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

WILLIAM D. BERG, ALAN R. KAUB, and BRUCE W. BELSCAMPER

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.

W.D. Berg and A.R. Kaub, University of Wisconsin, Madison, Wisc. 53706. B.W. Belscamper, Wisconsin Department of Transportation, Madison, Wisc. 53707.

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.

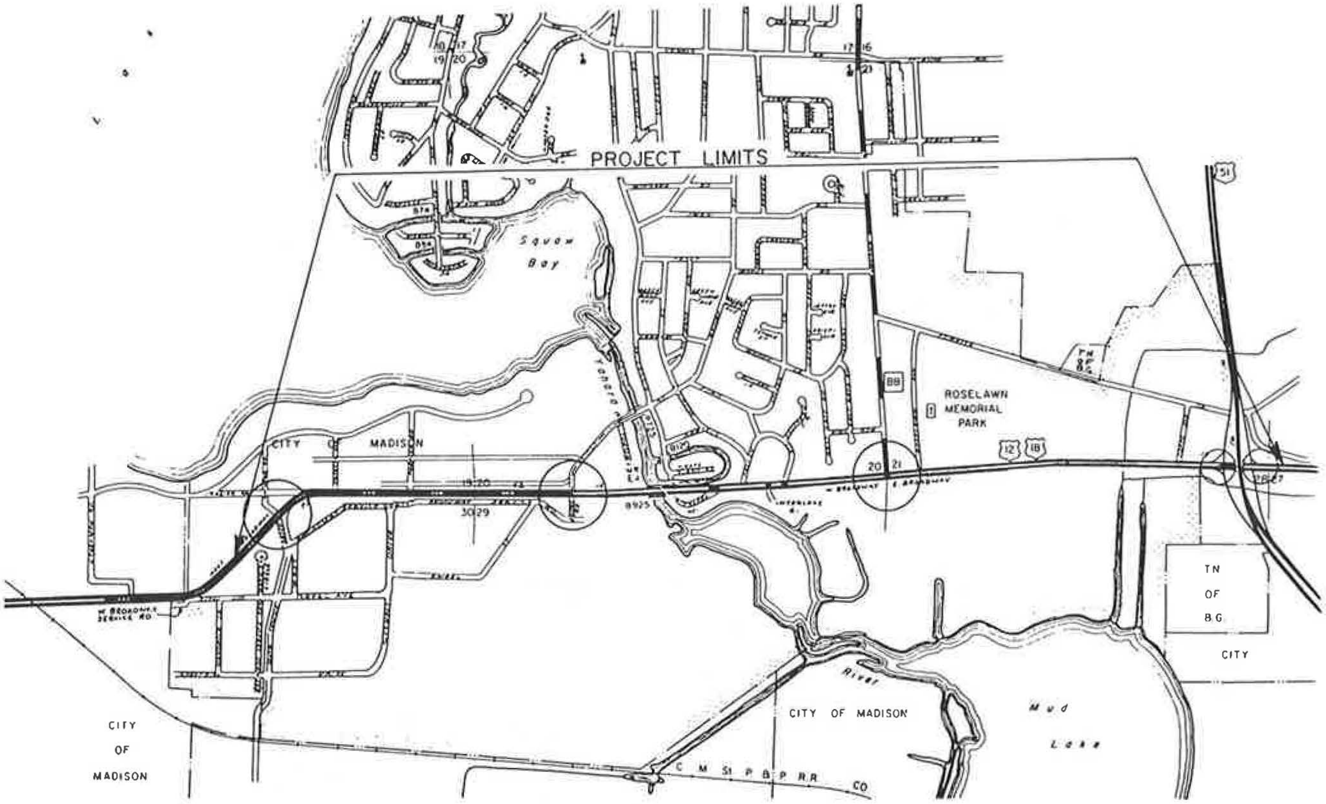

FIGURE 1 Beltline Highway study area.

