temporal and spatial extents.
- For the arterials, the control policy must provide signal settings with offsets that take into

account the presence of queues, even in the absence of spillback.

It is recognized that spillbacks are stochastic events that occur randomly over time and space because of fluctuating demand conditions. Definitionally, cross-street queues will overflow the available storage capacity and spill back into its upstream intersection whenever there are repeating net inflows over a sequence of signal cycles.

Control of Cross-Street Traffic To Reduce Spillback

To preempt spillback, the green time provided at the downstream intersection of cross streets must service these inflows. Thus, throughout its length where it acts as a collector of traffic, cross-street traffic must be provided with increasing values of green time. Stated another way, the revised control policy meters the inflow of vehicles along this cross street at its upstream end to permit the green time to be flared (increase gradually from one intersection to the next in the downstream direction).

Three parameters influence the flare of cross-street green time in the downstream direction:

- The number of arterial links affected by the spillback of a cross-street queue,
- The minimum acceptable spillback recurrence interval, and
- Available storage capacity on the cross street.

The number of arterial links affected by spillback of a cross-street queue is a function of the average speed of the shock wave along the arterial, the length of the arterial link, and the cycle length. The higher the number of arterial links affected by cross-street spillback (by virtue of higher shock wave speeds for fixed-link length and cycle length), the larger the green-time requirement for cross streets to reduce the adverse effects of spillback along the arterial.

Although it would be desirable to eliminate all possibility of cross-street spillback, one must accept as unavoidable a minimum frequency of spillback. To lower the frequency of spillback, stronger metering of cross-street traffic is required.

The available storage capacity in the cross street is essentially the block length (approximately 1/6 mile for the authors' networks) minus some adjustment for standing queue. The greater the available storage capacity, the smaller are the chances of spillback of queues. Thus, the cross-street green flare varies inversely with the block length.

In summary, the revised control policy is designed to improve traffic operations along arterial streets during periods of oversaturation by providing optimal offsets and increased green times along the cross streets. This apparent anomaly in the control policy is effective because it sharply reduces intersection blockage arising from overflow queues along the cross streets.

Control of Arterial Traffic

To provide arterial signal settings that offer offsets that are appropriate in the presence of moderate queues, the queue management control (QMC) concept was applied. QMC, a form of internal metering applied within a congested network, is designed to manage queue lengths to reduce the probability of spillback. QMC, implemented through signal coordination, is based on the following objectives:

1. Assure that every second of green time at the downstream node is fully utilized to maximize throughput.
2. Eliminate effective red (i.e., the time during a green phase when traffic is unable to move because of the presence of queues along the receiving arterial link).

It was indicated in this analysis that in the presence of moderate queues along north-south arterials, the optimal relative offsets along these arterials is approximately zero (simultaneous green). This condition, due primarily to the short link lengths along these arterials, also provided the opportunity for controlling the offsets along the cross streets so as to prevent, to a great extent, the onset of spillback conditions. The metering of cross-street traffic by increasing cross-street green times as one proceeds in the downstream direction, in an environment of simultaneous onset of the green phases servicing the arterial, provides the sought reverse progression (queue clearance) offsets along the cross streets.

The revised signal control policy was therefore responsive to the characteristics of the selected HTDSs:

- Metering of cross-street traffic to maintain a balance between demand and capacity.
- Reversing progression to limit the size of queues along the cross streets, thus limiting the exposure to spillback onto the arterial.
- Existence of near-optimal offsets along the arterials, given the presence of moderate queues on these short links.

Based on these analyses, revised values of the offsets and cycle splits were derived (9). These values were checked and adjusted through repeated simulation runs. It was also recommended that the placement of turn bays of adequate length on selected cross streets be established and enforced. These turn bays will somewhat reduce the capacity of a parking lane, but will ensure that delays experienced by turning vehicles encountering pedestrian traffic will not be transmitted to discharging vehicles moving through the intersection.

The recommended signal settings were then implemented for the Fifth Avenue arterial section. Before-and-after evaluation field studies were performed.

SIMULATION RESULTS

NETSIM was executed to simulate traffic operations with the existing signal timing and with the new control policy for two networks (Figures 1 and 2). The simulation results are given for a variety of traffic performance measures on individual links, on sections (sets of links), and on the network as a whole.

Fifth Avenue Arterial Network

The data in Table 1 indicate considerable improvement in traffic performance under the revised control scheme. The mean speed on the network increased by 31.3 percent (from 4.0 to 5.3 mph) despite an increase of 11.3 percent in the number of vehicle trips. Average delay and spillback durations were appreciably decreased. Link content was also decreased but to a somewhat lesser extent.

TABLE 1 Comparison of Simulated Traffic Performance on Fifth Avenue Arterial Network

Section	Vehicle Trips (hr)			Delay (veh-min)		
	Control Scheme		Change (%)	Control Scheme		Change (%)
	Current	Proposed		Current	Proposed	
5th Avenue, 63rd-55th streets	1,136	1,372	+20.8	992.9	725.5	−26.9
59th Street, 6th-Madison avenues	480	572	+19.2	808.0	580.9	−28.1
58th Street, 6th-Madison avenues	440	440	0	554.5	584.1	+5.3
57th Street, 6th-Madison avenues	620	684	+10.3	367.6	321.9	−12.4
57th Street, Madison-6th avenues	820	896	+9.2	860.7	698.3	−18.9
56th Street, 6th-Madison avenues	456	468	+2.6	508.1	247.0	−51.4
55th Street, Madison-6th avenues	440	472	+7.2	880.6	478.2	−45.7
Network	5,112	5,712	+11.7	5,326.2	4,367.4	−18.0

The traffic performance improved on the cross streets as well as on the Fifth Avenue arterial. The improved performance on Fifth Avenue resulted from the elimination of spillback from the cross streets, which demonstrates how congestion management of one component of traffic can benefit other components as well.

Link-specific comparisons of vehicle trips and spillback durations are given in Table 2. This table contains only the links that exhibited more than 15 percent difference in vehicle trips after implementing the proposed control scheme and experienced at least 50 sec of spillback during the simulated time period with either control scheme.

TABLE 2 Link-Specific Comparisons on Fifth Avenue Arterial Network

Artery	Control Scheme		Change (%)	Artery	Control Scheme		Change (%)
	Current	Proposed			Current	Proposed	
Vehicle trips (veh/hr)				Spillback duration (sec)			
5th Avenue				5th Avenue			
61st-60th streets	1,324	1,564	+18.1	61st-60th streets	281	157	−44.1
60th-59th streets	980	1,404	+43.3	60th-59th streets	491	0	−100.0
59th-58th streets	952	1,292	+35.7	61st Street			
58th-57th streets	1,048	1,340	+27.9	Madison-5th avenues	119	16	−86.6
57th-56th streets	1,016	1,240	+22.0	60th Street			
60th Street				Madison-5th avenues	575	295	−48.7
Park-Madison avenues	196	336	+71.4	59th Street			
Madison-5th avenues	268	544	+103.0	5th-Madison avenues	429	163	−62.0
5th Avenue-Plaza Street	512	708	+38.3	58th Street			
58th Street				6th Avenue-Plaza Street	180	0	−100.0
5th-Madison avenues	404	472	+16.8	Plaza Street-5th Avenue	411	79	−80.8
56th Street				5th-Madison avenues	363	0	−100.0
5th-Madison avenues	356	428	+20.2	55th Street			
55th Street				5th-6th avenues	353	0	−100.0
Madison-5th avenues	448	528	+17.9				
5th-6th avenues	388	500	+28.9				

TABLE 3 Comparison of Simulated Traffic Performance on Grid Network

Section	Vehicle Trips (hr)			Delay (veh-min)		
	Control Scheme		Change (%)	Control Scheme		Change (%)
	Current	Proposed		Current	Proposed	
8th Avenue, 31st-38th streets	1,932	1,932	0	567.7	633.6	+11.6
8th Avenue, 38th-44th streets	1,732	1,844	+6.5	252.7	515.0	+103.8
7th Avenue, 45th-38th streets	1,240	1,444	+16.5	959.6	629.5	−34.4
7th Avenue, 38th-32nd streets	924	1,416	+53.2	155.1	304.5	+96.3
Broadway, 45th-38th streets	1,152	1,340	+16.3	855.6	532.6	−37.8
Broadway, 38th-32nd streets	508	752	+48.0	296.6	272.3	−8.2
Avenue of the Americas, 32nd-37th streets	1,356	1,492	+10.0	666.2	429.2	−35.6
Avenue of the Americas, 37th-41st streets	904	1,688	+86.7	597.6	299.1	−49.9
42nd Street, 9th Avenue-Avenue of the Americas	620	624	+0.6	393.1	267.0	−32.0
42nd Street, Avenue of the Americas-9th Avenue	464	480	+3.4	160.0	194.6	+21.6
34th Street, 9th-5th avenues	604	624	+3.3	451.2	282.2	−37.5
34th Street, 5th-9th avenues	896	956	+6.7	1,242.5	1,058.2	−14.8
Network	10,816	12,144	+12.3	11,458.2	8,420.4	−26.5

Mean Speed (mph)			Content (vehicles)			Spillback Duration (sec)		
Control Scheme		Change	Control Scheme		Change	Control Scheme		Change
Current	Proposed	(%)	Current	Proposed	(%)	Current	Proposed	(%)
5.0	7.4	+48.0	78	63	−19.2	3,224	696	−78.4
2.6	4.2	+61.5	53	−38	−28.3	613	420	−31.5
3.4	3.3	−2.9	36	38	+5.5	854	79	−90.7
7.3	7.8	+6.8	24	20	−16.7	0	0	0
4.1	5.5	+34.1	57	46	−19.3	20	19	−5.0
3.9	8.2	+110.3	34	15	−55.9	0	0	0
2.2	4.2	+90.9	59	31	−47.5	522	215	−58.8
4.0	5.2	+30.0	419.2	353.1	−15.8	3,505	1,218	−65.2

Grid Network

The traffic performance on the grid network also improved considerably after simulated implementation of the proposed control scheme (Table 3). Mean speed on the network increased by 36.5 percent (from 6.1 to 8.3 mph), and vehicle trips increased by more than 13 percent. Delay and vehicle contents decreased by 32.0 and 14.5 percent, respectively. Spillback was almost eliminated from the network.

Number of vehicle trips increased significantly on both sections of 7th Avenue, on Broadway, and on the Avenue of the Americas. Other sections experienced relatively smaller increases in numbers of vehicle trips.

The average delay, mean speed, and vehicle content statistics indicate variability in performance from one section to another (see Table 3). Eight sections exhibited less delay with the revised timing policy, while four sections indicated higher delay. Nine of 12 sections showed higher speeds, while spillback duration over the network was reduced by an order of magnitude.

Overall, performance of the traffic on the grid network has improved, except on the 8th Avenue sections. On this arterial, although the number of vehicle trips increased slightly, the performance of traffic was adversely affected.

The adverse relationship between intersection spillback and traffic performance for systems that exhibit a congested environment is confirmed by these simulation results. Specifically, sharp reductions in intersection spillback and the consequent blockage of traffic translate into lower delay, higher speeds, and improved throughput.

