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 \( (t_b) \) 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 \( (t_a) \) and the end of the minimum green \( (t_b) \), the next pedestrian green would begin at time \( t_a \). Otherwise, the first pedestrian arriving after \( t_a \), for instance at \( t_b \), would induce another pedestrian green at time \( t_a \), 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

\[
N = \frac{3600}{(G_p + M + h_p \exp[-(S + M - T)\lambda_p])}
\]  

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
- \( \lambda_p \) = 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 \( \lambda_p \) 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

\[
D_p = \frac{N[3600/(N - G_p - M) + (S + M)^2]}{3600}
\]  

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 (3) 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 \((S + M)^2/(2(G_v + M))\).

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

\[
C = \frac{3600}{N}, \text{ average cycle length (s)};
\]

\[
G_v = C - G_p - 3.7, \text{ average vehicle effective green duration (s)};
\]
\[ Q_\text{p} = \text{single lane vehicle flow (vehicles/h)}; \]
\[ Q_s = \text{vehicle saturation flow (vehicles/h), taken as} \]
\[ 1800 \text{ vehicles/h for automobiles}; \]
\[ x = \frac{G_p}{C}, \text{ proportion of the average cycle length} \]
\[ \text{that is effectively green}; \]
\[ y = Q_s/C, \text{ saturation rate with respect to } Q_s; \]
\[ D_\text{v} = \text{average vehicle delay (s)}. \]

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

\[ D_v = \left[ \frac{C(1 - x)^2}{2(1 - xy)} \right] + \frac{3600y^2}{2Q_s(1 - y)} \]
\[ + 0.65(C/Q_s)(3600)^{1/3}y^{1/2} + 0.4 \quad (3) \]

can provide reasonable estimates of average vehicle delays whether signal controls are pre-timed 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 pre-timed system. As in the case of pedestrian delays, the 80th percentile vehicle delay is approximately equal to 2Dv.

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 vehicles discharging headways at the intersection. Greenshields’ distribution of discharging headways (3) is shown in the following table.

<table>
<thead>
<tr>
<th>Position of Vehicle in Queue</th>
<th>Discharging Headway (s)</th>
<th>Position of Vehicle in Queue</th>
<th>Discharging Headway (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>First</td>
<td>3.6</td>
<td>Fourth</td>
<td>2.4</td>
</tr>
<tr>
<td>Second</td>
<td>2.1</td>
<td>Fifth</td>
<td>2.2</td>
</tr>
<tr>
<td>Third</td>
<td>2.7</td>
<td>Sixth and up</td>
<td>2.1</td>
</tr>
</tbody>
</table>

Based on those headways, such a capacity (Qmax) can be estimated as

\[ Q_{max} = 5N - (3600 - NG_p - 14.2N)/2.1 \quad (4) \]

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_s) for an average cycle can be approximated by

\[ y_s = Q_s/Q_{max} \quad (5) \]

From the simulation analysis mentioned earlier, I found that when \( y_s \) 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 \( y_s \) 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 43.7 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 semactuated 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 5.48 may be attributable to the grouping of pedestrians who either use the same route or travel 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 $\lambda$, vehicles/s, average number of queuing vehicles per pedestrian green is approximately $G_p \lambda$. 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 $G_p\lambda$, vehicles to enter an intersection is considered as $4(G_p\lambda)$. An amber duration equal to pedestrian signal response time should be added to $4(G_p\lambda)$ to form a lower bound of minimum $M$.

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 2.** Average delay as a function of $G_p$ and $M$.

**Figure 3.** Vehicle arrival patterns.
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 $T_1$ 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 ($T_1$), $T$ can be estimated from

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

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 [8.1 m (20 ft)].

The problem of determining optimum settings for vehicle priority operation can be formulated as Min $D_{wm}$, which is subject to $4(G_s + W/V_p)$, $G_s = E + S$, $S = W/V_p$, and $E = 7$, where $D_{wm}$ represents the average delay of the heaviest single-lane 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 $G_s$ and $M$. Therefore, the best settings for any combination of pedestrian and vehicle flows call for the use of a minimum acceptable $G_s$ and a maximum allowable $M$: $E = 7$, $G_s = 7 + (W/V_p)$, 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 $y_s$ 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 signal settings 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 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 $4(G_s + W/M_{max})$, $G_s = E + S$, $S = W/V_p$, and $E = 7$, where $D_p - D_v$ denotes the absolute value of the difference between $D_p$ and $D_v$.

In this case, $G_s$ has to be set to 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 $G_s$ it is always possible to find an $M$ such that $D_v$ equals $D_p$. In the figure these combinations of $G_s$ and $M$ at which $D_p$ equals $D_v$ are represented by the intersects of $D_v$ and $D_p$ curves. It is clear that larger $G_s$ 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 $y_s$, the equity operations are relatively unfavorable to vehicle flows. Values of $y_s$ 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 $Q_m$, 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.
Min \( D_p Q_p + 1.5 \sum_{i=1}^{n} Q_p D_i \)  \( (7) \)

where \( n \) denotes the total number of vehicle traffic flow streams in conflict with pedestrians, and \( Q_{pi} \) and \( D_i \) represent the flow rate and the average delay of the \( i \)th vehicle stream respectively. \( D_i \) is given in Equation 2, and \( D_i \) 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 \( y \), 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.

<table>
<thead>
<tr>
<th>( Q_p )</th>
<th>( Q_1 )</th>
<th>( M )</th>
<th>( D )</th>
<th>( D_p )</th>
<th>( D_i )</th>
<th>( Y )</th>
</tr>
</thead>
<tbody>
<tr>
<td>W = 6 m, ( G_p = 13 ) s, ( T = 3.3 ) s</td>
<td>W = 9 m, ( G_p = 16 ) s, ( T = 3.6 ) s</td>
<td>W = 12 m, ( G_p = 18 ) s, ( T = 3.9 ) s</td>
<td>W = 15 m, ( G_p = 22 ) s, ( T = 4.1 ) s</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>100</td>
<td>200</td>
<td>0</td>
<td>27.5</td>
<td>2.2</td>
<td>0.14</td>
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<tr>
<td>400</td>
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Note: \( 1 \) m = 3.3 ft.

### Table 2. Optimum settings for equity operations.

<table>
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<th>( Q_p )</th>
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<th>( M )</th>
<th>( D )</th>
<th>( D_p )</th>
<th>( D_i )</th>
<th>( Y )</th>
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<tbody>
<tr>
<td>W = 6 m, ( G_p = 13 ) s, ( T = 3.3 ) s</td>
<td>W = 9 m, ( G_p = 16 ) s, ( T = 3.6 ) s</td>
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Note: \( 1 \) m = 3.3 ft.

### Table 3. Optimum settings for minimum total delay operations.

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<th>( Q_p )</th>
<th>( Q_1 )</th>
<th>( M )</th>
<th>( D )</th>
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<td>15.5</td>
<td>0.80</td>
<td>26.5</td>
</tr>
<tr>
<td>600</td>
<td>40.5</td>
<td>0.80</td>
<td>30.5</td>
<td>17.8</td>
<td>0.84</td>
<td>31.5</td>
</tr>
</tbody>
</table>

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.

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REFERENCES


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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 those 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: