□ 論 文 □

Signal Optimization for Oversaturated Arterials

과포화 간선도로의 신호 최적화

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요 약

일반적으로 교통수요가 용량보다 적으면 모든 교통량이 지체없이 신호교차로를 통과 할 수 있을 것이다. 이러한 비포화 상태에서는 어떻게 Delay나 Stop을 최소화시키느냐가 신호처리의 목적함수가 될 것이다. 그러나 교통수요가 용량보다 많아지면 신호교차로가 모든 교통량을 통과시키지 못하므로 시간이 갈수록 대기 행렬이 점점 길어질 것이다. 이러한 과포화상태에서는 늘어나는 대기 행렬을 조절하지 못하면 결국에는 Spillback이 상류 교차로로 확대되어 최악에는 교차로에서의 모든 방향의 움직임을 정지시키는 Gridlock상태로까지 악화 될 수 있다. 따라서 과포화 상태에서는 비포화 상태와는 달리 늘어나는 대기행렬을 조절하여 통과 교통량을 최대화 시키는 것이 신호처리의 목적 함수가 될 수 있을 것이다.

본 논문에서는 과포화시의 간선도로를 신호처리에 의해 일정한 대기행렬을 유지하므로써 시스템을 최적화 하는 알고리즘을 개발하였다. IMPOST(Internal Metering Policy to Optimize Signal Timing)는 논문에서 개발한 알고리즘을 C 언어로 프로그래밍한 model이다.

1. INTRODUCTION

Traffic congestion is one of the significant problems in most urban areas. It is no longer a characteristic only of big cities. Medium-sized cities and even smaller urban areas experience it. It adversely affects the economic growth of urban areas and contributes to the migration of industry. Unlike many other social problems -- poverty, hunger, low-quality education, homeless, drug-addiction -- traffic congestion is directly experienced everyday by millions of commuters. Thus, congestion is a societal problem which deserves more attention and research support than it has received in the past.

Because it is not generally feasible to physically expand the existing facilities, much effort has turned into developing improved control strategies to optimize the existing facilities. But most of the present concepts of optimization theory appear ineffective or invalid in oversaturated conditions because they deal only with undersaturated conditions. In general, when traffic demand is less than available capacity then, by definition, this undersaturated demand can be completely serviced without experiencing disruptive operating conditions. Under these circumstances, the overriding objective of a signal control policy is to minimize vehicle delay and stops.

Oversaturated networks, on the other hand, have many approaches which exhibit inadequate capacity relative to traffic demand. This excess of demand of relative to capacity produces standing queues which grow over time and can exhaust the storage capacity of the approach. Uncontrolled queue growth can physically block intersection, degrade the queue discharge process thereby reducing service rates and spread over a large portion of the system. This process impacts traffic on other approaches which may even be nominally undersaturated, radically reducing the productivity of the system and potentially causing gridlock. Under these circumstances, the concept of minimizing delay and stops is subordinate to the objective of maximizing productivity. In oversaturated conditions, the control policy should be aimed at managing the growth of queues to maximize the productivity of the roadway system. That is, the policy is designed to service as many vehicles through the specified roadway system, as possible, in a given period of time.

Gazis(6) has previously considered a saturated critical intersection and tried to optimize a control strategy to minimize the delay. This analysis is relevant towards the end of peak hour conditions, while queues still exist but when the traffic flows have fallen below their saturation rates. D. Longley(7) suggested that each controlled intersection, should adjust the green time split on the basis of queue length ratios on its various approaches. The function of a control system is to maintain the "queue unbalance" term at a zero value, irrespective of the traffic flows on the various approaches of the intersection. Such a control system will respond to changes in traffic flow rates and to factors affecting saturation flow rates across the intersection. Gartner and Little(8) developed a link performance function to express the loss incurred by platoons traveling through a signal-controlled intersection as a function of link offset. At flows that are close to capacity, a saturation deterrence function (SDF), which is developed in terms of the overflow queue was added in the objective function. Integer variables enter the formulation because of the periodicity of the traffic signals. The optimization problem was formulated as a mixed-integer linear program. The formulation optimizes simultaneously all the signal control variables of a network. Michalopoulos and Stephanopoulos(10) continued Gazis' work and tried to broaden the problem by making modifications in an algorithm. Overflow was handled by limiting queue lengths. They also formulated the interdependencies among intersections in the network and solved the equations that represented all intersections simultaneously. NCHRP 194(11) and OECD(12) all suggested the queue control method to reduce the probability of spillback in saturated conditions. And both reports well defined the definition and the mechanism of oversaturation. Mcshane and Roess(2), Lieberman(4,5,9) all suggested metering the number of vehicles allowed into the network to reduce the probability of spillback in saturated conditions.

There has been relatively limited research and no practical algorithms are readily available in signal optimization for oversaturated arterials. Since such algorithms are not available, traffic engineers may inappropriately use some of the currently available software such as PASSER, MAXBAND, or TRANSYT. These software are intended for undersaturated conditions, and as such, they are not appropriate to oversaturated conditions. The results obtained from such inappropriate use can be misleading and may lead to inefficient traffic operation and prolonged congestion.

The objectives of the new policy differ importantly from those of existing strategies; these differences reflect the essential disparities between the undersaturated and oversaturated flow domain, as discussed previously. This new control policy includes consideration of undersaturated approaches embedded within, or feeding traffic into, an oversaturated arterial.