FIELD STUDY

A before-and-after field study (10) was conducted on the Fifth Avenue network to evaluate the performance of traffic under the proposed control policy. Travel times were obtained using floating cars traveling along Fifth Avenue and along many of the cross streets intersecting Fifth Avenue. Traffic volumes were collected by observers stationed along Fifth Avenue.

The before-and-after data were collected for two weeks during April and May 1985. For each segment, more than 15 travel time runs were made during both midday (11:00 a.m. to 2:00 p.m.) and p.m. (3:00 p.m. to 6:00 p.m.) peak hours.

Table 4 shows average travel times on various segments of the Fifth Avenue arterial network both before and after the implementation of the proposed control policy. The network weighted mean is computed as

$$\text{Weighted mean} = \left(\sum_s n_s \bar{t}_s d_s\right) \Big/ \left(\sum_s n_s d_s\right) \quad (1)$$

where

n_s = number of trips on segment, s;

Mean Speed (mph)			Content (vehicles)			Spillback Duration (sec)		
Control Scheme		Change	Control Scheme		Change	Control Scheme		Change
Current	Proposed	(%)	Current	Proposed	(%)	Current	Proposed	(%)
10.9	10.2	−6.4	60	65	+8.3	44	36	−18.2
14.7	10.3	−29.9	33	52	+57.6	39	0	−100.0
5.5	8.5	+54.5	75	57	−24.0	314	128	−59.2
13.7	12.0	−12.4	18	33	+83.3	10	20	+100.0
5.8	9.2	+58.6	69	49	−29.0	181	11	−93.9
6.4	9.1	+42.2	22	25	+13.6	0	0	0
6.0	8.9	+48.3	54	41	−24.1	20	1	−95.0
3.9	10.5	+169.2	45	30	−33.3	2	25	+∞
8.5	10.9	+28.2	37	27	−27.0	0	0	0
12.4	11.3	−8.9	16	19	+18.8	94	25	−73.4
9.5	12.8	+34.7	44	31	−29.5	0	0	0
6.0	7.1	+18.3	104	93	−10.6	166	21	−87.3
6.1	8.5	+39.3	989	819	−17.2	2,592	286	−89.0

TABLE 4 Travel Time on Various Segments of the Fifth Avenue Arterial Network (floating car runs)

Sampling Period	Segment	Mean Travel Time (sec) Before	After	Difference	Percentage Difference
Midday	5th Avenue, 62nd-54th streets	133	170	+37	+27.8
	55th Street, Madison-6th avenues	252	169	-83	-32.9
	56th Street, 6th-Madison avenues	163	147	-16	-9.8
	57th Street, Madison Avenue-Broadway	322	263	-59	-18.3
	58th Street, 6th-Madison avenues	199	140	-59	-29.6
	59th Street, 6th-Madison avenues	229	206	-23	-10.0
Network weighted mean		233	194	-39	-16.7
p.m.	5th Avenue, 62nd-54th streets	162	112	-50	-30.9
	55th Street, Madison-6th avenues	287	187	-100	-34.8
	56th Street, 6th-Madison avenues	187	177	-10	-5.3
	57th Street, Madison Avenue-Broadway	298	219	-79	-26.5
	58th Street, 6th-Madison avenues	218	161	-57	-26.1
	59th Street, 6th-Madison avenues	248	267	+19	+7.7
Network weighted mean		234	180	-54	-23.1
All	5th Avenue, 62nd-54th streets	153	145	-8	-5.2
	55th Street, Madison-6th avenues	270	176	-94	-34.8
	56th Street, 6th-Madison avenues	176	162	-14	-8.0
	57th Street, Madison Avenue-Broadway	311	239	-72	-23.2
	58th Street, 6th-Madison avenues	209	153	-56	-26.8
	59th Street, 6th-Madison avenues	236	230	-6	-2.5
Network weighted mean		234	187	-47	-20.1

t_s = mean travel time (sec) on segment, s; and
d_s = segment length (ft).

The data in Table 4 show that the new control policy benefits the cross-street traffic the most. Those cross streets that were most congested before (e.g., 55th Street) exhibited the best improvements.

Fifth Avenue traffic also benefited, but to a somewhat lesser extent. Closer examination of the results indicates that when cross-street volumes were low (Table 5) and spillback did not occur, the current progressive signal timing provided better service than did the proposed simultaneous green indications. However, during the p.m. peak period, when cross-street volumes were much heavier, producing intermittent intersection spillback when the existing control was in force, the new signal timing performed better.

Traffic volume along 5th Avenue is slightly lower during the p.m. peak than during midday, while the cross-street volume is generally much higher during the p.m. peak (Table 5). Because the new control scheme produced improved results along 5th Avenue during the p.m. peak, relative to the existing timing, it thus confirms that the critical factor in expediting main street traffic movement is the treatment of controlling high-volume, cross-street traffic.

DISCUSSION

The revised control scheme was designed to reduce the frequency and temporal extent of intersection spillback by cross-street queues. This scheme is characterized by signal settings that are designed to expedite movement of cross-street traffic, yet provide near-optimal offsets and splits to the north-south arterials.

Currently, the one-way, north-south arterials are provided progressive signal offsets. In the absence of spillback by cross-street queues, the current signal pattern offers excellent service to north-south traffic. When traffic demand increases, however, such spillback blocks traffic along the arterials, disrupts progressive movement, and forms

TABLE 5 Volume Counts at Fifth Avenue (veh/hr)

Time Period	Intersection	Approach: Southbound Before	Southbound After	Eastbound Before	Eastbound After	Westbound Before	Westbound After
Midday							
11:30-12:00	5th Avenue and 59th Street	1,608	1,732	592	686		
12:05-12:35	5th Avenue and 58th Street	1,496	1,520	686	750		
12:40-1:10	5th Avenue and 57th Street	1,656	1,558	382	422	332	340
1:15-1:45	5th Avenue and 56th Street	1,616	1,594	478	482		
1:50-2:20	5th Avenue and 55th Street	1,544	1,560			584	616
p.m.							
3:30-4:00	5th Avenue and 59th Street	1,418	1,448	444	630		
4:05-4:35	5th Avenue and 58th Street	1,358	1,418	726	842		
4:40-5:10	5th Avenue and 57th Street	1,458	1,554	820	876	512	516
5:15-5:45	5th Avenue and 56th Street	1,494	1,398	624	650		
5:50-6:20	5th Avenue and 55th Street	1,546	1,470			492	550

standing queues in the presence of green signal indications. Thus, the progressive signal timing is effectively negated by these intersection blockages. It follows that any potential loss of progressive movement arising from the implementation of simultaneous green indications (i.e., zero relative offsets) is more than compensated for by the near absence of spillback. The net effect is beneficial.

The study discussed in this paper has led to the development of control policies that are designed expressly for servicing traffic in high-density areas during peak demand periods. By reducing spillback of cross-street queues within the high-density area, it has been shown that all traffic can benefit. Thus, the control scheme based on the objective of spillback avoidance has been shown to be more effective than the more conventional progressive movement policy in high-density environs. However, during off-peak hours when traffic volumes are low, the progressive movement policy is more effective.

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Case Study Evaluation of the Safety and Operational Benefits of Traffic Signal Coordination

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ABSTRACT

A high-volume urban arterial was analyzed to determine if rear-end accident frequency might be decreased by reducing the frequency of vehicular stops at five signalized intersections. The potentially most cost-effective technique for reducing the frequency of stops was to coordinate the signal controllers and permit the progressive flow of platoons of vehicles. The TRANSYT model was used to develop optimized timing plans for a hypothetical time-of-day signal control system. Detailed performance data for both the existing conditions and the proposed coordinated signal system were generated using the NETSIM model. Accident records were then analyzed and correlated with the estimated frequency of vehicular stops under existing conditions. The accident prediction model was used to estimate the safety impacts of the proposed signal coordination. Evaluation of the simulation output and accident prediction estimates revealed that a small reduction in the frequency of rear-end collisions should be possible if the traffic signals are coordinated. In addition, concurrent benefits would accrue in terms of reductions in frequency of stops and in delay.

The Madison Beltline Highway (U.S.H. 12 and 18) is the major circumferential route around the south side of the Madison, Wisconsin, metropolitan area. In addition to serving travel needs for the urbanized area, the highway also provides the principal direct link between southwestern Wisconsin and the Interstate system serving Chicago, Milwaukee, and Minneapolis-St. Paul. Average daily traffic is almost 45,000 vehicles at some locations, including 7 percent trucks. In the early 1970s, a major portion of the Beltline Highway was improved to freeway standards, which produced a 75 percent reduction in the overall accident rate. Reconstruction of the remaining approximately 5-mile segment of four-lane divided arterial has been delayed because of environmental issues and budgetary constraints. Currently, a total of five signalized intersections exists along the arterial segment, as shown in Figure 1.

Although recent geometric improvements have reduced the frequency of certain types of intersection-related accidents, a major safety problem continues to exist along the 3.7-mile arterial segment between Raywood Road and U.S. 51. Accident statistics from 1978 indicate that 234 accidents occurred in this section, with more than one-half being rear-end collisions resulting in 59 injuries. A study was therefore initiated to determine if rear-end accident frequency might be decreased by reducing the frequency of vehicular stops at the five signalized intersections (1). The potentially most cost-effective technique for reducing the frequency of stops was to coordinate the signal controllers and permit the progressive flow of platoons of vehicles. This could also be expected to reduce delay and fuel consumption.

The study was composed of four basic phases. First, the TRANSYT model (2) was used to develop optimized timing plans for a hypothetical time-of-day signal control system. The NETSIM model (3) was then used to generate detailed performance data for both the existing conditions and the proposed coordinated signal system. Next, accident records for the Beltline Highway were analyzed and correlated with the estimated frequency of vehicular stops under existing conditions. The final phase involved the use of the NETSIM evaluation data and the accident relationships to assess the potential effectiveness of implementing a coordinated traffic signal system.

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DEVELOPMENT OF OPTIMIZED TIMING PLANS

The 3.7-mile segment of the Beltline Highway between Raywood Road and the U.S. 51 interchange is a four-lane divided arterial with a 40-mph speed limit. Flow rates near or at saturation levels occur during the a.m. and p.m. peak periods. Large turning movements to and from the developed areas north of the Beltline Highway occur at Bridge Road, Monona Drive, and the U.S. 51 interchange ramps. Hourly volumes during the a.m. and p.m. peak hours are shown in Figure 2.

The intersections at Raywood Road, Bridge Road, and Monona Drive each operate under isolated volume-density traffic signal control. Separate turning lanes and signal phases are provided at each intersection. The ramps of the diamond interchange at U.S. 51 create two additional intersections, which operate under a single fixed-time controller that has vehicle detection capability for extending the green time on the Beltline approaches. Signal timing at the interchange ramps is designed to serve substantial left-turn movements and to prevent the development of queues of stopped vehicles between the intersections. Condition diagrams and signal phasing for each of the five intersections are shown in Figures 3-6. Although only 394 ft separate the two intersections at the U.S. 51 interchange, the spacing of the remaining intersections is substantial. The distances between Raywood Road, Bridge Road, Monona Drive, and the west ramp of the U.S. 51 interchange are 3,906 ft, 4,021 ft, and 4,209 ft, respectively.

