

Background Paper No. 9

SIGNALIZED INTERSECTION SPACING

prepared for the

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by the

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Background Paper No. 9

SIGNALIZED INTERSECTION SPACING

DISCLAIMER

This background paper represents the viewpoints of the authors. Although prepared for the Oregon Department of Transportation (ODOT), they do not represent ODOT policies, practices nor procedures.

GENERAL OBJECTIVE

This and other background papers were prepared for the purpose of stimulating discussion among interested individuals representing a variety of agencies having an interest in Oregon's highways.

SPECIFIC OBJECTIVE

The specific objectives of this background paper are to:

1. Summarize the information relative to the need for long, uniform signal spacing;
2. Identify the effects on traffic flow and progression when signalized intersections deviate from an adopted uniform spacing interval; and
3. Explore the problem of platooned flow through a signal system.

ACKNOWLEDGMENTS AND CREDITS

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WHAT DOES IT ALL MEAN?

Traffic Signal Synchronization

Synchronization of traffic signals to provide progression of traffic along an arterial has been shown to substantially reduce stopped delay and fuel consumption. The synchronized traffic signal system must be able to be timed to respond to different volume and travel speed conditions throughout the day (i.e., peak and off-peak periods) as well as temporal changes in turn and through volumes that occur over time.

Signal Spacing

The range of progression speeds and cycle lengths increase with increased signal spacing. For example, consider the following:

Spacing		Cycle Length	Progression Speed	
km	(mile)	(seconds)	km/h	(mph)
0.402	$\frac{1}{4}$	60	49	30
		120	25	15
0.804	$\frac{1}{2}$	60	95	58
		120	49	30

A 60-second cycle length which is appropriate for low-volume conditions results in a progression speed of only 49 km/h (30 mph) with signalized intersections spaced at 0.402 km (0.25-mile) intervals. A 120-second cycle, commonly used for high-volume conditions, results in a progression speed of only 25 km/h (15 mph). Maximum flow rate (vehicles per hour) as well as minimum fuel consumption occurs at speeds which are higher than can be achieved with efficient progression efficiency (progression band width divided by cycle length) of 60 seconds or longer. These progression speeds are slower than many drivers attempt to travel (i.e., average attempted speeds on urban arterials generally exceed 49 km/h in the off-peak and exceed 25 km/h in the in peak periods). Thus, short signal spacings, even at uniform intervals, will produce increased platoon dispersion. This will cause an increase in the variation in speed of vehicles in the platoon, reduce efficient progression, and substantially increase fuel consumption.

A uniform signal spacing of 0.804 km (0.50 mile) permits use of a long cycle length in peak periods while providing a progression speed which yields a reasonable traffic flow rate and fuel efficiency. The 0.804 km spacing also allows traffic engineers to utilize shorter cycle lengths during off-peak periods which yield high progression efficiency at an average speed which drivers will accept. Thus, limiting platoon dispersion in both peak and off-peak periods.

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WHAT DOES IT ALL MEAN? (Continued)

Platoon Dispersion

Yes, platoon dispersion occurs, but it is not as bad as it might seem. Platoon dispersion increases with increased travel time, not distance between signals per se. Platoon dispersion models show that greater dispersion occurs over a 0.402 km distance at, say, 30 km/h than over a 0.804 km distance at, say, 70 km/h (43 mph).

Signal spacing		Speed		Travel Time
km	(miles)	km/h	(mph)	(seconds) ⁽¹⁾
0.402	¼	30	18	48.2
0.804	½	70	43	41.3

(1) Calculated using SI units

Carroll Messer (33) a principal developer of PASSER II and a participant in development of the Highway Capacity Manual suggests that platoon integrity (little platoon dispersion) can be maintained for a travel time of up to two minutes between signals. Thus, at 30 km/h or 8.3333 m/s (18 mph or 26.46 fps) good platoon integrity may be obtained up to a distance of about one kilometer (0.8 mile). At higher speeds, platoon integrity may be maintained for even longespacings.

Less platoon dispersion, higher platoon densities and higher flow rates are achieved in larger, as compared to smaller, cities. This undoubtedly occurs because driver discipline improves when they become accustomed to the higher levels of “traffic pressure”. Moreover, driver performance in maintaining platoon integrity improves when excellent progression is provided and drivers become familiar with the progression timing. Observations on Telegraph Road in the Detroit metropolitan area and South Wadsworth in the Southwest Denver area indicate that very high levels of platooned flow is maintained over long distances. Experience has shown, for example, that one can drive Telegraph Road for a distance of 10 miles with little or no stopping.

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SIGNALIZED INTERSECTION SPACING

OVERVIEW

Introduction

Major arterials intended to provide a high degree of mobility and serve the longer trips. Since movement, not access, is their principal function, access management is essential in order to preserve capacity and safety. This principle was recognized in the 1984 edition of A Policy on the Geometric Design of Highways and Streets, (the AASHTO Green Book). Further, the adoption of functional design, in lieu of volume based design, represented a major change in the philosophy of planning and design of street and highways systems.

As a specific part of a pre-lease agreement, many commercial lessees and businesses demand that the developer obtain access commitments that include signals and direct access driveways. The transportation agency must be able to act by its standards and regulations in order to provide a safe and efficient highway.

Principal Discussion Topics

Topics of discussion involving this paper include the following:

1. Why long uniform signalized intersection spacing is essential to the efficient movement of traffic on major urban streets?
 2. What are the parameters which control the efficient flow of traffic?
 3. When is it essential that the selected long uniform interval be maintained? And when can deviations be accommodated?
-

Questions to be Addressed

Specific questions which need to be answered include the following:

1. What uniform signal spacing(s) should be adopted for Oregon highways in urban areas? Why?
2. Should the same spacing be used on all classes of highways? Why or why not?
3. What is the effect of deviating from the selected long uniform signal spacing, or multiples of this interval?
4. If it is necessary to deviate from the adopted signal spacing standard:
a) How much deviation may be allowed? b) Can compensation be

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SIGNALIZED INTERSECTION SPACING

OVERVIEW (Continued)

**Questions
to be
Addressed
(Continued)**

made so as to maintain efficient traffic progression when a signal location does not conform to the standard? And, if so how? c) What criteria are to be met when a deviation is proposed? d) What documentation is to be provided?

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Signalized Intersection Spacing

OPTIMAL SIGNAL SPACING

Controlling Parameters

The variables involved in the planning design and operation of signalized arterial streets are:

- Speed
 - Cycle length
 - Signal spacing
 - Efficiency of progression
-

Signal Spacing Considerations

Major arterials in developed urban areas experience very high travel demand during the morning and after work peak periods. Hence, capacity is always an issue once the property in the vicinity of the major street becomes fully urbanized. It has long been recognized that maximum flow rates are achieved when traffic is moving at a uniform speed of about 55 km/h (35 mph) to 65 km/h (40 mph). It might also be noted that fuel consumption and emissions are also minimum under these conditions. During off-peak conditions a major arterial should operate at speeds of between 70 to 85 km/h (45 to 55 mph). Thus, a signal timing plan must be able to provide efficient traffic flow at different speeds and operate efficiently within a range of cycle lengths. During off-peak conditions, a cycle of about 60 seconds is commonly employed. Longer cycle lengths are used during the high volume peak periods in order to minimize the “lost time” which occurs each time the signal indication is changed. This includes the delay between the beginning of the green indication and when the first vehicle enters the intersection as well as headway between successive vehicles as the queue begins to move and before the minimum headway is achieved.

The only variable is the efficiency of traffic progression (progression band width divided by cycle length). The higher the efficiency, the higher the capacity. Moreover, at high efficiencies, fewer vehicles are required to come to stop. Vehicle deceleration and acceleration as well as idling time are reduced; thus, the vehicle emissions, fuel consumption, and delay are minimized. Traffic operations personnel will need to be able to efficiently operate the signal system under a variety of conditions as traffic volumes increase as an urban area continues to develop, in response to different traffic volumes during peak and off-peak periods and to respond to temporal changes in the percentage of through turning traffic.

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OPTIMAL SIGNAL SPACING (Continued)

Optimum Spacing

Optimum combinations of speed and cycle length for 0.804 km (1/2-mile) 0.536 (1/3-mile) and 0.402 (1/4 mile) spacings were determined using the PASSER II-90 program to maximize the progression band width and provided the optimum cycle length. The resulting output was used to prepare the curves shown in Figure 1. Spacings for various cycle lengths and progression speeds are also given in Table 1.

Capacity will always be an issue on major urban arterials once an area develops. Therefore, the progression speed needs to be selected so that high flow rates (vehicles per hour) can be achieved. Maximum flow rates occur at speeds of about 65 km/h (40 mph). Moreover, the flow rate decreases when speeds fall below 55 km/h (30 mph). Thus, 50 km/h (30 mph) is the lowest speed if a major arterial is to accommodate high traffic volumes while limiting fuel consumption and emissions.

Even with 1/3 mile spacing and a cycle length of 90 seconds, speeds will be less than 30 mph. Progression at reasonable speeds can be achieved at short signal spacings such as at 0.402 km (1/4-mile) only so long as the traffic volumes are very low and short cycles (60 seconds or less) can be used. Inspection of Figure 1 reveals that a 60-second cycle will result in a progression speed of about 50 km/h (30 mph). Thus, it is evident from Figure 1 that long signal spacings increase the flexibility with which a signal system can be timed to accommodate different traffic demand at various times of the day as well as changing traffic volumes in future years.

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SIGNALIZED INTERSECTION SPACING

OPTIMAL SIGNAL SPACING (Continued)

**Optimum
Spacing
(Continued)**

**Table 1: Optimum Signal Spacing as a
Function of Speed and Cycle Length**

Cycle Length (seconds)	Speed, mph						
	25	30	35	40	45	50	55
	Feet						
60	1,100	1,320	1,540	1,760	1,980	2,200	2,420
70	1,280	1,540	1,800	2,060	2,310	2,590	2,830
80	1,470	1,760	2,060	2,350	2,640	2,940	3,230
90	1,650	1,980	2,310	2,640	2,970	3,300	3,630
100	1,840	2,200	2,570	2,940	3,300	3,670	4,040
110	2,020	2,420	2,830	3,230	3,630	4,040	4,440
120	2,200	2,640	3,080	3,520	3,960	4,400	4,840
Cycle Length (seconds)	Speed km/h						
	40	48	56	64	72	80	88
	Metres						
60	330	400	470	530	600	670	730
70	390	470	540	620	700	780	860
80	440	530	620	710	800	890	980
90	500	600	710	800	900	1,000	1,100
100	560	670	780	890	1,000	1,110	1,220
110	610	730	860	980	1,100	1,220	1,340
120	670	800	930	1,070	1,200	1,330	1,470

A 120-second cycle is commonly needed during peak periods in order to limit the time “lost” due to changing traffic signal indications (green, yellow, red) plus the “starting delay” experienced until successive vehicles on the queue/platoons enter the intersection at a uniform headway. As shown in Table 2, a 60-second cycle results a total lost time per hour which is about 15% of the hour. In other words, only about 3050 seconds of the 3600 seconds in an hour are used for moving traffic. A 120-second cycle has one-half the lost time and provides. It provides the ability to move about another 135 to 140 vehicles per hour ($276 \text{ sec}/2 \text{ sec per veh.} = 138 \text{ veh.}$) through the intersection. Cycle lengths longer than 120 seconds result in diminishing traffic throughout and substantially increase stopped delay (together with the additional wasted fuel and increased vehicular emissions). Thus, a 120-second cycle is commonly accepted as the longest cycle length for general application. Longer cycle lengths are used by some traffic engineers of congested intersections where traffic progression is not a consideration.

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OPTIMAL SIGNAL SPACING (Continued)

Optimum Spacing (Continued)

Inspection of Figure 1 (a vertical line at 120 seconds and a horizontal line at 30 mph) shows that the optimal signal spacing for 48 km/h (30 mph) and a 120-second cycle is 0.804 km (one-half mile). Inspection of Table 2 also shows that the optimal spacing is 2640 ft., or one-half mile. As will be discussed later, it is important to recognize that the optimal spacing is exactly 2640 ft. for a 120-second cycle and 48 km/h (30 mph) speed.

