A REAL-TIME FRONTAGE ROAD PROGRESSION ANALYSIS AND CONTROL STRATEGY

Carroll J. Messer, Robert H. Whitson, and J. D. Carvell, Jr.,
Texas Transportation Institute, Texas A&M University

Urban freeways frequently experience congestion due to normal peak-hour demands exceeding capacity and due to freeway incidents. It is proposed that, at least during these conditions, the adjacent frontage roads should be operated as major arterials to provide additional freeway capacity. A real-time, traffic-responsive frontage road progression analysis and control strategy that could be used in operating the frontage roads as a major traffic-carrying facility is presented. Previous computer control applications and future implementation of the strategy are discussed. The frontage roads are analyzed for progression as if the continuous, one-way frontage roads and diamond interchanges were combined to form a major two-way signalized arterial. To maximize frontage road progression, each interchange is assumed to operate on either a 3-phase or a 4-phase signal sequence. The progression optimization algorithm selects the phase sequence yielding the maximum progression. The traffic-responsive strategy using the 3- and 4-phase signal sequences is also described.

• TRAFFIC control theory and control systems have made significant advances in recent years. This progress is due in part to many research and operating agencies' working toward the common goal of improving the level of service provided the motoring public. As often occurs, progress brings change and, in fact, large changes may be required before any progress can occur.

Noticeable changes have occurred in traffic control concepts as well as in hardware implementation. The implementation of freeway ramp control systems to improve operations had modified, if not changed, the initially accepted view that freeways should be free of traffic signals. The beginning of an apparent widespread application of digital computers in traffic control has been noteworthy. As a result, significant changes have occurred in both method and mode of control.

Even though new traffic control technology and digital computers have been applied to freeways, the generally accepted view has remained that freeways should function as a prime mover of persons and goods. The land access or service function is still to be provided by other facilities, such as by continuous frontage roads when they are available. This is the generally accepted role for frontage roads where the freeway is not operating at or near capacity.

In many urban areas freeways operate during rush hours at or near capacity because of high traffic demands, and as a result traffic congestion frequently exists. The occurrence of an accident or stalled vehicle on the freeway will also cause considerable congestion and delay during many hours of the working day. When these types of freeway congestion occur, more on-freeway or near-freeway capacity would be helpful in reducing congestion and environmental pollution.

Frontage roads appear to offer considerable potential for relieving a significant amount of freeway congestion by increasing the use of the frontage roads by freeway motorists. However, to reach this objective would require that the frontage road operate, at times, like a major arterial and not in its traditional role as an access facil-

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ity. This unique dual role of operation is shown in the traffic movement versus access curve in Figure 1. The plot of the normal frontage road function would fall between the local and collector functions of traffic facilities: i.e., a high level of access in contrast to a low level of desired traffic movement. This high access point would describe the appropriate function for the frontage roads when the freeway is operating at a high level of service. However, when the freeway flow begins to experience congestion due to excessive demand or due to an incident, the frontage road should function as an alternate freeway route. During this time, the function of the frontage road would lie between the freeway and major arterial functions, as shown in Figure 1.

For the frontage road to be able to provide for the high-movement operation, it must be designed and operated to carry satisfactorily large traffic volumes at an acceptable level of service. The design should provide for one-way continuous frontage roads having 3 lanes in each direction of flow. The inside lane should be used for weaving with the freeway and be appropriately marked. The other two lanes should be free to move traffic, and parking should not be permitted during rush hours. Frontage road intersections should be of high-type design and have U-turn bays. From an operational viewpoint, the frontage road intersections (normally diamond interchanges) should be signalized, coordinated to provide progression along the frontage roads, and operated in a traffic-responsive mode to minimize delay and also provide an acceptable operating speed.

SCOPE

To provide the necessary high level of service for the frontage road traffic, a traffic-responsive signal control strategy that provides progression along the frontage roads is needed. This paper describes the theory and application of such a strategy. A one-way pair of frontage roads, as shown in Figure 2, is analyzed to find the best progression along both frontage roads while computing the green splits at each interchange in an effective, traffic-responsive manner considering all traffic using the interchange.

This control strategy has been developed within the Dallas freeway corridor research project conducted by the Texas Transportation Institute for the Federal Highway Administration in cooperation with the Texas Highway Department and the city of Dallas.

INTERCHANGE OPERATIONS

The traffic control strategy requires flexibility in signal operations. The traffic-responsive control strategy, while not actuated, requires that different signal phase sequences be implemented. Computer control at the diamond interchanges would probably be necessary. Two basic diamond interchange signal phasing schemes are considered for possible use at each interchange. These are the 3-phase with variable sequences and the 4-phase with overlaps. Progression analysis, to be described later, will determine which of these two basic phasing schemes should be used at each interchange so that maximum progression is obtained. It is assumed that traffic sensors are located on all approaches to each interchange such that demand volume counts are available for all movements in a real-time environment (e.g., 6-minute volume counts).

The length of green time given to an external approach movement to the interchange is determined, to the extent possible in all phasing schemes, in direct ratio to the movement's demand-to-capacity ratio. That is,

$$g_i \geq \frac{D_i}{S_i} C + L_i; \; g_i \geq M_i \tag{1}$$

where g_i is the green time used for movement i, D_i is the real-time traffic demand on movement i, S_i is the saturation (capacity) flow in vehicles per hour of green, C is the cycle length, and L_i is the total queue and amber lost time. It should be noted that g_i includes the amber time and must equal or exceed predetermined minimum movement times, M_i.

Three-Phase Variable Sequence

The basic 3-phase signal phase sequence is shown in the left section of Figure 3. The sequence begins (from the top) with both frontage roads receiving the green, followed by the two through-movement phases from the interchanging cross street. With this basic 3-phase sequence, the two frontage roads receive the same amount of green time. Therefore, this phasing arrangement is considered satisfactory only when both frontage road volumes are approximately the same. This would usually not be the case for the type of operation envisioned.

The basic 3-phase arrangement can be modified to produce phasing splits that are more responsive to volume variations on the two frontage roads. In order to favor the larger frontage road volume or to provide green times to the frontage roads in proportion to their demand volumes, two additional "3-phase" phasing sequences are used. These sequences are also shown in Figure 3. The phasing sequence that favors the "west-side" frontage road simply inserts an additional west-side frontage road phase into the basic 3-phase sequence.

The phasing arrangement used for favoring the "east-side" frontage road is slightly more complex. As in the previous sequence, an additional phase for providing more green time to the east-side frontage road is added just after the simultaneous frontage roads phase, as shown in the right section of Figure 3. However, the two major cross-street through-movement phases are reversed in this latter phase sequence. Reversing the order of the two through-movement phases provides smoother flow through the interchange and avoids short left-turning movements within the interchange. This variation in phase sequence can be effected with present computer control technology. To summarize the three phasing arrangements previously described, if the basic 3-phase sequence consists of phases A•B•C, then the favor-west-side sequence would be A•A1•B•C and the favor-east-side sequence would be A•A2•C•B. The appropriate sequence is automatically selected based on the level and distribution of frontage road traffic volumes.

Other considerations are necessary in 3-phase operation to promote smooth and orderly flow through the interchange. Traffic blockages of movements following the simultaneous frontage road phase may arise within the interchange area because of the simultaneous movement and storage of the conflicting left-turning movements from the frontage roads. The 3-phase sequence is particularly susceptible to this problem where the internal storage for left-turning vehicles within the interchange is small and left-turning volumes are high.

The following guidelines are offered to minimize the potential for blockages occurring because of simultaneous frontage road movements. No blockage problems are likely to occur until the smaller frontage road left-turning movement volume level reaches

$$q_L = 100 \frac{W}{C} \tag{2}$$

where q_L is the smaller frontage road volume in vehicles per hour, W is the available storage length in feet for vehicles within the interchange, and C is the cycle length in seconds. A 24-ft storage distance per vehicle and a peak flow rate factor of 1.5 were assumed. Thus, if the interchange storage distance were 120 ft and the cycle length 80 seconds, the critical left-turning volume, q_L, would be 150 vehicles per hour. By reducing the cycle length to 60 seconds, the critical volume level could be increased to 200 vehicles per hour. If the left-turning volume exceeds q_L, then 2 lanes for left-turning or a different signal phasing sequence should be considered.

When the critical volume level is reached, the maximum simultaneous frontage road phase, A_{max}, in seconds should not exceed

$$A_{max} = 4.0 + 0.09\ W \div T \tag{3}$$

where W is the interchange storage length and T is the decimal fraction of the inside frontage road lane volume turning left. A 2.1-second average vehicle headway was assumed. Thus, if W were 120 ft and T were 0.9, then A_{max} would be 16.0 seconds.