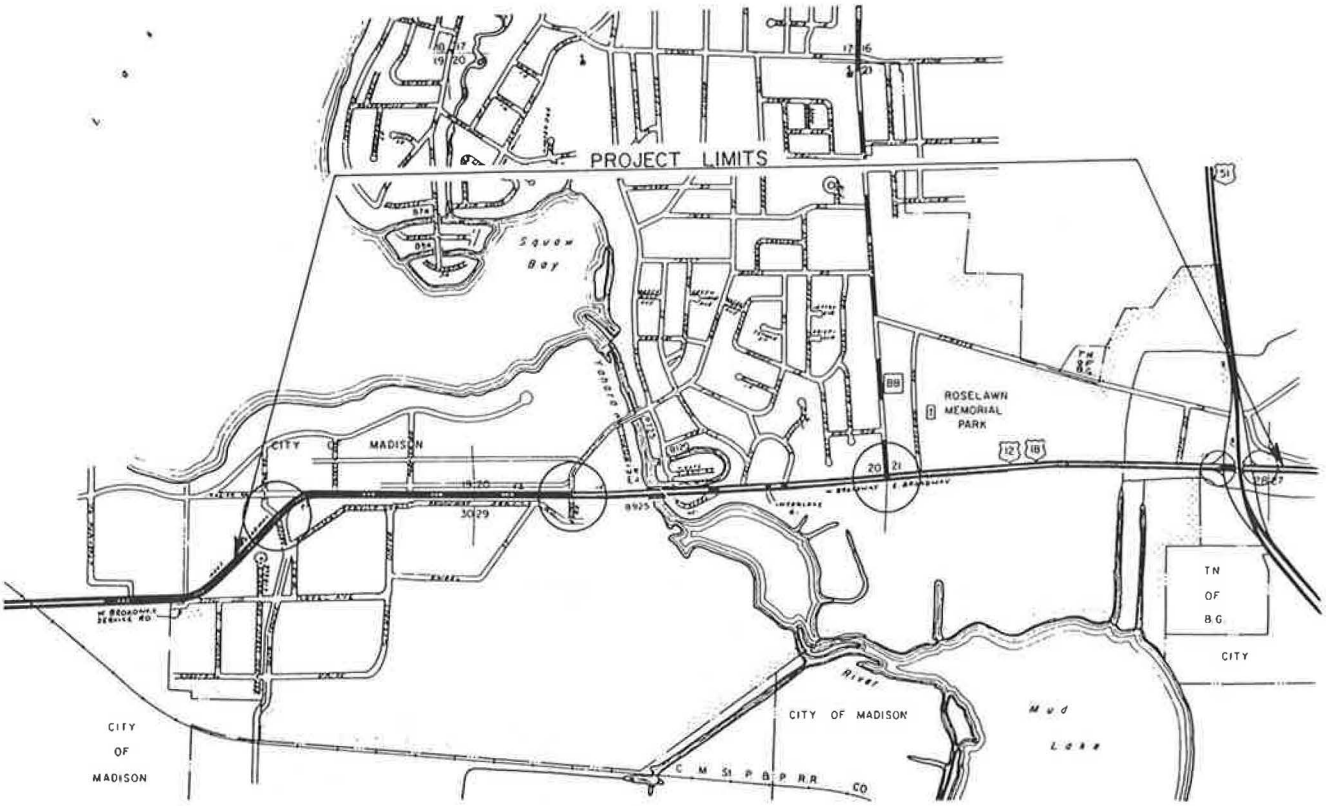

FIGURE 1 Beltline Highway study area.

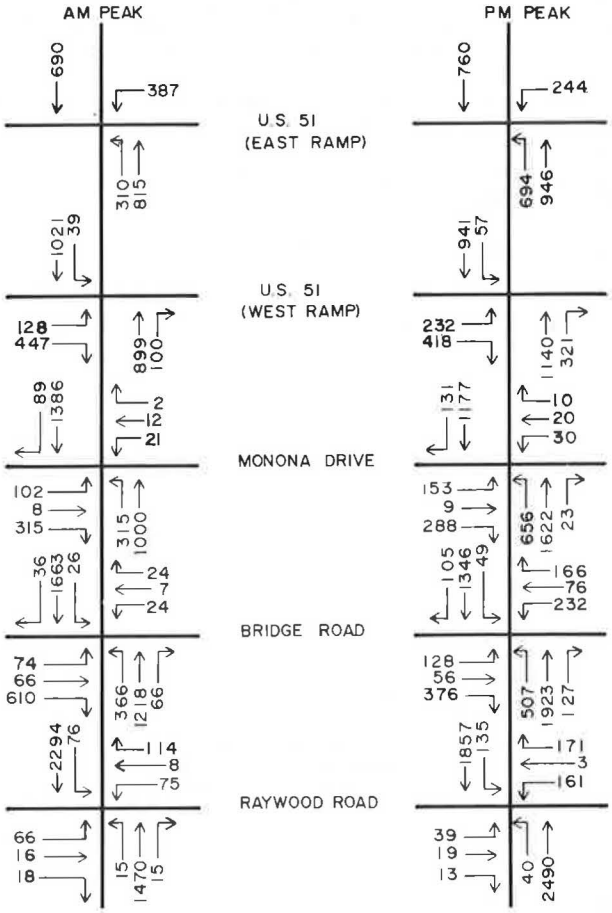

FIGURE 2 Volumes for a.m. and p.m. peak hours.

For the development of optimized signal-timing plans, five time periods were selected to represent the typical variation in traffic flow patterns on an average weekday:

a.m. peak:	7:30 to 8:30 a.m.
a.m. off peak:	9:30 to 10:30 a.m.
noon peak:	11:30 a.m. to 12:30 p.m.
p.m. off peak:	2:00 to 3:00 p.m.
p.m. peak	4:00 to 5:00 p.m.

Smoothed traffic volumes and turning movements were established for each time period by using data from a permanent recording station, machine counts, and manual intersection counts.

The TRANSYT model was then used to generate a set of optimized signal-timing plans for each of the five time periods. Although the existing signalized intersections included both traffic-actuated and fixed-time controllers, the proposed coordinated system would operate on a time-of-day basis, with each intersection having a traffic-actuated controller operating under a background cycle length. Because the TRANSYT model can only simulate a fixed-time control system, it was assumed that the TRANSYT-generated timing plans would produce offsets that would be optimal for average conditions. In effect, the optimal splits generated by TRANSYT were assumed to approximate the typical split that would result under actual field conditions with the controller allocating green time in proportion to demand.

Link-node diagrams and input data were prepared for each of the five time periods. Based on the traffic flow patterns and the geometrics at each intersection, signal-phasing sequences were specified as summarized in the following list.

1. Three-phase control with leading eastbound (EB) and westbound (WB) left-turn indications at Raywood Road.

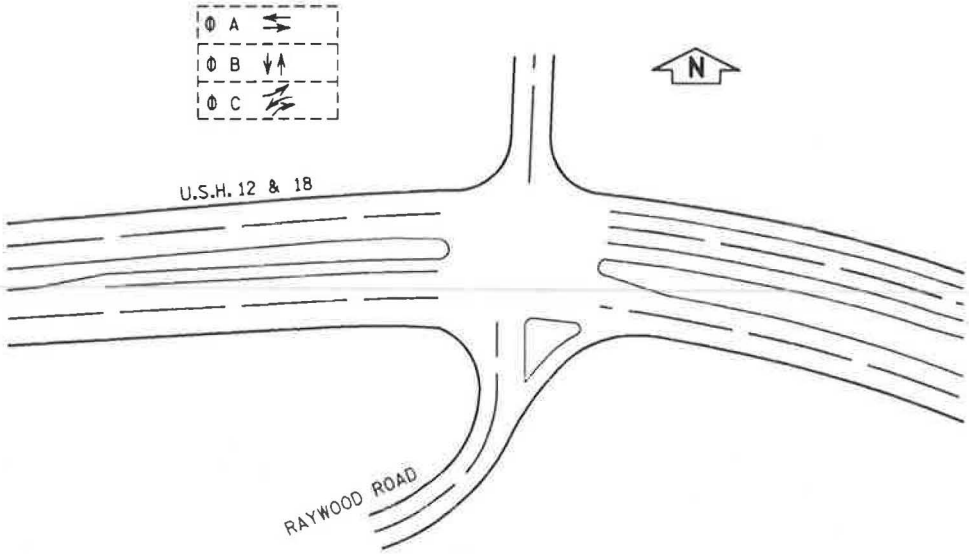

FIGURE 3 Condition diagram and signal phasing: Raywood Road.

2. Four-phase control with leading EB and WB left-turn indications at Bridge Road.
3. Three-phase control with leading EB left-turn phase at Monona Drive.
4. Four-phase control with leading WB left-turn phase and separate southbound right-turn phase at the U.S. 51 west ramp.
5. Three-phase control with leading EB left-turn phase at the U.S. 51 east ramp.

The TRANSYT model was then applied by using several different cycle lengths for each of the five time periods. Based on these data, the following optimal cycle lengths were identified.

Time Period	Optimum Cycle Length (sec)
a.m. peak	130
a.m. off peak	100
noon peak	100
p.m. off peak	100
p.m. peak	140

Further examination of the optimal splits and offsets revealed substantial similarity among the noon peak, and the a.m. and p.m. off-peak periods. It was therefore assumed for the purposes of the remaining study tasks that the proposed time-of-day control system would have three distinct timing plans: a.m. peak, p.m. peak, and off peak. The off-peak plan would be that which was generated based on noon-hour conditions.

Each of the timing plans was specified in terms of a background cycle length, offsets, and phasing sequence. The actuated controller settings for initiation and termination of each signal phase were established based on local practice and prior experience with the existing actuated controllers. Figures 7 and 8 show the progression bands for the arterial through movements during the a.m. and p.m. peak periods. The implied speeds of progression are approximately 40 to 45 mph.

TRAFFIC SIMULATION MODELING

Development of the TRANSYT-optimized traffic signal-timing plans was based on a macroscopic modeling of

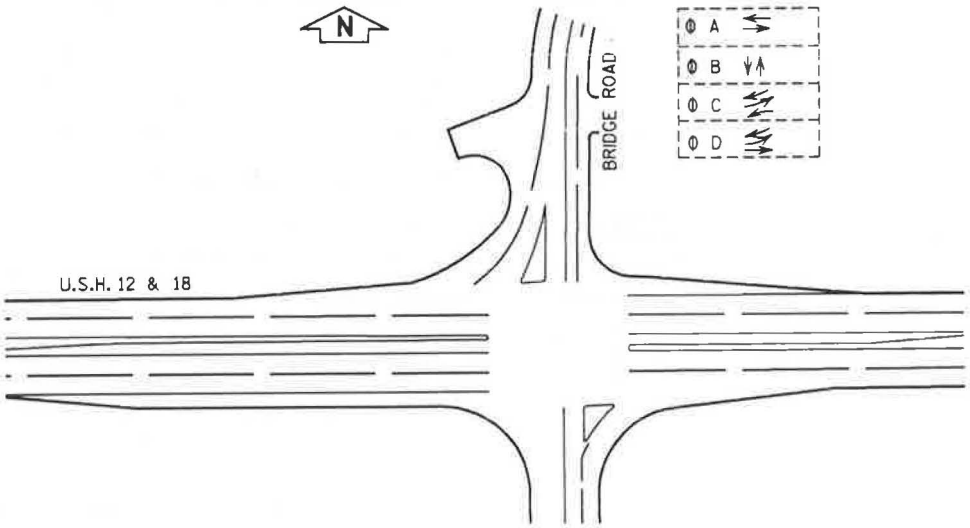

FIGURE 4 Condition diagram and signal phasing: Bridge Road.

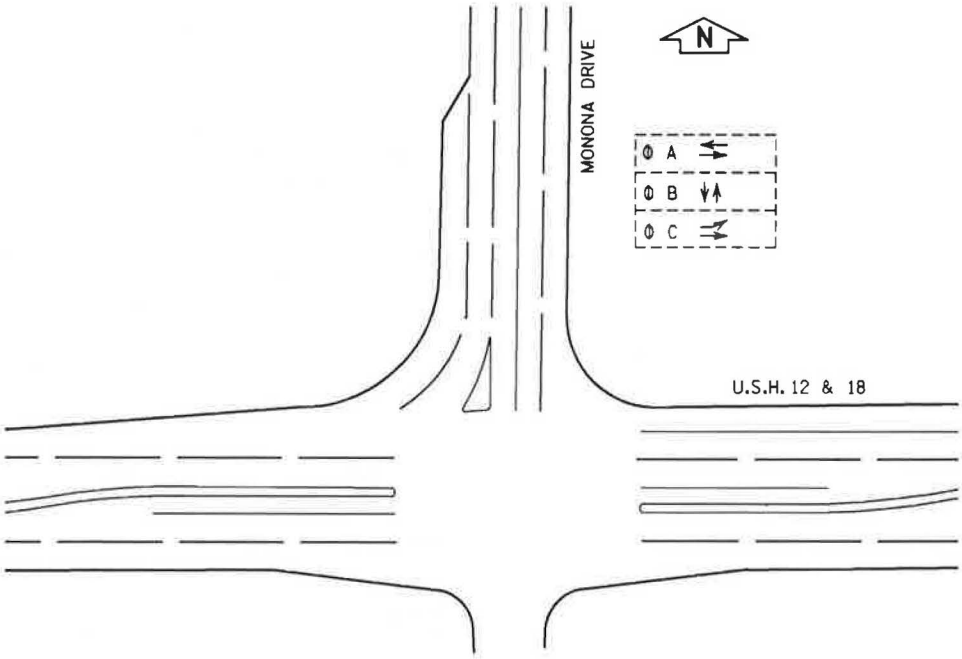

FIGURE 5 Condition diagram and signal phasing: Monona Drive.

traffic flow within the study network. Because both the existing and proposed signal systems included actuated controllers, the NETSIM model was used to generate the performance data, which would serve as the basis for evaluating the safety and operational benefits of signal coordination. Unlike the TRANSYT model, which assumes a fixed-time or average signal control plan, the NETSIM model simulates the functioning of the actuated controllers in response to a microscopic representation of the movement of individual vehicles within the network.

Model Verification and Calibration

Two important elements in using computer simulation models are verification and calibration: verification is the process of determining whether a simulation model performs as intended; calibration is the process of determining whether a simulation model accurately represents the real-world system. The objective of the calibration process is not to duplicate the existing traffic conditions exactly, but to eliminate major differences between the simulation

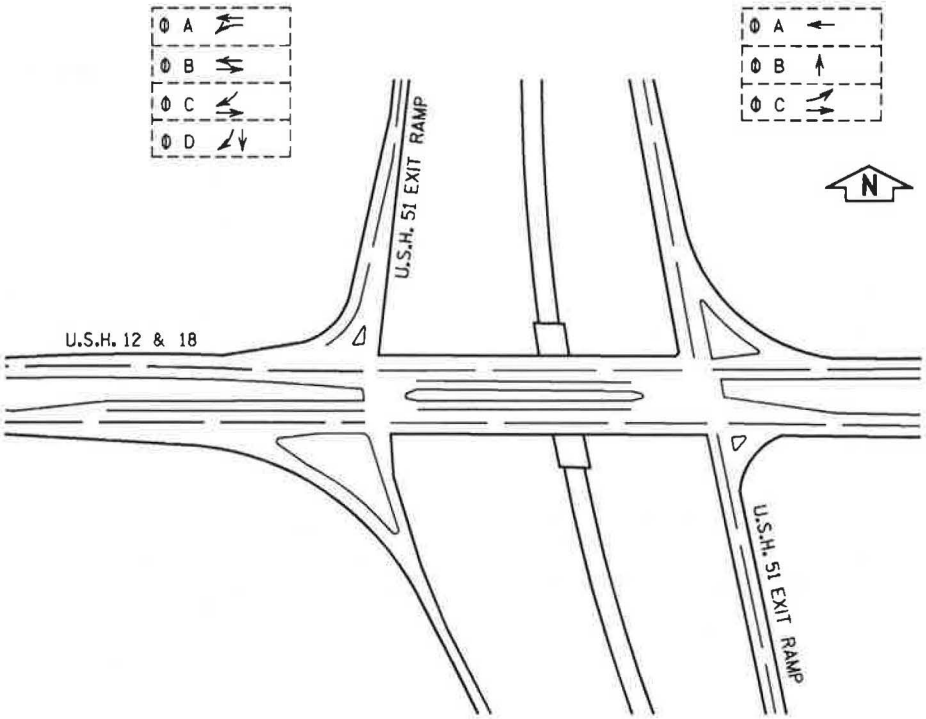

FIGURE 6 Condition diagram and signal phasing: U.S. 51 interchange.

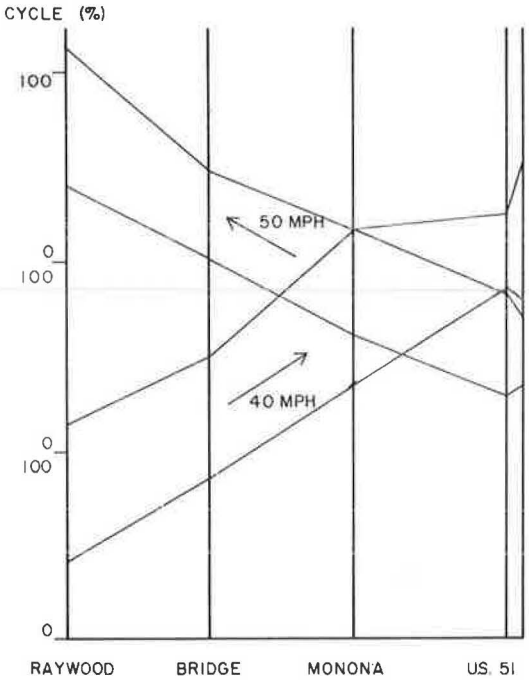

FIGURE 7 Signal progression bands for a.m. peak period.

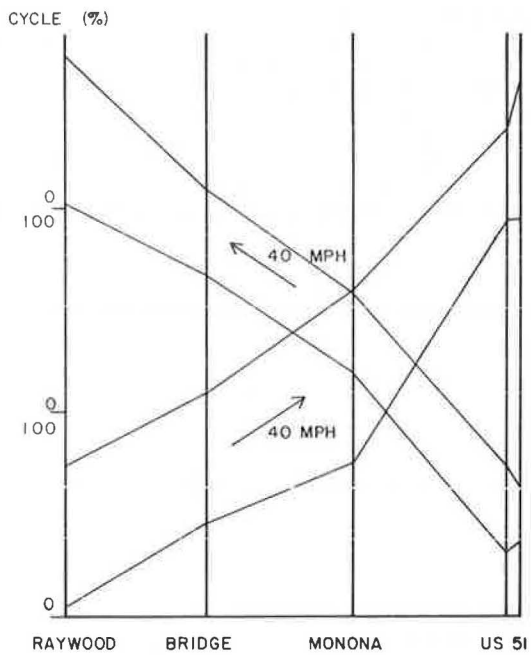

FIGURE 8 Signal progression bands for p.m. peak period.

results and the observed field data. If the agreement is acceptable, then it is said that the model is simulating the system.

Verification of the NETSIM model for the existing signal control system was performed by examining intermediate output data for successive 2-sec simulation intervals. The operation of the actuated signal controllers in the model was compared with the actual operating characteristics of the real-world controllers. In addition, the total number of vehicles discharged from selected links at the end of the simulation was compared with the smoothed traffic volumes, which were used as input to the model. Inconsistencies were eliminated by adjusting various input and embedded data.

After it was verified that the NETSIM model was reasonably simulating the logic of the existing traffic control system, the model was calibrated by further adjusting selected embedded parameters within the model until there was reasonable agreement between the simulated link speeds and queue lengths, and those observed in the field. For the proposed coordinated signal control system, the model verification and calibration process was principally confined to assuring that the simulated cycle length and offset relationships were consistent with the specified input values.

Generation of Performance Data

Because of the stochastic nature of the NETSIM model, the measures of effectiveness (MOEs) estimated in a given simulation run constitute a single random sample out of a population. Therefore, replications were required to establish a confidence interval for these MOEs. Because of project resource constraints, only three replications of each control system for the three time periods were made. Each replication was a simulation of a 15-min time interval using a different random number seed. The MOEs selected for evaluation were the average number of stops per vehicle, the average delay per vehicle, and the number of gallons of fuel consumed. Data were available for each link in the network as well as the network as a whole.

The MOEs from the simulation runs were then summarized and comparisons were made for stops per vehicle during the a.m. peak period, as shown in Table 1. Comparisons were made for each eastbound and westbound intersection approach link along the Beltline Highway as well as aggregate values for eastbound travel, westbound travel, all Beltline links (arterial), and the entire network including intersecting street approaches. The before data refer to the existing signal control system, and the after data refer to the proposed coordinated signal system. In addition to the estimated change in each MOE, a plus-minus value for the 90 percent confidence interval was calculated using a paired t-test (4). It is also noted in the table whether the change in the MOE value is statistically significant (yes = True, no = False).

TABLE 1 Stops per Vehicle: a.m. Peak (7-9 a.m.)

Location	Before	After	Reduction	+/–	Test
Eastbound					
Raywood Road	0.51	0.36	0.15	0.04	True
Bridge Road	0.46	0.54	–0.08	0.14	False
Monona Drive	0.38	0.35	0.03	0.11	False
U.S. 51 West	0.74	0.72	0.02	0.15	False
Subtotal[a]	2.40	2.25	0.15	0.40	False
Westbound					
U.S. 51 East	0.80	0.74	0.06	0.03	True
Monona Drive	0.64	0.39	0.25	0.16	True
Bridge Road	0.71	0.66	0.05	0.15	False
Raywood Road	0.86	0.57	0.29	0.25	True
Subtotal[a]	3.26	2.61	0.65	0.19	True
Arterial	2.83	2.43	0.40	0.16	True
Network	2.22	1.96	0.26	0.03	True

[a]Includes nonintersection links.

ACCIDENT RELATIONSHIPS

A basic hypothesis of the research was that the rate at which rear-end accidents occur is correlated with the frequency of stops at the signalized intersections. This hypothesis was tested by assembling accident data for a 3-year period and then using linear regression analysis to determine any statistically significant relationship between these data and the simulated stops-per-vehicle data generated by the NETSIM model.

Accident Data Base

Accident data for the years 1978 to 1980 were obtained from the Wisconsin Department of Transportation. The data were sorted by intersection approach link and time of day. Because the signal timing at the U.S. 51 diamond interchange ramps was designed to preclude the occurrence of stopped vehicles between the ramp intersections, only the external approaches to the intersection pair were used. This resulted in four intersection approaches for both eastbound and westbound travel. Only those accidents that involved a rear-end collision on one of the eight approach links during the 7-a.m.-to-6-p.m. weekday time period were used. These data were then stratified across the three time periods used for the traffic simulation modeling: a.m. peak (7-9 a.m.), off peak (9 a.m.-3 p.m.), and p.m. peak (3-6 p.m.). Finally, the rear-end accident rate for each link during each of the three time periods was calculated in terms of accidents per million entering vehicles, as shown in Table 2.

TABLE 2 Rear-End Accident Rates (no. of accidents per million entering vehicles)

Intersection	Time Period		
	a.m. Peak	Off Peak	p.m. Peak
Eastbound			
Raywood Road	0.86	0.34	2.25
Bridge Road	0.78	0.97	2.58
Monona Drive	2.44	2.04	0.89
U.S. 51 West	1.11	1.02	1.08
Westbound			
U.S. 51 East	3.84	1.94	1.79
Monona Drive	0.87	0.82	1.87
Bridge Road	1.89	0.51	2.85
Raywood Road	1.99	1.26	0.69

Accident Prediction Models

It was assumed that the rear-end accident rate on each intersection approach was linearly related to the typical frequency with which vehicles were forced to stop under the prevailing roadway and traffic conditions. The data on average stops per vehicle from the three sets of NETSIM simulation runs were used as values representative of what would occur throughout the three respective time periods. The 8 intersection approaches and 3 time periods resulted in a sample size of 24 data points.

Regression models with and without a constant term were analyzed and found to be similar. Because of its intuitive appeal, the following simple regression model was selected for subsequent application in evaluating the potential safety benefits of signal coordination:

$$A = 2.41S$$

where A is the number of rear-end accidents per million entering vehicles and S is the number of stops per vehicle on the intersection approach. The regression coefficient is statistically significant at the 5 percent level and the model explains 32 percent of the variance in rear-end accident rates. Although the variance explanation is not high, it is representative of the levels often found in accident prediction modeling. A plot of the data points about the regression line is shown in Figure 9.

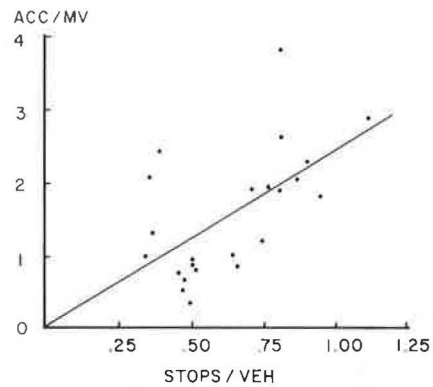

FIGURE 9 Rear-end accident prediction model.

IMPACTS OF TRAFFIC SIGNAL COORDINATION

Evaluation of the potential benefits of coordinating the traffic signals along the Beltline Highway was addressed in terms of both safety and quality of flow. The rear-end accident prediction model was used to estimate the change in accident rate that would result from the expected reduction in vehicle stops. Impacts on quality of flow were measured using the NETSIM estimates of changes in delay per vehicle, stops per vehicle, and annual fuel consumption during the 7-a.m.-to-6-p.m. weekday period.

Safety Impacts

The estimated annual reduction in rear-end accident frequency for each intersection approach and each time period is given in Table 3. The greatest benefits accrue to westbound traffic, in particular at

TABLE 3 Annual Reduction in Rear-End Accident Frequency

Intersection	Time Period		
	a.m. Peak	Off Peak	p.m. Peak
Eastbound			
Raywood Road	0.27	0.59	0.39
Bridge Road	(−0.17)	(−0.46)	(0.31)
Monona Drive	(0.05)	(−0.38)	(0.02)
U.S. 51 West	(0.03)	−0.38	0.16
Subtotal	(0.19)	(−0.63)	(0.89)
Westbound			
U.S. 51 East	0.05	0.15	(−0.04)
Monona Drive	0.48	0.41	(−0.18)
Bridge Road	(0.10)	0.72	0.91
Raywood Road	0.82	(0.14)	−1.11
Subtotal	1.45	1.41	(−0.42)
Total	1.64	(0.78)	(0.46)

Note: Numbers in parentheses are not statistically significant at the 10 percent level. Because of rounding, totals may not be exact.

TABLE 4 Measures of Effectiveness

	Reduced Stops per Vehicle		Reduced Delay per Vehicle		Reduced Fuel Consumed per Year	
	Number	Percent	No. of Seconds	Percent	No. of Gallons	Percent
Arterial						
a.m. peak	0.40	14.13	11.93	8.45	(1,629)	(0.48)
Off peak	(0.12)	(5.45)	(3.87)	(0.04)	(6,448)	(0.84)
p.m. peak	(0.09)	(2.93)	32.00	14.26	(208)	(0.04)
Network						
a.m. peak	0.26	11.71	(3.14)	(2.73)	(-3,647)	(-0.89)
Off peak	(-0.03)	(-1.78)	(-3.19)	(-4.17)	(603)	(0.07)
p.m. peak	(-0.03)	(-1.24)	(8.76)	(5.87)	(-10,941)	(-1.50)

Note: Numbers in parentheses are not statistically significant at the 10 percent level.

Bridge Road and Monona Drive. On a time-of-day basis, the 7-to-9-a.m. peak period would experience the largest reduction in frequency of rear-end accidents.

Overall, the projected reductions in accident frequency are relatively small. In some instances, an increase in the frequency of accidents is expected. Many of the estimates are not statistically significant because the confidence interval for the estimate includes the value of 0, or no effect. The low accident reduction projections were unexpected. In reviewing the prevailing roadway and traffic conditions along the Beltline Highway as well as the results of the simulation modeling, it was apparent that the combination of high traffic volumes and very large turning movements to and from the developed areas north of the Beltline Highway constrained the ability of the signal system to create vehicle platooning and maintain progressive flow. The large intersection spacing may also have permitted sufficient platoon dispersion to reduce some of the benefits of signal coordination.

Quality of Flow

The potential impacts of signal coordination on stops, delay, and fuel consumption are summarized in Table 4. Because the proposed signal coordination is designed to benefit traffic on the arterial links, the greatest impacts on quality of flow should appear on these links. The data in Table 4 indicate that the only statistically significant impacts are a 14 percent reduction in stops during the a.m. peak period, and reductions in delay of 8 percent during the a.m. peak period and 14 percent during the p.m. peak period. Although the remaining impacts are not statistically significant, negative impacts to arterial traffic are not expected.

The network performance data shown in Table 4 represent the aggregate impact of the proposed signal coordination on both the arterial links and the cross-street approach links at the signalized intersections. It is to be expected that some degradation in quality of flow would appear on the cross-street approaches because of the preferential treatment being applied to the arterial flow. The data in Table 4 indicate that the net impact on the entire network is generally favorable. The only statistically significant change is the estimated 12 percent reduction in stops during the a.m. peak period. Although there are a number of small negative impacts, none is statistically significant.

SUMMARY AND CONCLUSIONS

The findings of this research indicate that a small reduction in rear-end accident frequency on the arterial approaches to the case study intersections should be possible if the traffic signals are coordinated. In addition, concurrent benefits would accrue in terms of reductions in stops and delay to arterial traffic during the a.m. and p.m. peak periods. Although some reductions are projected to be as large as 14 percent, no significant changes are anticipated during the off-peak hours. It was expected that fuel consumption savings would be attainable; however, this is not supported by the data.

The potential impact of signal coordination on the case study network as a whole was found to be generally negligible. This indicates that much of the benefit that would accrue to the arterial traffic flow is offset by increases in number of stops and delay to the cross-street traffic. It can be reasonably assumed that with lower traffic volumes and turning movements, greater benefits should be possible because of the enhanced ability to create and maintain platooning of traffic. In this regard, a programmed construction of an adjacent freeway should divert sufficient traffic away from the existing facility to enable a coordinated signal system to achieve further improvements in safety and quality of flow.

Because of the case study approach used in the research, it is difficult to generalize the findings and conclusions. The hypothesis that a reduction in number of stops per vehicle can yield a concurrent reduction in rear-end accident rate was supported by the data. Similarly, the ability of signal coordination to improve quality of flow and reduce the frequency of stops, even under relatively high saturation levels, was demonstrated. The actual safety and operational benefits to be achieved through signal coordination will depend on the characteristics of the particular network.

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A Traffic Control System for the City of Kuwait

MUSA AL SARRAF, DAVID E. CASTLE, and WILLIAM G. MASON

ABSTRACT

The background of the rapidly rising demand for transportation in Kuwait is described and the main features of a proposed area traffic control (ATC) system are discussed. This system will coordinate traffic signals throughout most of the urban area of the city of Kuwait. It will also monitor car park usage, control car park guide signs, and provide a variety of data for use in traffic engineering and planning studies. The ATC system will operate in parallel with closed-circuit television and motorway (freeway) surveillance and control systems. Factors affecting preparation of specifications for international bidding are discussed, including the needs to specify functional requirements and to minimize misunderstandings between contractor and purchaser at an early stage. Finally, the importance of an appropriate organization and staffing are stressed if the new systems of control are to achieve maximum benefits. The ATC system is expected to play a key role in this organization and is an important element in the municipality's overall plan for traffic control and management in the future.

The municipality of Kuwait has overall responsibility for planning urban transport and the roadway system. In this role, the municipality commissioned a feasibility study and functional design project for an area traffic control (ATC) system to control and coordinate traffic signals in the urban area of the city of Kuwait.

The scope of the study was broad in nature in recognition of the integrated nature of modern traffic management. In addition to ATC functions, attention was focused on monitoring of operations at key multistory car parks in the central business district (CBD), a closed-circuit television system, and potential interaction with a proposed motorway surveillance and control system.

The efficiency of the existing organization of departmental responsibilities for traffic-related functions was also examined, with a view to maximizing the municipality's capabilities for effective traffic management in the future--a capability that will assume increasing importance as increasing travel demand places greater strains on the CBD and urban area road networks.

Summarized in this paper are some of the major findings of this study and an outline is given of the principal features of the system, which will play a central role in traffic management in the city of Kuwait during the next 10 to 15 years.

DEVELOPMENT OF THE URBAN AREA OF KUWAIT

Before describing the feasibility study and the ATC system in detail, it is useful to consider how the urban transport system as a whole has developed to its current form and to indicate the intended role of the ATC system as an integral part.

Pre-1950 Development

Urban development in Kuwait before 1950 occurred in the old town, bounded by a wall approximately along the line of the existing Soor Street. The population of the country at that time was approximately 100,000, the majority of whom lived in the old town. The road system in town was extremely limited and most movement was by foot, camel, or horse. Although the statistics for the period are not very accurate, it appears that less than 10,000 motor vehicles existed in the entire country.

Development in the 1950s and Early 1960s

The 1952 Kuwait master plan, shown in Figure 1, established a radial and ring system of roads extending from the boundary of the old town. This system included 1st Ring Road, 2nd Ring Road, and 3rd Ring Road, with radials converging on the town. The plan was based on a population of 1.25 million as a long-term growth target.

The radial and ring roads were all built to dual carriageway (four-lane divided roadway) standards with central reservations and generous rights-of-way, approximately 130 m (426 ft). A roundabout was constructed at almost all intersections on the Kuwait road system because it was the favored form of intersection in the United Kingdom at that time.

The decade from 1951 to 1961 was one of significant growth in population. By 1957 the population reached almost 200,000 and by 1961 had exceeded 300,000. Despite the rapid growth in population, the road network still provided a good level of service to traffic. This was primarily due to the very low rate of automobile ownership, that is, only 23,000 vehicles were registered in the country in 1960.

By 1965 the population had reached 475,000, and registered vehicles had almost quadrupled since 1960 to more than 80,000.

Transportation in the 1960s and 1970s

The development of the country had gone considerably beyond the limits foreseen in the original development plan. The municipality reacted to this development by extending the radial-ring development concept to include 4th Ring Road. However, it was clear that the original concept was out of scale with ac-

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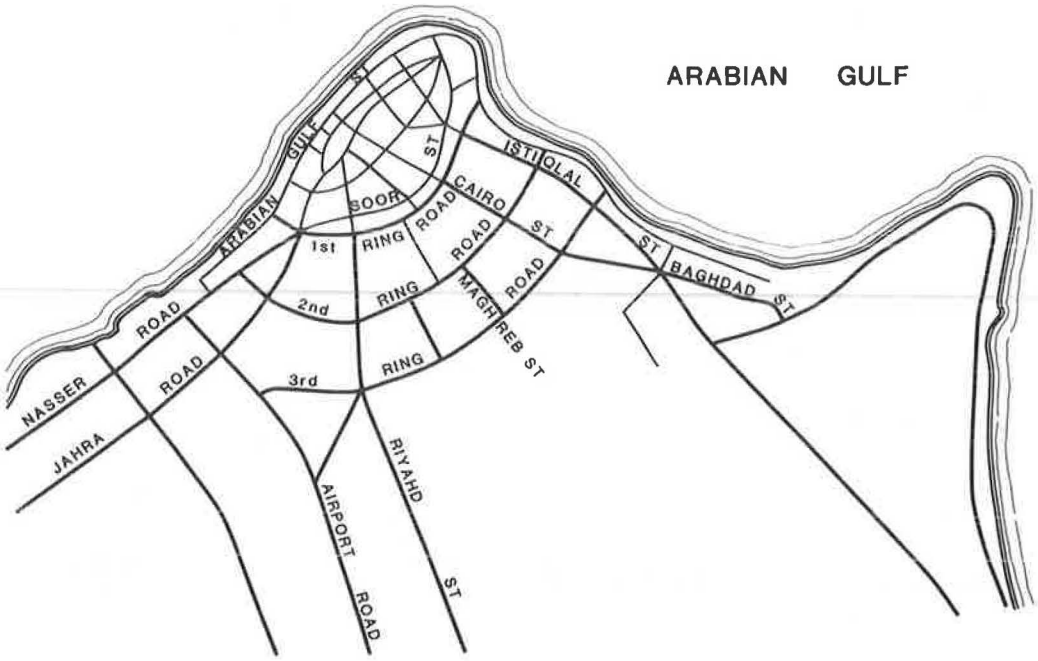

FIGURE 1 1952 Kuwait master plan.

tual and possible future developments. In 1968, the municipality initiated the development of a new master plan for Kuwait.

The new master plan predicted that a population level of 2 million would be reached between 1985 and 1997, along with corresponding increases in automobile ownership. Components of the 1971 master plan most relevant to the ATC project were as follows:

- The road system in the city of Kuwait Town should be restructured into a hierarchical system including new primary roads of motorway (freeway) standard at the highest level with secondary and tertiary roads to cater for shorter distance travel;
- A new (fifth) ring road should be built to motorway standard;
- Jahra Road from the city of Kuwait to Jahra in the west should be developed to motorway standard;
- Magreb Street should be upgraded to motorway standard and extended toward Ahmadi in the south to form the main connection to the proposed linear development down the eastern coast;
- Two other radials should be upgraded to motorway standard; and
- The population of the metropolitan area should be limited to 1.25 million, with additional development past this level being accommodated in new towns outside the metropolitan area.

The first comprehensive area traffic management plan for Kuwait was developed at the same time as the 1971 master plan. This traffic management plan introduced a system of one-way streets, related control of turning movements, improved parking layouts, replaced many roundabouts with signal-controlled intersections, and so forth. Although traffic signals had previously been applied at a few intersections in Kuwait, this plan clearly demonstrated their advantages of positive control in the urban area. Without doubt, this was the period during which the foundation of a system leading toward comprehensive ATC was set.

Design work began on the proposed motorway and expressway system during 1972 to 1973. Construction of the motorways and expressways began soon after the design effort. Consequently, much of the system is now open to traffic.

Effects of Population Growth

The problems of management and control of traffic in Kuwait are made more difficult by the population composition and rates of change. Table 1 shows the increases in the numbers of Kuwaitis and non-Kuwaitis from 1957 to 1980. It can be seen that while the Kuwaiti population increased during this period from 109,000 to 572,000 (this includes naturalization), the non-Kuwaiti population increased from 84,000 to 794,000.

TABLE 1 Population Growth, 1957-1980

Year	Kuwaiti	Non-Kuwaiti	Total
Total Population (1000s)			
1957	109.5	84.1	193.6
1961	153.7	155.0	308.7
1965	225.3	250.6	475.9
1970	336.7	397.7	734.4
1975	475.8	524.3	1,000.1
1980	572.3	794.6	1,366.9
Population Change			
1957-1961	44.2	70.9	115.1
1961-1965	71.6	95.6	167.2
1965-1970	111.4	147.1	278.5
1970-1975	139.1	126.6	245.7
1975-1980	96.5	270.3	366.8
Average Annual Percentage Rate of Increase			
1957-1961	8.8	16.5	12.4
1961-1965	10.0	12.8	11.4
1965-1970	9.6	9.7	9.7
1970-1975	5.9	5.7	5.8
1975-1980	3.8	8.7	6.5

Since 1961, the majority of the population has been non-Kuwaiti, that is, persons who come from a wide variety of backgrounds, nationalities, and social groups. Furthermore, the non-Kuwaiti population is continually changing because of migration into and out of the country. The 1980 census indicated that almost 50 percent of non-Kuwaitis had resided in Kuwait for less than 4 years. In such a dynamic situation, it will be appreciated that the scale of traffic growth is only one aspect of the problems affecting traffic control.

Because of a variety of factors, the municipality was convinced that traffic levels were likely to exceed the estimates of the 1971 master plan. Therefore, the municipality decided that an ATC system, in parallel with the work on the new primary network of motorways, could provide an overall improvement to the secondary network. This led to the municipality's commissioning Wilbur Smith and Associates to conduct the ATC feasibility study in 1980.

SCOPE OF STUDY

In recognition of the important role that traffic management has to play in meeting the severe transportation demands in a city such as Kuwait, the scope of the study was defined to cover a broad range of topics. In addition to a feasibility study and the preparation of functional specifications and plans for a comprehensive traffic control system, the scope encompassed recommendations on the creation of a traffic engineering department. For purposes of describing the scope of the study, the work can be divided into the following four categories:

- Data collection
- Definition of required facilities
- Preparation of specifications and plans
- Organization of a traffic management department

Each of these categories is described in the following subsections.

Data Collection

As is to be expected with a study of this nature, a wide variety of data was collected, including the following types:

- Manual intersection turning movement counts
- Directional machine traffic counts
- Speed-and-delay studies
- Driver behavior observations
- Vehicle occupancy counts
- Pedestrian movement counts
- Equipment inventory and intersection photographs
- Emergency vehicle operations
- Platoon dispersion studies

Most of the data collection surveys were conducted in a fairly conventional manner and are therefore not elaborated on further. However, the inclusion of platoon dispersion studies is somewhat unusual in a project of this nature and therefore will be further discussed in the following subsection.

Platoon Dispersion Studies

The distance between signalized intersections in certain areas of the Kuwait road network exceeds that range normally associated with deriving benefits from coordination of signal timings. However, because of the high standards of road construction, typically six-lane divided roadways with limited access to neighboring residential areas, traffic was observed to remain in platoons for distances of up to 2 km (6,560 ft).

Because of these observations, measurements were made of the rate at which platoons disperse on the roads in Kuwait. The data collection techniques used and subsequent analysis of data are described in more detail elsewhere (1). As a result of these efforts, it was concluded that

1. Traffic remains in platoons on the ring and arterial roads of Kuwait sufficiently to derive benefits from coordination of signals that are typically 1,000 to 2,000 m (3,280 to 6,560 ft) apart;
2. The normal range of values for the platoon dispersion factor (K) used in the TRANSYT signal-timing optimization program (2,3) is applicable to the ring and arterial roads of Kuwait as well as to the CBD road network; and
3. The optimized timings resulting from TRANSYT are not unduly sensitive to the value of the dispersion factor used, and in the majority of networks use of the program's default value (K35) will be adequate.

Definition of Required Facilities

The need to improve traffic management capabilities in Kuwait in terms of staff, organization, and the facilities available to the traffic engineer led to a wide range of functions being considered for inclusion in the system. These functions included

- Signal control and timing
- Emergency vehicle facilities
- Car park monitoring
- Closed-circuit television
- Motorway surveillance and control
- Data collection and retrieval

The extent to which each of these functions was incorporated into the functional design of the ATC system is described in the following subsections.

Signal Control and Timing

The continuing development of the city of Kuwait necessitated a system design that would be flexible to both short- and long-term changes in traffic demand. Accordingly, many intersections were specified to be equipped for vehicle actuation, with the central system having the capability to control, at any particular time, the mode of operation of each controller (pretimed, semi-actuated, or fully actuated operation). The flexibility to reorganize from the central computer the groups of intersections operating on a common cycle time (changing subarea boundaries) was also required.

Significant stand-alone capabilities were also required of controllers, including time-based coordination (cableless linking) to protect against failures of the data transmission system.

Alternative timing-plan selection techniques (time-of-day, traffic-responsive plan selection, and on-line optimization) were considered. The well-proven time-of-day technique was chosen. Although some additional benefits may be provided by an on-line optimization technique in the Kuwait CBD, it was not considered appropriate to specify such a

requirement in the initial system. The most well-proven and thoroughly tested technique of this nature is the SCOOT system, developed by the Transport and Road Research Laboratory in England and three British signal system companies (4). Because this technique is only available to British companies at the current time, it was not considered proper to specify it by name in a specification to be used for international bidding. To permit such facilities to be made available in the future, the specifications required that the system be capable of expansion to provide advanced traffic control techniques.

Emergency Vehicle Facilities

During discussions with fire service personnel, the desirability of preempting traffic signals was identified. Such preemption would assist emergency vehicles moving along predetermined routes, shown in Figure 2, by providing a green signal for these vehicles and by clearing traffic ahead of them. Accordingly, a preemption system was designed and specified in which the central computer directly controls each preempted controller via the normal communication lines. The fire preemption system is initiated by fire service personnel requesting 1 of up to 15 routes at a terminal located at the fire house. The terminal is connected to the central computer via the same type of communication lines as the controllers. In the initial system, two fire houses will be equipped with preemption terminals. The system will be capable of accommodating a total of at least six terminals.

A simpler, hard-wired form of preemption was required at two other fire houses. In these cases, a single intersection nearest the fire house is preempted when a button is pressed inside the fire station. Up to three exit plans will be provided for each fire house.

Car Park Monitoring

The construction and control of 23 multistory car parks in the CBD is a key element of the municipality's transport policy for the immediate and short-term future. Because these car parks will be widely distributed throughout the CBD, it was considered important to inform motorists of the location of car parks in which excess capacity exists at any point in time. Signs controlled by the ATC system will present information to the motorist sufficiently far in advance to direct him to an alternative car park if his first-choice car park is already full. Two types of signs will be used, which are shown in Figure 3. Providing this information should minimize the extent to which CBD congestion is exacerbated by motorists searching for a parking space.

The ATC system will receive data on the number of vehicles entering and leaving each car park and will control the informational signs. The ATC system will not, however, control the FULL signs immediately outside a car park or on each floor of the car park (if installed); these signs will continue to be controlled by the internal monitoring system of each car park.

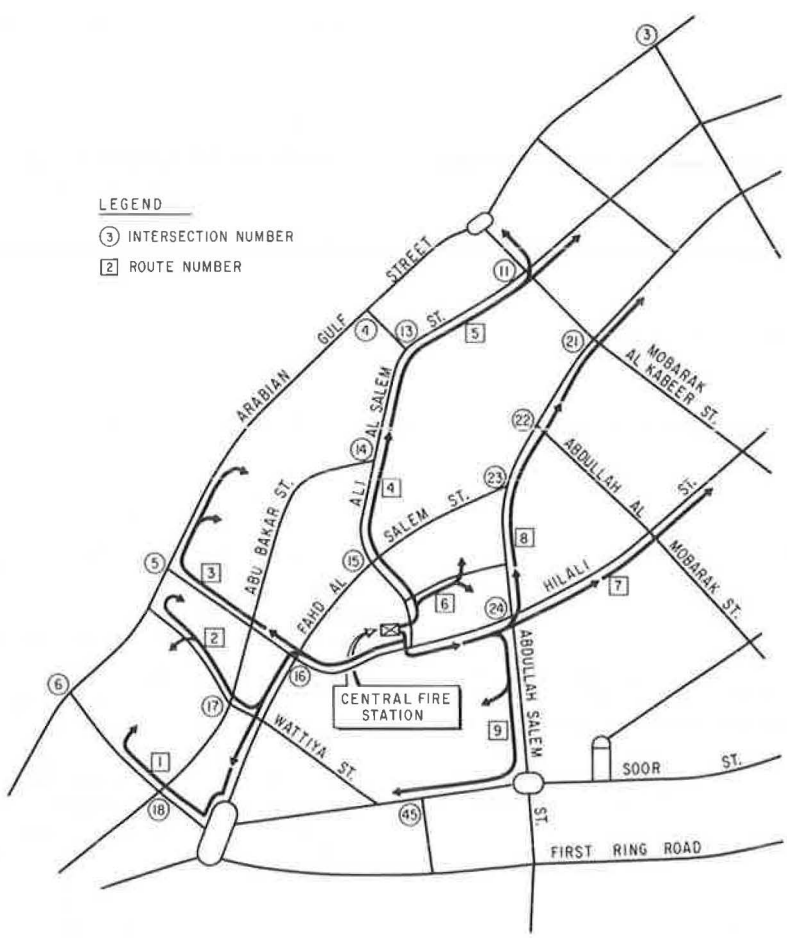

FIGURE 2 Proposed preemption routes for central fire station.

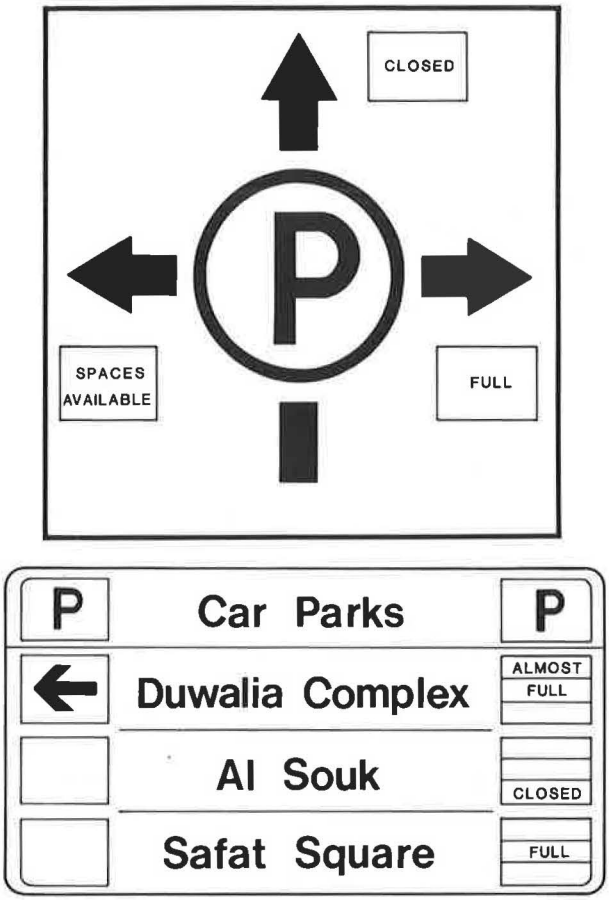

FIGURE 3 Car park information signs.

Monitoring of car parking performed by the ATC system is only one element of the municipality's car parking policy. If loss of roadway capacity caused by parked vehicles is to be minimized, several other points will also have a critical role to play: reduction of roadside parking, more rigorous enforcement of parking regulations, and control of car park fees.

Closed-Circuit Television

To enable system operators to monitor traffic conditions more effectively, a system of closed-circuit television (CCTV) cameras was designed to cover several key areas of the road network. A total of 12 cameras will be installed initially. These will be controlled by operators in the Traffic Control Center, who will view incoming pictures on any of the 12 wall-mounted television monitors and two desk-mounted monitors. The operator may also select that the output of any camera be recorded on a standard video cassette recorder for special studies, as necessary. A separate television monitor will be provided for use with the video cassette recorder. The CCTV system will be capable of expansion to incorporate up to 64 cameras and 16 wall-mounted monitors.

The CCTV system, while integrated into the traffic control system in an operational sense, will be provided by an entirely independent set of equipment. The CCTV system will be using its own transmission lines (coaxial cable), but may share some of the ducting used by the traffic control communication lines (25, 50, or 100 pairs of telephone-type cable).

As in most traffic systems of this nature, the use of CCTV will provide only one of a variety of means by which traffic problems will be brought to the attention of system operators. The ATC system has the capability of reporting when congestion is detected by unusually high occupancy at mid-block vehicle detectors. Direct communications will also be available from the ATC Center to the police and public transport agencies so that reports radioed in by police and bus drivers on the street can be relayed directly to traffic engineering staff. In addition, information from Kuwait's motorway surveillance and control (MSC) system will be readily available; this system is explained in the next subsection.

Motorway Surveillance and Control

At the time of the feasibility study in the early 1980s, Kuwait had already committed itself to a program of upgrading existing roads and constructing additional roads to provide a network of urban motorways. The principal features of this network were to be a new ring entirely surrounding the CBD, including both elevated and depressed roadway sections, together with a number of radial routes leading outward through the urban areas to neighboring towns and industrial complexes.

This motorway network will further improve the already high standard of roadway facilities in Kuwait and will eliminate a number of significant conflicting movements by grade separations. Nevertheless, the network will pose its own control problems, particularly regarding entry to and exit from the motorway system. The motorway's ability to rapidly move large volumes of traffic through the urban area is of little value if the street network in the vicinity of CBD exits is unable to absorb the demand without undue congestion. Design of the MSC system was not within the scope of the ATC feasibility study, and implementation of the motorway control system was expected to follow the ATC system at a later stage. However, the need for coordination of control activities between the two systems was recognized.

A number of possibilities concerning the integration of the MSC and ATC systems were reviewed. Coordination of control will be achieved principally at the operator level by the sharing of a common control room. The room will be of sufficient size to ensure that operators are not unduly distracted from their own duties. Nevertheless, this close proximity of operators controlling the two systems should enable the necessary feedback to be received concerning the impact on one system of action taken in the other. The central equipment of each system will operate independently of the other, although in due course some interchange of traffic count data may occur. In the street, the communication cables of the two systems will share ducting whenever practical.

Data Collection and Retrieval

Collection of a variety of traffic data and convenient presentation of such data to the end user were considered important requirements for achieving the overall objectives of providing an effective traffic management tool. Data to be collected range from short-term monitoring information on equipment failures, detector occupancy measurements, and car park status (full, almost full, or space available) to daily, weekly, and monthly reports and summaries of traffic volumes and car park usage.

Monitoring information will be presented by means of a wall map display operation, color graphics displays, and a combination of CRT and printed messages. The wall map display will provide a general overview of status. The color graphics displays will provide more detailed information and will be of particular value to operators and engineers because of the control available over the nature and level of detail of data presented. The optional weekly and monthly summaries of traffic flow and car park usage will be provided as printed reports; they are expected to be of use to planning and other departments, as well as to the traffic engineering department.

Preparation of Specifications and Plans

Specifications were prepared, to the extent possible, in a functional form to be suitable for international bidding. The specifications gave the contractor a certain degree of latitude in the final system design because detailing specific requirements could have inadvertently precluded some manufacturers from responding. In view of the functional nature of the specifications, the contractor will be fully responsible for designing, supplying, and installing a complete and operational ATC system, meeting the minimum requirements of the specifications.

Hardware Requirements

The functionality of the specifications applies to both hardware and software. In the case of hardware, certain minimum requirements were identified, together with specified performance characteristics such as the percentage of spare disk capacity, and CPU time and the maximum permitted response time to operator commands. Within these requirements, the contractor would then be responsible for designing and configuring a suitable hardware and software combination.

The computer configuration envisaged is shown in functional block diagram form in Figure 4. It features dual computer systems, one of which usually operates in a backup mode ready to take over if the primary computer fails. Peripherals essential for on-line operation are automatically switched to the on-line computer at all times. The primary storage medium is a disk, with magnetic-tape facilities being provided for archiving purposes.

Sufficient processing power and memory-disk capacity are required to enable the system to perform general engineering functions as background tasks, if necessary.

Software Requirements

The traffic engineering capabilities of a computerized signal system are defined by the system's software. To permit international bidding, no particular software package was specified by name, and requirements were defined in purely functional (although detailed) terms. The overall scope of the requirements is comparable to those included in systems in many other major cities around the world.

Using only time-of-day plan selection in Kuwait represents a simplification of software requirements, compared with other cities. On the other hand, the dual computer operation and car park monitoring requirements are less common.

Construction and Installation Standards

In addition to the functional plans and specifications for the ATC system hardware and software, the

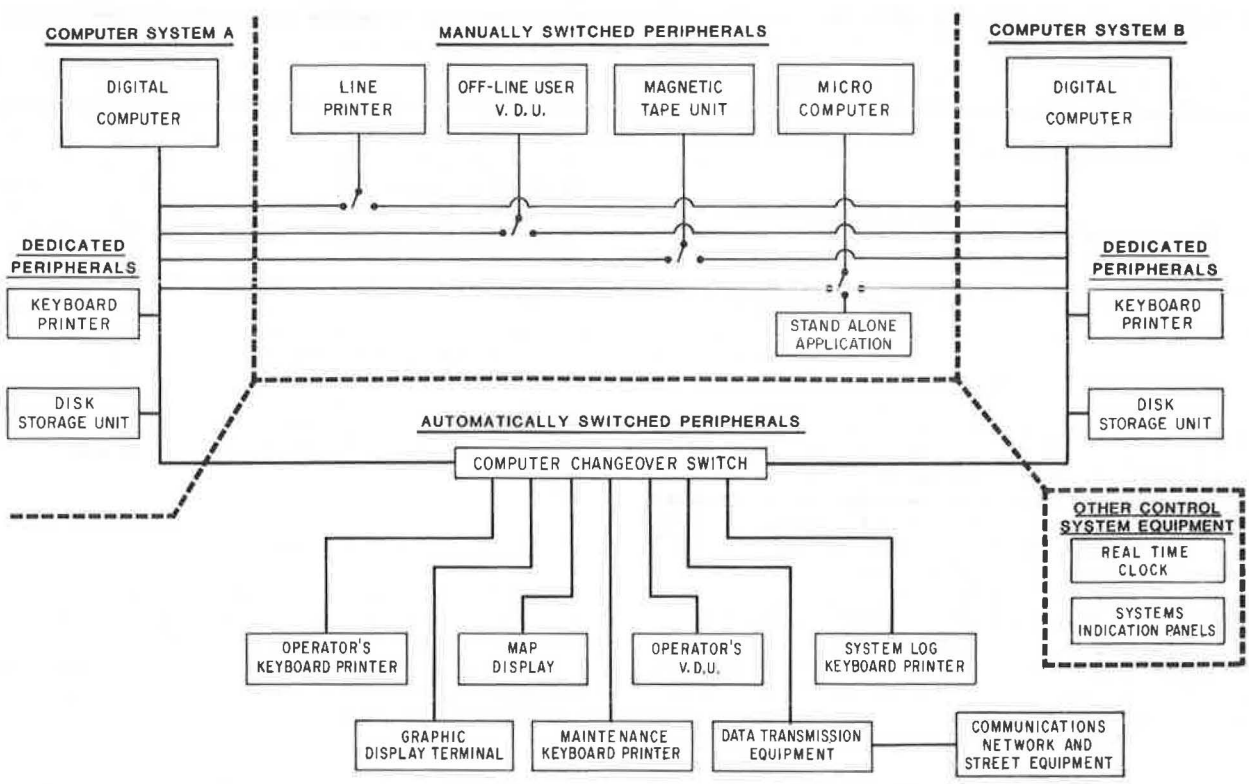

FIGURE 4 Functional block diagram form of central computer system.

practices and standards for construction of roadworks and installation of traffic signals and auxiliary equipment were defined in detail. To permit the contractor a measure of latitude, appropriate minimum standards and requirements for system implementation were developed. Detailed plans were prepared for roadway and intersection geometrics as well as for improvements to existing on-street signal hardware (signal heads, poles, conduit, detectors, etc.) and the construction of new signalized intersections.

Final System Proposal

As expected with a system of this nature, the specifications are necessarily detailed and lengthy, covering a wide range of equipment and services to be provided by the contractor, in addition to the computer hardware and software mentioned previously. Most sections of the specifications are along conventional lines, defining requirements for controllers, detectors, communications equipment, and so forth. However, one element of the specifications, while less common, is considered particularly important because of the international nature of the contract and the resulting problems of communications between the principal parties involved.

At the commencement of the contract, a 5-month-long final system proposal (FSP) period begins, during which the contractor is required to present a detailed proposal describing all aspects of the system to be provided. A preliminary submission is required after 2 months and a final submission after 5 months. Each submission is reviewed and discussed with the contractor in detail to ensure that the contractor fully understands and intends to comply with the specifications and to ensure that the purchaser fully understands the strengths and weaknesses of the proposed system.

The FSP period is typically one of intense discussion and brainstorming by all parties; this period is aimed at defining the final product in far more detail than can be placed in functional specifications or a bidder's technical proposal. The objective is to clearly define a system that both meets the requirements of the specifications and makes full use of the inherent strengths of the contractor's system philosophy.

Until the contractor's FSP is approved, no equipment may be ordered or manufactured. Although the FSP requirement may appear to result in the lack of any visible progress in system implementation for the first 5 months of the contract, this arrangement has proved most valuable on previous projects of this nature. Having an FSP period minimizes problems of system acceptance, both at the factory demonstration stage and on site following installation; it also should minimize wasted efforts on the contractor's behalf through avoidance of manufacturing of hardware or developing software for a system element that will ultimately not be accepted.

In a system as complex as a comprehensive computerized traffic signal system, it is inevitable that problems will arise during development, installation, and testing of the system. The requirement for an FSP period is based on the following premise: the earlier that problems, deficiencies, and misunderstandings are identified, the better for all concerned.

Organization of a Traffic Management Department

It is recognized that without appropriate staffing and organization of management, modern computerized systems for control of traffic signals and the motorway network will not in themselves provide the maximum benefits.

To date, management of many traffic-related functions in Kuwait has been somewhat fragmented between a number of ministries and the municipality. In some cases, the traffic function of a department has not been its primary area of responsibility and consequently may not always have received the attention it deserved. This has led to the situation in which traffic operations do not reflect the same level of management that is evident in the overall roadway network.

The impact of inefficient traffic operations has not been too severe to date because of the high standard and spare capacity of the roadway network. However, with increasing levels of vehicle ownership and travel demands, the need for more efficient usage of the road system is already apparent and will become increasingly apparent in the future.

To provide the necessary manpower and level of expertise required, the consultant recommended that (a) a single organization be responsible for dealing with future traffic problems, and (b) the wide-ranging goals of the nation's transportation and development plans, including the ATC system, be implemented. Overall responsibility for traffic management and control would be the primary concern and not merely a secondary activity of this proposed organization.

To facilitate the development of a strong traffic management organization, the ATC and MSC systems are to be housed in their own specially designed control center, with adequate office accommodations for management, engineering, and technical staff.

CONTINUING DEVELOPMENT IN KUWAIT

In 1983, the municipality commissioned the second overall review of the Kuwait master plan (the first review was conducted in 1977). Although these reviews resulted in modifications to the planned highway system, the overall principle of a hierarchical system was maintained, as shown in Figure 5. In the metropolitan area, the highway system has been extended to the west (5th Ring Road and 6th Ring Road), an outer bypass incorporated (running northwest from Shuaiba to Sulaibiya), and an intermediate connector added from this bypass, south of the airport, to Fintas (7th Ring Road).

According to the 1985 census, almost 1.7 million people currently reside in the metropolitan area, compared with the 1971 master plan projection of 1.25 million.

This review of the master plan, along with the data in the 1985 census, confirms the increasing importance of traffic control at all levels. As previously noted, an MSC system is already planned for the motorways and expressways. The ATC system, which will control signals within the CBD and on the main secondary roadway network, will interact with the MSC system and will form an integral part of Kuwait's overall traffic control and management program.

ACKNOWLEDGMENT

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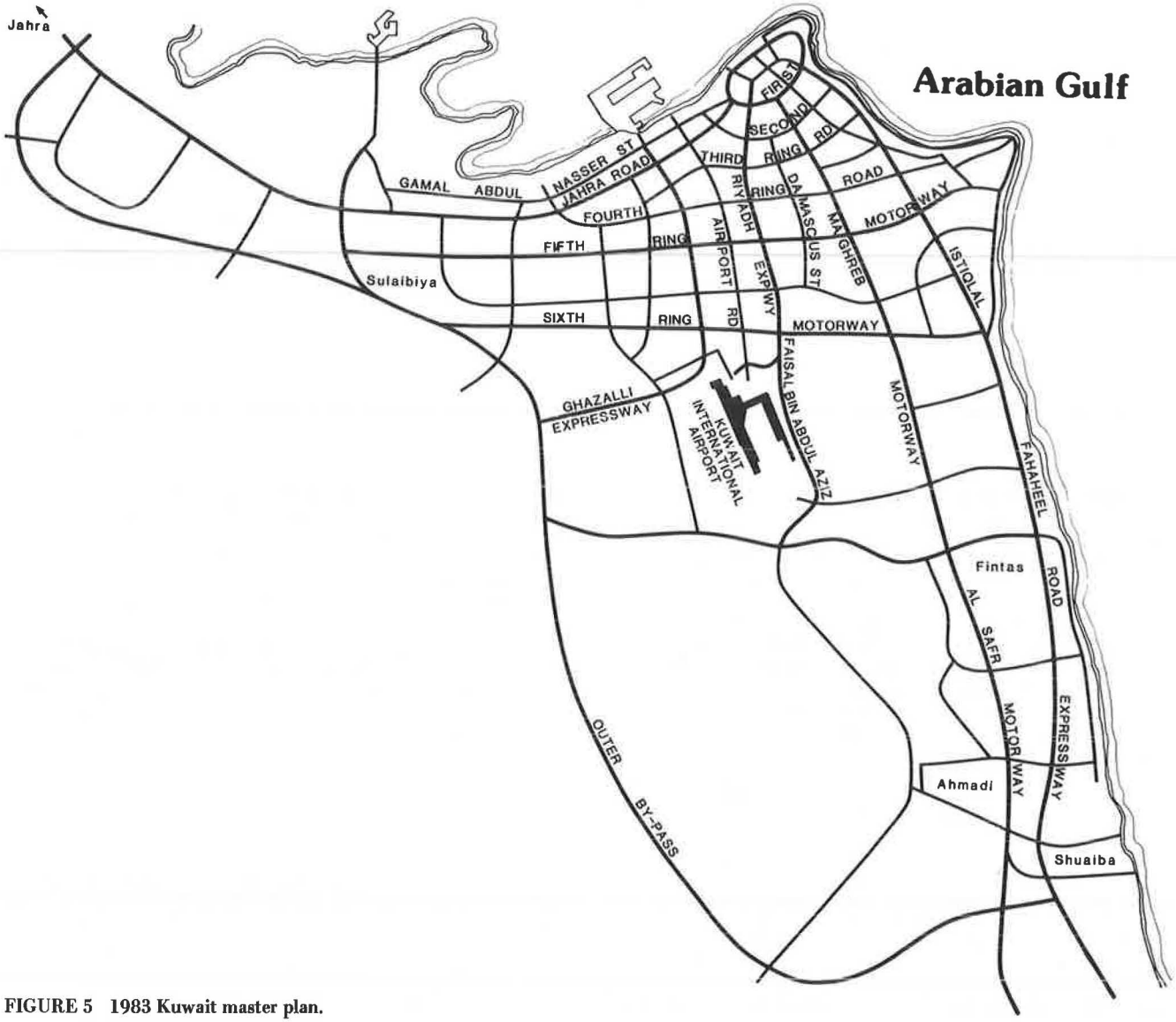

FIGURE 5 1983 Kuwait master plan.

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