**Table 2: Comparison of “Lost Time” per Hour for
60-Second and 120-Second Cycles**

Event	60-Second Cycle			120-Second Cycle		
	Number of Events per Cycle	Lost Time per Cycle (seconds)	Lost Time per Hour (seconds)	Number of Events per Cycle	Lost Time per Cycle (seconds)	Lost Time per Hour (seconds)
Yellow Interval	60	3	180	30	3	90
All-Red Clearance	60	2	120	30	2	60
Starting Delay ⁽¹⁾	60	4.2	252	30	4.2	126
Total Lost Time			552	276		
Lost Time as % of Hour			15.3%	7.7%		
Movement Time per Hour			3048 seconds	3324 seconds		

(1) Starting Delay

Position of Vehicle	Headway (sec)	Minimum	
		Headway (sec)	Delay (sec)
1	--	--	3.2
2	2.7	2.0	0.7
3	2.3	2.0	0.3
4	2.0	2.0	0
5	2.0	2.0	0
6	0	0	0
7	0	0	0
7	2.0	2.0	0
TOTAL			4.2

SIGNALIZED INTERSECTION SPACING

OPTIMAL SIGNAL SPACING (Continued)

Optimum
Spacing
(Continued)

**Figure 1: Relationship Between Speed,
Cycle Length and Signal Spacing**

Source: References (1) Stover, et al, 1991; (2) Stover & Koepke, 1988;
and (3) Marks, 1971.

SIGNALIZED INTERSECTION SPACING

OPTIMAL SIGNAL SPACING (Continued)

**Off-Peak
Cycle
Length**

A 0.804 km (1/2-mile) spacing enables the implementation of timing plans which will result in appropriate off-peak progression speeds at cycle lengths that are appropriate for use with off-peak traffic volumes. Inspection of Figure 1 shows that a 65-second cycle will provide progression at a speed of 85 km/h (55 mph) whereas an 80-second cycle provides progression speed of 70 km/h (45 mph).

Selection of the minimum cycle length based on off-peak volume may result in an inappropriately high speed once a long signal spacing has been established. In this case, a longer cycle length should be used. This will result in modest increase in delay to left-turning and crossing traffic on the minor cross street but will not affect traffic on the major street. For example, as shown in Figure 2, if a 60-second cycle length is chosen for the off-peak period on an arterial with signalized intersection spacing of (0.804 km) ½ mile, the speed necessary to provide progression is approximately (93 km/h) 58 mph. This speed is unacceptable on an urban arterial. Thus, the cycle length must be lengthened to provide for slower speeds. Also, if a speed of 80 km/h (50 mph) is desired, the appropriate cycle length is approximately 70 seconds.

**Progression
Efficiency**

As previously indicated, progression efficiency tends to increase as the cycle length increases because there are fewer phase charges per hour, hence “lost time” is reduced and the saving can be added to the progression band. As illustrated in Table 3, the maximum efficiency that can be achieved when there is no lead/lag minor or separate phase for turn movements is on the order of 45%. If a separate 10-second phase is provided for left-turns, a 60-second cycle will result in a progression efficiency of less than 30% (27% to 28% as given in Table 3). This will result in only about 1008 seconds of progression flow per hour (60 sec. cycle x 0.28 x 60 cycle/hr).

Where moderate to high-turn volumes are encountered, longer left-turn phases will be required and a 60-second cycle will result in even poorer progression efficiency. With a 120-second cycle, a 10-second left-turn phase will allow for a progression efficiency of nearly 40% (38% or 39% from Table 3). The number of seconds of progression is about 1368 seconds (120 sec. cycle x 0.38 x 30 cycles/hr. = 1368 sec.). This is an additional 300 seconds or 5 minutes per hour, or 37% more than with a 60-second cycle.

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SIGNALIZED INTERSECTION SPACING

OPTIMAL SIGNAL SPACING (Continued)

Progression
Efficiency
(Continued)

Figure 2: Selection of Peak Period Cycle Length Once Uniform Signalized Intersection Spacing Has Been Established

Source: Adapted from References (1) Stover, et al; (2) Stover & Koepke, 1988.

SIGNALIZED INTERSECTION SPACING

OPTIMAL SIGNAL SPACING (Continued)

Progression
Efficiency
(Continued)**Table 3: Maximum Progression Efficiency at Optimal Signal Spacing for Different Cycle Lengths, Exclusive Left-Turn Time, and Duration of Clearance Interval**

Cycle Length (Seconds)	Duration of Exclusive Left-Turn Phase (Seconds)	Clearance Interval (Seconds)	Potential Progression Efficiency ⁽¹⁾ (%)
60	0	3	45
	0	4	43
	10	3	28
	10	4	27
	15	3	20
	15	4	18
90	0	4	46
	10	3	36
	10	4	34
	15	3	30
	15	4	29
120	10	3	39
	10	4	38
	15	3	35
	15	4	34
	20	3	31
	20	4	30

(1) Progression band width divided by cycle length

Source: Reference (1) Stover, et al, 1991, p. 120.

However, cycle lengths longer than 120 seconds, even under very high volume conditions, are rarely desirable because the longer red phases stop more vehicles. The increased stopped delay more than offsets the benefits of the marginal saving on lost time as a percent of cycle length.

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SIGNALIZED INTERSECTION SPACING

OPTIMAL SIGNAL SPACING (Continued)

**Providing
Flexibility to
Respond to
Changing
Traffic
Conditions**

The flexibility to select continuous cycle lengths and progression speed while maintaining traffic progression increases as signalized intersection increases. The need to move large traffic volumes during peak periods is, or will become, a significant issue as urbanization occurs. Long and uniform signal spacing is the key to being able to provide efficient movement during high volume peak periods as well as off-peak periods in developed urban areas. The development of long and uniform signal spacing is also essential in smaller urban areas in order to be able to adjust to the increasing traffic volumes and changing traffic patterns that will occur as the area grows in population and employment.

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SIGNALIZED INTERSECTION SPACING

TRAFFIC SIGNAL SYNCHRONIZATION

**Closely
Spaced
Uncoordinated
Signals**

Chapter 9 of the Highway Capacity Manual (HCM) deals with isolated signals (a signal which is so far removed that traffic arriving at it is not influenced by any other signal). Chapter 11 of the HCM deals with capacity and level-of-service where there are a series of signals along the roadway which interact with each other. Signal timing optimization programs such as PASSER II and TRANSYT-7F are used to optimize the timing plans to obtain efficient traffic flow.

All of these procedures or models are designed to develop or evaluate the degree of progression. For example, Chapter 11 of the HCM involves the use of a progression adjustment section. One can not take this factor and apply it to situations where closely spaced traffic actuated signalized intersections indicates that their proximity results in situations which are “worse than would occur if traffic at each was random”. The sequence of the green phases at three successive signals may at times allow through traffic to move through the system. However at another time, no traffic may arrive at the downstream signal (signal #3) on the green. Subsequently the release of traffic at the upstream signals is such that the leading platoon departing signal #1 arrives at signal #3 so that there is no gap between the end of the platoon from signal #2 and the leading edge of the platoon from signal #3. As a result a long queue is stopped at signal #3 and the queue backup through signal #2.

An analogy using a bus transit system where strict adherence to the schedule for each bus is not followed. Bus #1 arrives at a bus stop early. Since it is early there are few passengers waiting and boarding time takes less than scheduled. Bus #1 arrives at successive bus stops even sooner and therefore picks up few passengers. When Bus #2 at the stop passengers which missed Bus #1 plus those passengers arriving for the scheduled stop of Bus #2 are waiting; boarding time takes longer than scheduled and Bus #2 runs later and later at subsequent stops.

The bus problem is avoided by strict enforcement of the schedule (not running ahead of schedule). Traffic signal synchronization addresses a similar problem. While the progression may not be ideal, it will maximize the use of the green phase for through traffic at each signal.

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SIGNALIZED INTERSECTION SPACING

TRAFFIC SIGNAL SYNCHRONIZATION (Continued)

System Criteria for Traffic Progression	Minor signals are often not spaced at the interval necessary to provide efficient progression at the desired speeds and cycle lengths. In such cases it is often possible to provide additional lanes at the intersection to accommodate the traffic with less cross-street green.
--	--

A “system” approach may also be used in setting the progression speed and efficiency as is the practice of Charlotte, North Carolina. The major arterial-to-major-arterial intersections establish the cycle length, progression speed and offsets. This establishes the major street green needed to maintain progression; the minor cross-street green plus yellow is the remainder of the cycle.

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SIGNALIZED INTERSECTION SPACING

DEVIATION FROM STANDARD INTERVAL

Issues and Problems

Once a standard interval for signalized intersection spacing has been selected, it must be recognized that it will be necessary to deviate from the spacing standard in some situations. It is also important to realize that any deviation will interfere with efficient movement (and this increases delay, fuel consumption and emissions) unless compensation can be made for the “misplacement”.

Effect of Deviation from a Uniform Interval

The progression efficiency curves shown in Figure 3 were developed from numerous PASSER II-90 runs. Approach volumes were assumed to be 10% left-turn, 80% through, and 10% right-turn. The exclusive left-turn phases result in the maximum progression efficiency to be less than 50%. (If there were no left-turns or if all left-turns were on green while yielding to opposing traffic, the maximum progression efficiency would be 0.50.) The PASSER runs were made at that spacing which would yield maximum progression efficiency for a given combination of speed and cycle length (such as 30 mph and 60 sec.; 30 mph and 120 sec.). Deviations in spacing were then made at 25 ft. intervals (2640; 2615 and 2665; 2590 and 2690; etc.) In addition to the 60-second and 120-second cycle lengths shown in Figure 3, runs were also made for a 90-second length.

Visual inspection of Figure 3 reveals the following:

- Optimal spacing for all speeds is longer at a 120-second cycle than a 60-second cycle.
- The effect of uniform signal spacings longer than optimal (i.e., optimal + 25 ft., optimal + 50 ft., etc.) is the same as uniform spacings less than optimal (i.e., optimal - 25 ft., optimal - 50 ft., etc.).
- The effect deviation from optimal spacing increases slightly as speed increases. This is evidenced by the wider “spread”, or width base, of the triangle.

The effect of a deviation from optimal spacing as a function of cycle length is not apparent.

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SIGNALIZED INTERSECTION SPACING

DEVIATION FROM STANDARD INTERVAL (Continued)

**Effect of
Deviation
from a
Uniform
Interval
(Continued)**

Figure 3a: 60-Second Cycle Length

Figure 3b: 120-Second Cycle Length

**Figure 3: Progression Efficiency for Various Speeds at Different
Cycle Lengths**

Source: Reference (1) Stover, et al, 1991.

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SIGNALIZED INTERSECTION SPACING

DEVIATION FROM STANDARD INTERVAL (Continued)

Effect of Cycle Length

The progression efficiency deteriorates less rapidly with the longer cycle lengths. For example, at 35 mph (56 km/h) the efficiency drops from about 33% to about 5% with a difference in spacing of ± 200 feet (91 metres) with a 120-second cycle it decreases from 34% to slightly more than 5% with a deviation of ± 400 feet (122 metres) from the optimal 3040 feet (927 metres) spacing. However, the rate of decrease is similar as is illustrated by the following for a 35 mph speed.

Cycle Length (Seconds)	Intersection Spacing (Feet)	Progression Efficiency
60	1540	0.31
	1340 or 1740	0.05
Deviation/Optimal Spacing = $200/1340 = 0.1299$, say 0.13		
120	3040	0.34
	2640 or 3440	0.08

$$\text{Deviation/Optimal Spacing} = 400/3040 = 0.1316, \text{ say } 0.13$$

Inspection of Figure 3 further shows that the effect of deviating from the optimal spacing is greater at 55 mph than at 35 mph for both 60 and 120-second cycles. Where as the ratio of the deviation divided by optimal spacing is about 0.13 at 35 mph it is about 0.17 at 55 mph.

Analysis of the PASSER output indicate that for each 1% a signal deviates from the uniform spacing mitigation in the extent of 2% of the cycle length must be added to the major street green in order to maintain progression on the major street.

Implications for Signal Spacing

Urban arterial street intersections will eventually experience capacity problems when the area becomes fully urbanized. Consequently, it is essential that major arterial-to-major-arterial intersections be spaced, as precisely as close as possible, to multiples of the optimum uniform interval (i.e., 1.608 km, 1 mile). Intermediate signalized intersections frequently experience less traffic on the intersecting cross-street. As a result, these intermediate signals may be located “out of position” to accommodate physical constraints or to match land holding and development patterns. The deviation can be compensated for by (a) increasing the green phase on the major arterial and decreasing the green time devoted to cross-street and/or, (b) the cross-street approach is flared

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DEVIATION FROM STANDARD INTERVAL (Continued)

**Implications
for Signal
Spacing
(Continued)**

to increase the number of lanes on the cross-street at its intersection with the arterial. As a precaution, additional right-of-way should always be acquired even though the need for flaring is not anticipated at the time of construction.

SIGNALIZED INTERSECTION SPACING

EFFECT ON TRAVEL RATE AND DELAY

Introduction

Frequent and/or non-uniform spacings of traffic signals constrain traffic flow and cause excessive delay. In research reported by Guinn (6) in 1967, traffic signal density (signals per mile) and traffic volume per lane were found to be critical variables affecting traffic flow on urban streets. Stover, et al (4) reported compatible findings in 1970. They concluded that operating costs and total user costs decreased as signal spacing increased. Also, longer spacings are needed as the volume per lane increases.

Delay

Staniewicz (12) estimated the delays resulting from reducing the through band based upon a 30 mph progressive speed, an unimpeded arrival by the first vehicle in the platoon, and 2.1-second arrival and departure headways (see Table 4).

Table 4: Illustrative Delays When Traffic Demand Exceeds Bandwidth Capacity

Volume Veh/Cycle/Lane	Volume-to- Capacity Ratio	Capacity of Through Band Veh/Cycle/Lane			
		3	6	9	12
3	.25	0	0	0	0
6	.50	17	0	0	0
9	.75	23	12	0	0
12 ⁽¹⁾	1.00	26	17	9	0

(1)Capacity assumed at 12 vehicles per cycle based on 29 seconds green per 60-second cycle.

First vehicle arrives unimpeded. 2.1 arrival and departure headways. Based progressive speed, 30 mph.

Source: Adapted from Reference (12)
Staniewicz & Levinson, 1995.

Delays will result whenever the approach volume exceeds the number of vehicles that can be accommodated in the through band. The volume-to-band capacity ratio is therefore more significant than the actual v/c ratio in influencing delays. For example, a volume of 9 vehicles per cycle would result in a 12-second delay when the band capacity is 6, while a volume of 6 vehicles per cycle would result in a 17-second delay when the through band is 3 vehicles per cycle. Thus, the data underscore the need for preserving the through band, since its reduction would increase delays even at moderate traffic volumes.

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SIGNALIZED INTERSECTION SPACING

PLATOON DISPERSION

**Platoon
Density**

-v-

**Platoon
Dispersion**

Density is the compactness of the platoon. It is perhaps most visible by observation of the gaps between successive vehicles in the platoon. Density can be measured as: 1) the gaps in seconds between the rear of a vehicle and the front of a following vehicle; 2) the headways (front bumper to front bumper of the following vehicle) in seconds; 3) the flow rate in vehicles per second by short time intervals, say, 10 or 15 seconds; and 4) the number of vehicles occupying a length of roadway and reported as a rate in vehicles per mile.

Platoon dispersion on the other hand is a deterioration in the platoon integrity that is a change in the compactness of a platoon as a function of time. Dispersion can be measured as: 1) a change in the length of gaps between the same pairs of vehicles as the travel downstream, 2) a change in headways, 3) a decrease in the flow rate, vehicles per second, within portions of the platoon (this is the most commonly used measure of dispersion), or 4) a decrease in the number of vehicles per unit of roadway in different portions of the platoon.

Platoon density is lower in small cities than in large urban areas where “traffic pressure” and congestion is higher. Platoon density is also higher in systems which have excellent progression with which drivers are familiar.

Platoon dispersion is less where: 1) the standard deviation of individual driver’s desired speeds is small (the vast majority of drivers want to travel at the same speed), 2) the speed of progression equals the driver’s desired average speed, and 3) left or right-turns do not disrupt the platoon (i.e., there are no opportunities for turns or left-turn and right-turn lanes are provided which allow the turning vehicles to diverge from the through traffic lane without disrupting following vehicles).

SIGNALIZED INTERSECTION SPACING

PLATOON DISPERSION (Continued)

**Measurement
and
Concepts of
Platoon
Dispersion**

Platoon dispersion can be measured, or defined in the following ways:

1. The gaps between successive vehicles. This measure is perhaps the most intuitively logical from casual observation of gaps in the front of the platoon and the gaps at the end of the platoon. Comparison of the average gap (or distribution of gaps) at the front of the platoon compared to the rear of the platoon can be used as a measure of dispersion within the platoon.
2. The difference in the time for passage of the first vehicle and last vehicle in a platoon released at an upstream signal compared to the difference in time between the passage of the first and last vehicle in the same platoon at a downstream signal. Dispersion is easily measured by this and was used by Messer, et al (22) and the Road Research Laboratory (31).
3. The flow rate (vehicles per second). This method can be used to measure dispersion by comparing the average flow rate at a downstream location with the average flow rate at the upstream signal at which the platoon was released. NCHRP Report 339 (29) utilizes this technique.

Dispersion has been found to be a function of the travel time from a signal to a downstream signal (or other downstream location) and the length of the platoon (22). The longer the travel time between signals, the greater the dispersion. This is intuitively logical since the longer the travel time, the more time (opportunity) there is for different drivers to deviate from the average travel time. **It is important to recognize that the dispersion is a function of the travel time, not distance per se.** This means that for a given distance between signals, there will be more dispersion of a slow speed, say 30 km/h (18 mph) than at a higher speed, say 60 km/h (36 mph).

The dispersion rate is found to be dependent upon the number of vehicles in the platoon; smaller platoons disperse at a faster rate than those with a larger number of vehicles (22, 32). This probably reflects that drivers feel less “traffic pressure” when there are fewer vehicles in the platoon and the greater opportunity for drivers to change lanes on multilane roadways since the fewer number of vehicles in a platoon also reflects a lower volume to capacity ratio. Nevertheless, although they disperse at a slower rate, large platoons experience more dispersion than smaller ones since there are more vehicles involved.

SIGNALIZED INTERSECTION SPACING

PLATOON DISPERSION (Continued)

**Measurement
and
Concepts of
Platoon
Dispersion
(Continued)**

Platoon dispersion has also been found to be a function of the coefficient of variation in speed which is

$$cv = \frac{s}{x}$$

where:

cv = the coefficient of variation

x = the average speed

s = the standard deviation of the vehicle speeds

$$= \left[\sum_{i=1}^n (x_i - x)^2 \right]^{1/2}$$

x_i = the speed of vehicle i

n = the number of vehicles observed

$$\sum_{i=1}^n = \text{the sum of all the squared differences } (x_i - x)^2$$

This relationship also implies that platoon dispersion will decrease as speed increases and hence for any given distance, the travel time decreases. This effect is produced since the standard deviation and the mean are not perfectly correlated (that is the standard deviation at an average speed, or progression speed, of 60 km/h is not twice the standard deviation for a speed of 30 km/h) if the speeds of vehicles in the platoon are not seriously disrupted by a turning vehicle leaving the platoon (the effect of a turning vehicle will be addressed later).

Ideal progression is illustrated in Figure 4. The progression band is equal to the green phase and is divided into equal line intervals of Δt . The flow rate, q_i , of each interval at the upstream signal is a constant and is equal to the constant flow rate, q_i , at the downstream intersection.

On two-lane highways, dispersion is to the front of the platoon as illustrated schematically in Figure 5. The time intervals, Δt , at the upstream intersection, i and the downstream intersection (j) are of the same deviation. The illustration also shows the following: 1) The flow rate, q_i , at the upstream intersection is constant for each of the intervals, 2) the length of time between the arrival of the first vehicle and the last at the downstream intersection is longer, and the first vehicles in the platoon arrive early (i.e., on the red phase), and 3) the arrival rate, q_i , at the downstream signal is not a constant, however it tends to become constant in the later intervals.

SIGNALIZED INTERSECTION SPACING

PLATOON DISPERSION (Continued)

**Measurement
and
Concepts of
Platoon
Dispersion
(Continued)**

This forward dispersion is typical of signalized two-lane highways where the offset to the downstream signal(s) is based on the average speed of traffic. This forward dispersion occurs because:

1. The average speed is highly influenced by the few drivers to elect to travel at a slower speed than that desired by most drivers.
2. While the slow driver may be at front or rear of the platoon or anywhere in between, on average over several platoons will tend toward the middle.
3. Drivers preceding the slow vehicle are not inhibited by it and can drive at the average speed. Thus, they will arrive “early” on the red phase at the downstream signal. Vehicles following the slow vehicle “bunch up” behind it.

On multilane highways dispersion is to the rear of the platoon as illustrated in Figure 6. This occurs because:

1. Following vehicles are not as impeded by a preceding slow vehicle as they are on a two-lane highway because of lane change opportunities. Hence, the desired speed of most drivers tends to be similar to the average speed upon which the offset to the downstream signal is based.
2. Some drivers following the slow vehicle will not have an opportunity to change lanes, or will elect not to. These vehicles will be traveling at a speed slower than the progression speed, and hence the dispersion to the rear.

Platoon dispersion has also been found to be affected by city size. This is undoubtedly due to drivers in large cities are experienced with driving a shorter and more uniform headway because high-volume capacity conditions exist for several hours each day. This phenomenon is similar to that experienced in the 1950's when hourly counted traffic volumes in L.A. began to substantially exceed the volumes in other cities as well as exceeding the then accepted maximum theoretical capacities. Thus, platoon dispersion is less and saturation flow rates are higher in Portland than they are in Corvallis. However, the projected growth in Oregon's population will undoubtedly result in increased congestion in the smaller urban areas and, in turn, less platoon dispersion in future years.

SIGNALIZED INTERSECTION SPACING

PLATOON DISPERSION (Continued)

Measurement and Concepts of Platoon Dispersion (Continued)

In absence of auxiliary turn lanes for left and right turns, a turning vehicle will cause following vehicles to decelerate or even come to a stop. This will create large gaps in the platoon. The effect on platoon dispersion is illustrated in Figure 7. As shown, the flow rate for one or more intervals, Δt , in the progression band will be reduced and part of the platoon “shifted” to intervals following the progression band. Vehicles turning at a few midblock locations will seriously disrupt or even destroy the platoon.

Platoon Dispersion Models

A considerable amount of research has been devoted to the development or calibration of platoon dispersion models for traffic models such as NETSIM and TRANSYT-7F (14 through 27). Dispersion models based on this research is also used in signal timing optimization programs such as PASSER II (22). Other research (28) has focused on the effect of platoon dispersion on stopped delay at traffic signals and application of the progression adjustment factors of the Highways Capacity Manual procedure.

The dispersion algorithms used in TRANSYT, PASSER II and proposed in NCHRP Report 339 are schematically illustrated in Figures 8, 9 and 10 respectively. The TRANSYT algorithm assumes a flow rate equal to the saturation flow rate, s , except for the interval, or intervals, at the end of the platoon departing the upstream signal. The flow rate, q_b , at the downstream signal is a variable and may be different for each interval. Both forward and rear dispersion is assumed with the dispersion to the rear being the greater. A recursive procedure is used to calculate the dispersion by short time intervals Δt . PASSER II assumes a uniform flow rate, q_j , is assumed at the downstream signal with possible forward and rearward dispersion. Unlike TRANSYT, the flow rate within the platoon is average over the dispersed arrival period at the downstream locations.

The dispersion model proposed in NCHRP Report 339 is an extension of the model proposed by Rouphail (23). It assumes a uniform flow rate, q_u , departing the upstream signal. It assumes that secondary flows (left-turns and right-turns) at the upstream signal and other entering traffic is small compared to the primary flow (through traffic at the upstream signal). It also assumes that the secondary flows can be distributed uniformly throughout the cycle. The model calculated the most dense portion of the platoon and defines the progressed flow as the platoon window. This

SIGNALIZED INTERSECTION SPACING

PLATOON DISPERSION (Continued)

**Platoon
Dispersion
Models
(Continued)**

procedure recognizes that the most dense portion of the platoon does not always move at exactly the same speed as the average speed. Referring to Figure 10, q_w is the average flow rate in vehicles per second (vps) inside the platoon window and q_p is the average flow rate in vps outside the platoon window.

The Road Research Laboratory, England, (34), developed a simple platoon dispersion model which is the predecessor for the algorithm used in TRANSYT. As modified by Messer, et al (22) to use similar terms, this equation is:

$$\begin{aligned} PD_{ij} &= 1.0 + 0.00667 t_{ij} \\ \text{where: } PD_{ij} &= \text{the platoon dispersion between an upstream} \\ &\quad \text{signal, } i, \text{ and downstream signal } j. \\ t_{ij} &= \text{the travel time between signal } i \text{ and signal } j \end{aligned}$$

Application of this model clearly shows that the platoon dispersion for signals paced at 0.402 km (1/4-mile) and a speed of 30 km/h (18 mph) is the same as for signals at 0.804 km (1/2-mile) and a speed of 60 km/h (36 mph) since the travel time is the same (48.24 seconds) in both cases. (Note: SI calculations, calculations using U.S. units will yield slightly different results due to rounding.) The platoon dispersion in both cases is $PD_{ij} = 1.0 + (0.00667)(48.24) = 1.32$. Thus, the platoon is expected to take about 32% longer (time from front of first vehicle in the platoon to the rear of the last) for the platoon to pass a point 48 seconds downstream than it to pass through the upstream signal.