2. INTERNAL METERING POLICY FOR OVERSATURATED ARTERIALS

Internal Metering Policy (IMP) formurated only for oversaturated arterial with two-phase operation was originally developed by Edward Lieberman(9). The objectives of paper are to enhance the previous research and extend to oversaturated arterials with multi-phase, left-turn operation. The reader is encouraged to refer to NCHRP 3-38(3), Volume 2 for further detail.

The objctives of a traffic control policy designed expressly for oversaturated flow conditions on signalized networks, differ intrinsically from those appropriate for undersaturated flow environments. The goal of the IMP is to manage the growth of queues to maximize the productivity of the roadway system: to service as many vehicles as possible in a given period of time.

This goal is met if the control policy satisfies the following objectives:

Ⅰ. Control queue formation to prevent (or minimize the frequency and extent of) spillback into intersections.

Ⅱ. Fully utilize all available green time at the highest service rate.

Ⅲ. Effectively utilize existing roadway storage capacity.

Ⅳ. Provide equitable service to competing traffic streams.

Ⅴ. Minimize the number of vehicle stops.

Since oversaturated networks often exhibit a mix of undersaturated and oversaturated elements, it is also necessary for the policy to accommodate undersaturation conditions, and to satisfy the following objective:

Ⅵ. Minimize delay along the undersaturated elements of the network.

Control Queue Formation

By definition, oversaturation occurs when traffic demand exceeds the service capability of at least one section of the roadway system. This arises when either the capacity of the system is below the levels that can be achieved with effective traffic management and control, or when the high demand for service cannot be accommodated in a timely fashion even when effective control is applied.

In either case, the excess of demand relative to "supply" (or "operational" capacity) creates an inventory of delayed vehicles in the form of overflow queues. These queues will continue to grow with the passage of time so long as demand exceeds operational capacity. In fact, it is the uncontrolled growth of these overflow queues that acts to reduce the operational capacity below the levels of potential roadway capacity, thus exacerbating the imbalance between traffic demand and the system' s capability to service this demand.

This reduction in service rate is due to the physical presence of long queues which may:

- o Reduce queue discharge service rates as a human (driver) response to the presence of overflow queues downstream which require the discharging vehicles to stop after a short distance, thus limiting acceleration and, therefore, discharge rates. This mechanism has been observed in the field.
- o Physically block intersections. The immediate effect of this situation is to reduce service rates to zero for some portion of the green phase, thereby creating additional overflow queues upstream which serve to spread the reduction in operating capacity, elsewhere. Such queue growth can cause "gridlock" which can bring traffic movement to a halt over much of the roadway system.

The first objective of IMP is to:

- o Limit the length of overflow queues on approaches, to the extent possible, consistent with the need to encourage high rates of service during the green phase.
- o Avoid the onset of spillback, or at least reduce its frequency and extent to a minimum.

Fully Utilize Green Time at High Service Rates

Even if the control policy controls queue formation, there is no guarantee that high service rates can also be realized. Thus, the control policy must:

- o Utilize the entire green time by avoiding gaps in demand during the green phase.
- o Create coherent platoons to maximize queue discharge service rates throughout the green phase.

Utilize Existing Storage Capacity

Traffic movement should be managed so that the available storage capacity on the approaches is fully utilized subject, of course, to the constraints imposed by the need to satisfy the first two objectives.

Provide Equitable Service

In an oversaturated traffic environment, the control policy must allocate service capacity to meet some system-wide concept of equity. Under what circumstances should one traffic stream be provided preferred treatment relative to another?

Minimize Vehicle Stops

A saturated environment is, by definition, characterized by stop-and-go operation. Since vehicle stops are related to rear-end accidents, it follows that a control policy for this environment should include this objective.

Minimize Delay on Undersaturated Approaches

Given a heterogeneous mix of oversaturated and undersaturated approaches on a network, the policy must be designed to maximize productivity on the oversaturated approaches and to minimize delay on the undersaturated approaches where capacity is not a limiting constraint. Thus, the policy must accommodate different design criteria for these two classes of approaches, within the context of an integrated, coherent policy.

2.1 Formulation for Oversaturated Arterials with Two-Phase Left-Turn

This section presents the underlying concepts of IMP applied to a single approach, in both descriptive and mathematical form. These concepts will be extended to system-wide formulations. Figure 1 is a schematic of traffic flow on an approach (link (B,A)) in the presence of a sizeable standing queue at the stop-line at the beginning of the green indication there. This formulation assumes saturated flow conditions along the arterial approaches to intersections A and B. The demand on the cross street approaches to intersection B corresponds to some known or estimated value, V_c, which may be above, or below, saturation levels.

For this "unit problem" the cycle length, C; the green phase duration G_A at intersection A servicing this arterial approach; the free-flow speed of vehicles within the incoming platoon, v_f; the queue-discharge wave speed, u; the approach length, L; the number of lanes on this approach, $(LN)_A$, and on the cross street approach(es) to intersection B, $(LN)_C$; and the queue start-up lost time, s;

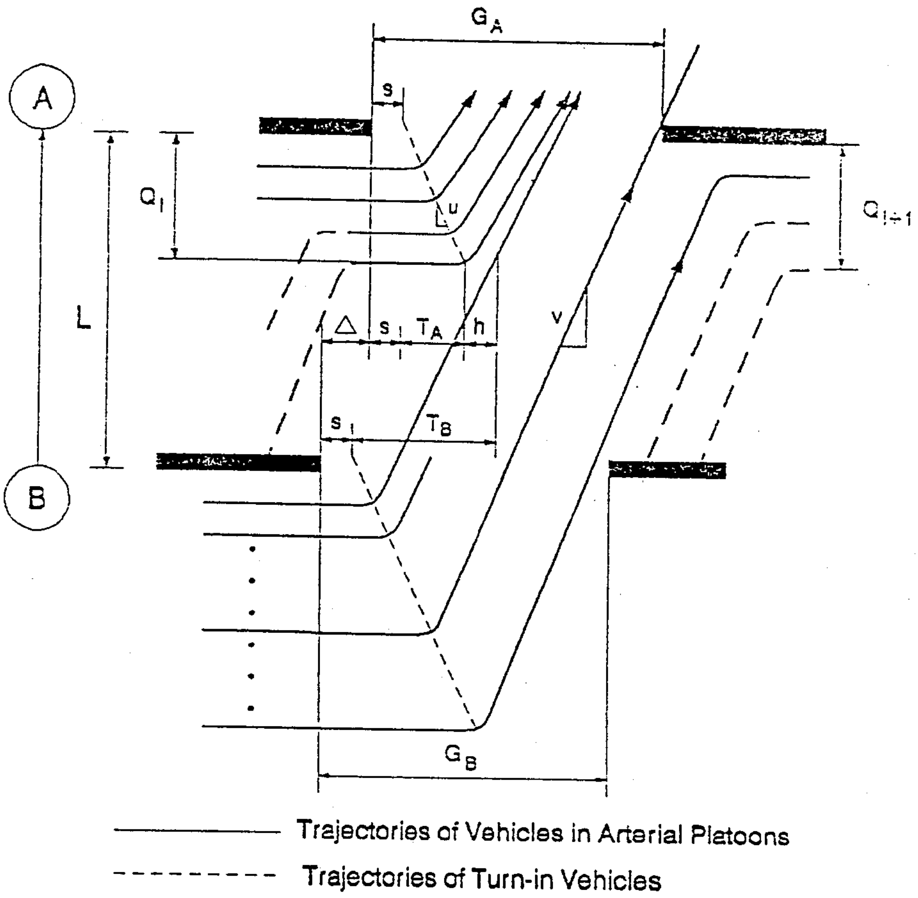


Figure 1. Schematic of a Saturated Flow Environment on a Single Arterial Link, (B,A)

are assumed to be known or previously calculated.

The IMP determines the control parameters, Green Phase Duration at Intersection B and the Relative Offset between the start of the green phases at intersections B and A (G_B and Δ, respectively) that will satisfy the first five objectives cited above, to the extent possible.

To satisfy Objectives II and V, we wish to set the relative offset, Δ, so that:

The lead vehicle in the incoming platoon released from the arterial approach to intersection B arrives at the tail of the queue on the approach to intersection A within one headway, h, of the last queued vehicle as it begins to discharge.

Note that if the lead vehicle of the incoming platoon arrives at the tail of the queue as described above, and reaches the stop-line during this green phase, it will certainly discharge across the stop-line close behind its lead vehicle. Many trailing vehicles in this platoon, however, will generally remain dispersed.

Referring to Figure 1, we can write,

$$S_B + T_B = \Delta + S_A + T_A + h,$$

where, $T_B = \dfrac{(L - Q_i)}{v}$; *and* $T_A = \dfrac{(Q_i - L_v)}{u}$

and the mean speed, v, can be computed as described subsequently.

Substituting for T_A and T_B yields (for $S_A = S_B$):

$$\Delta = T_B - T_A - h, \text{ or}$$

$$\Delta = \frac{(L-Q_i)}{v} - \frac{(Q_i - L_v)}{u} - h = \frac{L}{v} - \frac{Q_i(u+v)}{uv} - h + \frac{L_v}{u} \quad (1)$$

where,

G_N = Green phase duration, sec., at intersection, N; N = A or B. This duration includes Green, Yellow and All-Red (if any) intervals.

h = Mean queue discharge headway, sec;

L = Approach length, feet

L_v = Effective mean vehicle length within a standing queue, feet

Q_i = Queue length in cycle, i, at the time the first vehicle in the incoming platoon reaches the tail of this queue, feet. This queue contains all the vehicles on the approach to intersection A, when the signal servicing the upstream approach to inter section B, changes to green.

r_i = Q_i/L, ratio of queue length to approach length.

s = Phase lost time: start-up lost time plus the portions of the change intervals that do not service the discharging queue.

T_A = Elapsed time, sec., from start of queue discharge at the beginning of the arterial green phase at intersection A, until the start of movement of the last vehicle queued on the arterial approach to intersection A.

T_B = Elapsed time, sec., from start of queue discharge at the beginning of green time at intersection B, to the time when the lead vehicle in the dis charging queue on the arterial approach to inter section B, reaches the last vehicle in the dis charging queue on the approach to intersection A, as it starts to move, with a separation head way of h sec.

u = Mean speed of queue discharge wave propagat ing upstream, fps;

v = Mean speed of lead vehicle in incoming platoon, fps;

Δ = Relative signal offset, sec;

As discussed in more detail author' s dissertation, r is a "free parameter" which must be assigned a design value",

r_o, the desired ratio of queue length:approach length. Substituting r_o into (1) yields:

$$\Delta_o = \frac{L}{v}[1 - r_o\frac{(u+v)}{u}] - h + \frac{L_v}{u} \qquad (1a)$$

That is, for stated geometrics and *operating* conditions, the optimal signal offset, Δ_o, depends on the *desired* value of standing queue length, r_o.