Four-Phase Overlap

The other basic phasing arrangement considered at each interchange is the 4-phase overlap operation (1, 2). Although this phasing scheme is widely used, few publications are available that describe strategies that could be used for real-time control (3, 4). The basic 4-phase with overlap operation is shown in Figure 4. The lengths of the offsets, or overlaps (ϕ_4 and ϕ_8 in Figure 4), depend primarily on the travel times from one frontage road to the other. Usually, the offsets are the same length but may be different, to reflect grades, locations of stop lines, etc. Phases 2 and 5 are the overlap phases. Movements 1, 4, 5, and 8 are used to compute the phase associated with each movement. Minimum movement times also must be satisfied for each movement.

While operating in a progressive system, the cycle length, C, at each intersection must be the same throughout the system. To generate this cycle at each interchange having overlaps ϕ_4 and ϕ_8, the following green (green plus amber) movement requirements must be satisfied:

$$g_1 + g_3 + g_4 = C \tag{4}$$

$$g_5 + g_7 + g_8 = C \tag{5}$$

$$g_3 + g_7 = C - \phi_4 - \phi_8 \tag{6}$$

where the subscripts of the green movements refer to the movement numbers shown in Figure 4, C is the cycle length, and ϕ_4 and ϕ_8 are the eastbound and westbound offsets respectively. Equations 4 and 5 reflect the requirement that the sum of the conflicting green times at each intersection must add to one cycle. Equation 6 describes the overlap operational requirement and links the two intersections together to operate as an interchange. As indicated, the sum of internal left-turn greens must equal a constant value for a given cycle since the overlaps are fixed.

The green times provided within the interchange cannot be established independently at each intersection because of the requirements placed by Eq. 6 on the two internal left-turn green times. Since the sum of these two green times is predetermined, they must be proportioned so that the time remaining within the cycle at each intersection for moving traffic into the interchange is in proportion to the green time needed at both intersections. This is accomplished by computing the east-side left-turn green, g_7, from

$$g_7 = \frac{P_1 + P_4}{P_1 + P_4 + P_5 + P_8} \cdot [C - \phi_4 - \phi_8] \tag{7}$$

where P_4 is the demand/capacity ratio of movement 4, etc. This green time is computed before any other time within the interchange. Equation 6 is then solved for the other internal left-turn green, g_3. After the internal left-turn greens are computed, the portion of the cycle remaining at each intersection, as given by Eq. 4 or 5, is allocated to the other two movements in proportion to their respective demand-to-capacity ratios using Eq. 1.

The following example is presented to illustrate the interdependency of the interchange equations and their operational characteristics. Assume that the west-side frontage road demand (movement 1) increases while all others remain the same. The desired increase in the green time of g_1 would be provided in the following manner: Since the demand-to-capacity ratio, P_1, would increase, the east-side internal left-turn green, g_7, as computed from Eq. 7, would be larger. It follows from Eq. 6 that the west-side left-turn green, g_3, would be smaller than before, which provides in itself additional green time for the west-side frontage road (movement 1).

The left-turn green time computed from Eq. 7, g_7, must fall within the bounds

$$M_7 \le g_7 \le C - M_5 - M_8 \tag{8}$$

Figure 1. General movement and access functional relationships.

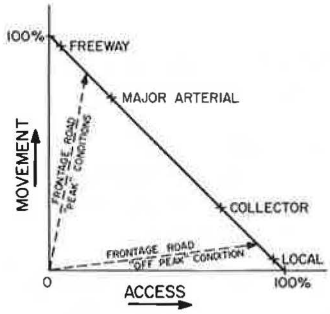


Figure 2. A frontage road progression analysis and control area.

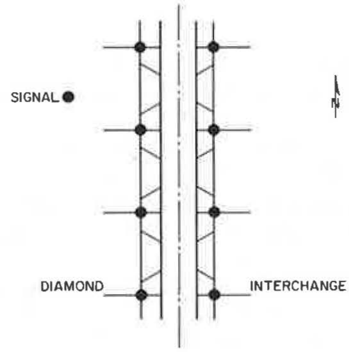


Figure 3. Three types of 3-phase signal sequences.

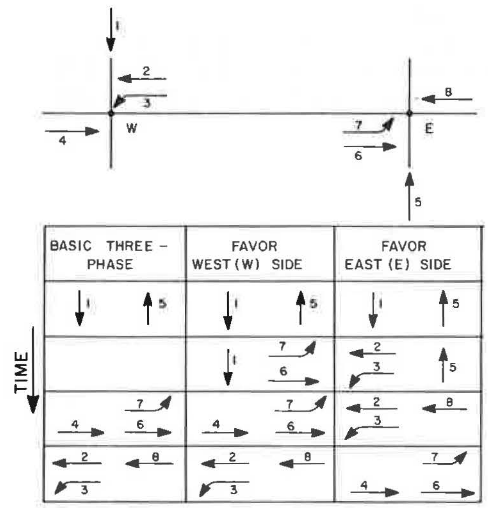


Figure 4. Phasing and interval lengths for 4-phase overlap operation.

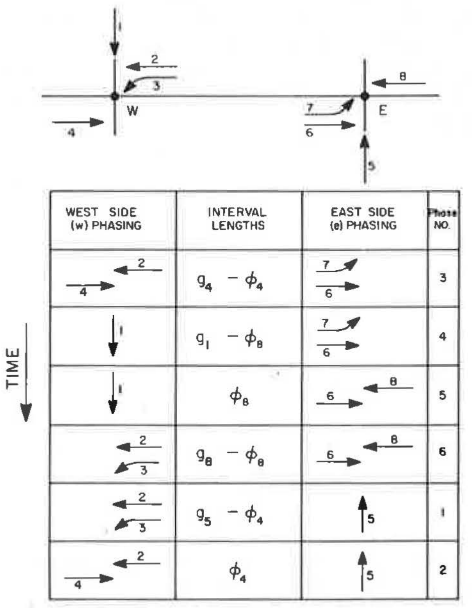
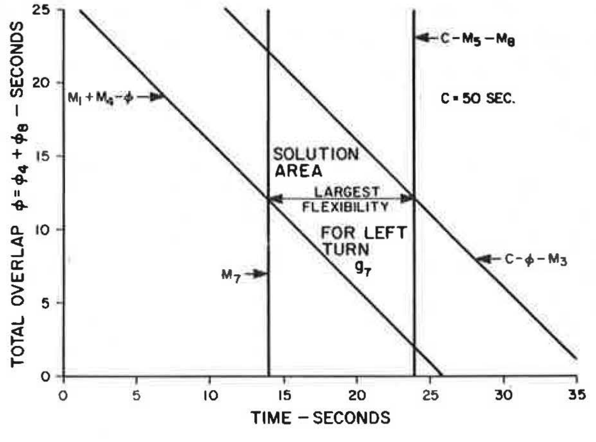


Figure 5. Overlap and minimum green time relationships in 4-phase operations.



and

$$M_1 + M_4 - \phi \le g_7 \le C - \phi - M_3 \tag{9}$$

to ensure that adequate time is available for the remaining movements at the two intersections after the left turn at each intersection is computed. Minimum-movement greens are given by M_i and $\phi = \phi_4 + \phi_8$.

Figure 5 shows the allowable range of overlaps for the 4-phase scheme when a relatively short cycle length of 50 seconds is used. The allowable range of overlaps is defined by the upper and lower limits of the solution area for the left-turning movement, g_7. Minimum greens (M_i in Figure 5) of 12 seconds are assumed for the frontage roads and 14 seconds for all other movements. For the minimum values chosen, most normal diamond interchanges can operate at a 50-second cycle since the overlaps will be from 5 to 10 seconds in each direction for a total overlap of 10 to 20 seconds.

Two other important items are evident from Figure 5. First, minimum greens should not be selected without knowing their effects on signal operation. If the minimum greens are large and the cycle length is short, a condition may arise where it is not possible to compute satisfactory movement lengths for the interchange. Second, there exists an optimal overlap that gives the greatest variation or flexibility in signal phase allocation for a given set of minimum greens. In Figure 5, this optimum total overlap is 12 seconds, or an overlap of 6 seconds in each direction.

The converse point of view is also relevant. For a given interchange with a fixed total offset ($\phi = \phi_4 + \phi_8$ in Figure 5) and symmetrical minimum greens, there exists an optimal combination of minimum greens for maximum phase flexibility. That is, from Figure 5,

$$C - M_5 - M_8 = C - \phi - M_3 \tag{10}$$

yields

$$M_5 + M_8 - M_3 = \phi_{opt} \tag{11}$$

and

$$M_1 + M_4 - \phi = M_7 \tag{12}$$

yields

$$M_1 + M_4 - M_7 = \phi_{opt} \tag{13}$$

For the design under consideration, the minimum greens M_1 and M_5 equal 12 seconds, and M_3, M_4, M_7, and M_8 equal 14 seconds. As a consequence, Eqs. 11 and 13 are equivalent. Thus $\phi_{opt} = 12 + 14 - 14 = 12$ seconds. The optimal phase flexibility location is independent of cycle length, although increasing the cycle length increases the allowable solution area and range of feasible overlaps. However, increasing the cycle length increases the sum of the two internal left turns, from Eq. 6, which may cause unsatisfactory operation by reducing external movement capacities.

Progression Optimization

The progression that is maximized is the sum of the progression bands along both frontage roads. The two one-way frontage roads are analyzed as if they were combined to form a single two-way arterial street with the interchange considered to be an intersection having multiphase variable-sequence signal operation. The progression optimization theory used is described in detail in a previous publication (5) on progression optimization for multiphase variable-sequence signals on arterial streets. Only the concepts necessary for converting the arterial progression theory to analyze the frontage road progression analysis problem will be described.

It is assumed that each interchange in a frontage road progressive system can use either the variable 3-phase or the 4-phase overlap signal operation. The progression program will select one of these two types of operation for each interchange such that the total progression along both frontage roads is maximized. Thus, some interchanges may use 3-phase operation whereas others use 4-phase. As traffic conditions change, an interchange may switch from one type of signal phase operation to the other.

The main reason for considering both the 3-phase and 4-phase operation is that usually one or the other will give good frontage road progression. If only one were available, progression might not be possible. This occurs because of the differences in starting times of the green signal for the two frontage road movements. As used in the arterial progression program (5), these differences in starting time of the progressive through movements are called the relative offsets, r_{ij}, of progressive movement j with respect to progressive movement i, with elapsed time being positive.

As shown in Figure 6, the relative offset, r_{15}, for the 3-phase operation is zero. The frontage road green times are shaded to indicate that they are the progressive through movements. From Figure 3 it can be observed that the relative offset of the frontage road greens for 3-phase operation is zero regardless of the phase variation used. That is, both frontage road greens begin at the same time in all three cases.

The relative offset of the frontage road greens for the 4-phase overlap operation is also shown in Figure 6 and is shown to have a value of about one-half cycle, which is in the normal range of values. By referring to Figure 4, it can be shown that the relative offset of movement 5 with respect to movement 1, r_{15}, is given by

$$r_{15} = g_1 + g_8 - \phi_8 \tag{14}$$

Assuming representative values for g_1 of 16 seconds, g_8 of 22 seconds, and ϕ_8 of 8 seconds, then the offset r_{15} would equal 30 seconds, or about one-half of a normal cycle length.

EXAMPLE PROBLEM

An example frontage road progression problem was analyzed to illustrate the operation of the program. Four interchanges were assumed to exist in the frontage road progressive system, and 3-phase or 4-phase overlap operation was assumed possible at each interchange. Traffic and geometric data were assumed. Interchange and progression speed data were as given in Table 1. Cycle lengths from 50 to 70 seconds were evaluated in 1-second increments to find the best possible progression.

The results of the progression analysis revealed that the most efficient (5) progression exists at a 60-second cycle length, as shown in Figure 7. The optimal efficiency was found to be 20 percent; i.e., 20 percent of the cycle is available for progression along the frontage roads. However, as also shown in Figure 7, the attainability (5) of the progression solution is 100 percent; i.e., the progression bands are limited only by the size of the green phases and cannot be improved unless the frontage road green times are enlarged.

Table 1 also shows the optimal signal phase sequence selected for each interchange for the given conditions. Three-phase operation with the east-side frontage road being favored was used at interchange No. 1, 3-phase operation with only simultaneous frontage road greens at interchange No. 2, 4-phase with overlaps at interchange No. 3, and 3-phase with the east-side frontage road favored at interchange No. 4. The optimal progression offsets are also given in Table 1.

The optimal progression time-space diagram is shown in Figure 8. Note in Figure 8 the differences in the location of the frontage road greens. As expected, the three, 3-phase sequences have their greens starting at the same time. The 4-phase overlap operation used at interchange No. 3 has a relative offset of about one-half cycle between the start of the frontage road greens. It can be observed from the time-space diagram that progression would not have been possible if all of the interchanges had been forced to use only 3-phase operation. Further analysis has revealed that the same is true if all interchanges had to use only 4-phase overlap operations. From a

Figure 6. Locations of frontage road progressive movements in 3- and 4-phase sequences.

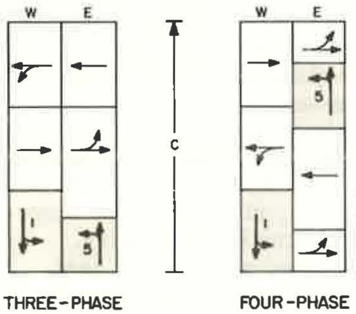


Table 1. Progression results for example problem at optimal cycle length of 60 seconds.

Interchange Number	Spacing	Speed	Optimal Phasing	Optimal Offset (seconds)
1	—	40 fps (12.2 m/s)	3φ - E	0
2	1,200 ft (368 m)	40 fps (12.2 m/s)	3φ - S	30
3	1,800 ft (550 m)	40 fps (12.2 m/s)	4φ	15
4	600 ft (183 m)	40 fps (12.2 m/s)	3φ - E	30

Figure 7. Progression efficiency and attainability variation with cycle length.

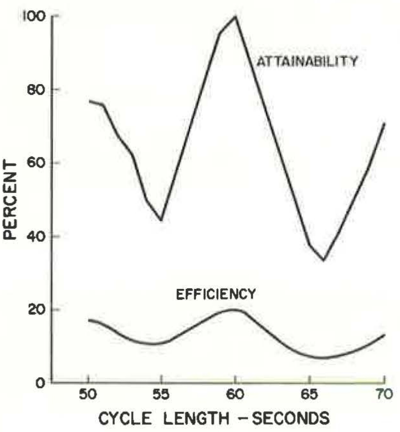
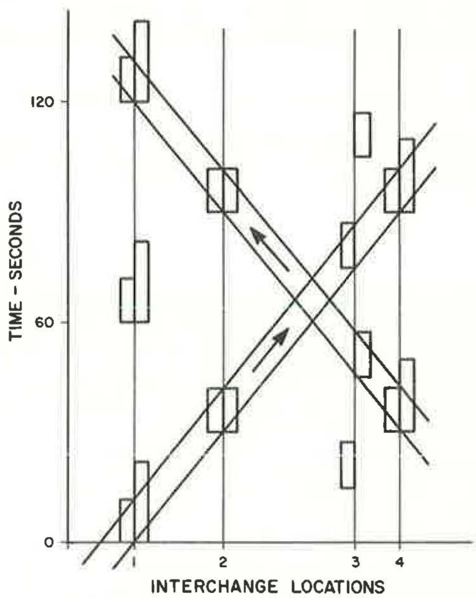


Figure 8. Optimal frontage roads progression solution.



progression point of view, this fact illustrates the need for having more than one type of phasing possible at an interchange.

IMPLEMENTATION

This frontage road progression analysis and control strategy is planned for implementation, testing, and evaluation within the Dallas corridor research project. Computer control will be provided at 15 interchanges in 3 subsystems. All aspects of the control strategy previously described will be used in this computer control system.

Previous real-time computer control using sections of this control strategy indicate that the overall frontage road control strategy presented should be effective. The real-time progression strategy was used on an arterial computer control system in Dallas (5). Real-time diamond-interchange computer control using 4-phase overlap phasing has been operated in both Dallas (5) and Houston (6), with the latter also providing one-way frontage road progression through two diamond interchanges. All of these previous real-time computer control systems have been successful.

ACKNOWLEDGMENT

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The contents of the paper reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect views or policies of the Federal Highway Administration. This paper does not constitute a standard, specification, or regulation.

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DISCUSSION

Joseph M. McDermott, Illinois Department of Transportation

The concept of a traffic-responsive signal control strategy providing progression along freeway frontage roads recognizes the important role of frontage roads as part of urban transportation corridors. Continuous, one-way frontage roads integrated into signalized, coordinated diamond interchanges offer the highest level of efficiency, capacity, and operational flexibility for handling urban freeway overloads and for distributing interchange traffic.

Unfortunately, the implementation of frontage road control strategies will often be limited to subsystems defined by each frontage road discontinuity, since many cities do not enjoy continuous routes. In the Chicago area, for example, over 70 percent of the existing expressway mileage lacks frontage roads. However, one of the Chicago area "subsystems" illustrates the interplay between freeway and frontage road and points out some of the operational variables that should be considered as part of the overall control strategy.

Figure 9 shows a section of the Dan Ryan Expressway. The inbound 4-lane expressway roadway expands to 6 lanes downstream. A 3.5-mile stretch (95th Street to 67th Street) of continuous, one-way frontage road (State Street) has 11 signalized intersections, feeds 6 metered entrance ramps, and empties 5 exit ramps. The city operates the fixed-time frontage road signals to provide progression for the inbound morning rush period. The frontage roads are discontinuous at either end of the subsystem, due to changes in horizontal expressway alignment as well as railroad grade separations and other physical constraints.

In 1966 the Illinois Department of Transportation initiated ramp metering inbound to alleviate freeway congestion caused by overloading near the last inbound entrance merge prior to the frontage road discontinuity (7). Over 1,300 vehicles had been using this one entrance in the morning peak hour. A comparison of travel times on the frontage road and the expressway (Figure 10) showed that the quickest inbound route during normal expressway operations included use of the frontage road followed by expressway entry at the last entrance ramp (71st Street). A study of these ramp users showed that 15 percent had previously been on the freeway and had exited to bypass the congestion. Many other drivers, although not previously on the freeway, bypassed upstream entrance ramps to similarly reduce travel time. The net effect of having too much traffic entering at one ramp was prolongation of the freeway congestion, causing more traffic to use the last ramp, etc., etc.

Ramp metering cut the ramp volume down to about 700 vph, reduced expressway congestion, and improved both freeway and frontage road through-travel times, all by delaying and diverting entrance-ramp users. The experience demonstrates the potential of frontage road progression for handling through bypass traffic as well as the imbalances that can result when the freeway problem is not internal to the frontage road bypass.

The importance of locating freeway incidents as part of the frontage road control strategy should not be overlooked. It may be advantageous to have traffic-responsive control only where needed for incident bypass and not along the whole corridor. It also may be advantageous, under some conditions such as complete freeway blockages, to have capability for extended progression on only one frontage road.

There are other operational variables affecting applications of the control strategy that must be considered. Some of these, such as pedestrian signals, could force longer cycle lengths and reduce progression flexibility. It is common in the Chicago area, for example, to have considerable pedestrian traffic at diamond interchanges, as well as bus stops on internal diamond approaches, to serve rail-transit stations located in freeway median strips. Other important variables include the presence or lack of U-turn bays and left-turn pockets, variable numbers of lanes, ramp metering queues, and parking controls.

The authors are to be complimented for their work thus far. The implementation, testing, and evaluation proposed will determine if the strategies can be tailored to fit day-to-day operational situations. As part of an overall corridor control system, one can envision a freeway surveillance and control system interfaced with traffic-responsive alternate routes and integrated with on-freeway and off-freeway driver information systems.

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7. Fonda, Roy D. An Analysis of Short-Term Implementation of Ramp Control on the Dan Ryan Expressway. Chicago Area Expressway Surveillance Project, Rept. 20, Illinois Department of Transportation, 1968. Abridged in Highway Research Record 279, 1969, p. 157.

Figure 9. Dan Ryan Expressway interchanges, 95th Street to 63rd Street.

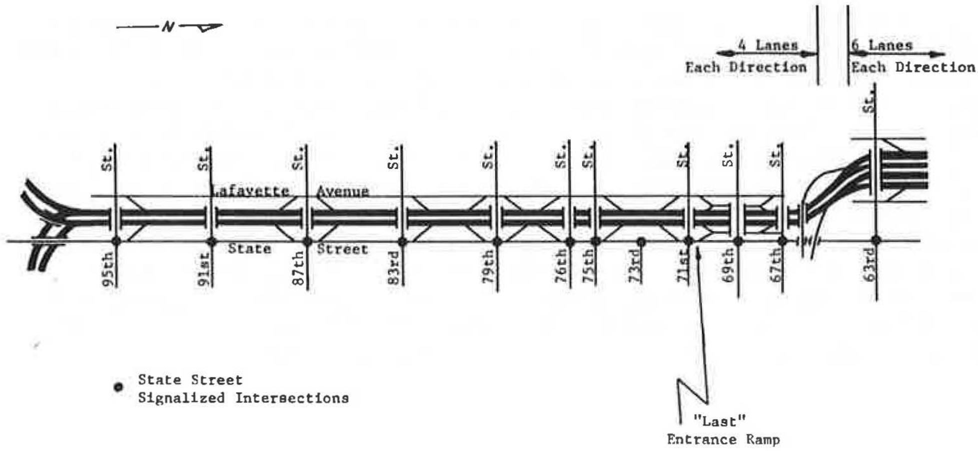
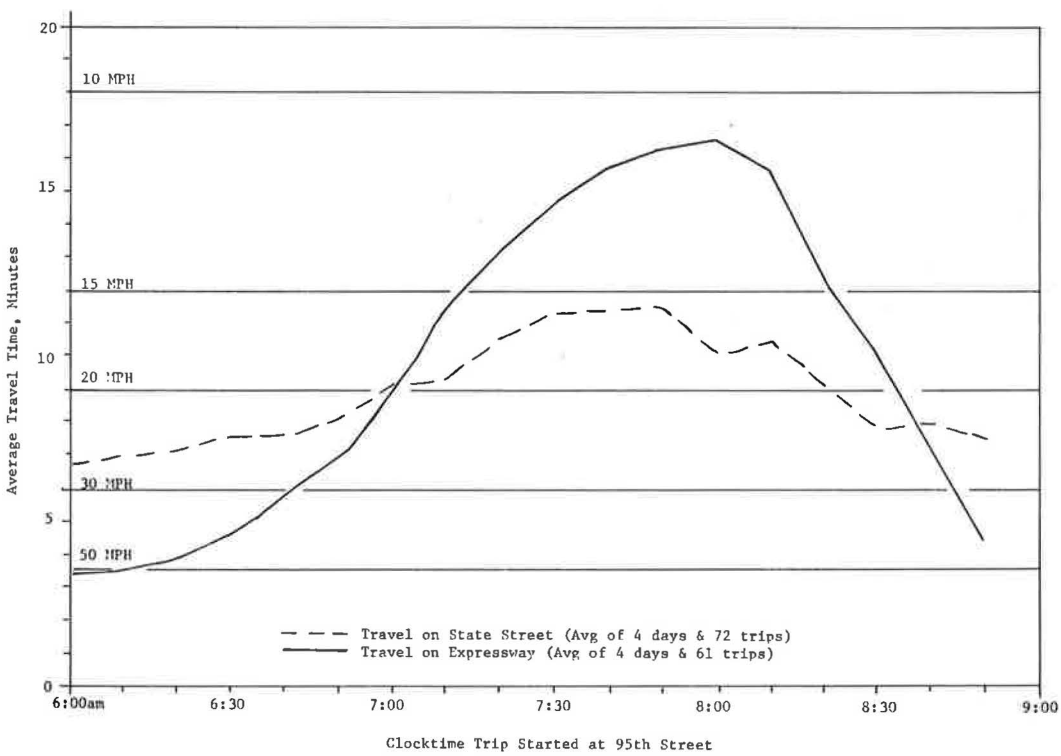


Figure 10. Individual trip travel times (before ramp metering), 95th Street to 71st Street, 3.0 miles.



AUTHORS' CLOSURE

The authors wish to express their appreciation to McDermott. His comments are constructive and informative and provide a meaningful addition to the paper. We would like to take this opportunity to add a few closing remarks to this discussion.

McDermott has presented a freeway-frontage road subsystem in Chicago to which the frontage road control strategy presented in this paper could be applied. Problems of freeway ramp control just upstream of a discontinuous frontage road were noted. Several frontage road discontinuties along the Gulf Freeway in Houston are now being eliminated in recognition of the rising importance of frontage road utilization.

McDermott's summary statement also expresses our position that what is really desired as a future goal is to develop an urban freeway corridor management and control system wherein the freeway, frontage roads, and adjacent arterials are operated as a system to provide the maximum possible utilization of these facilities. We hope that this paper has contributed, in some way, toward meeting this goal.