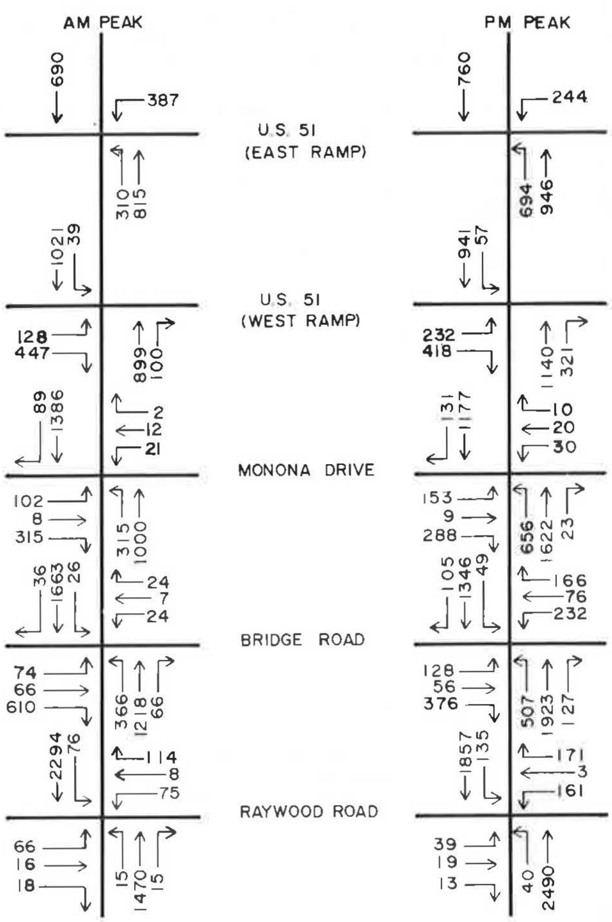

FIGURE 2 Volumes for a.m. and p.m. peak hours.

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.

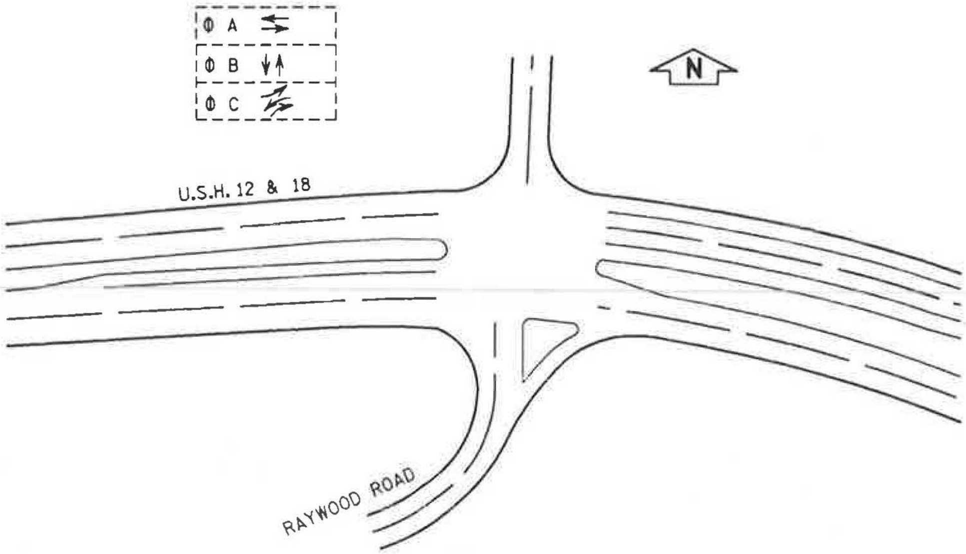

FIGURE 3 Condition diagram and signal phasing: Raywood Road.

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

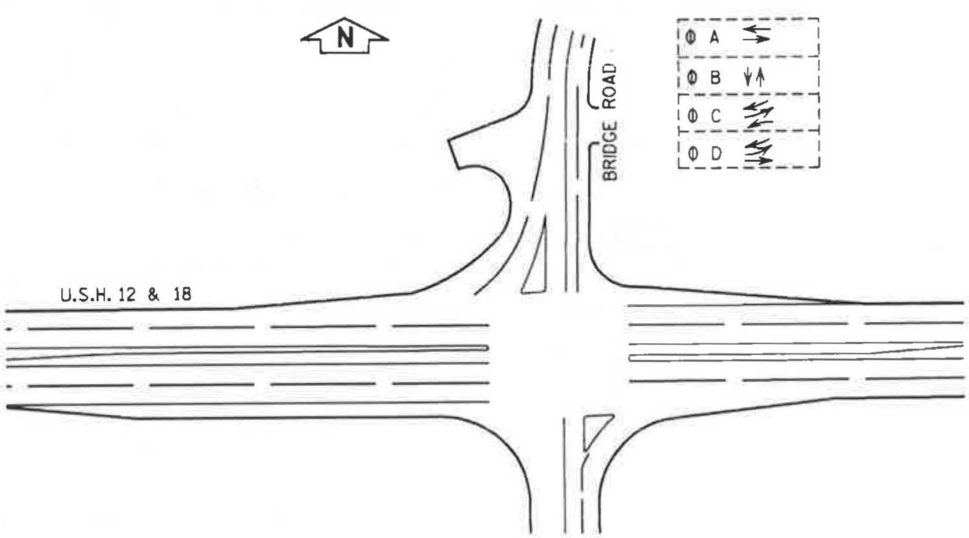

FIGURE 4 Condition diagram and signal phasing: Bridge Road.

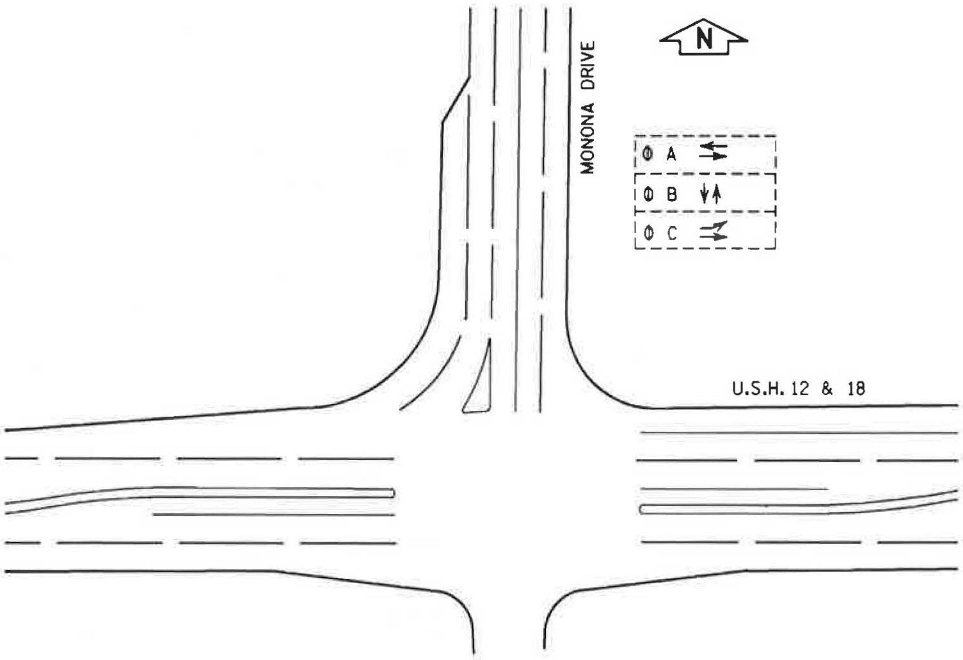

FIGURE 5 Condition diagram and signal phasing: Monona Drive.

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

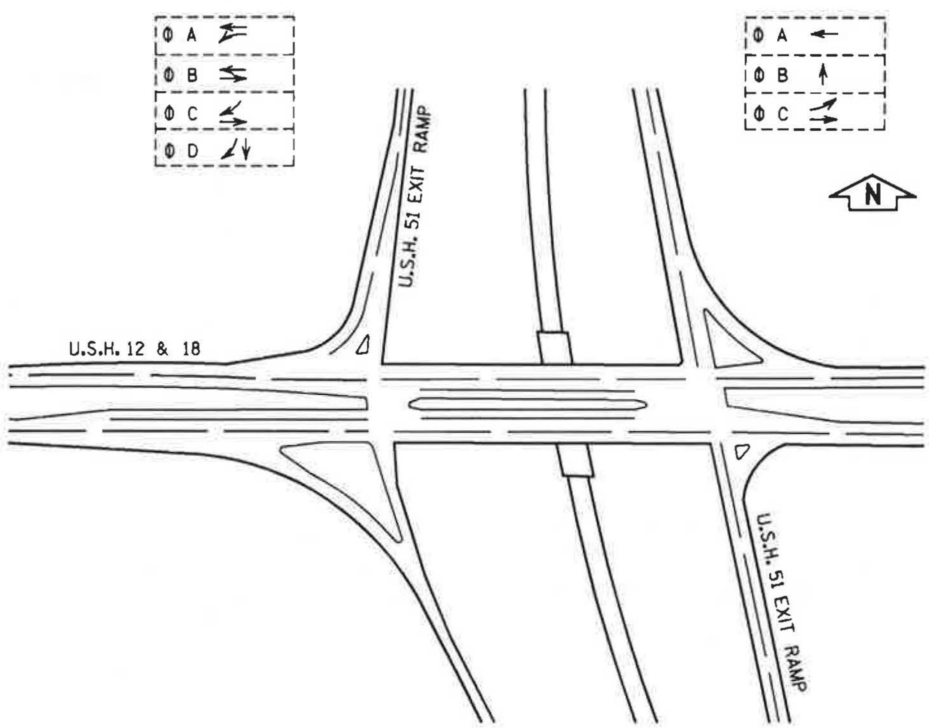

FIGURE 6 Condition diagram and signal phasing: U.S. 51 interchange.

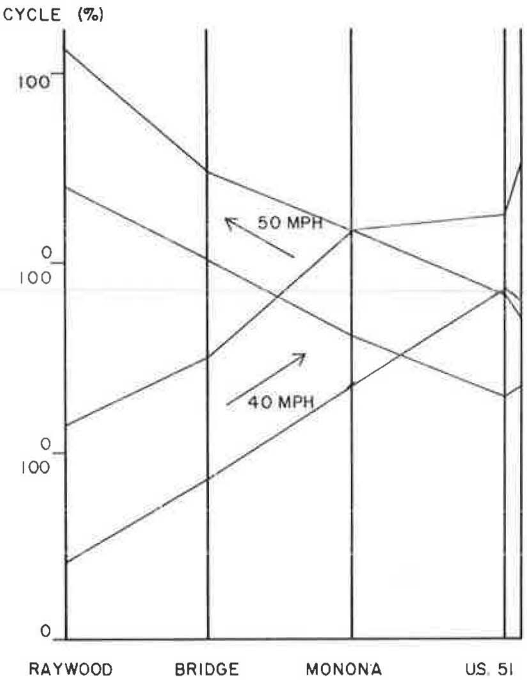

FIGURE 7 Signal progression bands for a.m. peak period.

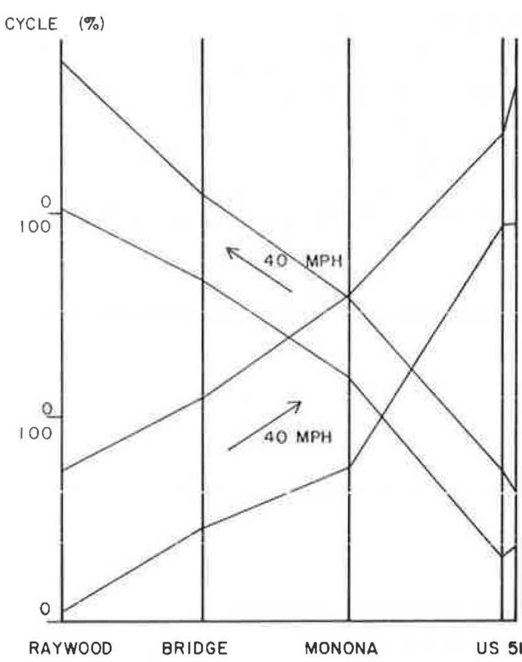

FIGURE 8 Signal progression bands for p.m. peak period.

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.

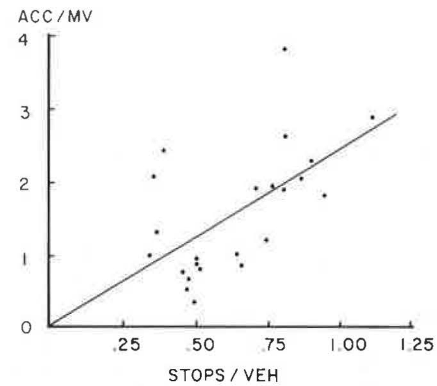

FIGURE 9 Rear-end accident prediction model.

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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