Under pre-timed signal control, Objective I can be met (or approached) only if the mean queue length is stable over time. (Of course, fluctuations in demand and service rates will cause fluctuations in queue length; this property was addressed in Chapter 4 of author' s dissertation.)

Therefore, since the demand for service through intersection B is assumed to exceed the capacity there, it is necessary to *meter* the number of vehicles entering the approach (B, A) so that, on average, there is an input-output balance on the approach to intersection A from one cycle to the next. Specifically, the green time, G_B should control the inflow to the approach to equal the outflow from this approach, over the cycle.

To avoid undue detail for this development, we assume that the mean discharge headway, h, at B is same as at A. Then, *for saturated conditions* on the arterial approach to intersections A and B, the per-lane stable (*i.e.*, static) conservation expression for approach (B,A) is:

$$\frac{Q_{i+1}}{L_v} = \frac{Q_i}{L_v} - \frac{(G_A\text{-}s)}{h_A} + \frac{(G_B\text{-}s)}{h_B}(1-P_B)\ N_c P_c \frac{(LN)_c}{(LN)_A} \qquad (2)$$

where,

C = Cycle Length, sec.

$(LN)_A$ = number of lanes on the arterial approaches to intersections A and B (assumed equal at this time)

$(LN)_c$ = total number of lanes on the cross street approach(es) to intersection B

P_B = Proportion of total traffic along arterial approach to intersection B, that leaves the arterial to turn onto the cross street(s)

P_c = Proportion of cross street traffic (weighted mean of both cross street approaches at intersection B) that turns onto arterial link, (B,A)

N_c = Average number of cross street vehicles per-lane seeking service in one cycle.

Maintaining a stable queue implies, $Q_{i+1} = Q_i$. Asserting this condition in equation (2), assuming a constant value of h and solving for G_B yields:

$$G_B = \frac{G_A - P_{B}s - X_c(C\text{-}s)p_c\frac{(LN)_c}{(LN)_A}}{1 - P_B - X_c p_c\frac{(LN)_c}{(LN)_A}} \qquad (3)$$

where X_c is the demand:capacity ratio on the cross street approach(es), a function of N_c, G_B and C.

This equation holds only if Objectives I and II are met. Since X_c depends on G_B, some iteration is required. Practical considerations require that $P_B \leq \frac{(LN)_c}{(LN)_A}$, and P_B and P_c must be "reasonable" in that the denominator of equation (3) does not approach zero.

Review of equations (1) and (3) reveals that the offset, and the arterial phase duration, G_B, depend on *disparate sets of parameters.* However, the signal offset and split, despite this apparent "separation of variables", are actually *strongly* coupled by the requirements that the mean queue length, Q, be stable over time, (*i.e.*, $Q_{i+1} = Q_i$) and that there be full productive utilization of arterial green time at both intersections (*i.e.*, that saturation conditions prevail on the arterial approaches) and that the incoming arterial platoon from intersection B merge smoothly with the discharging queue on the approach to intersection A. That is, Objectives I, II and V are interdependent:

The offset controls the mean queue length at the beginning of the green phase and maintains the productivity of the system *under the condition* that the phase duration at the upstream intersection maintains a stable queue by balancing input-output, on average, from cycle to cycle.

The arterial green phase duration at the upstream intersection controls the queue stability *under the condition* that the offset maintains the productivity of the system by controlling the queue length.

Thus, these two control parameters under the Internal Metering Policy perform different, but interdependent functions, within the congested environment. The fact that, mathematically, is independent of the phase duration, G, is an important property that will be exploited in the corridor control policy formulation.

2.2 Formulation for Oversaturated Arterials with Multi-Phase Left-Turn

This section presents additional formulation for oversaturated arterial with multi-phase left-turn operation.

2.2.1 Improved Formulation

The number of per-through-lane vehicles per cycle in queue on the approach at the beginning of the arterial green developed for oversaturated arterial with two-phase left-turn includes the number of left-turn vehicles because two-phase arterial doesn' t have exclusive left-turn phase and left-turn bay. By separating left-turn movement, however, the number of per-through-lane vehicles per cycle in queue on the approach at the beginning of the arterial green is different from that of developed for two-phase arterial. In other words, under the multi-phase operations, the number of per-through-lane vehicles per cycle in queue has to be subtracted the number of left-turn vehicles per-through-lane per cycle from that of developed for two-phase operations. Therefore, the term of $(1 - P_B)$ in all equation developed for two-phase has to be changed to $(1 - P_B)(1 - P_A^L)$ except for the equation related shock wave speed. Where, P_A^L is the proportion of left-turn vehicles at downstream intersection.

2.2.2 Additional Formulation

<u>Left -Turn Duration</u>

Left-turn treatment refers to the right-of-way designation to left-turn vehicles. Three potential left-turn treatments are considered:

- Protected, in which left-turning vehicles have exclusive right-of -way during the phase,
- Permissive, left-turning vehicles have no exclusive phase, crossing the intersection in gaps of the opposing traffic stream, and
- Protected-Permissive, a sequenced combination of the previous two treatments.

In internal metering policy, constraints expressed for δ_i, $\overline{\delta_i}$ which provide for protected left-turn movements when needed must be developed. A protected left-turn movement is needed when:

a) Opposing flow is saturated and subject left-turn volume exceeds NLS x NCYC

where, NLS = Number of left-turn sneakers per cycle

NCYC = Number of cycles per hour

b) Opposing flow is undersaturated and subject per - lane left-turn volume exceeds

$N_{LT,perm}(1) = \{\{(g - N_{Lr} \times H_{Lr} - s) / H_{L,perm} + NLS\}$

where, g = Green time servicing opposing volume

$N_{t,r}$ = Number of thru, right-turn vehicles serviced in on-coming direction per cycle

$H_{t,r}$ = Mean discharge headway for thru, right-turn vehicles

s = Start-up lost time

$HL_{,perm}$ = Mean discharge headway for left-turners with no opposing traffic, but during permitted phase

or

Determine an estimate of $N_{LT,perm}(2)$ based on Case 6 of Table 9-12 in the 95HCM. Here we assume that a left-turn bay is provided; this implies that P_{LT} = 1.0 and

for V0 =	0	200	400	600	800	1000	1200	1220+
f_{LT} =	0.86	0.78	0.70	0.61	0.49	0.36	0.20	0.18

Assert a saturation flow rate 1800 vph.

Then, $N_{LT,perm}(2) = (1800 \times f_{LT} \times (g-s)) / 3600$

$= ((g-s) \times f_{LT}) / 2$

Select the the lower value for $N_{LT,perm}$: $N_{LT,perm}(1)$ or $N_{LT,perm}(2)$

Then, $N_{LT,prot}$ is calculated like this.

$N_{LT,prot} = N_{LT} - N_{LT,perm}$

where, $N_{LT,prot}$ = Number of left-turners serviced per cycle during the protected green phase

N_{LT} = Number of left-turners per cycle

$N_{LT,perm}$ = Number of left-turners serviced per cycle during the permitted green phase

Then, L or $\overline{L}$ is calculated like this.

L or $\overline{L} = N_{LT,prot} \times H_{LT} + s$

where,

L = Required green time to service protected outbound left-turners

$\overline{L}$ = Required green time to service protected inbound left-turners

H_{LT} = Mean left-turn discharge headway during protected phase

Left-Turn Phase Sequence

Phase sequence is a critical timimg element for signalized arterials. This section presents a generalization that permits the optimization program to select when the left turn phase will occur with respect to the through green at any signal. The left turn green can be chosen to lead or lag, whichever gives the most total productivity.

Referring to Figure 2, there are four possible left-turn phase patterns. For each, a relationship between r, $\overline{r}$, δ, $\overline{\delta}$, L, $\overline{L}$ was established.

Where, r = Total red time facing outbound traffic flow, per cycle

$\overline{r}$ = Total red time facing inbound traffic flow, per cycle

δ = Elapsed time from start of outbound green phase to start of inbound green phase

$\overline{\delta}$ = Elapsed time from start of outbound red phase to start of inbound red phase

L = Time allocated for outbound left green

$\overline{L}$ = Time allocated for inbound left green

Table 1 summarizes the constraints for each possible left-turn phase pattern.

Table 1. Constraints for Left-Turn Phase Pattern

Pattern	δ
1. (leading/lagging)	$\geq$ L
2. (lagging/leading)	$\leq -\overline{L}$
3. (dual leading)	$= L - \overline{L}$
4. (dual lagging)	$= \overline{r} - r$

To select one pattern that will maximize the productivity among four patterns mixed integer program was employed. In this case, we have two equality constraints and two inequality constraints. To solve this case, firstly, we have to convert equality constraint to inequality constraint. Secondly, we have to rearrange the direction of inequality to get that every inequality equation are larger than zero. Thirdly, we can introduce big number (M) and binary variable(yi). And, finally,we have to add another constraint (y1+y2+y3+y4=3) to solve mixed integer program because this constraint chooses one yi to have zero value.

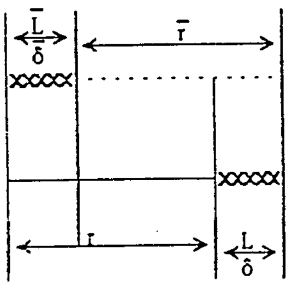
Table 2. Converted Constraints to Solve MILP

Original Constraints	Converted Constraints
$\delta \geq L$ (leading/lagging)	$\delta - L + M \cdot y1 \geq 0$
$\delta \leq -\overline{L}$ (lagging/leading)	$-\delta - \overline{L} + M \cdot y2 \geq 0$
$\delta = L - \overline{L}$ (dual leading)	$\delta - L + \overline{L} + M \cdot y3 \geq 0$ $-\delta + L - \overline{L} + M \cdot y3 \geq 0$
$\delta = \overline{r} - r$ (dual lagging)	$\delta - \overline{r} + r + M \cdot y4 \geq 0$ $-\delta + \overline{r} - r + M \cdot y4 \geq 0$
	y1+y2+y3+y4=3 (yi:binary variable)

Now the original constraints are converted like Table 2.

2.3 Formulation for the IMP Control of Two-Way Arterials

IMPOST (Internal Metering Policy to Optimize Signal Timing) is designed to provide "optimal" signal settings for signalized two-way arterials, to maximize system pro-



--------- inbound
_______ outbound
XXXXX protected left-turn

Note:

$\delta, \overline{\delta} > 0$

$\delta + r = \overline{\delta} + \overline{r} \rightarrow \delta = \overline{\delta} + \overline{r} - r$

$\overline{L} + \overline{r} = L + r \rightarrow L = \overline{L} + \overline{r} - r$

In this pattern, δ and $\overline{\delta}$ are positive. Therefore, two conditions, $\delta > L$ and $\overline{\delta} > \overline{L}$, should be satisfied.

However, if $\delta > L$, $\overline{\delta} > \overline{L}$ is automatically satisfied.

$$\overline{\delta} + \overline{r} - r > \overline{L} + \overline{r} - r$$

$$\overline{\delta} > \overline{L}$$

Therefore, in this pattern, the constraint is $\delta > L$.

Figure 2. Four Left-Turn Phase Patterns (Page 1 of 4)

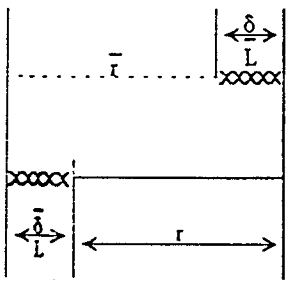
ductivity, for oversaturated, undersaturated and mixed traffic conditions. While the formulation for control of undersaturated approaches does not explicitly minimize delay, the objective of minimizing the adverse effects of platoon dispersion, acts to minimize delay.

In prior sections we have shown how it is possible to apply the IMP to one-way arterials. In fact, several formulations were presented. All formulations, however, were designed to maximize productivity by satisfying, to the extent practicable, the six stated IMP Objectives.

It is well-known that for "closed" signal systems (i.e., those network configurations exhibiting closed loops), the **"signal closure condition"** will likely preclude the possibility of assigning optimal offsets to each approach. For the IMP, an additional condition exists. The optimal arterial green phase duration at an intersection is generally one value when considering one direction of travel along an arterial, and another value when considering the other direction of travel. Thus, to satisfy the IMP, different arterial green times must be assigned at an intersection. This requirement also influences the signal closure condition.

The signal closure condition (for a single green phase servicing each approach at an intersection) simply requires that the sum of offsets around every closed loop equals some integer-multiple of the common signal cycle length. Consequently, one or more of these offsets will, in general, depart from the true optimal value.

Because of the need to satisfy the closure condition for



---------- inbound
________ outbound
XXXXX protected left-turn

Note:

$\delta, \bar{\delta} < 0$

$\delta + r = \bar{\delta} + \bar{r} \rightarrow \delta = \bar{\delta} + \bar{r} - r$

$\bar{L} + \bar{r} = L + r \rightarrow L = \bar{L} + \bar{r} - r$

In this pattern, δ and $\bar{\delta}$ are negative. Therefore, two conditions, $\delta < -\bar{L}$ and $\bar{\delta} < -L$, should be satisfied.

However, if $\delta < -\bar{L}$, $\bar{\delta} < -L$ is automatically satisfied.

$\bar{\delta} + \bar{r} - r < \bar{r} - r - L$

$\bar{\delta} < -L$

Therefore, in this pattern, the constraint is $\delta < -\bar{L}$.

Figure 2. Four Left-Turn Phase Patterns (Page 2 of 4)

two-way arterials, the procedures which implement IMP on one-way arterials (that have no closed loops) are not directly applicable to the two-way arterial nor, of course, to a grid system of streets. Consequently, the IMPOST formulation was developed.

Figure 3 is a schematic of the time-space diagram and of the signal phasing, respectively, which will be used as a basis for developing an Internal Metering Policy to Optimize Signal Timing (IMPOST) for two-way arterials. The symbols shown there are defined below:

L = Approach Length, feet (stop-line to stop-line)

C_a = System-wide Cycle Length, seconds

g_i, g_{i+1} = Durations of green phases servicing traffic travelling in the <u>outbound</u> direction on approaches to intersections i and i+1, respectively.

$\overline{g_i}, \overline{g_{i+1}}$ = As above, but servicing traffic travelling in the <u>inbound</u> direction.

δ_i, δ_{i+1} = Offsets between the start of the green phase servicing <u>outbound</u> traffic and the start of the overlapping green phase servicing inbound traffic, at intersections i and i+1, respectively.

Δ_i = Offset between the start of the green phase servicing <u>outbound</u> traffic at intersection, i, and the start of the green phase servicing outbound traffic at intersection, i+1.

$\overline{\Delta_i}$ = Offset between the start of the green phase servicing <u>inbound</u> traffic at intersection, i+1, and the

