Optimizing Settings for Pedestrian-Actuated Signal Control Systems

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A pedestrian-actuated traffic signal control system has been implemented at intersections where most traffic conflicts are between vehicle flows and pedestrians. When properly implemented, such a system can process traffic flows more efficiently than a pretimed control system. It is not yet clear, however, how a pedestrian-actuated system can be best utilized, and misuse is not uncommon. This study provides a basis for proper use of a pedestrian-actuated system by characterizing the performance of the system at isolated intersections in terms of traffic delay. Three optimization problems of signal setting—the minimization of (a) average vehicle delay, (b) total delay of pedestrians and vehicle riders, and (c) the difference between average pedestrian delay and average vehicle delay—are discussed.

The operation of a pedestrian-actuated control system is fairly straightforward (Figure 1). When the first pedestrian arrives at an intersection at time t_1 and pushes a button to change the signal, it takes the control a response time of T s to switch the signal from red to green for the pedestrian. Response time can be considered as equal to vehicle amber time. Subsequent arrivals of pedestrians between time t_1 and time t_2 have no influence on the control.

The pedestrian green, totaling G_p s, consists of a WALK duration of E s and a DON'T WALK duration of S s. After the end (t_4) of the pedestrian green, the signal changes to vehicle green for a minimum of M s.

Amber time is treated as green time in this study and is included in minimum vehicle green duration M. If there are pedestrian arrivals between the start of DON'T WALK (t_3) and the end of the minimum green (t_5), the next pedestrian green would begin at time t_5. Otherwise, the first pedestrian arriving after t_5, for instance at t_6, would induce another pedestrian green at time t_7 and cause an overall vehicle green duration of G_v s. Thus, vehicle green duration is a function of pedestrian arrivals.

A pedestrian-actuated control system can be evaluated by measures of performance, perhaps the most pertinent of which is traffic delay. For pedestrians delay is a function of pedestrian flow patterns and signal settings, while for vehicles it is a function of vehicle flow patterns, signal settings, and the sequence of pedestrian green intervals.

Assuming that traffic arrival is random, I derived the following formulas for estimating traffic delays. These formulas are then applied to three signal setting optimization problems: (a) minimizing average vehicle delay, (b) minimizing total delay of pedestrians and vehicle riders, and (c) minimizing the difference between average pedestrian and average vehicle delays.

SYSTEM PERFORMANCE

Frequency of Pedestrian Green Interval

The performance of a pedestrian-actuated signal control system is governed largely by the frequency of pedestrian green intervals. For randomly arriving pedestrians the frequency measured in terms of the number of pedestrian green intervals per hour can be determined analytically as

$$N = 3600/\left\{G_p + M + h_p \exp[-(S + M - T)\lambda_p]\right\} \quad (1)$$

where

- N = the number of pedestrian green intervals per hour;
- G_p = pedestrian green duration (s);
- M = minimum vehicle green duration (s);
- T = pedestrian signal response time (s);
- h_p = pedestrian headway (s);
- S = pedestrian DON'T WALK duration (s); and
- λ_p = pedestrian flow rate (persons/s).

In Equation 1 I also assumed that pedestrians arriving in DON'T WALK durations would wait for the next green interval. This equation indicates that as λ_p increases N approaches a fixed value of $3600/(G_p + M)$, at which an actuated signal pattern becomes the same as a pretimed pattern with a cycle length equal to $G_p + M$.

Average Pedestrian Delay

Average pedestrian delay with random arrivals can be determined from

$$D_p = N[T(3600/N - G_p - M) + (S + M)^2/2]3600 \quad (2)$$

where D_p is the average pedestrian delay in seconds per pedestrian. Delay suffered by a pedestrian is measured from the time of arrival at an intersection to the start of the first pedestrian green.

Results of a recent simulation analysis of pedestrian-actuated signal control systems (1) reveal that the 80th percentile pedestrian delay is approximately twice as long as the average delay for a given combination of flow and signal settings.

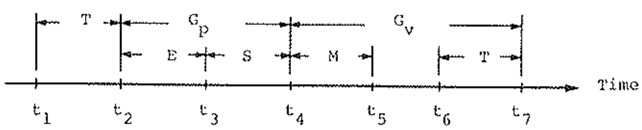
Compared to a pretimed signal system with a pedestrian DON'T WALK duration of S s, a pedestrian green of G_p s, and a vehicle green plus amber duration of M s, average delay has an expected value of $(S + M)^2/2(G_p + M)$. Therefore, it can be concluded from Equations 1 and 2 that the average delay incurred by a pedestrian-actuated signal is always less than that expected from a pretimed signal.

Average Vehicle Delay

Vehicles are delayed by signal controls because queues form at red lights. To relate delays to traffic flows and signal settings, let

- C = 3600/N, average cycle length (s);
- G_e = C - G_p - 3.7, average vehicle effective green duration (s);

Figure 1. Timing of pedestrian-actuated signal control.



Q_v = single lane vehicle flow (vehicles/h);
Q_s = vehicle saturation flow (vehicles/h), taken as 1800 vehicles/h for automobiles;
x = G_e/C, proportion of the average cycle length that is effectively green;
y = Q_vC/G_eQ_s, saturation rate with respect to Q_s; and
D_v = average vehicle delay (s).

Based on the same simulation analysis I found that with the above definitions Webster's formula (2)

$$D_v = [C(1-x)^2]/[2(1-xy)] + 3600y^2/[2Q_v(1-y)] - 0.65[C/(Q_v/3600)^2]^{1/3}\, y^{(2+5x)} \tag{3}$$

can provide reasonable estimates of average vehicle delays whether signal controls are pretimed or pedestrian actuated. It can be concluded from Equation 3, however, that a pedestrian-actuated control system will always result in fewer delays than a pretimed system. As in the case of pedestrian delays, the 80th percentile vehicle delay is approximately equal to $2D_v$.

Equation 3 should be used with discretion, because it does not reveal the actual performance of a control system under all circumstances. This requires further explanation regarding the interaction between vehicle flows and the vehicle-processing capacity of an intersection. This capacity, measured in terms of the maximum number of vehicles that can pass through the intersection in an hour, is a function of signal settings and characteristics of vehicle discharging headways at the intersection. Greenshields' distribution of discharging headways (3) is shown in the following table.

Position of Vehicle in Queue	Discharging Headway (s)	Position of Vehicle in Queue	Discharging Headway (s)
First	3.8	Fourth	2.4
Second	3.1	Fifth	2.2
Third	2.7	Sixth and up	2.1

Based on those headways, such a capacity (Q_{max}) can be estimated as

$$Q_{max} = 5N - (3600 - NG_p - 14.2N)/2.1 \tag{4}$$

where 14.2 represents the total time in seconds needed for the first five queuing vehicles to enter the intersection.

The average saturation rate (y_a) for an average cycle can be approximated by

$$y_a = Q_v/Q_{max} \tag{5}$$

From the simulation analysis mentioned earlier, I found that when y_a has a value greater than about 0.8 average delay has a tendency to become a function not only of flow rate but also of the sequence of arriving headways. The performance of a control system may therefore be unstable. In other words, for a given flow rate, estimates of average delays based on different field samples either have a large variation or may become time dependent (increase with respect to time). Then Equation 3 may not provide a good estimate. Whenever possible, signal settings should be chosen in such a way as to avoid a y_a value greater than 0.8.

The system performance represented by Equations 1, 2, and 3 is characterized by several features that dictate the requirements of optimum signal settings. First, as depicted in Figure 2, average vehicle delay for a given G_p decreases at a decreasing rate with respect to M, while average pedestrian delay increases at a more or less constant rate with respect to M. Second, for a given M, average vehicle delay increases while average pedestrian delay decreases with respect to G_p. And, finally, when M is smaller than G_p, average vehicle delay increases rapidly when M is reduced.

Validity of the Assumption of Random Arrivals

To test the validity of the assumption that traffic arrives randomly, data related to pedestrian and vehicle flow patterns were collected at several locations in Potsdam, New York (see Figures 3 and 4).

