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Research Project: Develop Real-Time, Multimodal, Traffic Adaptive Control System

16. Abstract

This project evaluated alternative control sequences and settings for the actuated, three-phase diamond interchange. The settings evaluated include the minimum green interval, maximum green interval, and passage time. The objective of this project was to develop guidelines for establishing controller settings that would generally yield low-delay (if not lowest) operation for the range of volumes encountered during a typical weekday. This objective was achieved through a combination of theoretical analysis and an examination of diamond interchange phasing and traffic flow patterns. As a result of this analysis, efficient controller settings were defined and their performance verified using computer simulation.

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ACTUATED CONTROLLER SETTINGS FOR THE DIAMOND INTERCHANGE WITH THREE-PHASE OPERATION

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Research Report 01/01
Research Project: Develop Real-Time, Multimodal, Traffic Adaptive Control System

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SUMMARY

This report describes the investigation and development of actuated controller settings for a diamond interchange operating with a three-phase sequence. The settings considered include the minimum green interval, maximum green interval, and passage time. The investigation was limited to isolated interchanges controlled by a single actuated controller.

The objective of this research project was to develop guidelines for establishing controller settings that will generally yield low-delay (if not lowest) solutions for the range of flow rates encountered during a typical weekday. This objective was achieved through a combination of theoretical analysis and an examination of diamond interchange phasing and traffic flow patterns.

The research focused on the diamond interchange operating with a "three-phase" phase sequence. The phrase "three-phase," is a legacy term that implies the concurrent service of the two cross-road movements, the two left-turn movements, and the two frontage road movements during common phases as well as a cross-road left-turn phase that lags (or follows) that of the conflicting cross-road through movement.

The findings from this project led to four conclusions regarding actuated diamond interchange operation. First, there exists an equilibrium cycle length C_{eq} for all actuated intersections and interchanges. If this cycle length exceeds the minimum-delay pretimed cycle length C_o by 30 percent then the delay-reducing benefits of actuated operation are minimal relative to a well-timed pretimed operation. Second, if the equilibrium cycle length is less than 1.3 C_o then actuated operation will yield lower delay than pretimed operation. Third, the unique phase relationships dictated by the interchange geometry and basic three-phase sequence introduce the need for minimum and maximum green intervals that promote efficient cross-road progression and limit the frequency of queue spillback. Finally, the basic three-phase sequence offers similar performance to the extended three-phase sequence implemented in the Eagle EPAC 300 controller (i.e., Alternate Sequence 17).

Based on this research project, procedures were developed and recommended for use in timing an actuated three-phase diamond interchange. These procedures can be used to determine the minimum green and maximum green settings for all interchange phases. They can also be used to estimate the delay to the individual interchange traffic movements.

INTRODUCTION

Overview

This report describes the investigation and development of actuated controller settings for a diamond interchange operating with a three-phase sequence. The settings considered include the minimum green interval, maximum green interval, and passage time. The investigation was limited to isolated interchanges controlled by a single actuated controller.

The remaining sections of this report document the findings and conclusions from this research project. The first section describes existing technology as it relates to selecting actuated controller settings and the implications of these settings on delay. The discussion in this section will include actuated-control concepts as they relate to signalized intersections and interchanges. This discussion includes intersections because of the numerous operational similarities between intersections and interchanges. A subsequent section examines two variations of three-phase control. One variation represents the "basic" three-phase sequence, and the other variation represents an "extended" three-phase sequence. The extended sequence is intended to accommodate unbalanced off-ramp traffic demands more efficiently than the basic sequence. Finally, guidelines are described for determining efficient actuated controller settings for the basic three-phase sequence.

Research Objective

The objective of this research project was to develop guidelines for establishing controller settings that will generally yield low-delay (if not lowest) solutions for the range of flow rates encountered during a typical weekday. This objective was achieved through a combination of theoretical analysis and an examination of diamond interchange phasing and traffic flow patterns.

Research Scope

The research focused on the diamond interchange operating with a "three-phase" phase sequence. The phrase "three-phase," is a legacy term that implies the concurrent service of the two cross-road movements, the two left-turn movements, and the two frontage road movements during common phases as well as a cross-road left-turn phase that lags (or follows) that of the conflicting cross-road through movement. All other references to the term "phase" in this report will refer to the movement numbering convention established by the National Electrical Manufacturers Association (NEMA). This convention assigns one uniquely numbered controller timing function to each left-turn and through traffic movement. Figure 1 shows the generally recognized relationship between timing function numbers (or phases) and interchange traffic movements.

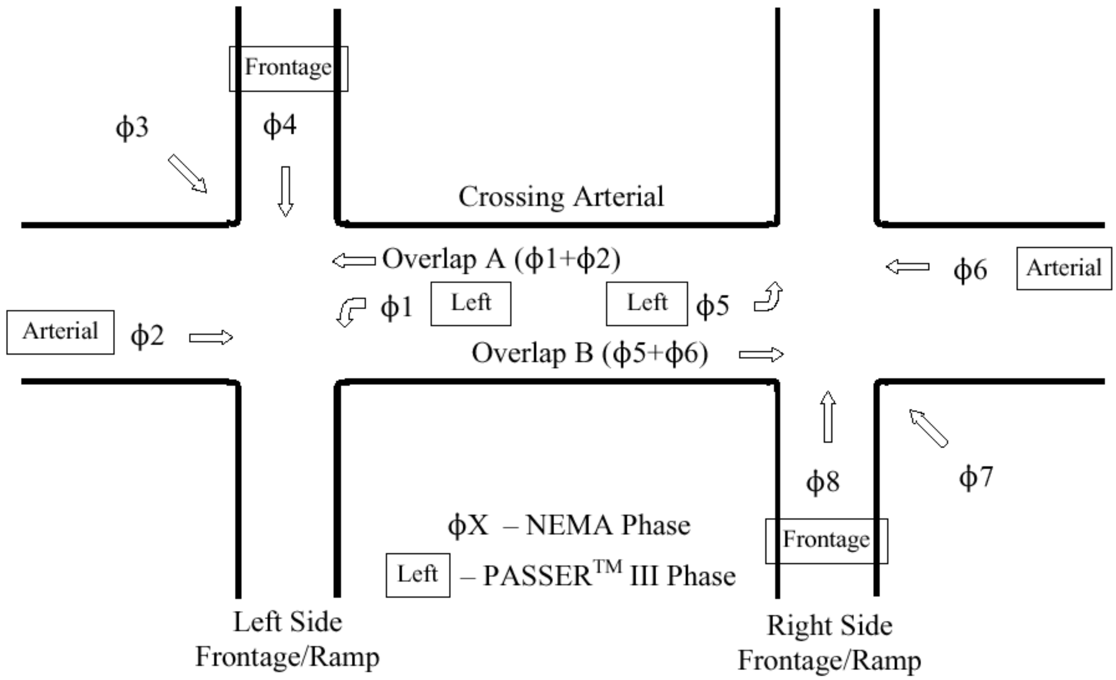


Figure 1. NEMA phasing for diamond interchanges.

EVALUATION OF ALTERNATIVE CONTROL STRATEGIES

Overview

This section describes a review of basic concepts related to actuated interchange operation. Initially, it presents descriptions for two commonly-used variations of the three-phase sequence. Then, it examines the factors affecting the performance of an actuated phase, starting with the examination of the effect of phase duration and cycle length on intersection delay. The findings from this examination are used to define phase durations and cycle lengths associated with minimum delay. Next, this section presents various techniques for determining maximum and minimum green interval settings that result in low (or lowest) delay operation. Finally, it extends these techniques to the diamond interchange.

Three-Phase Diamond Interchange Phasing

Two types of three-phase operation were evaluated for this research. They are:

1. basic three-phase sequence, and
2. extended three-phase sequence (i.e., overlapped left-turn and off-ramp phase).

Both of the three-phase sequences listed above are commonly used at diamond interchanges. The former sequence requires the simultaneous termination of Phases 1 and 5 (cross-road left-turn) whereas the latter sequence allows for the continuation of either Phase 1 or 5 and the concurrent service of the non-conflicting off-ramp phase (e.g., Phase 1 and Phase 8). The extended three-phase sequence results in lower delays (relative to the basic sequence) when off-ramp traffic demands are unbalanced. The phasing for the basic and extended three-phase sequences is shown in Figures 2 and 3, respectively. Note that the ring structure for both sequences is such that each ramp junction is uniquely controlled by the phase timing functions in one ring.

Other variations of the extended three-phase sequence exist. One such sequence requires the simultaneous termination of Phases 1 and 5 but, after that, allows for the concurrent service of either Phase 4 or 8 and the cross-road through phase at the other ramp terminal (e.g., Phase 4 and Phase 6). Again, the flexibility of the extended sequence is intended to result in lower delays (relative to the basic sequence) when off-ramp traffic demands are unbalanced. The extended sequence shown in Figure 3 was selected over other variations because it is explicitly implemented in the Eagle EPAC 300 controller (which is a controller commonly used by the research sponsor).

Actuated Controller Settings for Signalized Intersections

This section explores the relationship between actuated controller operation and intersection delay. This exploration establishes several fundamental relationships between traffic demand, phase duration, cycle length, and delay. These relationships are used to develop maximum and minimum green interval durations that insure low (or lowest) delay for the range of volume conditions that occur during the typical day.

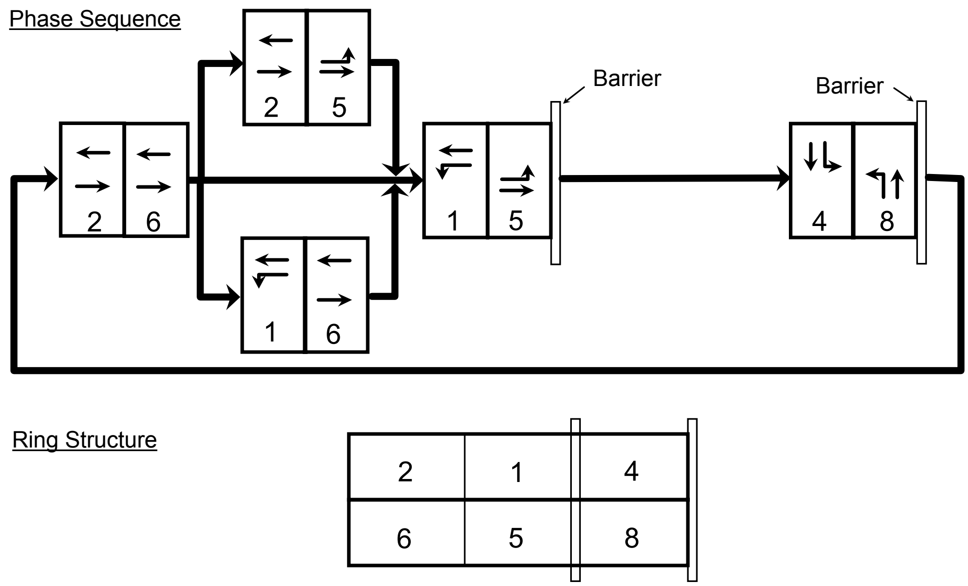


Figure 2. Basic three-phase diamond interchange phase sequence.

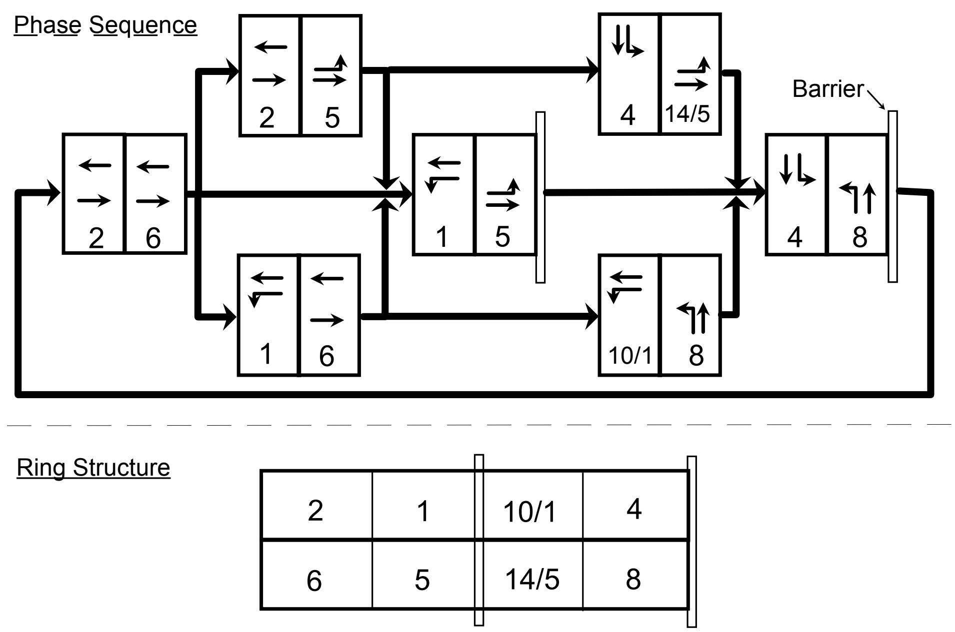


Figure 3. Extended three-phase diamond interchange phase sequence implemented in the Eagle EPAC 300 controller.

Green Interval Duration

The green interval duration for a given phase is computed as the sum of the effective green and the total lost time, less any time needed for the change interval. This relationship is expressed as:

$$G = g + l_t - Y \qquad \textbf{(1)}$$

where:

G = green interval duration, s;
l_t = total lost time (= $l_s + l_e$), s;
l_s = start-up lost time (typically, 2.0 s), s;
l_e = end lost time (= Y when phase gaps-out; otherwise, = Y - 2.0), s;
g = effective green time (= g_p or g_{eq} for pretimed and actuated control, respectively), s; and
Y = yellow and all-red intervals (i.e., change interval) (typically, 3.0 to 5.0 s), s.

Equation 1 applies to both pretimed and actuated phases, depending on whether the effective green time equals g_p or g_{eq}. Equations for each term are described in the following paragraphs. It should also be noted that, if an actuated phase ends because a large gap between vehicles occurs (i.e., it gaps out), the end lost time increases to equal the duration of the change interval.

Effective Green Time for Pretimed Control. For a pretimed phase, Appendix II of the *Highway Capacity Manual* (*HCM*) (*1*) indicates that analysts can use the following equation to estimate the effective green time:

$$g_p = \frac{\frac{v}{s}\,(C - L)}{\sum (\frac{v}{s})_{ci}} \qquad \textbf{(2)}$$

where:

g_p = effective green time for a pretimed phase, s;
C = cycle length, s;
v = traffic demand volume, veh/h;
s = saturation flow rate (typically, 1,800 veh/h/ln), veh/h;
L = total lost time per cycle, computed as the sum of l_t for each critical phase, s; and
$\sum (\frac{v}{s})_{ci}$ = summation of flow ratios (= v/s) for all critical phases, i.

Effective Green Time for Actuated Control. For an actuated phase, the time needed to clear a waiting queue and the subsequent time that arrivals can extend the green together define the effective green time. The effective green duration for an actuated phase is computed as:

$$g_{eq} = g_s + g_e \qquad \textbf{(3)}$$

where:
g_{eq} = equilibrium effective green time for an actuated phase, s;
g_s = queue service time, s; and
g_e = green extension time, s.

The quantity predicted by Equation 3 is called an "equilibrium" green time because its length is not constrained by the minimum or maximum green interval settings on the controller. The interval duration is dictated solely by traffic demand. If the minimum green setting is large, relative to the equilibrium interval duration, then it would constrain the resulting interval duration such that it had a large, non-equilibrium value. In contrast, if the maximum green setting is small then it would constrain the resulting interval duration such that it had a small, non-equilibrium value.

The *HCM* (*1*) describes a series of equations that can be used to estimate the queue service time g_s and the green extension time g_e for an actuated phase. Specifically, the queue service time is computed as:

$$g_s = f_q \frac{v\,(C - g_{eq})}{s - v} \tag{4}$$

where:
g_s = queue service time, s;
f_q = 1.08 - 0.1 $(G / G_{max})^2$, and;
G_{max} = maximum green interval setting, s.

The factor f_q is described by Akcelik (*2*) as a factor that accounts for randomness in arrivals. It equals 1.0 when G_{max} is 12 percent larger than G. This factor is assumed to equal 1.0 for this research.

The green extension time g_e is computed as:

$$g_e = \frac{e^{\lambda\,(h_{\max} - \Delta)}}{\alpha\, q} - \frac{1}{\lambda} \tag{5}$$

where:
g_e = green extension time, s;
q = arrival flow rate (= v/3600), veh/s;
h_{max} = maximum allowable headway, s;
λ = non-platoon flow rate, veh/s;
Δ = minimum platoon headway (see Table 1), s;
α = fraction of non-platoon vehicles (= $e^{-b\Delta q}$), and;
b = bunching factor (see Table 1).

Two of the variables needed for Equation 5 are computed using the following equations:

$$h_{max} = PT + \frac{D_1 - D_n + L_d + L_v}{V_a} \qquad \textbf{(6)}$$

$$\lambda = \frac{\alpha\, q}{1 - \Delta\, q} \qquad \textbf{(7)}$$

where:

PT = passage time setting, s;
D_1 = distance to the leading edge of the advance detector furthest from the stop line, ft;
D_n = distance to the leading edge of the advance detector nearest the stop line, ft;
L_d = length of an advance loop detector (typically, 6 ft), ft;
L_v = detected vehicle length (typically, 18 ft for passenger cars), ft; and
V_a = vehicle approach speed, ft/s.

Finally, values for two variables, *b* and Δ, can be obtained from Table 1.

TABLE 1 Variable values for Equations 5, 6, and 7

Traffic Lanes Served	Min. Platoon Headway, Δ	Bunching Factor, b
1	1.5	0.6
2	0.5	0.5
> 2	0.5	0.8

The flow rate used in Equations 5 and 7 is dependent on the controller ring structure and phase sequence. For phases that do not end at a barrier (as shown in Figures 2 and 3), the *phase* flow rate is used to compute the corresponding green extension time. In contrast, phases that precede a barrier are usually programmed to operate using the "simultaneous gap-out" feature of the actuated controller. As a result, the green extension time of any phase that precedes a barrier is dependent on the *combined* flow rate of all phases that precede the barrier. For example, the ring structure in Figure 2 shows that the volume of the Phase 2 through movement would dictate the Phase 2 green extension time; however, the combined volume of the Phase 1 and the Phase 5 movements would dictate the green extension time for Phase 1 (and Phase 5). Table 2 lists the appropriate flow rates to be used in Equations 5 and 7 to estimate the green extension time for a three-phase signal where the left-turn phases lag the adjacent through phases.

TABLE 2 Flow rate and traffic lane components for Equations 5 and 7

Components	Phase [1, 2]					
	1	2	4	5	6	8
Arrival flow rate:	$q = q_1 + q_5$	$q = q_2$	$q = q_4 + q_8$	$q = q_1 + q_5$	$q = q_6$	$q = q_4 + q_8$
Traffic lanes:	$n = n_1 + n_5$	$n = n_2$	$n = n_4 + n_8$	$n = n_1 + n_5$	$n = n_6$	$n = n_4 + n_8$

Note:
1 - Assumes three-phase operation with lagging left-turn phases.
2 - q_i = arrival flow rate served by Phase *i*; n_i = number of lanes served by Phase *i*.

Maximum Allowable Headway. The maximum allowable headway represents the largest time interval between detector calls that can still extend the green indication for the subject phase. A larger time between vehicle calls would result in gap-out of the phase. A shorter time between calls will extend the phase. If the phase extends until the maximum interval duration is reached, a "max-out" occurs.

Both long and short maximum allowable headways are undesirable. Long values increase the likelihood of max-out which negates the delay-reduction potential of actuated control. Short values of the maximum allowable headway can result in early termination of the phase (i.e., phase gap-out before the waiting queue is served). Passage time settings and detector lengths recommended by Lin (*3*) associate a maximum allowable headway less than about 2.5 s with early phase termination.

The relationship between maximum allowable headway, detector distance, and approach speed represented by Equation 6 applies to the case where the stop line detector is "inactive" during the green interval. This operation can be invoked at the controller for most modern controllers. By being rendered inactive, the stop line detector does not needlessly extend the green to vehicles that are already at the intersection. Equation 6 also assumes that the detectors are operating in presence mode. Other variations of Equation 6 exist for pulse mode and for active stop line detection; these equations are described by Bonneson (*4*) in the *Manual of Traffic Detector Design*.

Equation 6 was used to compute the maximum allowable headway for typical detector designs used by the Texas Department of Transportation at three-phase diamond interchanges, as described by Vengler *et al.* (*5*) in the *PASSER™ III-98 Application and User's Guide*. The details associated with each design vary with approach speed. Table 3 lists these details and the resulting maximum allowable headways.

TABLE 3 Maximum allowable headway for typical detector designs[1]

Approach Speed (mph)	Distance to third loop (ft)	Distance to second loop (ft)	Distance to first loop (ft)	Passage Time (s)	Max. Allowable Headway (s) [2]
30	100	n/a	n/a	2.0 to 3.0	2.5
35	135	n/a	n/a	2.0 to 3.0	2.5
40	170	n/a	n/a	2.0 to 3.0	2.4
45	210	330	n/a	2.0	4.2
50	220	350	n/a	2.0	4.1
55	225	320	415	1.2	3.8

Notes:
n/a - detector not used.
1 - an additional, 6' x 40' stop line detector is also included for all approach speeds.
2 - maximum allowable headways are based on the shorter passage time listed in Column 5 and the following assumptions: detectors have presence-mode operation and an inactive stop line detector during the green interval.

Sensitivity Analysis. Figure 4 shows the green extension time g_e obtained from Equation 5. The trends in this figure suggest that maximum allowable headways larger than 2.5 s are needed to extend the green by any significant amount. The trends also suggest that maximum allowable headways longer than 6 s may extend the green by an undesirably large amount when associated with higher traffic demands.

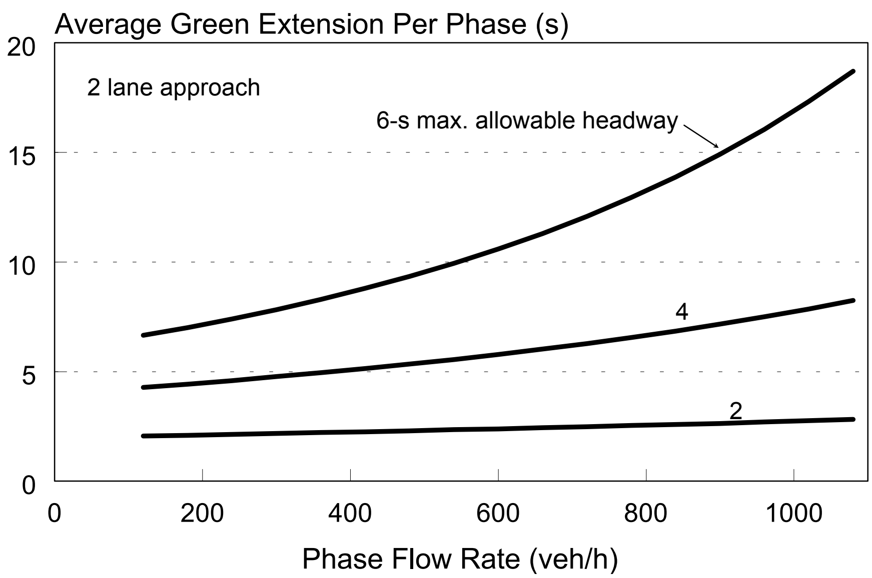


Figure 4. Effect of maximum allowable headway and flow rate on green extension time.

Delay

The *HCM* (*1*) provides the following equation for computing the delay to pretimed and actuated movements at isolated intersections:

$$d = d_1 + d_2 \tag{8}$$

where:

- d = average control delay, s/veh;
- d_1 = delay due to uniform arrivals; s/veh; and
- d_2 = delay due to random and oversaturation queues (i.e., incremental delay), s/veh.

The two delay terms are computed using the following two equations:

$$d_1 = \frac{0.5\ C\ (1 - \frac{g}{C})^2}{1 - Min(1,X)\ \frac{g}{C}} \tag{9}$$

$$d_2 = 900\ T \left[(X - 1) + \sqrt{(X - 1)^2 + \frac{8\ k\ I\ X}{c\ T}} \right] \tag{10}$$

where:

- X = volume-to-capacity ratio (= v / c);
- T = duration of analysis period, hours;
- k = incremental delay factor;
- I = upstream filtering/metering adjustment factor; and
- c = capacity (= $s\ g / C$), veh/h.

The *HCM* (*1*) indicates that the incremental delay factor k ranges in value from 0.04 to 0.50 for actuated phases and is constant at 0.5 for pretimed phases. For actuated phases, the value of k is computed using the following equations:

$$k = (1 - 2\ k_{min})\ (X - 0.5) + k_{min} \quad : \quad k \geq k_{min},\ k \leq 0.5 \tag{11}$$

$$k_{min} = -0.3745 + 0.3537\ PT - 0.091\ PT^2 + 0.0089\ PT^3 \tag{12}$$

Equation 12 is a regression equation developed for this research based on values reported in Table 9-14 of the *HCM* (*1*). The trend associated with Equations 11 and 12 indicates that low values of k (and hence, low values of incremental delay) are realized when the volume-to-capacity ratio and the passage time have small values.

It is interesting that the k factor obtained from Equation 11 is not dependent on the maximum green interval setting G_{max}. Theoretically, this factor should approach zero (as would the incremental delay term) as the maximum green interval is increased. This argument is based on the fact that the incremental delay term is intended to account for delays due to "phase failure" (i.e., waiting vehicles left unserved at the end of the phase). A phase that never maxes-out should never have a phase failure and hence, should not incur incremental delay. Because the k factor from Equation 11 never equals 0.0, it probably reflects "typical" maximum green settings (e.g., settings that are only slightly larger than those needed to serve peak traffic demands).

Cycle Length

Cycle Length for Pretimed Control and Actuated Control with Frequent Max-Out. Webster (*6*) derived the following expression for estimating the minimum-delay cycle length for pretimed control:

$$C_o = \frac{1.5\,L + 5}{1 - \sum_i (\frac{v}{s})_{ci}} \tag{13}$$

where:

C_o = minimum-delay cycle length for pretimed control, s; and

$\sum (\frac{v}{s})_{ci}$ = summation of flow ratios (= v/s) for all critical phases, i.

This equation predicts the cycle length that yields the lowest delay based on consideration of both the uniform and incremental delay terms. It applies to pretimed signal phasing; however, it may also apply to actuated phasing if frequent max-outs occur. Its application to phases that frequently max-out is allowed because such phases incur both uniform and incremental delay.

As discussed previously, an actuated phase with a maximum green setting large enough to ensure phase termination by gap-out would likely only incur uniform delay d_1. Examination of the equation for predicting this delay term (i.e., Equation 9) indicates that the lowest delay is achieved when the cycle length is minimized. However, reducing the cycle length for an actuated phase is difficult as the cycle length is dependent on many factors (as suggested by Equations 3, 4, and 5). One obvious method for reducing the cycle would be through the imposition of a maximum green setting, however, this would increase the frequency of max-out which would defeat the purpose of this action (i.e., delay reduction). Other methods of reducing the cycle length (e.g., reducing the minimum green interval, reducing the maximum allowable headway, etc.) have the potential to reduce the cycle length without increasing delay; however, they too have lower limits that, if exceeded, can lead to increased delay.

Cycle Length for Actuated Control with Infrequent Max-Out. The duration of an undersaturated, actuated phase with a large maximum green is dictated by the time needed to serve the queue plus the time needed to achieve gap-out. This phase duration is the equilibrium phase duration (as computed using Equations 1 and 3). Similarly, the sum of the equilibrium phase durations is the equilibrium cycle length. The equilibrium cycle length can be estimated using Equations 3, 4, and 5 to compute the equilibrium phase length for each of the critical phases. Combining these equations results in the following equation for estimating the equilibrium cycle length for actuated control (as previously reported by Akcelik (*7*)):

$$C_{eq} = \frac{L + \sum_i g_{e,i}\,(1 - v_{ci}/s_{ci})}{1 - \sum_i (\frac{v}{s})_{ci}} \qquad \textbf{(14)}$$

where:

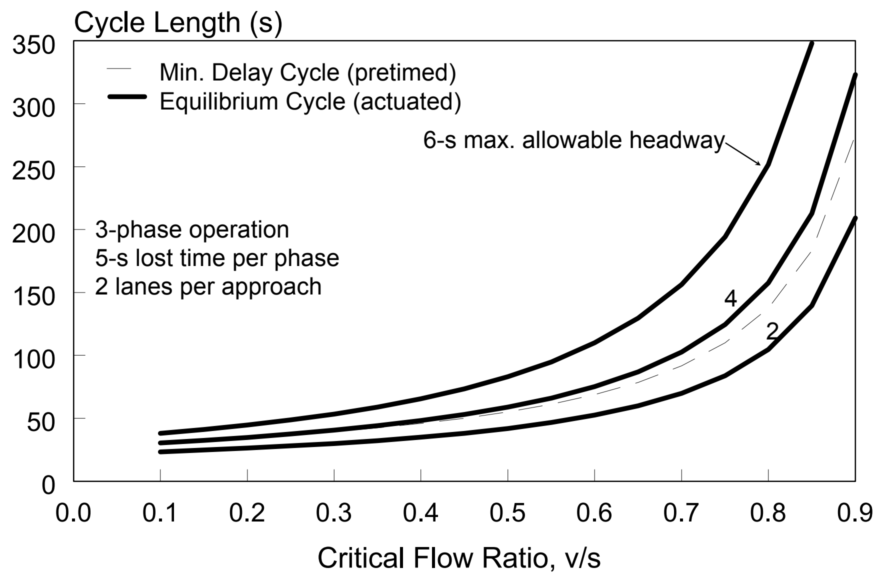
C_{eq} = equilibrium cycle length for an actuated intersection, s; and
$g_{e,i}$ = green extension time for critical Phase *i* based on values and variables in Tables 1 and 2, s.

Equation 14 is based on the assumption that the equilibrium effective green g_{eq} for each critical phase *i* is larger than its minimum green setting and smaller than its maximum green setting. It is also based on the assumption that a call on a conflicting phase is received by the controller before the end of the minimum green interval. This latter assumption is valid when the combined volume on all conflicting phases exceeds about 350 veh/h (i.e., $\approx \Sigma\,(v/s)_{ci} = 0.2$). At this volume level, the resulting call headway (≈ 10 s) is about equal to the typical minimum phase duration (i.e., minimum green plus change interval). In other words, a conflicting call should be received before the end of the minimum green interval when the critical flow ratio at an intersection exceeds 0.2. At lower flow ratios, there will be significant "dwell" time (i.e., no call on any phase) in the signal cycle that will increase the cycle length beyond that predicted by Equation 14.

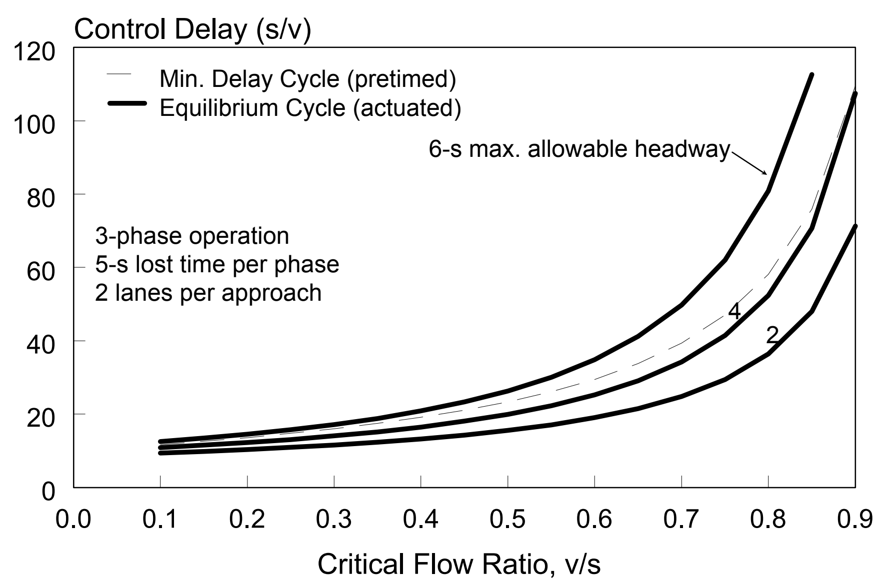
Sensitivity Analysis. Equations 1 through 14 were used to examine the relationship between the sum of critical flow ratios, cycle length, and delay for both pretimed and actuated intersections. Figure 5 shows the results of this evaluation.

The solid lines in Figures 5a and 5b illustrate the effect of flow ratio and maximum allowable headway on the equilibrium cycle length and delay ($d = d_1$), respectively, for actuated operation. The dashed line in each figure illustrates the same relationships for the minimum-delay cycle length and delay ($d = d_1 + d_2$) for pretimed operation. Because equilibrium is achieved when the maximum green settings are large enough to preclude max-outs, only the uniform delay term d_1 (evaluated at the equilibrium effective green time and cycle length) was used to estimate the delay for actuated operation.

The trends shown in Figure 5 show that both the minimum-delay and equilibrium cycle lengths increase in an exponential manner with the critical flow ratio. For a given critical flow ratio, the equilibrium cycle length tends to be less than the minimum-delay cycle length when the maximum allowable headway is less than about 3.7 s. In contrast, the delay associated with actuated control is less than that associated with pretimed control when the critical flow ratio is less than 0.85 and the maximum allowable headway is less than about 4.5 s.



a. Effect of flow ratio and maximum allowable headway on equilibrium cycle length.



b. Effect of flow ratio and maximum allowable headway on intersection delay.

Figure 5. Equilibrium cycle length and delay for actuated intersections.

The data used to develop Figure 5 were also examined to determine the relationship between cycle length and delay. Figure 6 shows the results of this investigation.

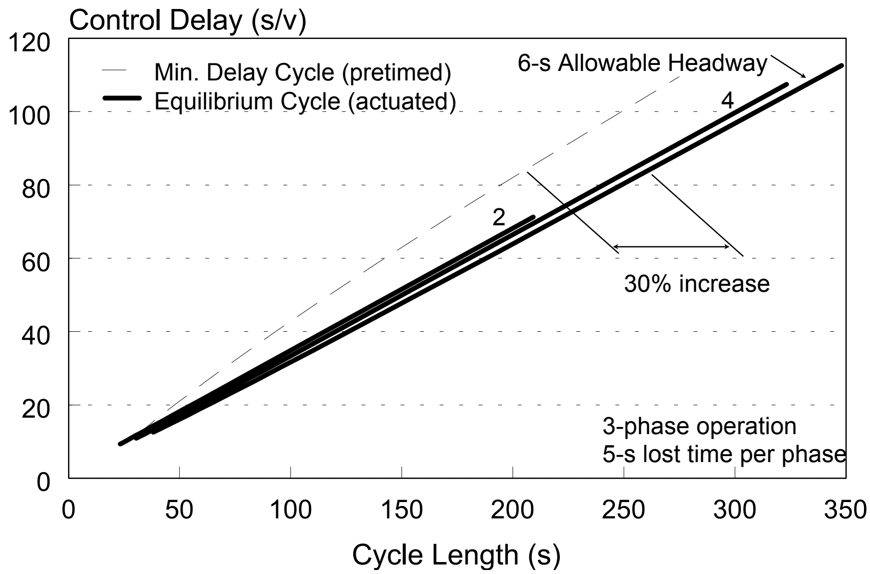


Figure 6. Relationship between cycle length and delay.

The trends in Figure 6 show that the equilibrium cycle length C_{eq} that yields the same delay as the minimum-delay cycle length C_o is about 30 percent larger than the minimum-delay cycle, regardless of the magnitude of the maximum allowable headway. This relationship implies that an actuated intersection will operate with lower delay than a pretimed intersection when: (a) C_{eq} is less than 1.3 C_o and (b) the maximum green setting is sufficiently large as to preclude the occurrence of max-outs.

Findings. If it is assumed that a pretimed intersection is set to operate at the minimum-delay cycle length and have phase durations determined using Equations 1 and 2, the following statements can be made:

1. An actuated intersection with: (a) a maximum allowable headway of about 4.5 s or less, (b) an equilibrium cycle length C_{eq} less than 1.3 C_o, and (c) maximum green settings so large that the phases never max-out, would operate with less delay than a pretimed intersection.

2. An actuated intersection that does not satisfy the conditions in Statement 1 will operate less efficiently than a pretimed intersection. However, the efficiency of the pretimed intersection could be matched if the maximum greens were set equal to the equivalent pretimed green phase durations (as determined by Equations 1 and 2). Of course, this would result in frequent max-out and equivalent pretimed operation.

3. The trend lines in Figure 5 suggest that maximum allowable headways larger than 5.5 s are not likely to produce an actuated operation that is more efficient than pretimed, even for low traffic volumes.

One condition in Statement 1 is that the maximum green setting G_{max} is sufficiently large as to preclude any occurrence of max-out. Guidelines by Kell and Fullerton (*8*) suggest that maximum green settings that are at least 25 percent larger than the minimum-delay green interval G_o are likely to have this quality (i.e., $G_{max} > 1.25\ G_o$, or $1.3\ G_o$ when rounded). The minimum-delay green interval duration G_o would be obtained from Equations 1 and 2 when $C = C_o$.

Maximum Green Setting

Three different equations for computing the maximum green interval have been defined in the literature. All three equations are based on the minimum-delay pretimed green interval duration G_o. As noted previously, Kell and Fullerton (*8*) recommend setting the maximum interval G_{max} equal to G_o after inflating it by 25 to 50 percent (i.e., $G_{max} = 1.25\ G_o$ to $1.5\ G_o$). In contrast, Lin (*3*) recommends setting the maximum interval equal to G_o after adding 10 s (i.e., $G_{max} = G_o + 10$). Finally, Skabardonis (*9*) recommends computing the maximum green interval using the following equation:

$$G_{\max} = G_o + \frac{3600\,X^2}{4\,(1 - X)\,s} \tag{15}$$

Equation 15 adds 0.0 s to G_o at $X = 0$ and 9 s to G_o at $X = 0.95$; hence, it is a little more conservative than Lin's approach.

The findings from the previous section suggest the following equation for determining the maximum green interval setting for actuated phases:

$$\begin{aligned} &\textit{If } C_{eq} < 1.3\,C_o \quad \textit{then } G_{\max} = \textit{Larger of}: [G_{\min} + 10,\quad 1.3\,G_o] \\ &\qquad\qquad \textit{otherwise } G_{\max} = \textit{Larger of}: [G_{\min},\quad G_o] \end{aligned} \tag{16}$$

where:

G_{min} = minimum green interval setting, s; and
G_o = minimum-delay pretimed green interval duration (where G_o is obtained from Equations 1 and 2 when $C = C_o$).

The factor 1.3 multiplied by G_o in this equation is one of several possible values. As mentioned previously, Kell and Fullerton (*8*) suggest that factors between 1.25 and 1.50 are acceptable. The theoretic analysis in the preceding paragraphs suggests that any value larger than 1.3 is equally acceptable. The issue of determining the "best" factor value is more closely examined in a subsequent section of this report.

Equation 16 is consistent with the recommendation of Kell and Fullerton (*8*) and the recommendation of Lin (*3*). In contrast, it is contrary to the recommendation of Skabardonis (*9*). Skabardonis recommends increasing G_{max} with increasing volume; whereas, Equation 16 recommends that the maximum green setting *not* exceed G_o when volumes are sufficiently high as to result in $C_{eq} > 1.3\ C_o$. Skabardonis' recommendation is likely to increase the frequency of max-out during low-volume scenarios and increase delays (due to long cycle lengths) during high-volume scenarios, relative to operations produced by application of Equation 16.

Minimum Green Setting

The minimum green setting can vary depending on the type of service provided to pedestrians. If there is no pedestrian activity, or if there is activity and it is provided a call button, then the minimum green setting can be based solely on driver expectancy. In this regard, driver expectancy requires a minimum green interval of 4 to 8 s in duration. Minimum green interval durations (especially those that exceed about 0.75 G_o) should be avoided as they tend to increase delay.

When pedestrians are present and they are not provided a call button, the minimum green interval setting should be based on pedestrian crossing time requirements. Procedures for estimating the minimum green setting under these circumstances are described by Venglar *et al.* (*5*).

Actuated Controller Settings for Signalized Interchanges

The internal traffic movements at an interchange (Phases 1, 5, Overlap A, and Overlap B) create unique situations that require consideration when setting the minimum green and maximum green interval settings for actuated interchanges. These situations relate to the physical separation of the two ramp junctions and the interaction of their associated phases. For example, arrivals to Phase 2 occur throughout the cycle and are randomly dispersed. In contrast, arrivals to Phase 5 and Overlap B are platooned and lag the start of Phase 2 by a length of time equal to the travel time between ramp junctions. The interaction between these "associated" phases adds additional constraints on their duration in order to minimize phase failure, queue spillback, and delay.

This section describes additional considerations for the minimum green and maximum green settings as they relate to the signalized diamond interchange operating with actuated control and a three-phase sequence.

Minimum Green Setting

Left-Turn-Phase Minimum Green Setting. One advantage of three-phase operation is its tendency to provide concurrent two-way progression along the cross road. However, as the distance between ramp junctions increases, it becomes more difficult to maintain this progression because the internal travel time (i.e., the travel time between ramp junctions) eventually exceeds the volume-based duration of the combined cross-road phases (i.e., 1 + 2 or 5 + 6). Figure 7 illustrates this problem.

Figure 7 illustrates a situation where the travel time *tt* for Phase 2 traffic exceeds the duration of Phases 5 + 6. As a result, the last portion of the Phase 2 platoon arrives after the end of Phase 5 (and Overlap B). The consequence of this "late" arriving platoon is a recurring residual queue of vehicles that must wait the duration of Phase 4+8 before receiving the green indication.

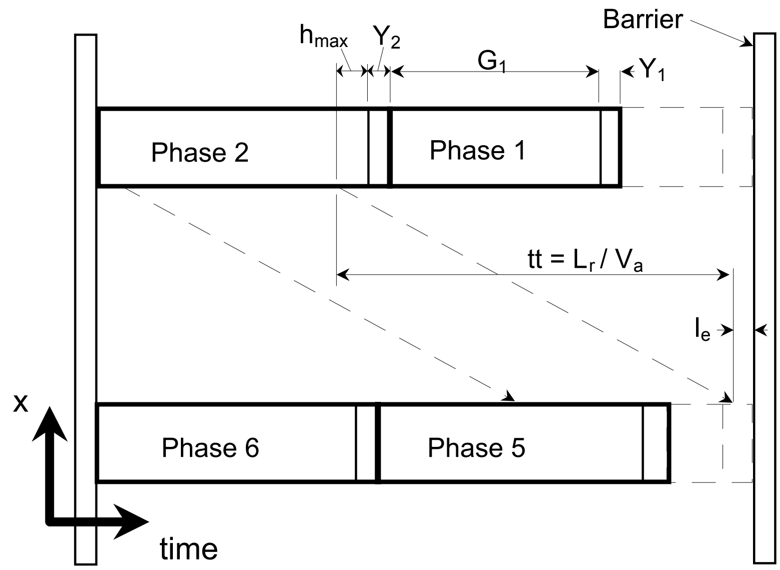


Figure 7. Relationship between cross-road phase duration and internal travel time.

The solution to this problem with late arrivals is found by adjusting the duration of Phase 1. At first glance, this solution might seem misdirected as the problem is related to Phases 2 and 5. However, the predominant use of the "simultaneous gap-out" feature in the actuated controller dictates that Phases 1 and 5 end together. Thus, by ensuring that Phase 1 has a length that is shorter than the travel time, Phase 5 is indirectly extended such that the platoon can proceed uninterrupted through the interchange. The mathematical relationship that relates the minimum Phase 1 time to the travel time is:

$$G_1 > \frac{L_r}{V_a} + l_{e,5} - h_{\max,2} - Y_1 - Y_2 \tag{17}$$

If it is assumed that all change intervals Y_i are equal, that the maximum allowable headway for Phase 2 $h_{max,\,2}$ is about 3 s, and that the end lost time for Phase 5 $l_{e,\,5}$ is equal to Y - 2.0, then Equation 17 reduces to the following equation for the minimum green setting:

$$G_{\min,lt} = \frac{L_r}{V_a} - Y - 5 \tag{18}$$

where:

$G_{min,\,lt}$ = minimum green setting for the cross-road left-turn phase (1 or 5), s;
L_r = distance between ramp terminals (measured from stop line to stop line), ft;
V_a = vehicle approach speed, ft/s; and
Y = yellow and all-red intervals (i.e., change interval) (typically, 3.0 to 5.0 s), s.

The relationships shown in Figure 7 suggest that Equation 18 needs to be applied only to the "shorter" of the two left-turn phases (i.e., Phase 1). The larger phase (i.e., Phase 5) is also extended and, presumably, there are no late arrivals from Phase 6. However, there is still the possibility of late arrivals from Phase 6 when Phase 5 is relatively short, yet Phase 6 and 5 combined are "critical" (i.e., their combined flow ratio exceeds that of Phases 2 and 1). Hence, the following rule is coined regarding the use of Equation 18:

Minimum Green Rule 1. The minimum green setting computed in Equation 18 should be applied to the critical left-turn phase and the left-turn phase with the smaller flow ratio.

In some instances, Rule 1 will dictate the imposition of a minimum green on both left-turn phases. However, it is possible that the critical left-turn phase also has the smaller flow ratio. In this instance, Equation 18 will only be applied to this one left-turn phase.

Cross-Road Through-Phase Minimum Green Setting. As with the left-turn movements, the distance between the ramp junctions (when combined with the associated phase/ring interaction) dictates special consideration for the cross-road through-movement phases. Specifically, at wider diamond interchanges, it is possible that vehicles departing the upstream stop line on Phase 6 will not reach the detectors at the downstream ramp terminal in time to extend Phase 2 or call Phase 1. This problem is also possible for travel in the opposite direction as it relates to Phases 2, 5, and 6. The result is that Phases 1 and 5 are skipped which disrupts progression and results in traffic queuing in the lanes between ramp junctions.

The aforementioned problem is most likely to occur when ramp spacing is large, cross-road through demands are balanced, and U-turn traffic is light. The resulting delays are likely to be large and increase with increasing frontage road volume. This problem can be avoided when the duration of Phase 2 or 6 exceeds the travel time to the first downstream detector. The following equation can be used to determine a minimum green setting for Phase 2 or 6 that eliminates this problem:

$$G_{\min,th} = l_s + \frac{L_r - D_{1d}}{V_a} \qquad \textbf{(19)}$$

where:

$G_{min,\,th}$ = minimum green setting for the cross-road through phase (2 or 6), s;
D_{1d} = distance between the downstream stop line and the leading edge of the advance detector furthest from this stop line (as measured on the cross road between ramp junctions), ft; and
l_s = start-up lost time (typically, 2.0 s), s.

If either Phase 2 or Phase 6 has a green that exceeds that described by Equation 19, then Phases 1 and 5 will not be skipped. Applying the value from Equation 19 to the critical phase (i.e., the phase with the larger flow ratio) is ideal because it generally has the greater need for green time. The benefit of this choice is that any increase in delay to other movements (as are typically incurred when a large minimum green is imposed) is minimal. In summary, the following rule is coined regarding the use of Equation 19:

Minimum Green Rule 2. The minimum green setting computed in Equation 19 should be applied to the critical cross-road through phase.

Sensitivity Analysis. The minimum green settings implied by Equations 18 and 19 are shown in Figure 8 for a range of ramp spacings. For both equations, an absolute minimum green interval of 5 s (based on driver expectancy) was imposed. In general, the minimum green increases with increasing ramp spacing. It is smaller for higher approach speeds. For Phases 2 and 6, the minimum green setting ranges from 5 to 22 s for the ramp spacings shown. For Phases 1 and 5, the minimum green setting ranges from 5 to 15 s. The next section discusses the maximum green trend line.

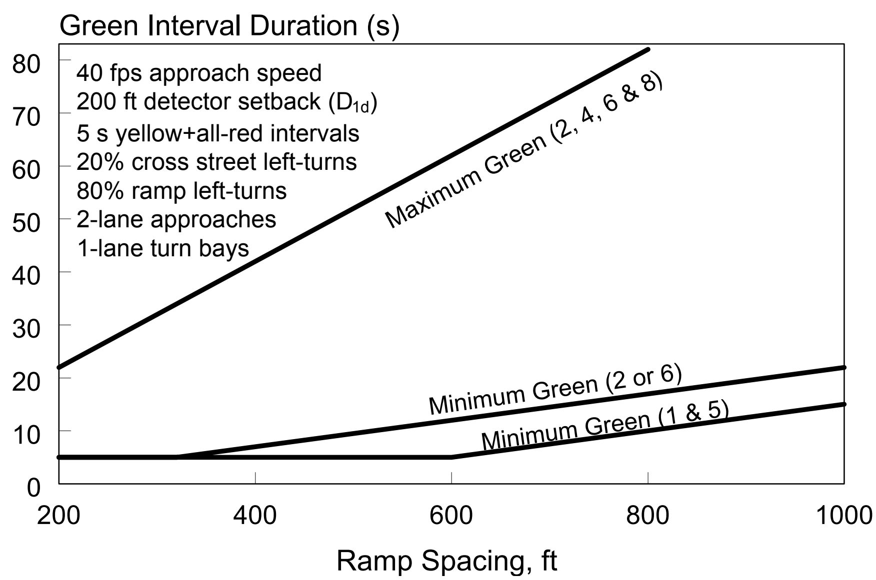


Figure 8. Effect of ramp spacing on maximum and minimum green settings.

Maximum Green Setting

The nature of three-phase operation and the limited storage area between the ramp terminals also impose some limitations on the maximum duration of the external phases (i.e., Phases 2, 4, 6, and 8). These phases cannot be so long as to have vehicles queue on the internal street segment and spill back into the upstream ramp terminal. The maximum green settings for these phases are described in this section.

Cross-Road Through-Phase Maximum Green Setting. The maximum green setting for Phases 2 and 6 should be set such that the left-turn vehicles that queue at the downstream ramp junction do not overflow the left-turn storage space provided. The maximum interval that would prevent this from occurring can be computed as:

$$G_{\max,th} = \frac{n_{lt}\, L_b\, 3600}{p_{ltcs}\, n_t\, L_q\, s} + l_s \qquad \textbf{(20)}$$

where:

$G_{max,\,th}$ = maximum green setting for a cross-road through phase (2 or 6), s;
n_{lt} = number of exclusive left-turn lanes in the opposing direction of travel;
n_t = number of through lanes on the cross road in the subject direction of travel;
p_{ltcs} = ratio of left-turn volume in the opposing cross-road traffic stream to total volume in the subject direction of travel (i.e., $p_{ltcs} = v_1 / v_2$ or $p_{ltcs} = v_5 / v_6$);
L_b = length of the lane(s) for exclusively storing left-turn vehicles, ft;
L_r = distance between ramp terminals (measured from stop line to stop line), ft;
L_q = length of a traffic lane occupied by a stopped vehicle (typically, 25 ft), ft;
s = saturation flow rate per lane (typically, 1,800 veh/h/ln), veh/h/ln; and
l_s = start-up lost time (typically, 2.0 s), s.

This maximum green setting should be imposed on both through movements.

Off-Ramp-Phase Maximum Green Setting. The maximum green setting for Phases 4 and 8 should be set such that vehicles that turn left from the ramp and queue at the downstream ramp junction do not overflow the storage space provided on the lanes between ramp terminals. The maximum interval that would prevent this from occurring can be computed as:

$$G_{\max,rp} = \frac{n_t\, L_r\, 3600}{p_{ltrp}\, n_{rp}\, L_q\, s} + l_s \qquad \textbf{(21)}$$

where:

$G_{max,\,rp}$ = maximum green setting for the off-ramp phases (4 and 8), s;
n_{rp} = number of lanes on the off-ramp;

n_t = number of through lanes on the cross road in the subject direction of travel; and
p_{ltrp} = fraction of left-turn vehicles in the off-ramp traffic stream.

This maximum green setting should be imposed on both off-ramp movements.

Sensitivity Analysis. Figure 8 shows the maximum green settings implied by Equations 20 and 21 for a range of ramp spacings. The left-turn bay length is assumed to equal half the distance between ramps. By coincidence, the variable values assumed for this illustration yield the same maximum green settings from Equations 20 and 21.

In general, the trend line in Figure 8 indicates that the proposed maximum green increases with increasing ramp spacing. Based on the assumed values shown in the figure, the maximum green setting ranges from 22 to 82 s for ramp spacings that range from 200 to 800 ft. As maximum green settings less than 40 s are likely to be too short for most "busy" interchanges, the trends in Figure 8 suggest that three-phase operation may not be well suited to ramp spacings less than 400 ft.

Cycle Length

Cycle Length for Pretimed Control and Actuated Control with Frequent Max-Out. The previous section that dealt with signalized intersection cycle lengths described an equation derived by Webster (*6*) for estimating the minimum-delay cycle length C_o (i.e., Equation 13) for pretimed intersections. An equivalent equation is not available for the diamond interchange. However, the PASSER III software package is used to compute cycle length yielding the least delay. This software is used to determine the minimum-delay cycle length for several interchange volume scenarios (described in the next section).

As a means of evaluating the potential applicability of Equation 13 to interchanges, the minimum-delay cycle length obtained from PASSER III is compared with that predicted by Equation 13. The critical flow ratio used in Equation 13 is based on consideration of the flow ratios for each phase and the ring structure shown in Figure 2. Figure 9 compares the resulting minimum-delay cycle lengths with those obtained from PASSER III.

The trends shown in Figure 9 suggest that the minimum-delay cycle length predicted by Equation 13 is roughly equivalent to that obtained from PASSER III. However, there is some variation in cycle length as a result of ramp spacing. Specifically, the results show that Equation 13 overestimates the minimum delay cycle length by about 20 percent for ramp spacings of 360 ft. In contrast, it underestimates the minimum-delay cycle length by about 8 percent for ramp spacings of 720 ft. It is likely that Equation 13 would be most applicable to ramp spacings of 600 ft. From this analysis, it is concluded that Equation 13 can predict the minimum-delay cycle length for a diamond interchange with three-phase, pretimed control (or actuated control with frequent max-out), provided that ramp spacings are between 400 and 800 ft.

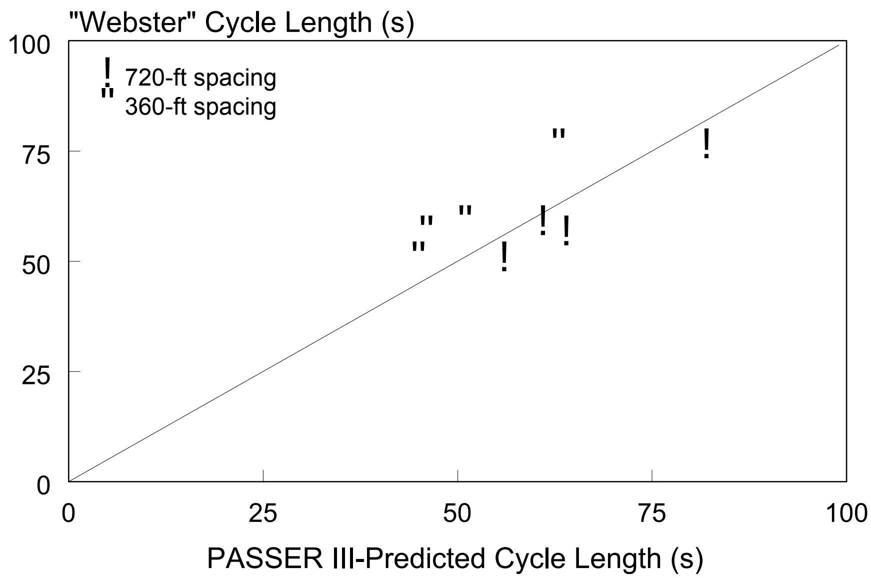


Figure 9. Comparison of minimum-delay cycle lengths from PASSER III and Equation 13.

Cycle Length for Actuated Control with Infrequent Max-Out. In a preceding section, an equation (i.e., Equation 14) was described as suitable for predicting the equilibrium cycle length for an actuated intersection. This equation is based on several assumptions, one of which is that the vehicles arrive randomly to each phase. However, this is not so for Phases 1, 5, Overlap A, and Overlap B at a diamond interchange.

In the case of Phase 1 and Overlap A, vehicles from Phase 6 arrive in concentrated platoons approximately *tt* seconds (where, *tt* = the travel time between ramp junctions) after the start of Overlap A (= Phase 2). If these platooned vehicles arrive *before* the end of Phase 2, then the equilibrium cycle computed by Equation 14 would be applicable to the diamond interchange. If these platooned vehicles are still arriving *after* the end of Phase 2, then they will extend Phase 1– much as would a waiting queue of left-turn vehicles. An equation for computing the equilibrium cycle length for this latter situation is described in the remainder of this section.

If the Phase 6 platoon arrives after the start of Phase 1, the resulting queue clearance time for Phase 1 can be estimated as:

$$g_{s,1} = l_{s,6} + g_{s,6} + \frac{L_r}{V_a} - (G_2 + Y_2) \quad \geq 0.0 \qquad \textbf{(22)}$$

where:

$g_{s,1}$ = queue service time for the cross-road left-turn Phase 1, s;
L_r = distance between ramp terminals (measured from stop line to stop line), ft;
V_a = vehicle approach speed, ft/s; and
G_2 = green interval for Phase 2, s.

The second term in Equation 22, queue service time for Phase 6 $g_{s,6}$, can be obtained by combining Equations 3 and 4. The resulting equation is:

$$g_{s,6} = (v/s)_6 \ (C - g_{e,6}) \qquad \textbf{(23)}$$

Finally, after the Phase 6 platoon clears, random arrivals from Phase 6 extend Phase 1 (and Overlap A) until gap-out or max-out. If Phase 1 ends by gap-out, Equation 5 is used to compute its green extension time $g_{e,1}$. However, the arrival flow rate q used in Equation 5 is equal to the Phase 6 flow rate (i.e., $q = v_6/3600$). Also, the number of lanes n in the movement includes those in the left-turn lane and those serving the cross-road through movement (i.e., $n = n_{th,6} + n_{lt,1}$).

The equilibrium effective green for Phase 1 is estimated by combining Equations 22, 23, 3, and 5 (using the Phase 6 flow rate in Equation 5). Similarly, the effective green time for Phases 2 and 4 is computed using Equations 3, 4, and 5. Finally, these effective green times are combined with Equation 1 and summed to equal the equilibrium cycle length. As an alternative, these equations were combined to obtain the following equation for computing the equilibrium cycle length:

$$C_{eq,1} = \frac{L + [g_{e,c}\,(1 - v_c/s_c)]_{4,8} + [g_e\,(1 - v/s)]_6 + L_r/V_a - l_e}{1 - (v_c/s_c)_{4,8} - (v/s)_6} \qquad \textbf{(24)}$$

where:

$C_{eq,1}$ = equilibrium cycle length for an actuated interchange when Phase 6 platoons arrive during Phase 1, s.

The second term in both the numerator and denominator of Equation 24 is computed using the flow characteristics of the critical off-ramp phase (i.e., the phase with the larger flow ratio). The third term in the numerator and denominator is based on the flow characteristics of Phase 6, regardless if it is a critical phase. The green extension time g_e for Phase 4 or 8 and for Phase 6 is computed using the flow rate and traffic lane equations listed in Table 2.

Using a development similar to that described in the preceding paragraphs, the equilibrium cycle length based on Phase 5 arrivals can be computed as:

$$C_{eq,5} = \frac{L + [g_{e,c}\,(1 - v_c/s_c)]_{4,8} + [g_e\,(1 - v/s)]_2 + L_r/V_a - l_e}{1 - (v_c/s_c)_{4,8} - (v/s)_2} \qquad \textbf{(25)}$$

where:

$C_{eq,5}$ = equilibrium cycle length for an actuated interchange when Phase 2 platoons arrive during Phase 5, s.

The third term in both the numerator and denominator of Equation 25 is based on the flow characteristics of Phase 2, regardless if it is a critical phase.

Finally, the equilibrium cycle length for the three-phase diamond interchange can be estimated as the larger of the equilibrium cycle lengths computed from Equations 14, 24, and 25. That is:

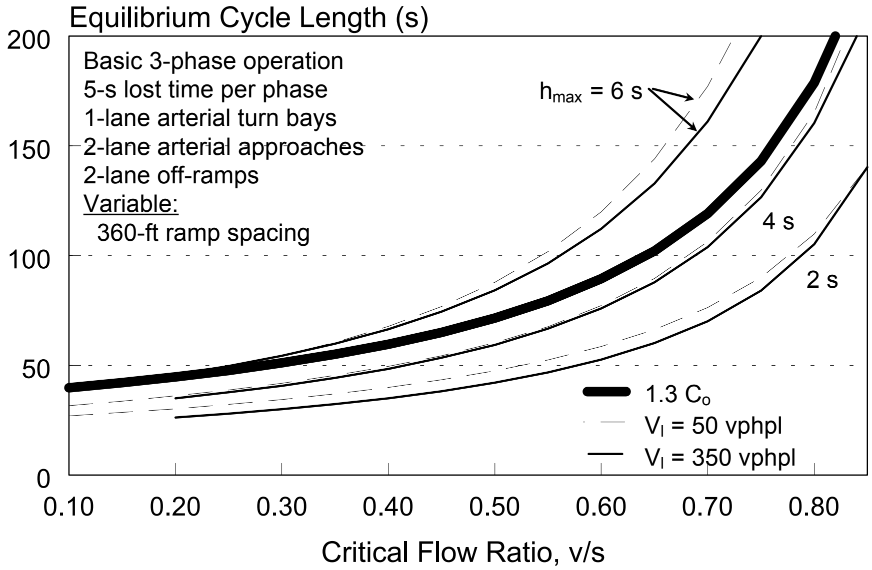
$$C_{eq,d} = Larger\ of{:}\ [C_{eq},\ C_{eq,1},\ C_{eq,5}] \qquad \textbf{(26)}$$

where:
$C_{eq,d}$ = equilibrium cycle length for an actuated interchange, s.

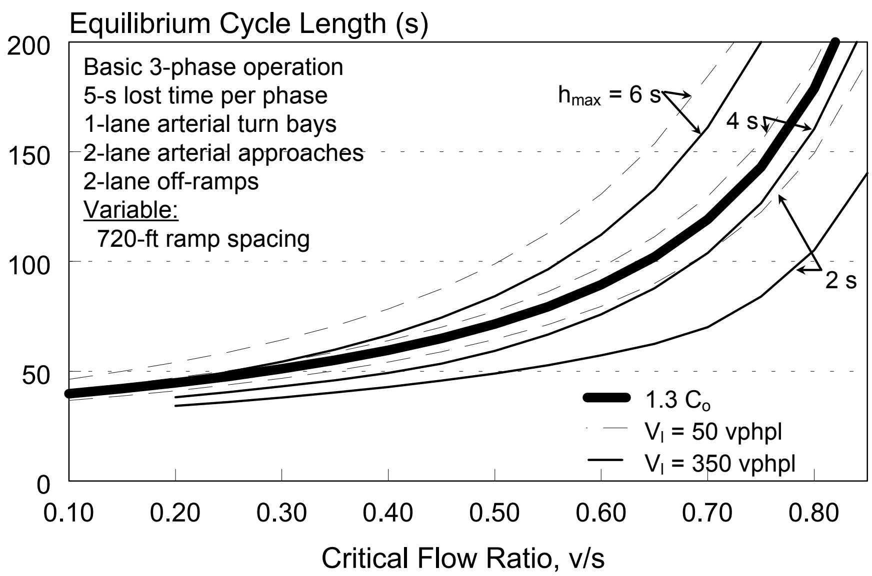
Sensitivity Analysis. The effect of flow ratio, left-turn volume, ramp spacing, and maximum allowable headway on interchange equilibrium cycle length was examined. Figure 10 illustrates the results of this examination. The trends shown are similar to those previously found with regard to intersection operations (as shown in Figure 5).

As shown in Figure 10, the equilibrium cycle length increases exponentially with increasing flow ratio. It also increases with maximum allowable headway. It may increase with ramp spacing, but only if the left-turn volume is light (e.g., 50 veh/h). The equilibrium cycle length is insensitive to ramp spacing when left-turn volumes are heavy because the heavy left-turn (combined with simultaneous gap-out of the lagging left-turn phases) tends to extend the left-turn green much more so than does ramp spacing. In other words, when left-turns are heavy, Equation 14 dictates the equilibrium cycle length; when it is light and ramp spacing is large, Equation 24 or 25 dictates the equilibrium cycle length.

The thick trend line in Figure 10 represents the minimum-delay cycle length C_o inflated by 30 percent. Findings in the previous section indicated that equilibrium cycle lengths in excess of 1.3 C_o offer little benefit from actuated control (relative to a well-timed pretimed controller). The trends shown in Figure 10 suggest that interchange detector designs that produce maximum allowable headways in excess of 4.5 s may offer little benefit from actuated operation, regardless of demand volume. In fact, the limiting maximum allowable headway may only be 4.0 s when left-turn volumes are light (say, 100 veh/h or less) and ramp spacings are large (say, 500 ft or more).



a. Equilibrium cycle length for 360-ft ramp spacing.



b. Equilibrium cycle length for 720-ft ramp spacing.

Figure 10. Effect of flow ratio, left-turn volume, and ramp spacing on equilibrium cycle length.

EVALUATION OF ACTUATED CONTROL MODES AND CONTROLLER SETTINGS

Overview

This section describes an evaluation of two, three-phase actuated control modes and an evaluation of several of the controller settings developed in the previous section. The evaluation considered a range of volume and ramp spacing scenarios for an otherwise typical diamond interchange with frontage roads. This section describes the scenarios and the evaluation procedure. Then, the results of the comparison of the two control modes are described, followed by the results of the proposed controller setting evaluation.

Analysis Approach

The approach used to evaluate the alternative control modes and proposed controller settings is based on the use of several volume scenarios and ramp spacings that collectively embrace a range of typical three-phase diamond interchange operating conditions. These conditions were simulated using the CORSIM simulation model Version 4.3 with the Hardware-in-the-Loop system developed by the Texas Transportation Institute. This system is described elsewhere by Koonce *et al.* (*10*). It allows the simulator to interact in real time with an actual controller. In this instance, a NEMA-based Eagle EPAC 3000 controller was used to control the signal in the simulated interchange.

Figure 11 shows the interchange geometry used for all analyses and scenarios. As indicated by this figure, the frontage road/off-ramp and the cross-road through movements had two through lanes in each travel direction. Both cross-road left-turn movements have a single-lane left-turn bay. The free-flow speed was set in CORSIM as 30 mph which resulted in a 40-ft/s approach speed. A 100-ft presence-mode detector loop was used in each lane of each approach. When combined with a passage time of 0.0 s, this design yielded the same operation and a slightly larger maximum allowable headway (2.7 s) than that listed in Table 3. The distance between ramp junctions (i.e., the ramp spacing) was varied among two values (i.e., 360 ft and 720 ft).

Table 4 lists the volume scenarios used for the evaluation. Collectively, these scenarios present a wide range of volume level and demand patterns commonly found at a diamond interchange. It was assumed that 10 percent of all ramp left-turn vehicles would turn left again at the downstream ramp junction (i.e., complete a U-turn). Right-turn activity was not explicitly considered. The maximum green settings were established at a value 50 percent larger than the minimum-delay pretimed green interval duration G_o (where G_o is obtained from Equations 1 and 2 when $C = C_o$). The minimum green setting was set equal to 8 s for all phases and scenarios.

Comparison of Alternative Control Modes

As discussed in a previous section, two alternative, three-phase control modes were evaluated for this research. These two modes are: (1) basic three-phase sequence, and (2) extended three-phase sequence (i.e., overlapped left-turn and off-ramp phase).

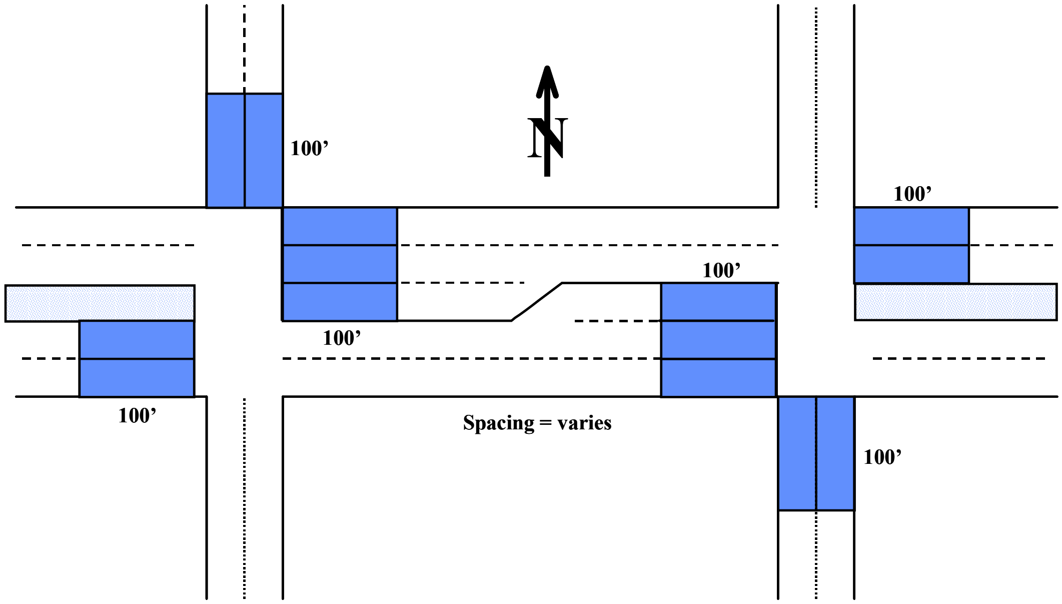


Figure 11. Diamond interchange geometry used for evaluation.

TABLE 4 Volume scenarios used for evaluation process

Scenario (run duration)	Characteristic	Phase					
		1	2	4	5	6	8
		WB Left	EB Thru	SB Ramp	EB Left	WB Thru	NB Ramp
A Heavy EB (1 hour)	Volume, veh/h	98	1450	600	360	700	6
	Max. Green, s	11	45	21	20	33	21
	Left-turn vol, %			17			17
B Heavy EB & WB (1 hour)	Volume, veh/h	102	1200	600	108	1200	600
	Max. Green, s	11	41	21	11	40	21
	Left-turn vol, %			17			17
C Heavy NB & WB (1 hour)	Volume, veh/h	133	700	600	110	1100	850
	Max. Green, s	14	34	31	11	39	31
	Left-turn vol, %			17			41
D Light (2.5 hours)	Volume, veh/h	10	40	60	11	40	6
	Max. Green, s	11	11	11	11	11	11
	Left-turn vol, %			17			17
E Heavy Through (1.5 hours)	Volume, veh/h	135	1050	1051	63	1226	1225
	Max. Green, s	13	47	54	11	54	54
	Left-turn vol, %			5			10

Note:
1 - underlined values denote the heavier traffic movements for each scenario (if any).

The two three-phase control modes identified in the preceding paragraph are commonly used at diamond interchanges. The "basic" sequence requires the simultaneous termination of Phases 1 and 5 (cross-road left-turn). The "extended" sequence allows for the continuation of either Phase 1 or 5 and the concurrent service of the non-conflicting off-ramp phase (e.g., Phase 1 and Phase 8). Presumably, the flexibility of the extended three-phase sequence would result in lower delays (relative to the basic sequence) when off-ramp traffic demands are unbalanced.

The evaluation consisted for simulating each control mode using the five analysis scenarios. Each scenario was simulated with a ramp spacing of 360 ft and again with a ramp spacing of 720 ft. The duration of each simulation varied, depending on an estimate of the desired precision in the predicted phase durations. The run durations developed from this analysis varied from 1.0 to 2.5 hours depending on the volume scenario; specific durations are listed in Column 1 of Table 4.

The basic three-phase sequence was implemented on the Eagle controller using its "full-function EPAC" mode. This mode uses eight-phase NEMA logic that has traditionally been used for signalized intersection control; only six of these phases are used to control the interchange.

The extended three-phase sequence was implemented using the Eagle controller's "Alternate Sequence 17." This alternate sequence is one of several sequences available in the Eagle controller that satisfy the Texas Department of Transportation's Texas Diamond Intersection Control specification. Alternate Sequence 17 has preset (and unchangeable) settings for the controller's detector logic, ring structure, and phase sequence control logic.

For the extended sequence, the detector design was altered from that shown in Figure 11. This action was undertaken to make the resulting maximum allowable headways equivalent among the two control modes. Alternate Sequence 17 has special detector logic that disables the stop line detectors for Phases 2, 4, 6, and 8 when the respective phase is green. It also has a default passage time of 1.0 s for all phases. To achieve a maximum allowable headway equivalent to that used for the basic sequence (i.e., 2.7 s), the 100-ft, long-loop detector on each approach was replaced by two 40-ft loops. On the external approaches (i.e., Phases 2, 4, 6, and 8), the first loop was located at the stop line and the leading edge of the second loop was located 100-ft upstream. On the internal through-movement approaches, the first loop was located at the stop line and the leading edge of the second loop was located 200 ft upstream. The location of this upstream detector was based on the "typical" detector design described by Venglar *et al.* (*5*). Although a stop line loop on the internal segment is not shown by Venglar *et al.* in the typical design, Alternative Sequence 17 is programmed to use such a detector (probably to avoid trapping left-turn vehicles from Phases 4 or 8 at the internal stop lines) so it was included in the design.

Figures 12 and 13 show the results of the evaluation. Figure 12 compares the delay for each phase, averaged across all volume scenarios. Figure 13 compares the delay for each volume scenario, as averaged across all phases. The trends shown in these figures suggest that there is no significant difference in the performance between the two control modes. The results shown in Figures 12 and 13 are for a 720-ft ramp spacing; however, nearly identical results were obtained for the 360-ft spacing.

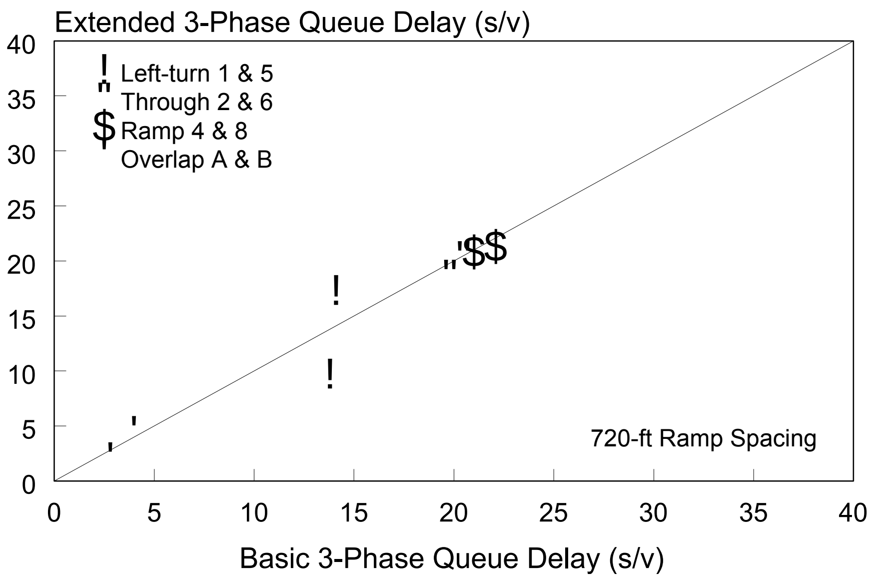


Figure 12. Comparison of basic and extended three-phase sequences based on phase delay.

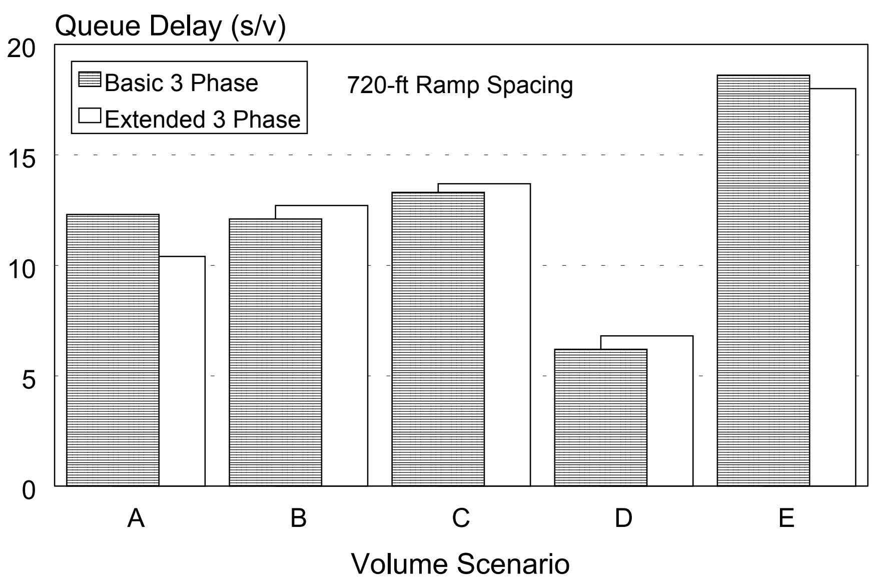


Figure 13. Comparison of basic and extended three-phase sequences based on volume scenario.

Evaluation of Proposed Controller Settings

This section describes the results of an evaluation of the minimum and maximum green settings developed in a previous section. This evaluation is based on an examination of delays that result from various setting values as applied to a diamond interchange using the basic three-phase sequence and an actuated controller.

The delay reported in this section is "control delay" to be consistent with that obtained from the delay equations provided in the *HCM* (*1*). The queue delay obtained from the CORSIM output was converted to control delay by increasing it by 30 percent (i.e., control delay = 1.3 x queue delay). This conversion constant was derived from a report by Showers and Courage (*11*) that compared CORSIM and *HCM* delays.

Minimum Green Setting

Left-Turn-Phase Minimum Green Setting. For this examination, the duration of the left-turn phase was varied over a range of typical values. Each specific phase duration evaluated was obtained by setting the minimum and maximum green intervals at a common value. It is recognized that this action defeats the purpose of actuated control; however, its intent was to examine the potential disruption to platoon progression that could result from "short" left-turn phases (i.e., those whose green duration is less than that predicted by Equation 18). The results of this evaluation are shown in Figure 14 as they relate to volume scenario "e." Similar trends were found for the other volume scenarios.

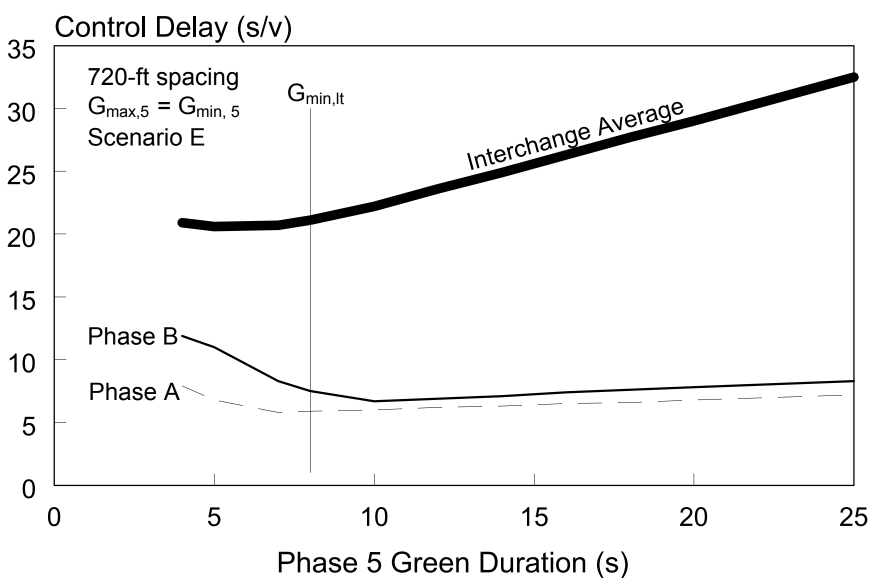


Figure 14. Effect of left-turn phase duration on delay.

The trends shown in Figure 14 offer evidence of the significant delays that can occur to the cross-road through movements at the downstream ramp junction (served as Overlaps A and B) *when* the minimum green for Phase 5 is short. For the 720-ft ramp spacing evaluated, the minimum green setting $G_{min,\,lt}$ for Phase 5 is estimated as 8.0 s (based on a 5-s change interval and a 40 ft/s approach speed). As shown in Figure 14, this setting is associated with near-minimum delays (and good progression) for the cross-road through movement and near-minimum delay for the interchange.

Cross-Road Through-Phase Minimum Green Setting. For this examination, the minimum green setting for one cross-road through phase was varied over a range of typical values. To demonstrate the potential problem fully, volume scenario "c" was modified to eliminate the U-turn volumes and to make the cross-road volumes balanced. For this latter modification, the Phase 6 volume was reduced from 1,100 veh/h to 700 veh/h and the minimum green setting for Phase 2 was set at 5.0 s. Figure 15 shows the results of this evaluation.

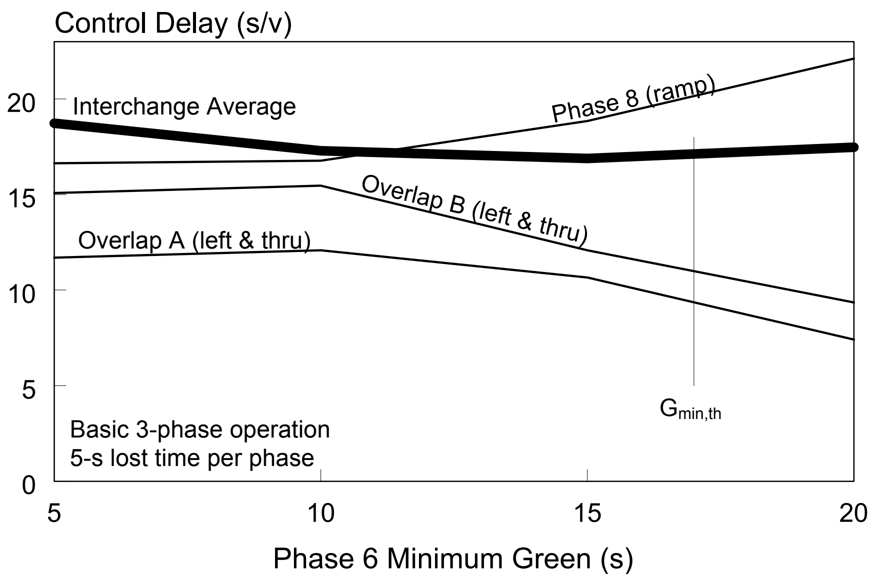


Figure 15. Effect of through-phase minimum green setting on delay.

As the trends in Figure 15 indicate, the duration of Phase 6 has some impact on the delay to the progressed cross-road movements (as they travel through the downstream ramp junctions during Overlap A or B). The delays to Phases A and B generally decrease as the minimum green setting is increased; however, the delays to other interchange movements increase as a result of the longer cycle length. It appears that interchange delay is lowest when the minimum green is about 15 s, which is slightly less than the minimum green setting of 17 s obtained from Equation 19 (with l_s = 2.0 s, D_{1d} = 100 ft, L_r = 720 ft, and V_a = 40 ft/s).

Maximum Green Setting

The guidance provided in Equation 16 regarding the maximum green setting was evaluated to determine if it could yield low-delay operation. For this examination, the maximum green setting was varied over a range of reasonable values. The maximum green setting that yielded the least delay was then compared with the minimum-delay pretimed green interval duration G_o (where G_o is obtained from Equations 1 and 2 when $C = C_o$). This comparison was undertaken because previous research had indicated that there was a correlation between G_o and the G_{max} that produced minimum delay. Specifically, the ratio $f = G_{max} / G_o$ was computed for each maximum green setting considered in the evaluation.

The equilibrium cycle length was also determined for each volume scenario to determine if the relationship between G_o and G_{max} changed depending on whether C_{eq} was greater or less than 1.3 C_o (as suggested by the development of Equation 16). However, it was discovered that all five volume scenarios produced an equilibrium cycle length C_{eq} less than 1.3 C_o. In order to overcome this limitation, the passage time was increased from 0.0 s for all scenarios. For scenario "e," it was found that increasing the passage time to 1.0 s increased the resulting equilibrium cycle length such that it exceeded 1.3 C_o. Similar passage times produced the same result for the other scenarios.

The results of the evaluation are shown in Figure 16, as they relate to volume scenario "e." The delay shown is a volume-weighted average of all interchange movements. The dashed trend line relates to a passage time of 0.0 s; the thick trend line relates to a passage time of 1.0 s. Similar trends resulted from the other volume scenarios.

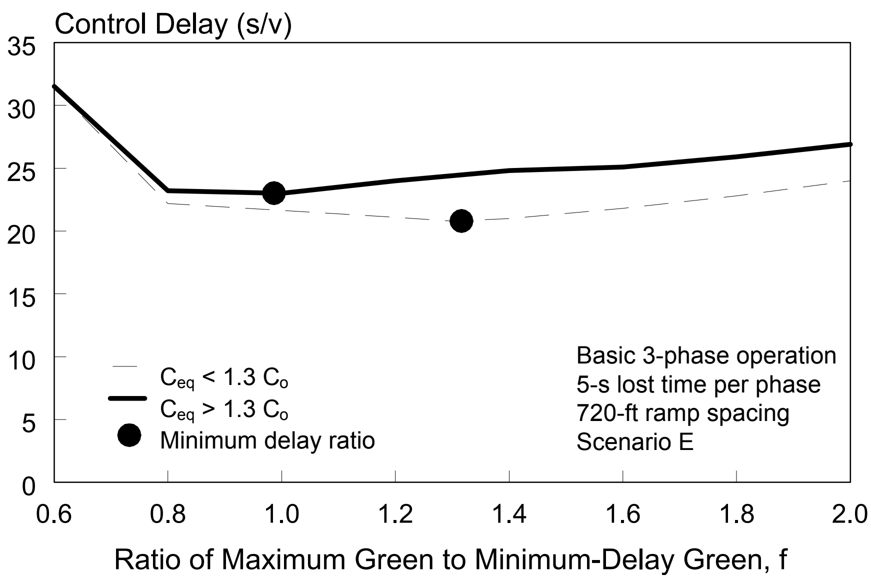


Figure 16. Effect of maximum green setting on delay.

The trends shown in Figure 16 suggest that delay varies with the maximum-green/minimum-delay-green ratio *f*. When C_{eq} is less than 1.3 C_o, the minimum delay ratio occurs at about 1.3 which falls in the range recommended by Kell and Fullerton (*8*) (i.e., 1.25 to 1.50). The ratios associated with minimum delay for the other volume scenarios were found to range from 1.3 to 1.8, with only a small increase in delay for values larger than 1.8.

When the passage time was increased such that C_{eq} was larger than 1.3 C_o, the minimum delay ratio occurred at about 1.0, as shown in Figure 16. This finding is consistent with the rationale used in developing Equation 16. Specifically, this equation indicates that the maximum green interval should equal the minimum-delay pretimed green interval duration G_o when the equilibrium cycle exceeds 1.3 C_o. The rationale is based on the observation that, if C_{eq} exceeds 1.3 C_o, the most efficient operation is achieved when the interchange phase durations are constrained such that an equivalent pretimed operation is achieved.

Cycle Length

As a final examination, the accuracy of the equilibrium cycle length equation (i.e., Equation 26) was evaluated. An accurate version of this equation is needed in order to apply the proposed procedure for determining the maximum green setting, as described by Equation 16.

For this examination, the five volume scenarios were simulated at 360-ft and 720-ft ramp spacings. In each instance, the maximum green setting for each phase was set to 190 s; a value that was sufficiently large as to allow the interchange to achieve equilibrium (i.e., no phase terminated by gap-out). The minimum green setting for each phase was set to 5.0 s. The results of this examination are shown in Figure 17.

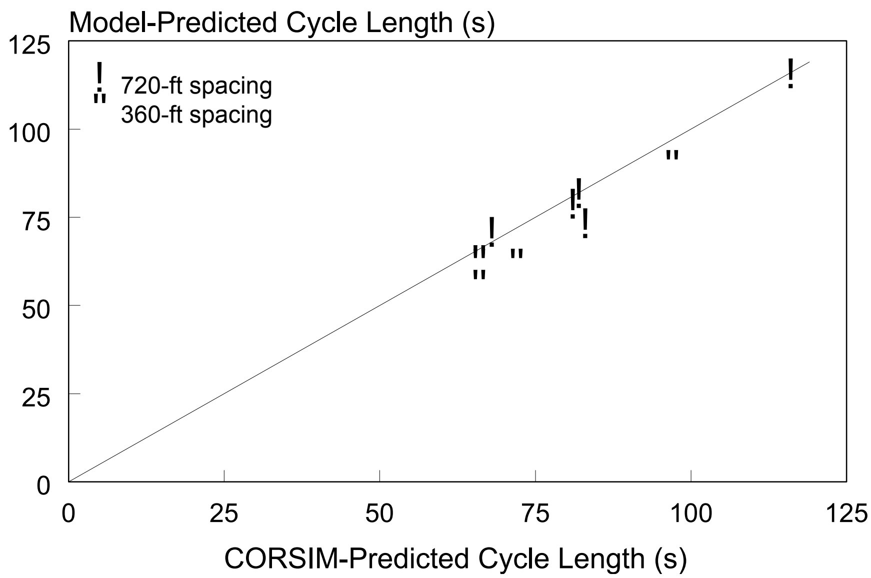


Figure 17. Relationship between CORSIM-predicted and model-predicted equilibrium cycle lengths.

The trends shown in Figure 17 indicate that Equation 26 can predict the equilibrium cycle length with reasonable accuracy. Equation 26 predicted an equilibrium cycle length for the low-volume scenario (i.e., scenario "d") that was much lower than that obtained from CORSIM. On closer examination, it was noted that the sum of critical flow ratios (i.e., $\Sigma\ (v/s)_{ci}$) was about 0.03, which was much lower than the limit of 0.20 discussed with regard to Equation 14. Specifically, critical flow ratios below 0.20 violate the assumptions made when developing Equation 26 (and 14). Critical flow ratios below about 0.20 are associated with significant "dwell" time during the phase and result in an increased cycle length. This dwell time can be estimated using the following equation:

$$\Delta C_{eq} = \frac{3600}{v_1 + v_2 + v_4 + v_5 + v_6 + v_8} \qquad \textbf{(27)}$$

where:
ΔC_{eq} = additional cycle time due to dwell, s; and
v_i = traffic demand volume for Phase i, veh/h.

Thus, for scenario "d," the equilibrium cycle length shown in Figure 17 represents the sum of C_{eq} and ΔC_{eq}, as obtained from Equations 26 and 27, respectively.

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

The findings from this project led to the following conclusions regarding actuated diamond interchange operation:

1. There exists an equilibrium cycle length C_{eq} for all actuated intersections and interchanges. If this cycle length exceeds the minimum-delay pretimed cycle length C_o by 30 percent then the delay-reducing benefits of actuated operation are minimal relative to a well-timed pretimed operation. When C_{eq} exceeds 1.3 C_o, efficient operation can be realized when the maximum green setting is limited to the minimum-delay pretimed green interval duration G_o (i.e., $G_{max} = G_o$).

2. If the equilibrium cycle length is less than 1.3 C_o then actuated operation will yield lower delay than pretimed operation.

3. The unique phase relationships dictated by the interchange geometry and basic three-phase sequence introduce the need for minimum and maximum green intervals in order to promote efficient cross-road progression and limit the frequency of queue spillback.

4. The basic three-phase sequence offers similar performance to the extended three-phase sequence implemented in the Eagle EPAC 300 controller (i.e., Alternate Sequence 17). An advantage of the basic three-phase sequence is that it can be implemented using any NEMA TS-2-based controller.

Recommendations

Based on this project, the following equations are recommended for use in timing an actuated diamond interchange with a basic three-phase sequence:

Minimum Green Settings

Left-Turn Phase. The following equation can be used to estimate the minimum green setting for the left-turn movements at a diamond interchange:

$$G_{\min,lt} = \frac{L_r}{V_a} - Y - 5 \quad \textbf{(28)}$$

where:

$G_{min,\,lt}$ = minimum green setting for the cross-road left-turn phase (1 or 5), s;
L_r = distance between ramp terminals (measured from stop line to stop line), ft;
V_a = vehicle approach speed, ft/s; and
Y = yellow and all-red intervals (i.e., change interval) (typically, 3.0 to 5.0 s), s.

The minimum green setting computed in this equation should be applied to the critical left-turn phase (i.e., that phase with the larger flow ratio) *and* the left-turn phase with the smaller flow ratio. In some cases, this may be the same phase.

Cross-Road Through Phase. The following equation can be used to estimate the minimum green setting for the through movements at a diamond interchange:

$$G_{\min,th} = l_s + \frac{L_r - D_{1d}}{V_a} \tag{29}$$

where:

$G_{min,\,th}$ = minimum green setting for the cross-road through phase (2 or 6), s;
D_{1d} = distance between the downstream stop line and the leading edge of the advance detector furthest from this stop line (as measured on the cross road between ramp junctions), ft; and
l_s = start-up lost time (typically, 2.0 s), s.

The minimum green setting computed in this equation should be applied to the critical cross-road through phase.

All Phases. The minimum green setting for all interchange phases should be set as small as possible. However, this setting should not be so short that it violates driver expectancy. Values between 4 and 8 s are consistent with driver expectancy needs and should be used as absolute minimums for all phases. Moreover, a minimum green setting between 4 and 8 s should be used whenever the value obtained from Equation 28 or 29 is less than this range.

Maximum Green Settings

All Phases. The following equation can be used to estimate the maximum green setting for interchange phases:

$$\begin{aligned} &If\ C_{eq} < 1.3\ C_o \quad then\ G_{\max} = Larger\ of\text{: } [G_{\min} + 10,\quad 1.3\ G_o] \\ &otherwise\ G_{\max} = Larger\ of\text{: } [G_{\min},\quad G_o] \end{aligned} \tag{30}$$

where:

G_{max} = maximum green setting, s;
C_{eq} = equilibrium cycle length for an actuated controller, s;
C_o = minimum-delay cycle length for pretimed control (from PASSER III or Equation 13), s;
G_{min} = minimum green interval setting, s; and
G_o = minimum-delay pretimed green interval duration (where G_o is obtained from Equations 1 and 2 when $C = C_o$).

Equation 26 (described previously in this report) should be used to estimate the equilibrium cycle length needed to determine the maximum green setting.

Cross-Road Through Phase. The maximum green setting for Phases 2 and 6, as determined from Equation 30, should not exceed the value obtained from the following equation:

$$G_{\max,th} = \frac{n_{lt}\, L_b\, 3600}{p_{ltcs}\, n_t\, L_q\, s} + l_s \qquad \textbf{(31)}$$

where:

$G_{max,\,th}$ = maximum green setting for a cross-road through phase (2 or 6), s;
n_{lt} = number of exclusive left-turn lanes in the opposing direction of travel;
n_t = number of through lanes on the cross road in the subject direction of travel;
p_{ltcs} = ratio of left-turn volume in the opposing cross-road traffic stream to total volume in the subject direction of travel (i.e., $p_{ltcs} = v_1 / v_2$ or $p_{ltcs} = v_5 / v_6$);
L_b = length of the lane(s) for exclusively storing left-turn vehicles, ft;
L_r = distance between ramp terminals (measured from stop line to stop line), ft;
L_q = length of a traffic lane occupied by a stopped vehicle (typically, 25 ft), ft;
s = saturation flow rate per lane (typically, 1,800 veh/h/ln), veh/h/ln; and
l_s = start-up lost time (typically, 2.0 s), s.

This maximum green setting should be imposed on both through movements.

Off-Ramp Phase. The maximum green setting for Phases 4 and 8, as determined from Equation 30, should not exceed the value obtained from the following equation:

$$G_{\max,rp} = \frac{n_t\, L_r\, 3600}{p_{ltrp}\, n_{rp}\, L_q\, s} + l_s \qquad \textbf{(32)}$$

where:

$G_{max,\,rp}$ = maximum green setting for the off-ramp phases (4 and 8), s;
n_{rp} = number of lanes on the off-ramp;
n_t = number of through lanes on the cross road in the subject direction of travel; and
p_{ltrp} = fraction of left-turn vehicles in the off-ramp traffic stream.

This maximum green setting should be imposed on both off-ramp movements.

Delay Computation

If the equilibrium cycle length C_{eq} is less than 1.3 C_o and the maximum green setting exceeds the minimum delay pretimed green interval duration G_o by 30 percent or more (i.e., $G_{max} \geq 1.3\ G_o$) then the delay for an actuated movement can be computed using Equation 9 (i.e., $d = d_1$). If these conditions are not satisfied or if the movement has pretimed control, then the delay should be computed using Equations 9 and 10 (i.e., $d = d_1 + d_2$) with the incremental delay factor k in Equation 10 set equal to 0.5.

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