A much more complicated model developed by Messer, et al (22) considers the number of vehicles in the platoon as well as travel time. It gives the same platoon dispersion when the number of vehicles in the average platoon is 13.8 (22, p. 92). For shorter platoons the Messer, et al model yields a greater dispersion and less dispersion for longer platoons.

It is to be noted that the constant of 0.00667 is derived from research (22, 34) which is three decades old. The data used by Messer were collected in College Station/Bryan, Texas in 1974-75. At the time the combined population of the twin cities, including students at Texas A&M University was poor to nonexistent. Changing driver behavior in response to longer traffic volumes, higher levels of congestion, and increased familiarity with signal systems times for progression flow suggests that the value of the constant is too large and thus overstates platoon dispersion – probably by a substantial margin. The platoon smoothing equation, $F = 1/(1 + abt)$ discussed later is a better approach.

SIGNALIZED INTERSECTION SPACING

PLATOON DISPERSION (Continued)

**Platoon
Dispersion
Models
(Continued)**

**Figure 4: Schematic Illustration of
Ideal Progression Without Platoon Dispersion**

SIGNALIZED INTERSECTION SPACING

PLATOON DISPERSION (Continued)

Platoon
Dispersion
Models
(Continued)

**Figure 5: Schematic Illustration of Forward Platoon Dispersion,
Typical Where Progression Speed is Based
on Average Speed**

SIGNALIZED INTERSECTION SPACING

PLATOON DISPERSION (Continued)

Platoon
Dispersion
Models
(Continued)

Figure 6: Schematic Illustration of Dispersion to the Rear of the Platoon Typical of Multilane Highways

SIGNALIZED INTERSECTION SPACING

PLATOON DISPERSION (Continued)

Platoon
Dispersion
Models
(Continued)

Figure 7: Schematic Illustration
of the Effect of Turning Traffic on Platoon Dispersion

SIGNALIZED INTERSECTION SPACING

PLATOON DISPERSION (Continued)

**Platoon
Dispersion
Models
(Continued)**

**Figure 8: Schematic Illustration of the Dispersion Algorithm Used in
TRANSYT**

SIGNALIZED INTERSECTION SPACING

PLATOON DISPERSION (Continued)

**Platoon
Dispersion
Models
(Continued)**

**Figure 9: Schematic Illustration of the Dispersion Algorithm Used in
PASSER II**

SIGNALIZED INTERSECTION SPACING

PLATOON DISPERSION (Continued)

Platoon
Dispersion
Models
(Continued)

Figure 10: Schematic Illustration of the Dispersion
Algorithm Proposed in NCHRP Report 339

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SIGNALIZED INTERSECTION SPACING

PLATOON DISPERSION (Continued)

**Platoon
Dispersion
Models
(Continued)**

The commonly used platoon dispersion model developed by Robertson and used in TRANSYT 31 and elsewhere, such as NCHRP Report 339 is:

$$F = \frac{1}{1 + a b t} \quad (\text{Eq. 1})$$

where:

- F = platoon smoothing factor, a measure of platoon dispersion
- a = platoon dispersion rate
- b = ratio of leading edge of platoon arrival time at a downstream location divided by the average travel time
- t = the travel time in seconds between the upstream signal (departure point i) and a downstream signal (or other location j)

Robertson elected to assume 0.80 as the value of b ; this value is built into the TRANSYT model. Therefore, the value of 0.80 has been commonly used and Equation 1 is commonly defined as:

$$F = \frac{1}{1 + a t}$$

where:

- F and a are defined as above
- t = 0.8 time the average journey time over which the platoon dispersion is being calculated = ~~0.8~~

Selection of the platoon dispersion rate, b , is important because it controls the calculated dispersion, particularly when the value of b is calculated using the value of a . Values of a and b which have been reported are given in Tables 5 and 6.

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SIGNALIZED INTERSECTION SPACING

PLATOON DISPERSION (Continued)

Platoon Dispersion Models (Continued)

**Table 5: Parameter Values Recommended in NCHRP Report 233
and in TRANSYT-7F Manual**

NCHRP 233		TRANSYT-7F Manual		Roadway Characteristic	Description of Conditions
α	β	α	β		
0.5	0.8	0.5	0.8	Heavy friction	Combination of parking, moderate to heavy turns, moderate to heavy pedestrian traffic, narrow lane width; traffic flow typical of urban CBD
0.37	0.8	0.35	0.8	Moderate friction	Light turning traffic, light pedestrian traffic, 11-to-12-ft. lanes, possibly divided; typical of well-designed CBD arterial
0.24	0.8	0.25	0.8	Low friction	No parking, divided, turning provisions, 12-ft. lane width; suburban high-type arterial

Source: Reference [\(29\)](#), NCHRP Report 339

Table 6: Summary of Platoon Dispersion Studies

Best-Fit Parameter Value		Description of Conditions	Reference
α	β		
0.20	0.80	Three-lane dual carriageway; suburban high-type arterial, England	Collins and Gower
0.24	0.80	Typical suburban arterial roadway with two lanes in each direction; turn lanes provided, Toronto, Canada	Lam
0.40	0.80	Three-lane dual carriageway with 10-15 percent commercial vehicles; reasonable freedom for overtaking, Manchester, England	Sneddon
0.63	0.80	Two-way road 35 ft. wide with two narrow lanes in the direction studied; 2-3 percent commercial vehicles; severely restricted overtaking, Manchester, England	Sneddon
0.60 ^a 0.70 ^b	0.63 ^a 0.59 ^b	Single carriageway 33 ft. wide on 5 percent downgrade; subject to 30-mph speed limit and clearway regulations during peak periods; but volume of 12 vehicles/h in direction studied, Sheffield, England	El-Reedy and Ashworth
0.50	0.80	Characteristics ranging from single-lane flow with heavy parking and very restricted overtaking to multilane flow with no parking and relatively free overtaking, England	Robertson, Hillier and Rothery
0.21	0.97	Two-lane urban streets, low friction, Lincoln, Nebraska	McCoy, et al
0.15	0.97	Four-lane urban divided streets, low friction, Lincoln, Nebraska	McCoy, et al