Figure 3a depicts a vehicle arrival pattern identified at a distance 42.7 m (140 ft) upstream from an intersection. The rate of arrival is light and affected only slightly by an upstream intersection controlled by a pedestrian-actuated signal. Figure 3b presents a vehicle arrival pattern of moderate flow rate at a location 76.2 m (250 ft) upstream from an intersection. This arrival pattern is not noticeably affected by a remote intersection upstream. On the other hand, the vehicle arrival patterns shown in Figures 3c and 3d respectively were observed 213.4 m (700 ft) downstream from an intersection controlled by a semiactuated vehicle signal. The flow rates at the time of data collection were moderately heavy, and interactions between vehicles were substantial because of the build-up of queues downstream.

Regardless of the dissimilarities in flow conditions, chi-square tests show that all the four arrival patterns conform to Poisson distributions at the 5 percent significance level, as does the pedestrian arrival pattern shown in Figure 4. The relatively high chi-square value of 8.14 compared to the critical value of 9.48 may be attributable to the grouping of pedestrians when friends or relatives arrive together.

From the above observations, traffic arrival patterns at an isolated intersection can be reasonably assumed to be random. When they are not random, such as when a heavy vehicle flow or a forced flow prevails, Equations 1, 2, and 3, because they demand minimum data input in the form of flow rates, are still good for finding optimum signal settings.

OPTIMUM SETTINGS

Optimum settings for a traffic signal control system will be dictated by the purpose of the control, which may vary from one locale to another. Nevertheless, the following are probably the most important purposes:

1. Minimizing average vehicle delay,
2. Minimizing the differences between average delay of vehicle riders and that of pedestrians, and
3. Minimizing total rider and pedestrian delays.

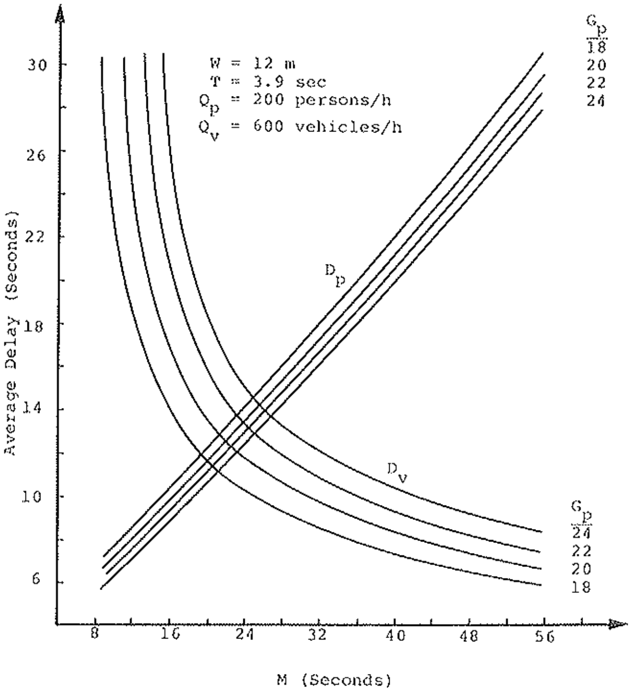
The signal controls based on these will be referred to as vehicle priority operation, equity operation, and minimum total delay operation respectively.

Vehicle Priority Operation

The rationale behind minimizing average vehicle delay hinges on the fact that they waste time, increase vehicle operating costs, and pollute the air. Therefore, as long as a control does not bring about excessive pedestrian delays, average vehicle delay may be minimized.

Several factors should be taken into account. First, in setting minimum vehicle green, M should be long enough to allow those vehicles stopped by a pedestrian green to cross the intersection during their green. For a pedestrian green of G_p s and a vehicle flow of λ_v vehicles/s, average number of queuing vehicles per pedestrian green is approximately $G_p\lambda_v$. The green time required for these vehicles to enter an intersection can be estimated from Greenshields' data on vehicle discharging headways, which on the average decrease from a high of 3.8 s for only one queuing vehicle to a stabilized value of about 2.1 s for a queue of considerable length. As a conservative measure, an average of 4 s/vehicle is allowed in the following analysis. Thus, green time required for $G_p\lambda_v$ vehicles to enter an intersection is considered as $4(G_p\lambda_v)$. An amber duration equal to pedestrian signal response time should be added to $4(G_p\lambda_v)$ to form a lower bound of minimum M.