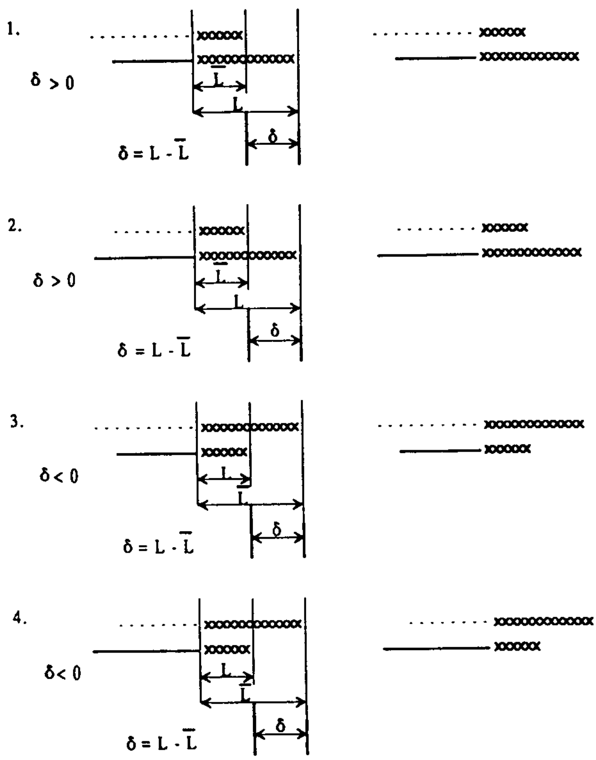


Figure 2. Four Left-Turn Phase Patterns (Page 3 of 4)

start of the green phase servicing inbound traffic at intersection. i.

r_i, r_{i+1} ; = Durations of red phases, as defined above for $\overline{r_i}, \overline{r_{i+1}}$ $g_i, g_{i+1}, \overline{g_i}, \overline{g_{i+1}}$ respectively. In general, $r = 1 - g$; $\overline{r} = \overline{1 - g}$

The loop constraint (*i.e.,* closure condition) for each pair of approaches spanning between intersections i and i+1 is (refer to Figure 3)

$\Delta_i + g_{i+1} + r_{i+1} + \delta_{i+1} + \overline{\Delta_i} - \overline{r_i} - \overline{g_i} - \delta_i = I_i$

which can be written as:

$\Delta_i + \delta_{i+1} + \overline{\Delta_i} - \delta_i = I_i$ where I_i is zero or an integer

The loop constraint for dual leading or dual lagging is exactly the same as that of leading/lagging left-turn phase described above.

2.4 Optimization of Arterial Control Parameters

As shown in Figure 4, the arterial analysis consists of a front-end process wherein all of the geometric properties of the arterial are defined. The next step consists of performing the IMP analysis and computing the system-wide cycle length, C_o; the values of arterial green durations at each intersection, G; the values of bounds on queue length, r_0; and on signal offset, Δ. The next step is to set up the constraint relations and then executing the MILP soft-

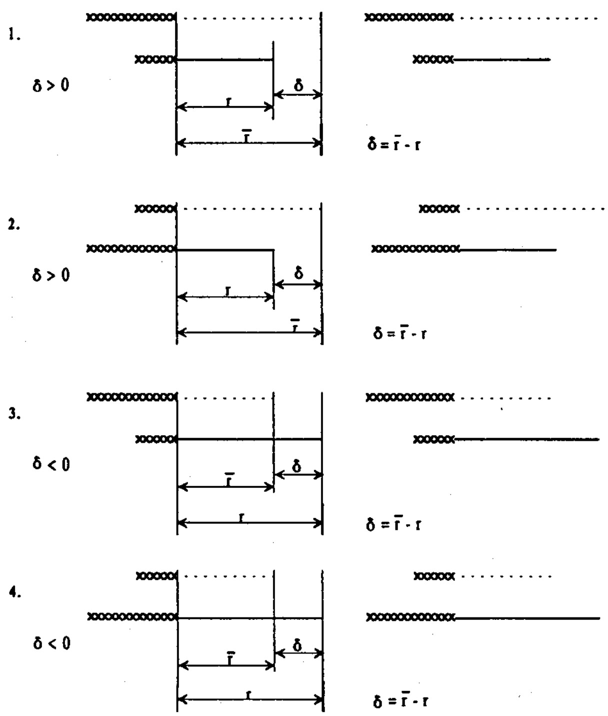


Figure 2. Four Left-Turn Phase Patterns (Page 4 of 4)

ware to perform the IMPOST analysis to obtain the solution vector which yields the signal offsets, Δ and δ, and the associated queue lengths, r0. The complete formation of IMPOST was presented in author' s dissertation.

3. CONCLUSIONS AND RECOMMENDATIONS

3.1 Conclusions

The results presented describe the relationships among the signal parameters (phase duration, phase sequence, cycle length and offset); the approach geometrics (approach length and number of lanes); the arterial and cross street traffic volumes and turn movements; traffic operations (speed, queue formation) and the response of the traffic environment (extent of starvation and spillback, and loss of productivity).

For oversaturated networks servicing two-way traffic, the IMP concepts were applied in the development of a Mixed-Integer Linear Programming (MILP) formulation named IMPOST for two-way arterials. The IMPOST formulation is applicable to all kinds of signalized traffic

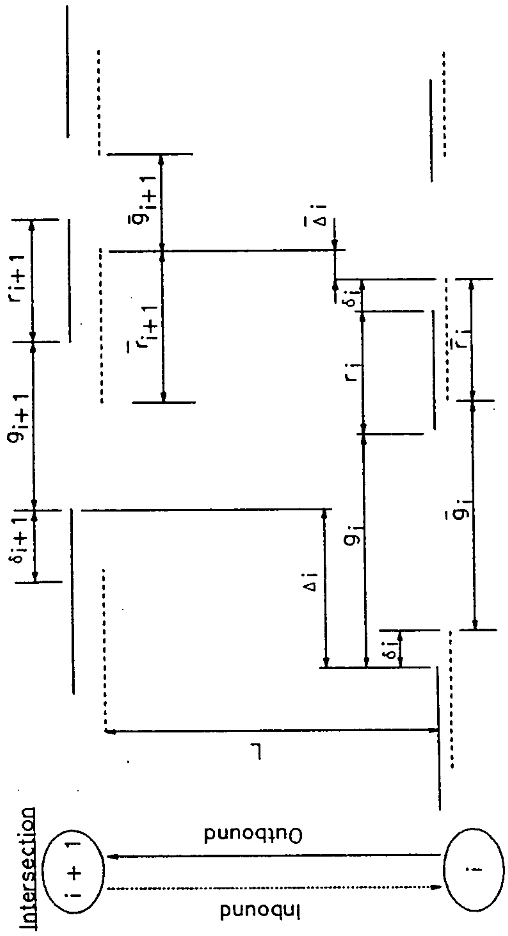


Figure 3. Time-Space Diagram for the IMPOST Algorithm

environments including those consisting only of undersaturated approaches, those of only oversaturated approaches and those exhibiting a mix of undersaturated and oversaturated approaches. The IMP/IMPOST formulations expressly treat all signal control parameters and queue length as dependent [solution] variables.

The IMP policy and IMPOST formulation represent a meaningful departure from current practice in the development of signal timing policies. Specifically, this approach focuses on the need to **manage queue formation** and **maximize productivity** in controlling an oversaturated environment.

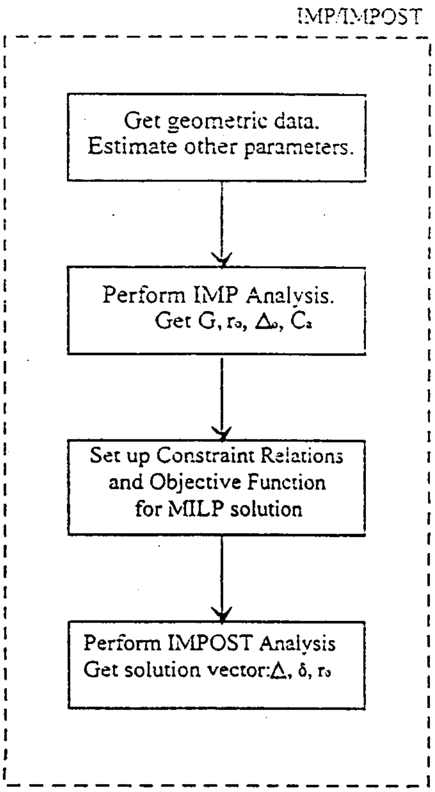


Figure 4. Arterial Analysis Procedure

3.2 Recommendations

As is the case for any new policy, it is necessary to refine, calibrate and test the policy in a laboratory environment. Such an activity should be staged to:

- Get feedback from simulation experiments so as to refine the formulation and enhance it, as needed.
- Apply the CIC at an oversaturated network. Gather before/after data, analyze the results; refine the model, as needed.
- Apply IMP for an oversaturated, one-way arterial. Again, the test program must allow for data collection, analysis, policy refinement and retest.
- Apply IMPOST to an oversaturated, two-way arterial.

The evaluation should proceed, stage by stage, in a progressive manner. If this policy proves to be beneficial, then it should be extended in several ways:

- Develop diversion strategies and integration system capable of interfacing with both freeway ramp control and arterial traffic signal control systems.
- Inclusion of cycle length as a dependent variable should be addressed. Much of the experience gained with MAXBAND model can be used here.
- Extend the IMPOST formulation to accommodate grid network configurations.
- Improve the IMPOST for real-time purpose.

REFERENCES

1. 1985 Highway Capacity Manual. TRB Special Report 209. Transportation Research Board, National Research Council, Washington, D.C., 1985.

2. William R. McShane and Roger P. Roess, "Traffic

Engineering." Prentice Hall Polytechnic Series in Transportation, 1990.

3. Rothenberg, M.J., "Urban Congestion in the United States: What Does the Future Hold?", ITE Journal, Vol. 55, No. 77, July 1985.

4. Lieberman, E.B., et al, "Congestion-Based Control Scheme for Closely Spaced, High Traffic Density Networks", Transportation Research Record 1057, TRB, 1986.

5. Rathi, A.K. and Lieberman, E.B., et al, "Effectiveness of Traffic Restraint for a Congested Urban Networks: Simulation Study", Transportation Research Record 1232, TRB, 1989.

6. D. C. Gazis. "Optimum Control of a System of Oversaturated Intersections", Operation Research, No. 12, 1964.

7. D. Longley. "A Computer Strategy for a Congested Computer - Controlled Traffic Network", Transportation Research, Vol. 2, 1968.

8. N. H. Gartner, John D. C. Little, and Henry Gabbay. "Optimization of Traffic Signal Settings by Mixed-Integer Linear Programming: Part 1 and Part 2", Transportation Science, Nov. 1975.

9. "Internal Metering Policy for Oversaturated Networks", Volume 1 and 2. NCHRP Report 3-38(3), June 1992.

10. P. G. Michalopoulos and G. Stephanopoulos. "Optimum Control of Oversaturated Intersections", Traffic Engineering and Control, Vol. 19, No. 5, 1978.

11. Pignataro, et al , "Traffic Control In Oversaturated Street Networks", NCHRP Report 194, Transportation Research Board, Washington, D.C., 1978.

12. OECD, "Traffic Control in Saturated Conditions", OECD, Paris, 1981.