Source: Reference [\(29\)](#), NCHRP Report 339

SIGNALIZED INTERSECTION SPACING

PLATOON DISPERSION (Continued)

**Platoon
Dispersion
Models
(Continued)**

The sites studied by McCoy, et al were located in Lincoln, Nebraska. Lane widths were 12 or 13 ft., level alignment, level grade, no parking, speed limit 35 mph on 2-lane sections, 45 mph on 4-lane sections, there were no or a limited number of driveways. These values of a and b are more representative of the traffic flow friction on U.S. urban arterials than the studies conducted in England. Moreover, it is important to note that McCoy, et al (21) using Lincoln, Nebraska data found the value of b to be 0.97. This is considerably larger than the commonly assumed value of 0.80 which is embedded in the TRANSYT model. This means that the leading edge of the platoon is traveling at about the same speed as the average speed. Or more specifically, the travel time of the leading edge of the platoon is 97% of the average travel time as opposed to 80%. Thus, there is much less forward platoon dispersion than assumed in the TRANSYT model. This is as would be expected with disciplined drivers familiar with a signal system providing reasonably good progression.

It is also likely that the value of a will be larger as drivers become experienced in driving in high-volume, high-speed synchronized traffic signal systems. Thus, platoon dispersion can be expected to be less in large urban areas and to decrease with experience. Observation of platooned flow on South Wadsworth (Denver metropolitan area) and Telegraph Road (Detroit metropolitan area) indicate that drivers can and do maintain good platoon integrity (little platoon dispersion) when experienced in driving on high-volume highways having long signal spacings with good progression timing.

SIGNALIZED INTERSECTION SPACING

SAFETY

Introduction

A number of studies have researched the effect of access density (number of access points per mile) on crash rates. A few of these have investigated the effect of the number of signalized intersections per mile. Some have developed regression equations in which signalized access density is one of several independent variables such as the number of unsignalized intersections, commercial drives and/or total driveways per mile, ADT, posted speed, urban area population (a proxy for development activity and presumably access volumes) and cross-section.

**Signal
Density**

Figure 11 shows that crash rates increase as signalized intersection density (the number of signals per mile) increases. Inspection reveals that crash rates tend to be higher on 4-lane roadways having a continuous two-way left-turn lane (TWLTL) than those with raised medians. It is also readily apparent that those with TWLTL's also tend to have a greater range of signal density. The higher crash rates on roadways with TWLTL's are likely different development patterns along 4-lane divided versus 5-lane (4 traffic lanes plus a TWLTL) and especially the complex conflict pattern of turning and through movements that occur with a TWLTL as compared to a roadway having a raised median.

**Regression
Equations**

Various researchers have developed regression equations for crash rates or for the number of crashes per mile. These equations have been developed with specific data base – usually an individual state. Therefore, the specific equations are not applicable to Oregon highways. Nevertheless, it is of interest to note that most of the models include signals per mile as an independent and that these models indicate that crashes increase as the number of signals increase.

SIGNALIZED INTERSECTION SPACING

SAFETY (Continued)

Regression
Equations
(Continued)

Figure 11: Effect of Signal Density on Crash Rates

Source: Reference (13) Squire & Parsonson, 1989.

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SIGNALIZED INTERSECTION SPACING

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SIGNALIZED INTERSECTION SPACING

APPENDIX A: EQUATIONS FOR TWO-WAY PROGRESSION

The relationship between cycle length, signal spacing and progression speed for maximum progression efficiency with two-way traffic are well established. They were used by Marks (3) in developing curves for cycle lengths of up to 100 seconds. The equations are:

$$s = \frac{v \ c}{1.362}$$

where: s = spacing in feet
 v = speed in mph
 c = cycle length in seconds

or
$$m = \frac{v_l \ c}{7.2}$$

where: m = spacing in metres
 v_l = speed in km/h
 c = cycle length in seconds

The above equations can be used to develop the same curves as generated by the PASSER runs as shown in Figure 1. However, PASSER, or similar signal timing programs must be used to generate curves such as shown in Figure 3.

Background Paper No. 9

SIGNALIZED INTERSECTION SPACING

APPENDIX B: COMPENSATION FOR DEVIATION FROM STANDARD SIGNAL INTERVAL

The following are examples of the calculations using the PASSER output to calculate the compensation in green time due to a deviation in signal spacing.

60-second cycle, 35 mph

Spacing (ft)	(1) Progression Efficiency	(2) Cycle Length (sec)	(3) = (1) x (2) Major Street Green (sec)
1340	0.31	60	18.6
1340 ± 200	0.05	60	3.0

- Deviation in spacing = $200/1340 = 0.1299$, say 0.13 or 13%
- Time “lost” on major street = $18.6 - 3.0 = 15.6$ sec
- Connection needed to restore major street progression:
 $15.6 \text{ sec} = (0.13 \text{ deviation})(60 \text{ sec cycle})(x)$
 $x = 2$ or 2% of cycle for each 1% deviation

120-second cycle, 35 mph

Spacing (ft)	(1) Progression Efficiency	(2) Cycle Length (sec)	(3) = (1) x (2) Major Street Green (sec)
3040	0.34	120	40.8
3040 ± 400	0.08	120	9.6

- Deviation in spacing = $400/3040 = 0.1316$, say 0.13 or 13%
- Time “lost” on major street = $40.8 - 9.6 = 31.2$
- Connection needed to restore major street progression:
 $31.2 \text{ sec} = (0.13 \text{ deviation})(120 \text{ sec cycle})(x)$
 $x = 2$ or 2% of cycle for each 1% deviation

120-second cycle, 55 mph

Spacing (ft)	(1) Progression Efficiency	(2) Cycle Length (sec)	(3) = (1) x (2) Major Street Green (sec)
4840	0.35	120	42.0
± 825	0.02	120	2.4

- Deviation in spacing = $825/4840 = 0.117$ or 17%
- Time “lost” on major street = $42.0 - 2.4 =$
- Connection needed to restore major street progression:
 $39.6 \text{ sec} = (0.17 \text{ deviation})(120 \text{ sec cycle})(x)$
 $x = 1.94$, say 2.0 or 2% of cycle for each 1% deviation