Figure 2. Average delay as a function of G_p and M.



It is not clear what the maximum delay most pedestrians are willing to tolerate is. To maintain the spirit of a pedestrian-actuated control system, however, an upper bound should be imposed on M. This upper bound is denoted as M_{max} and is limited to 60 s.

In addition, there are several concerns in setting a pedestrian green duration G_p. For WALK duration, the Manual on Uniform Traffic Control Devices (4) set a minimum of 7 s to allow waiting pedestrians to enter an intersection. Abrams and Smith (5) have recently indicated that, except when pedestrian queues are longer than 15 people (an unlikely event for intersections controlled by a pedestrian-actuated system), 7 s is sufficient. This should be considered a lower bound because too short a WALK duration tends to create confusion and stress on pedestrians.

On the other hand, DON'T WALK duration should permit pedestrians to clear an intersection before vehicles are released. Thus, if W denotes the maximum width of an intersection and V_p is a design speed for pedestrians, then minimum DON'T WALK duration should be at least equal to W/V_p. In consequence, the minimum requirement of G_p is $7 + (W/V_p)$. When a G_p longer than the minimum is preferred, it would be desirable to set

Figure 3. Vehicle arrival patterns.

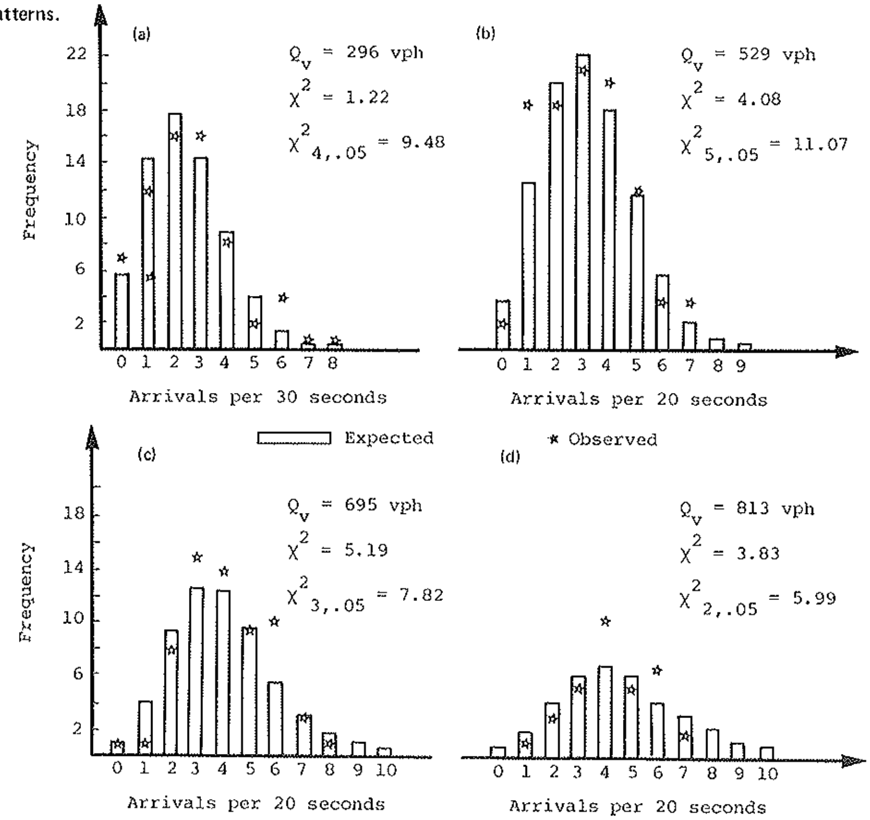
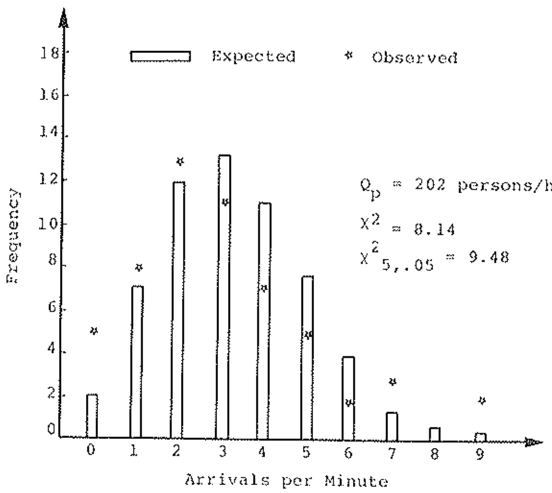


