Figure 7. Probability of stopping during yellow interval versus accepted deceleration rate.

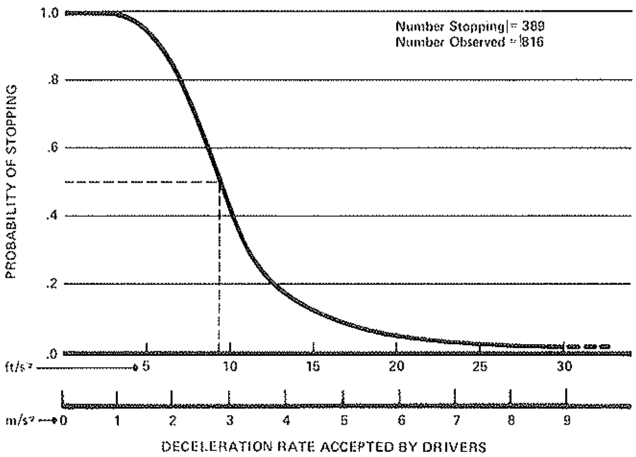
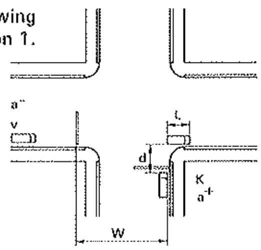


Figure 8. Intersection showing parameters used in Equation 1.



the vehicles stopping most quickly is 2.0 m/s^2 (9.7 ft/s^2). Drivers confronted with a close decision during the yellow interval will accept a deceleration rate of 2.0 m/s^2 (6.5 ft/s^2) 85 percent of the time.

Method of Determining the Length of the Clearance Interval

The minimum length of the clearance interval can be calculated to adequately serve drivers' needs and to meet law enforcement purposes by using Equation 1.

It should be noted that the time calculated from this equation is that needed for clearance. This can be provided by the yellow interval in combination with an all-red interval. Using this technique, a city or county can standardize the length of the yellow interval (say, 3.6 s for 60-s cycle phase) and provide additional clearance time with the all-red interval. If this equation is to be correctly applied, engineers should conduct field studies in their own locations to determine local values for the unknown parameters.

The reader should note that the terms of Equation 1 are not new and that various permutations of them have been recorded in the literature since 1929 (3). The value of Equation 1 is that it is theoretically correct and includes all parameters involved in the clearance decision. Engineers should develop probability of stopping versus time charts similar to Figures 1-7 for their own cities. In this way the decision time $[R + (V/2a^-)]$ can be computed by how drivers actually behave in the area being studied. The time deduction for cross-flow acceleration needs to be applied with caution, and a value of zero should be used if light jumping is possible (yellow interval visible to waiting traffic).

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Optimization of Pretimed Signalized Diamond Interchanges

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This paper describes a computer program that can determine the best strategy for a pretimed signalized diamond interchange to minimize the average delay per vehicle. This program, PASSER III, is one of a series of signalization programs developed for the Texas State Department of Highways and Public Transportation. All basic interchange signal phasing sequences, including all possible patterns from lead-lead, lag-lead, lead-lag, and lag-lag phasings, are evaluated by the program. Interchange performance is evaluated by using average vehicle delay; exterior delay is calculated by Webster's delay equation; and interior delay is determined from deterministic delay-offset techniques. Minimum delay analyses of 18 sample problems were made. Many signalization phasing patterns were found to provide optimum operation over the set of problems evaluated. While four-phase overlap and three-phase timing plans were normally found to provide good operation, other signalization patterns may produce even better operation.

The signalized diamond interchange is a critical facility for providing high performance levels along urban freeway corridors. Efficient movement of traffic through the interchange and the quality of service provided motorists depend to a large measure on the type of signalization used. However, there seem to be differences of opinion regarding the best way to signalize a diamond interchange. This is probably because no efficient methodology for analyzing the problem has been proposed, although numerous researchers have made significant attempts at providing guidelines for improving diamond interchange signalization.

This paper describes a computer program that can determine the best strategy for a pretimed signalized diamond interchange to minimize the average delay per vehicle. This program, named PASSER III, is one of a series of signalization programs (1, 2) developed for the Texas State Department of Highways and Public Transportation.

Munjal (3) presented a systematic discussion of diamond interchange signalization, and he and Fitzgerald (4) reported on diamond interchange simulation programs. Much discussion in the literature has addressed the relative merits of four-phase overlap signalization compared with other types of phasing patterns. One paper (5) has contributed to this discussion and, perhaps, to the confusion. We hope that our paper will provide a tool that can be used by traffic engineers to accurately analyze their interchange problems and that can eliminate the need to rely on debatable guidelines.

The basic problem is how to determine, for a given set of traffic demands, the best pattern for pretimed signals at a diamond interchange. If the traffic volumes are those presented in Figure 1, Poisson arrivals are assumed for exterior traffic flow. All arterial through movements and frontage roads have two-lane approaches, whereas the two interior left-turn volumes are serviced by one-lane left-turn bays of adequate storage capacity. All geometrics and volume assumptions are arbitrary.

SIGNAL PHASING

Let us now look at the left intersection of a diamond interchange shown in Figure 2 and see how many different signal phases with no conflict among movements this intersection can have. Phase A is when the off-ramp and the left-turn traffic from the arterial are stopped and the straight through traffic is moving. Phase B results when the traffic from the off-ramp is given a green signal; all other movements at this intersection must be stopped at this time. Phase C occurs when the outbound arterial left-turn traffic is given a green signal, and all the incoming conflicting traffic feeding the diamond at this intersection stops. There are no additional phases at this intersection. There are only three similar phases on the right intersection of the interchange; these form the basis for all the possible phasing patterns. Any phases for pedestrians, as well as the amber phases for motorists, have been excluded from these and from all phasing patterns discussed in this paper.

Munjal (3) has shown that the phase order of left and right intersections can be ABC or ACB independently of one another. Order ABC was called leading left turns and order ACB lagging left turns. Thus there are only four possible basic interchange phasing codes (sequences) that can be generated (Figure 3). Munjal's equivalent descriptions are as follows:

Phase Code	Left Phase Order	Right Phase Order	Munjal's Description
1	ABC	ABC	Lead-lead
2	ACB	ABC	Lag-lead
3	ABC	ACB	Lead-lag
4	ACB	ACB	Lag-lag

All of the possible signal phasing patterns that an engineer might devise can be developed by using these basic phase codes and then varying the offset between the two intersections from zero to one cycle length. In this paper, the offset is defined as the time difference in seconds between the start of left basic phase A and the end of right basic phase B.

An example of how an interchange signal phasing pattern results from a given interchange phasing code and offset is presented in Figure 4. Phase code 1 and an arbitrary offset have been selected.

SIGNAL TIMING

Signal green times are usually calculated independently of the interchange phase code selected by PASSER III as if the two intersections were also independent of one another. It is possible to override this basic operating procedure and force selected movements to have equal green times, although this reduces the efficiency of the interchange. One other exception is permitted, the four-phase with overlap signal calculation requirements (5).

The green times of phases A, B, and C of Figure 2 are calculated, in the independent mode of operation, using Webster's formula (6):

$$G = (y/Y) \cdot (C - L') + L \tag{1}$$

where

- G = phase green on approach (s);
- y = q/s;
- q = approach volume (vehicles/s);
- s = approach saturation flow (vehicles/s green);
- Y = sum of all y at intersection;
- C = cycle length (s);
- L = intersection lost time; and
- L' = sum of intersection phase lost times (s).

Messer and Berry (5) have shown that a formula similar to Equation 1 should be used to calculate green times for four-phase overlap signalization (a special case of interchange phase code 1). In this case, green times on the four external approaches to the interchange are calculated from

$$G = (y/Y) \cdot (C + \phi - L') + L \tag{2}$$

where G is the green phase on exterior approaches in seconds and ϕ is the sum of interchange overlap (travel times) in seconds.

EXTERIOR DELAY

The performance of a diamond interchange is evaluated primarily on the basis of average delay for all vehicles using the interchange. At the beginning of the analysis procedure, delays on the four exterior approaches to the interchange are first calculated by Webster's delay equation (6)

$$d = [C(1-\lambda)^2]/[2(1-\lambda x)] + \{x^2/[2q(1-x)]\} - 0.65(C/q^2)^{1/3}\, x^{(2+5\lambda)} \tag{3}$$

where

- d = average vehicle delay for exterior approach movement (s/vehicle);
- q = approach movement flow rate (vehicles/s);
- λ = proportion of cycle green for approach movement; and
- x = signal saturation ratio, qC/gs; $g = G - L$ (s).

A total of 14 separate exterior movements are analyzed for delay, 1 for each identifiable turning movement from the exterior approaches. The two arterial approaches have 3 movements: right turn, through on arterial, and through then left turn within the interchange. The two ramps (frontage roads) have 4: right turn, through, left turn then through on the arterial, and left turn then left turn within the interchange (a ramp U-turn).

Figure 1. Signalization and approach volumes at a diamond interchange.

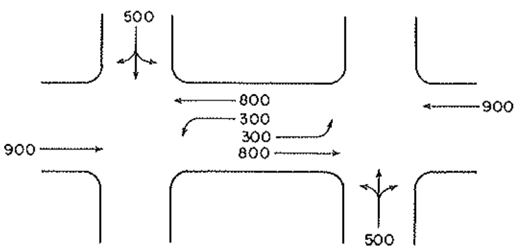


Figure 2. Three basic phases at left intersection of interchange.

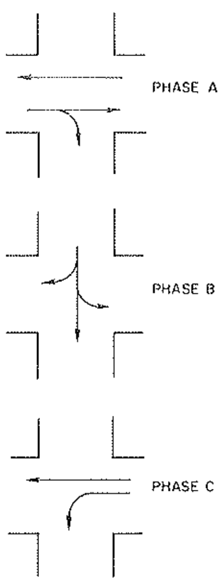
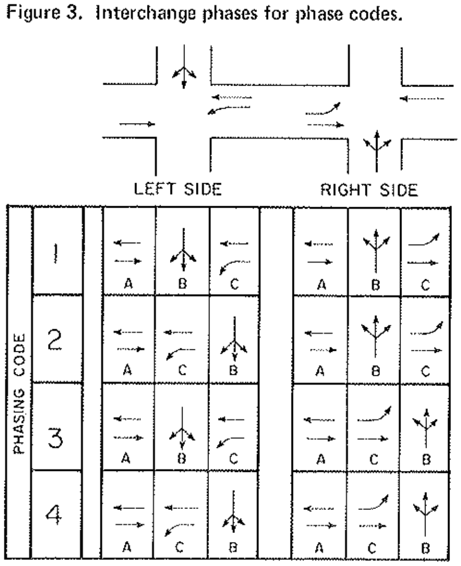


Figure 3. Interchange phases for phase codes.



INTERIOR DELAY

Vehicle delays that occur within the interchange are calculated by a version of the deterministic delay-offset technique (7). Several excellent papers have described applications of this technique to signalized intersections (8, 9, 10). Documentation and validation of PASSER III is likewise available (2).

Figure 5 (9, p. 16) shows how Gartner defined a traffic link as a section of street carrying a traffic flow movement in one direction between two signalized intersections. Delay is incurred at the downstream signal of the link where traffic exits from the link. The offset across any link may be defined as the time difference between the starting point of green phase A at the upstream signal of the link and the starting point of the next green phase at the downstream signal. A link is a directional quantity, assuming the direction of traffic flow along it. Gartner described the flow of traffic through the link's exit signal and developed the computational procedure for obtaining a delay-offset relationship (9, pp. 13-15).

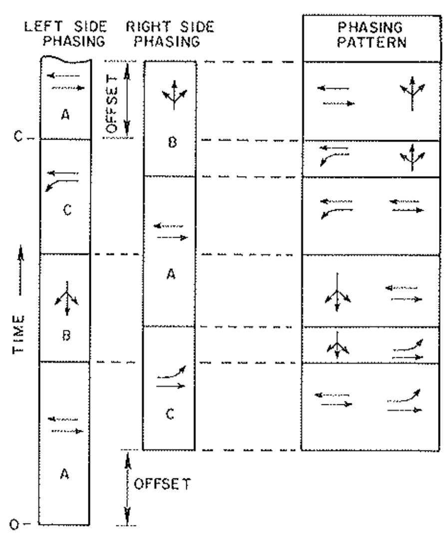
INTERCHANGE DELAY

Average delay per vehicle using the diamond interchange is calculated by combining the effects of exterior and interior interchange delays. For an otherwise given set of geometric, volume, and signalization inputs, interchange delay changes only as the offset between the two intersections is varied, as illustrated in Figure 6.

Figure 6 was developed from the volume data in Figure 1 with an assumed U-turn volume of 150 vehicles/h on both ramps, a 70-s cycle length and a 14-s travel time between the two intersections. Interchange delays were calculated by interchange phase code 1 (ABC:ABC). Delay is observed to drop to a minimum delay value at a 14-s offset and then to begin to rise beyond this minimum delay offset. Also shown in Figure 6 is the component of interchange delay occurring within the interchange. External delay remained constant.

Figure 7 shows the variation in maximum queue lengths that would occur on the interior left-turn and through lanes for the left-to-right (eastbound) arterial as a function of offset. Queue storage capacities, although unlimited in all our analyses, are important input constraints on the PASSER III program.

Figure 4. Development of diamond interchange phasing pattern from phasing ABC:ABC and offset.



INTERCHANGE PHASING ANALYSES

In addition to the four basic interchange phasing codes, a fifth one was studied. This code, 1A, represents a special case of the normal code 1 (lead-lead) interchange phase. The phase sequences are the same for code 1 (ABC:ABC), but the four external green times are calculated (see Equation 2) to total

$$C + 2 \cdot \text{travel time} \tag{4}$$

The popular four-phase with overlap signal phasing results if the offset between the two intersection signals is selected to be the same as the travel (overlap) time (3).

Figure 5. Traffic links connecting pair of adjacent signalized intersections.

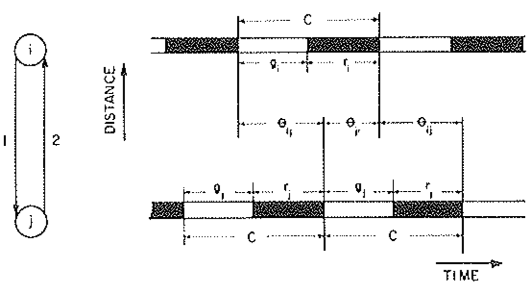


Figure 6. Variation in interchange delay with offset for phase code 1.

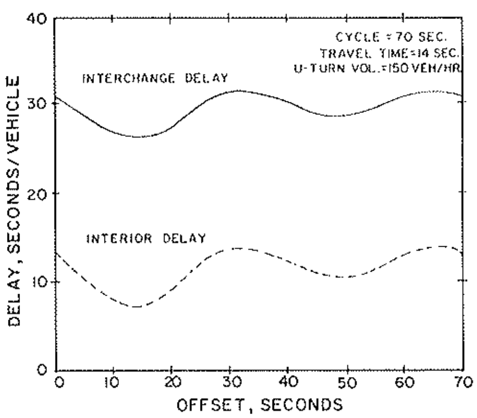
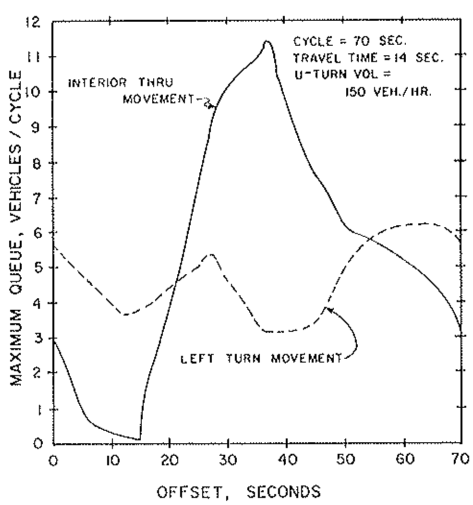


Figure 7. Variation in maximum queue length with offset for phase code 1.



The performance of four-phase with overlap can be determined in Figure 8 from the delay curve of interchange phase code 1A at an offset of 14 s, which, as might be expected, results in the minimum delay for this set of conditions. Other offsets increase the average interchange delay. It should be noted that the normal unimpeded travel time between the two intersections is assumed to remain constant at 14 s, regardless of the offset selected. In the real world, motorists may adjust their travel time slightly depending on the offset. If a queue forms on a movement in the interior of the interchange, a queue start-up delay or signal lost time is also assumed to occur (11).

A comparison of the performances of two different types of interchange phasing arrangements, 1A (ABC:ABC) and 4 (ACB:ACB), can be made from Figure 8. Minimum delay for code 4 occurs at 0- and 70-s offsets, which are the same because the cycle length is also 70 s. A 0-s offset for 4 results in a three-phase, lag-lag interchange signal phasing pattern. In his subjective review of diamond interchange signal phasing arrangements Munjal (3) concluded that there are two preferred sets of phasing patterns: four-phase with overlap and three-phase, lag-lag patterns. For this sample problem, the PASSER III outputs indicate that these two patterns would operate well. More important, however, is the fact that the phasing patterns that give minimum delay can be determined.

Although interchange phase codes 1 or 1A and 4 may be able to generate good operating conditions if the proper offset for each is selected, there are other basic interchange phasing arrangements that might provide even better results. Until all of these phase codes have been considered, the best interchange phasing pattern cannot be selected.

An example of the performance of all five interchange phasing codes is presented in Figure 9. For this problem, codes 1, 1A, and 4 will provide relatively good operation at their respective minimum delay offsets. Codes 2 and 3 do not perform as well as the others, and their performance curves, in the middle range of delay values, are not as responsive to differences in offset. A total of 350 different interchange timing plans were analyzed to generate the results presented in Figure 9. A manual analysis would not be practical, and a detailed microscopic simulation may not be economical.

These delay results tend to support the previously discussed general guidelines that the four-phase with overlap (lead-lead) and the lag-lag are the generally preferred signalization strategies. This general guide-

Figure 8. Performance of phase codes 1A and 4.

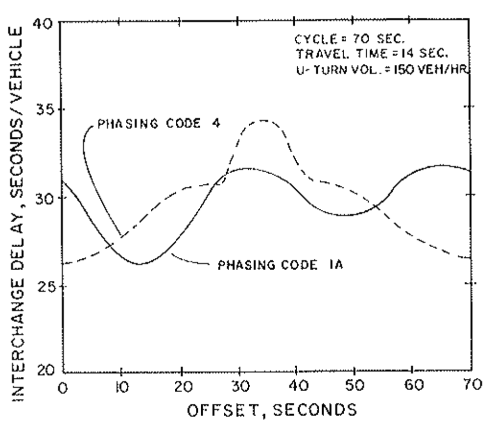


Figure 9. Variation in interchange delay with offset for five phase codes.

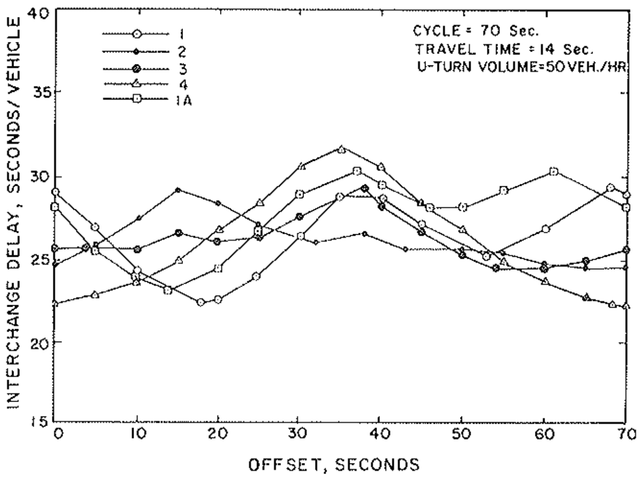


Table 1. Minimum delay for interchange and phase codes 1 and 1A for 50 vehicles/h U-turn volume.

Travel Time (s)	Cycle Length (s)	Minimum Interchange Delay (s/vehicle)		
		Optimum Phasing	Code 1	Code 1A
6	60	20.85	21.89	23.95
6	70	23.17	25.33	27.52
6	80	25.68	27.85	31.03
10	60	19.92	20.30	21.28
10	70	22.92	23.61	24.81
10	80	25.95	26.93	28.31
14	60	19.35	19.35	20.00
14	70	22.05	22.22	23.07
14	80	24.80	25.42	26.31

Table 2. Minimum delay for interchange and phase codes 1 and 1A for 150 vehicles/h U-turn volume.

Travel Time (s)	Cycle Length (s)	Minimum Interchange Delay (s/vehicle)		
		Optimum Phasing	Code 1	Code 1A
6	60	22.75	25.32	26.65
6	70	25.02	28.87	29.97
6	80	27.71	31.81	33.75
10	60	23.93	23.54	24.10
10	70	26.99	27.24	28.04
10	80	28.72	30.85	31.94
14	60	22.57	23.40	22.82
14	70	26.28	26.28	26.28
14	80	29.35	29.35	29.83

Table 3. Minimum delay phase codes for 18 interchange signalization problems.

Travel Time (s)	Cycle Length (s)	Phase Codes by U-Turn Volume	
		50 Vehicles/h	150 Vehicles/h
6	60	2, 3	2, 3
6	70	2, 3	2, 3
6	80	2, 3	2, 3
10	60	4	1
10	70	4	2, 3
10	80	4	2, 3
14	60	1	4
14	70	4	1, 1A
14	80	4	1

line may be useful, but it does not indicate which of the two is the better. As the following study results will show, the other phasing codes may operate better under a different set of conditions.

MINIMUM DELAY STUDIES

A number of geometric, signalization, and traffic flow studies will be presented to demonstrate PASSER III program features and to illustrate the need for a thorough investigation of available performance of design and signalization options. Delay performance curves similar to those in Figure 9 were developed for 18 basic signalization problems.

Throughout the studies, we used the interchange external volumes in Figure 1 and held them constant. These volumes result in exterior volume-capacity ratios of about 0.8. The turning movement variations within the interchange allowed ramp (frontage road) U-turn volumes of 50 and 150 vehicles/h. A U-turn volume in excess of 100 may be considered large (12). The three interchange spacings selected for study gave 6, 10, and 14-s running travel times between the two intersections. We thought this range would include most signalized diamond interchanges; we have successfully tested the program calculations against field data from an interchange having slightly higher travel times (2). Last, cycle lengths of 60, 70, and 80 s were analyzed. Five interchange phasing codes (1, 1A, 2, 3, and 4) were analyzed for all possible offsets in 1-s increments, and a minimum delay was then selected for each of the 18 problem sets.

Results

Minimum delay results for the 50 vehicles/h U-turn volume problems are presented in Table 1. Table 2 contains the minimum delay results for U-turn volumes of 150 vehicles/h and shows minimum interchange delay and minimum delays for phase codes 1 and 1A. The minimum delay offsets (not shown) for phase codes 1 and 1A in all cases would provide signal phasings in the four-phase with overlap family.

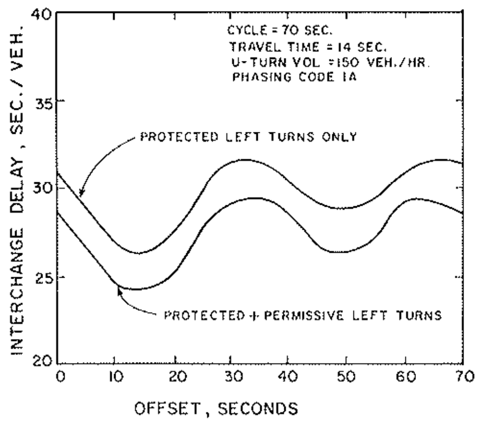
The minimum delay interchange phasing codes for all 18 signal problems studied are given in Table 3. Our most important finding was that every one of the possible interchange phasing codes produced a minimum delay solution in at least one of the 18 problems studied (determined from Table 3). As travel time increases (the distance between the ramps increases) from 6 to 14 s, the interchange phase code that provides minimum delay also changes.

Discussion

If the results of this study are as descriptive of the real world as we believe them to be, the varying opinions traffic engineers express concerning the relative merits of different diamond interchange phasing schemes seem to have been justified. For example, a four-phase can be better than a three-phase scheme in some cases; in other cases three phases are better than four. However, another phasing pattern may be better than either three or four.

We believe PASSER III removes the guesswork from selecting the best minimum delay signal phasing pattern at a pretimed diamond interchange. A total of 6300 interchange phasing options were analyzed to find the minimum delay phasing codes (shown in Table 3) and their respective interchange phasing patterns. This analysis was done at a total computer cost of $25 on the local university computer system running on the lowest

Figure 10. Reduction in interchange delay from addition of permissive left turns.



computer job priority level. A higher priority run (8 a.m. to 5 p.m.) would have cost only $100. The PASSER III program will automatically select the best interchange phasing pattern.

Some of the literature might be interpreted to suggest that four-phase with overlap signalization has unusual advantages over other types. It is true that it does have some good features, for example, arterial progression, but no diamond interchange signal phasing pattern has mystical powers, not even four-phase with overlap. It is simply a lead-lead phasing arrangement that is timed to perfect progression for the front of the two arterial through platoons. As intersection spacing and travel time increase, the green times on the external movements at both intersections must be increased to maintain the perfect progression of the arterial through movements. This increase will result in an obvious increase in external signal capacity. Increasing external green times reduces the signal capacity and green times of the interior left-turn phases. In the standard lead-lead phasing arrangement (code 1), greens are split at the two intersections in proportion to the volumes at each intersection. Increasing the spacing does not change the green split, but progression may not be as good. As the previous results show, it is difficult to estimate what net effects these features will have on total average interchange delay.

PERMISSIVE LEFT TURNS

A number of states have begun using the protected left-turn phase at signalized intersections (left-turn arrow) followed by a permissive left-turn phase (left turn legal on circular green if clear) in order to increase high-volume intersection capacity. Texas has also begun using the protected-plus-permissive left-turn phasing at some critical diamond interchanges. This type of control effectively provides some left-turn capacity on the arterial through phase (Figure 2, phase A). This type of signalization may completely change preferred phasing patterns engineers are accustomed to using and may also change the minimum delay interchange phasing patterns for a given signalization problem (Table 3).

The PASSER III computer program can analyze the protected-plus-permissive phasing concept in either the leading or lagging phase sequence. The effects of opposing queues and traffic flow are considered. A mathematical model of this process has been developed by Fambro, Messer, and Anderson in a paper in this Record.

An example of the reduction in overall interchange delay that would occur if a permissive left-turn phase were added to phase A at both ramp intersections can be determined from Figure 10. In this case, an overall reduction in delay of approximately 2 s/vehicle would result. A much higher reduction in delay for the interior left-turn vehicles, where the capacity is increased, occurs when maximum queue lengths are shortened.

SUMMARY

The results of this study show that the best minimum delay, pretimed diamond interchange signal phasing pattern can be estimated using PASSER III. While signalization guidelines and preferred signal phasing patterns are helpful, their usefulness is limited and their performance uncertain. A detailed analysis of all pretimed signalization options can now be performed efficiently. We hope that at least some of the issues that have clouded diamond interchange signalization can now be analyzed.

ACKNOWLEDGMENTS

The research reported in this paper was performed during a project on the development of a frontage road level of service evaluation program (PASSER III) conducted by the Texas Transportation Institute and sponsored by the Texas State Department of Highways and Public Transportation in cooperation with the Federal Highway Administration, U.S. Department of Transportation. We wish to thank Herman E. Haenel and Elmer A. Koeppe of the Texas State Department of Highways and Public Transportation for their technical inputs and constructive suggestions throughout the project. The contents of this paper reflect our views, and we only are responsible for the facts and accuracy of the data presented. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This paper does not constitute a standard, specification, or regulation.

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Virginia's Crash Program to Reduce Wrong-Way Driving

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Over a 4-year period beginning in 1970, wrong-way incidents and accidents were reduced on Virginia's interstate highways by 50 percent and on noninterstate four-lane divided highways by 70 percent. However, since 1975 an upward trend has been observed on interstate roads, while the downward trend has continued on noninterstate roads. This paper discusses the following engineering measures taken to reduce wrong-way driving: using reflectorized pavement arrows on ramps, eliminating pavement flares, providing stop lines across exit ramps near junctions with crossroads, continuing the pavement edge line across exit ramps, continuing double yellow lines on two-lane divided crossroads opposite exit ramps, reducing crossover width across exit ramps, adding guidance to local drivers on new interchanges, informing the driver of the geometry of the intersection before he or she enters it, and providing guidance for drivers at T-intersections without a crossover.

A survey of wrong-way incidents by the Virginia Department of Highways and Transportation and the Virginia State Police was initiated in June 1970 and has continued since, except for December 1970 to June 1971. The data collected show that until June 30, 1976, a total of 114 wrong-way accidents involving 54 deaths and 120 injuries had occurred on interstate highways. In the 167 accidents reported on other four-lane divided highways during the same period, 33 were killed and 173 injured.

Fatalities and injuries caused by wrong-way driving on interstate highways and four-lane divided highways in Virginia were compared with total accident fatalities and injuries on major highways in the state during 1970 to 1976. This comparison showed that although wrong-way accidents were relatively few compared to the total number of accidents they were exceptionally severe. The data showed that the fatality rate per wrong-way accident was 31 times greater than that for other types of accidents on interstate highways and 10 times that for other types on four-lane divided highways. The data also showed that the injury rate was 2.9 times that for other types of accidents on interstate highways and 2.3 times the rate for these on four-lane divided highways.

However, as shown below, the wrong-way incidents and accidents could not be related to the total accidents on a statewide basis for interstate and other four-lane divided highways. Table 1 gives the vehicle kilometers, total accidents, wrong-way incidents, and wrong-way accidents for each calendar year since 1970 for interstate, arterial, and primary highways.

These data show that on interstates the total number of accidents during 1972 was 1947 billion vehicle kilometers (1515/billion vehicle/miles) of travel. In 1973 the total dropped to 868 (1389), a decrease of 8.3 percent from 1972, which was possibly accomplished by the legislation effective in June 1972 that reduced the blood alcohol content (BAC) level from 0.15 percent to 0.10 percent for a presumption of drunk driving and stipulated a mandatory revocation of the driver's license for a period of 6 months for persons convicted of driving while intoxicated (DWI). Later, in December 1972, breath tests were introduced to make conviction for drunk driving easier.

On interstate roads in 1973 enforcement of these regulations sharply cut the total anticipated accident rate. In 1974 on the interstate highways the total number of accidents decreased to 639 billion vehicle kilometers (1022/billion vehicle/miles), a reduction of 26.4 percent from 1973. This shift might have been caused largely by the energy crisis of 1973 to 1974 and its accompanying reduction in the speed limit to 88.5 km/h (55 mph).

As shown in Figure 1, there was a considerable dip in wrong-way incidents and accidents at the beginning of 1973, possibly because of fear of DWI conviction. Later in 1973 the trend reversed and did not seem to be affected by the new legislation and reduced speed limit. The 26.4 percent reduction in total accidents during 1974 was apparently not reflected in the figures for wrong-way incidents and accidents (Figure 1 and Table 1).

From 1970 to 1973, when the total travel and accidents were increasing, incidents and accidents either remained constant or decreased. Since 1974, when total travel and accidents again increased, wrong-way incidents and accidents also tended to increase, a reversal of the relationship between total accidents and wrong-way incidents and accidents. Therefore there is no apparent relationship between vehicle kilometers of travel or total accidents and wrong-way incidents or accidents on interstate highways.

On arterial and primary highways total travel increased until 1973. In 1974 it decreased, probably be-