Figure 4. Pedestrian arrival pattern.



the DON'T WALK duration to its minimum of W/V_p and to allocate the rest of G_p to WALK.

Finally, pedestrian signal response time T, which may be regarded as vehicle amber time, should allow vehicles to clear an intersection before the green is turned over to pedestrians. According to the Traffic Engineering Handbook (6), T can be estimated from

$$T = r + V/(2a) + (W + L)/V \qquad (6)$$

where

r = perception-reaction time (1 s);
V = approach speed of clearing vehicle;
a = deceleration rate of clearing vehicle [4.57 m/s^2 (15 ft/s^2)];
W = intersection width; and
L = length of vehicle [6.1 m (20 ft)].

The problem of determining optimum settings for vehicle priority operation can be formulated as Min D_{vm}, which is subject to $4(G_p\lambda_v) + T \leq M \leq M_{max}$, $G_p = E + S$, $S = W/V_p$, and $E \geq 7$, where D_{vm} represents the average delay of the heaviest single-lane vehicle traffic flow that pedestrians interfere with. Its value is given by Equation 3.

The solution to this optimization problem is straightforward, because, as shown in Figure 2, average vehicle delay increases and decreases with respect to G_p and M. Therefore, the best settings for any combination of pedestrian and vehicle flows call for the use of a minimum acceptable G_p and a maximum allowable M: $E = 7$, $G_p = 7 + (W/V_p)$, and $M = M_{max}$.

Table 1 shows an example of vehicle priority operation based on V_p = 1.1 m/s (3.5 ft/s), V = 40 km/h (25 mph), and M_{max} = 60 s. The operations are characterized by the following features:

1. Average pedestrian delay (D_p) is overwhelmingly greater than the average vehicle delay (D_v);
2. Average pedestrian delay of about 30 s for all combinations of intersection widths and traffic flows implies an 80th percentile delay of about 60 s; and
3. Values of y_a for the ranges of intersection widths and traffic flows considered are well below 0.8. Thus, the signal settings would result in stable system performances with respect to vehicle delay, meaning that average delay is unlikely to grow with respect to time.

If the average pedestrian delay and the corresponding 80th percentile delays are excessive, then a smaller value of M_{max} should be used. For example, with an M_{max} of 40 s, the average pedestrian delay shown in Table 1 could be reduced by about 10 s, but average vehicle delay would not be increased by more than 4 s. The corresponding 80th percentile delay would have a more acceptable value of about 40 s. Use of a smaller M_{max}, however, has the danger of leading to an unstable system performance with respect to vehicle delays when both pedestrian and vehicle flows are heavy.

Equity Operation

If a control system is intended to permit fair use of an intersection by pedestrians and motorists, then signals can be set to minimize the discrepancy between average pedestrian delay and average vehicle delay. For vehicle flows, the needs of the heaviest traffic flow stream are the most important concern. Lighter streams affected by pedestrians may be ignored.

The problem of finding optimum settings for equity operations can be formulated as Min$(D_p - D_v)$, which is subject to $4(G_p\lambda_v) + T \leq M \leq M_{max}$, $G_p = E + S$, $S = W/V_p$, and $E = 7$, where $(D_p - D_v)$ denotes the absolute value of the difference between D_p and D_v.

In this case, G_p has to be set at its minimum requirement to ensure that a unique solution exists and that combined delay of pedestrians and vehicles be kept as low as possible. This constraint is essential because, as can be seen from Figure 2, for any G_p it is always possible to find an M such that D_v equals D_p. In the figure these combinations of G_p and M at which D_p equals D_v are represented by the intersects of D_p and D_v curves. It is clear that larger G_p induces longer delays.

The approximate solutions to the above optimization problem with different combinations of intersection widths and traffic flows are shown in Table 2. The following observations can be made by comparing the equity operations with the vehicle priority operations under the same conditions.

1. Equity operations bring about substantial reduction in average pedestrian delays. They result in less total delay at light vehicle flows and greater total delay at heavy vehicle flows.
2. As indicated by the larger values of y_a, the equity operations are relatively unfavorable to vehicle flows. Values of y_a greater than 0.8 at heavy vehicle flows suggest that an equity operation here is undesirable.
3. For a given combination of intersection width and vehicle flow, the optimum settings of M are not sensitive to pedestrian flow rate.
4. For a given combination of Q_p and Q_v, the optimum settings of M increase by no more than 2.5 s for every additional 3.1 m (10 ft) of intersection width.
5. The 80th percentile delays range about 7 to 40 s for the various combinations of intersection widths and traffic flows. Therefore, from the viewpoint of pedestrians, equity operations are preferred to vehicle priority operations.

Minimum Total Delay Operation

When travel time is the dominant concern in the operation of a signal control system, it would be desirable to minimize total pedestrian and vehicle delay. Assume that vehicle occupancy rate is 1.5 riders/vehicle; the best settings can be identified by finding the solution to the problem

$$\text{Min}\left(D_pQ_p + 1.5\sum_{i=1}^{n} Q_{vi}D_{vi}\right) \tag{7}$$

where n denotes the total number of vehicle traffic flow streams in conflict with pedestrians, and Q_{vi} and D_{vi} represent the flow rate and the average delay of the ith vehicle stream respectively. D_p is given in Equation 2, and D_{vi} can be determined from Equation 3.

The objective function given above should be subjected to the same constraints as those described in the case of vehicle priority operation.

A complete treatment of this optimization problem is impossible because optimum settings are partially dictated by the number of vehicle traffic streams that have to be taken into account. Table 3 provides an insight into the nature of this type of operation and shows the best settings and their implications for only one vehicle traffic stream. The operations have the following characteristics:

1. G_p should be set at its minimum requirement;
2. Optimum settings of M increase with respect to intersection width and vehicle flow rate for a given pedestrian flow rate;
3. Values of y_s are all under 0.8 for the ranges of intersection width and traffic flows considered, and stable performance of the control systems with respect to vehicle delays can be expected; and
4. Optimum settings are comparable to those of the vehicle priority operations at heavy vehicle flows. And, at light vehicle and pedestrian flows, they are similar

Table 1. Optimum settings for vehicle priority operations.

			W = 6 m, G_p = 13 s, T = 3.3 s			W = 9 m, G_p = 16 s, T = 3.6 s			W = 12 m, G_p = 19 s, T = 3.9 s			W = 15 m, G_p = 22 s, T = 4.1 s		
Q_p	Q_v	M (s)	D_p (s)	D_v (s)	Y_s	D_p (s)	D_v (s)	Y_s	D_p (s)	D_v (s)	Y_s	D_p (s)	D_v (s)	Y_s
100	200	60	27.5	2.2	0.14	30.0	2.9	0.15	30.4	3.7	0.15	30.8	4.5	0.16
	400			2.7	0.28		3.6	0.29		4.5	0.30		5.5	0.33
	600			3.5	0.42		4.5	0.44		5.6	0.46		6.8	0.50
	800			4.5	0.56		5.8	0.59		7.1	0.61		8.6	0.66
200	200	60	29.4	2.3	0.14	30.6	3.1	0.15	32.1	3.9	0.16	33.4	4.8	0.16
	400			3.0	0.29		3.8	0.30		4.8	0.31		5.8	0.34
	600			3.8	0.43		4.8	0.45		6.0	0.47		7.2	0.50
	800			4.9	0.57		6.2	0.60		7.6	0.62		9.2	0.68
400	200	60	29.6	2.4	0.14	30.9	3.1	0.15	32.3	3.9	0.16	33.5	4.8	0.16
	400			3.0	0.29		3.8	0.30		4.8	0.31		5.8	0.34
	600			3.8	0.43		4.8	0.45		6.0	0.47		7.2	0.50
	800			4.9	0.57		6.2	0.60		7.6	0.62		9.2	0.68

Note: 1 m = 3.3 ft.

Table 2. Optimum settings for equity operations.

		W = 6 m, G_p = 13 s, T = 3.3 s				W = 9 m, G_p = 16 s, T = 3.6 s				W = 12 m, G_p = 19 s, T = 3.9 s				W = 15 m, G_p = 22 s, T = 4.1 s			
Q_p	Q_v	M (s)	D_p (s)	D_v (s)	Y_s	M (s)	D_p (s)	D_v (s)	Y_s	M (s)	D_p (s)	D_v (s)	Y_s	M (s)	D_p (s)	D_v (s)	Y_s
100	200	7.0	3.6	3.6	0.17	9.0	4.9	4.8	0.18	10.0	6.2	6.1	0.20	11.0	7.6	7.5	0.22
	400	10.5	4.4	4.4	0.34	12.0	5.8	5.8	0.36	13.5	7.4	7.3	0.39	14.5	9.0	8.9	0.42
	600	14.5	5.6	5.5	0.50	16.0	7.2	7.1	0.54	17.5	9.0	8.9	0.57	19.0	10.9	10.7	0.61
	800	18.5	7.0	7.0	0.66	20.5	9.0	9.0	0.70	22.5	11.1	11.3	0.74	25.0	13.5	13.8	0.81
200	200	11.5	5.4	5.4	0.23	12.5	7.0	6.9	0.25	13.0	8.5	8.6	0.28	14.0	10.3	10.1	0.30
	400	14.5	6.6	6.5	0.43	15.5	8.3	8.2	0.47	16.5	10.2	10.1	0.51	17.5	12.0	11.9	0.55
	600	17.5	7.9	8.0	0.62	19.0	10.0	10.1	0.66	20.5	12.1	12.3	0.70	22.5	14.5	14.3	0.73
	800	22.5	10.3	10.2	0.76	25.0	13.0	12.9	0.80	27.5	15.6	15.7	0.83	30.0	18.3	18.6	0.85
400	200	13.0	6.6	6.6	0.28	13.5	8.1	8.2	0.31	14.0	9.7	9.8	0.34	14.5	11.2	11.5	0.36
	400	15.5	7.7	7.9	0.51	16.5	9.5	9.6	0.54	17.5	11.4	11.4	0.57	18.5	13.2	13.1	0.60
	600	19.0	9.4	9.4	0.69	20.5	11.5	11.5	0.71	22.0	13.5	13.6	0.74	23.5	15.6	15.7	0.76
	800	24.0	11.8	11.9	0.80	26.5	14.4	14.4	0.82	29.0	17.0	16.7	0.84	31.5	19.6	19.1	0.85

Note: 1 m = 3.3 ft.

Table 3. Optimum settings for minimum total delay operations.

		W = 6 m, G_p = 13 s, T = 3.3 s				W = 9 m, G_p = 16 s, T = 3.6 s				W = 12 m, G_p = 19 s, T = 3.9 s				W = 15 m, G_p = 22 s, T = 4.1 s			
Q_p	Q_v	M (s)	D_p (s)	D_v (s)	Y_s	M (s)	D_p (s)	D_v (s)	Y_s	M (s)	D_p (s)	D_v (s)	Y_s	M (s)	D_p (s)	D_v (s)	Y_s
100	200	6	3.4	3.6	0.17	7	4.4	4.8	0.19	8	5.6	6.2	0.20	9	6.9	7.7	0.22
	400	6	3.4	4.5	0.34	11	5.5	5.9	0.37	8	5.6	7.7	0.40	9	6.9	9.4	0.44
	600	13	5.1	5.6	0.50	20	8.8	6.8	0.52	33	10.2	7.5	0.52	44	23.4	8.0	0.51
	800	36	14.8	5.8	0.61	51	24.7	6.3	0.60	60	30.4	7.1	0.61	60	31.9	7.6	0.63
200	200	6	3.5	5.9	0.25	7	4.8	7.7	0.29	8	6.3	9.5	0.32	9	8.0	11.4	0.36
	400	6	3.5	7.7	0.50	12	6.8	8.9	0.51	17	10.4	9.9	0.50	21	13.8	10.9	0.50
	600	22	10.1	7.1	0.58	28	14.5	8.0	0.57	34	19.0	8.8	0.57	38	22.4	9.9	0.57
	800	37	17.7	6.9	0.65	45	23.2	7.7	0.65	52	28.1	8.5	0.65	58	32.5	9.4	0.65
400	200	6	3.6	9.1	0.43	7	5.1	11.5	0.50	9	7.3	12.6	0.49	16	9.1	14.2	0.50
	400	13	6.5	8.9	0.56	16	9.3	9.8	0.55	18	11.5	11.1	0.56	19	13.4	12.8	0.59
	600	22	10.8	8.2	0.63	25	13.7	9.5	0.64	28	16.5	10.6	0.64	30	18.8	12.1	0.66
	800	32	15.7	7.3	0.70	37	19.6	9.3	0.71	41	22.9	10.5	0.71	45	26.2	11.7	0.72

Note: 1 m = 3.3 ft.

to those of the equity operations.

In general, minimum total delay operation is more desirable than vehicle priority operation and equity operation because it avoids long pedestrian delays that can result from vehicle priority operation at light vehicle flows and vehicle delays similar to those of vehicle priority operation. And, finally, M_{max} can be chosen to produce acceptable average pedestrian delays at heavy vehicle flows and to alleviate the risk of unstable system performance.

SUMMARY

This paper characterizes the performance of a pedestrian-actuated signal control system in terms of traffic delays, describes the three types of signal operations, and discusses the validity of the assumption that traffic arrives randomly. This was verified by data collected for isolated intersections at Potsdam.

Vehicle priority operation may result in excessive pedestrian delays when long minimum vehicle green durations are chosen. Shorter vehicle greens, on the other hand, risk unstable system performances with respect to vehicle delay. Equity operation is preferable from the viewpoint of pedestrians, but it may bring about unstable system performance when heavy vehicle flows are present. Minimum total delay operation reduces the negative features of the other two operations and is thus more desirable.

Regardless of the types of operation, pedestrian green duration should always be set at its minimum requirement, which is 7 s plus the time needed for a pedestrian to cross the intersection. The setting of minimum vehicle green duration, on the other hand, depends on intersection width and traffic flow. In general, broader intersections and heavier vehicle flows require longer minimum vehicle green duration.

ACKNOWLEDGMENT

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Level of Service at Signalized Intersections

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In the 1965 Highway Capacity Manual levels of service at signalized intersections are related to load factor, which was intuitively judged the best level of service indicator available. Load factor has, however, presented such problems as its insensitivity to low service volume, absence of any rational basis for defining breakpoints, and difficulty in identifying loaded cycle. A rational method for quantifying the different levels of service at signalized intersections was therefore needed. In our research we used a road-user opinion survey that involved depicting and rating different traffic situations at a carefully selected single signalized intersection. Over 300 drivers rated randomly arranged film sequences of two types—a driver's view (microview) and an overall view (macroview) of an intersection—and evaluated these films, segment by segment, in terms of appropriate levels of service. Field studies and the attitude survey provided data for the development of two psychophysical models. Statistical analysis indicated that average individual delay correlated better with level of service rating than with measured load factor and encompassed all levels of service. Of all parameters affecting levels of service, load factor was rated highest by road users.

In the 1965 Highway Capacity Manual (HCM) (1) the concept of level of service was introduced for both uninterrupted flow conditions and street intersections with signalized control. Level of service is "a qualitative measure of the effect of a number of factors, which include speed, travel time, traffic interruptions, freedom to maneuver, safety, driving comfort, convenience and operating costs." There are stops at intersections, so speed cannot be the appropriate measure of level of service; an operational index called load factor (LF), then, was used to determine the various levels of service at signalized intersections. HCM defined this index as a "ratio of the total number of green signal intervals that are fully utilized by traffic during the peak hour to the total number of green intervals, for that approach during the same period." The different levels of service are identified alphabetically from A (free flow) to F (forced flow or stop-and-go conditions), based on the value of LF as follows: