Travel time predictions in urban networks

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Abstract

In-vehicle information, collective and individual dynamic route guidance, congestion management, and incident detection are all systems requiring predictions of short-term link travel times. Hobeika et al. (1993) were among the early researchers who provided a complete approach for predicting travel times in urban networks under incidents based on anticipatory traffic conditions. This paper extends the early work to include travel time predictions under lane(s) closure for construction and maintenance. The algorithm for lane closure relies on the real time dynamic data obtained from sensors and uses a macroscopic traffic input-output model for calculating delays and consequently travel times. The algorithm has been tested and verified using the well-known CORSIM simulation model developed by Federal Highway Administration.

1 Introduction

The prediction of travel times in urban road networks is a basic component of many traffic monitoring and control systems. In-vehicle information, collective and individual dynamic route-guidance, congestion management and incident detection are all systems requiring predictions of travel times on the given network. Such predictions could also make a useful contribution to bus management and passenger information systems.
Historically, most forecasting models in the transportation area have tended to focus on short-term traffic-flow prediction for use in the operation and control of traffic signals and ramp meters (Okutani and Stephanedes, 1984; Davis and Nihan, 1991; Dougherty and Kirby, 1993). Since the late 1980s, the estimation and prediction of short-term link travel times has become increasingly important. This is in recognition of the fact that for distributed Route Guidance Systems (RGS) to be successful, the calculated routes should be based not only on historical and real-time travel time but also on anticipatory link travel time information.

Hobeika, A.G., et al. (1993) were among the early researchers who provided a complete approach for predicting travel times in urban networks under incidents based on anticipatory traffic conditions. The approach encompassed three levels of analysis. At level 1, the incident clearance time and the total vehicular delays are estimated based on incident characteristics and on real-time traffic data obtained from surveillance systems after the incident is detected and verified. If the delay estimation is beyond a certain threshold, the approach activates level 2 analysis, which searches for the best candidate routes to divert the drivers onto based: 1) on the best feasible diversion routes, 2) on the anticipated demand in the next one hour, and 3) on the traversed travel times on each of the candidate routes under future traffic conditions. Once the diversion route(s) is selected and the route advisory to the public is provided, the traffic control strategies on the arterial system are optimized to take care of the diverted traffic. These last actions are conducted at the level 3 analysis.

The focus of this paper is to present the procedure for determining travel times in an urban network under the incident/lane closure situation. The incident/road closure algorithm is based on the 6-year research conducted at Virginia Tech, which resulted in the Wide Area Incident Management System (WAIMS). WAIMS builds on the 3-year data collection efforts of incidents conducted in Northern Virginia with the help of local and state police and VDOT personnel. The collected data provided an important component of the travel time prediction algorithm, which is the time needed to clear an incident based on incident characteristics and emergency response personnel and equipment. The collected data, which contained 3,000 incidents, has been classified using the Classification and Regression Tree (CART) statistical package, and its outputs for determining clearance times were documented in 1996 [1].

The prediction of clearance time feeds into the macroscopic input-output model developed by Morales (1987) and Lindley (1987) and represented here in Figure 1. The figure is used as a basis for delay and travel time estimations. It shows the constriction of flow due to an incident at a highway link as indicated by the decrease in roadway capacity (C2), where C2 is less than C1, and C1 represents the normal capacity flow of the roadway. This reduction in capacity stays until the incident is completely cleared, at which time the get away capacity of the roadway returns to its normal capacity C1.
During the incident response and clearance times, the queue of traffic builds up and stays that way until the incident is cleared. After that, the queue starts to dissipate until the time to normal flow is reached, which is at the end of the recovery period. The difference between the arrival rate of flow and the departure rate of flow gives the number of vehicles in the queue and also estimates the individual delay experienced by a driver as shown in Figure 1.

2 The bottleneck algorithm

The bottleneck algorithm deals with traffic conditions under lane closure and under reduction in lanes due to incidents and other constrictions due to geometric features. In order to understand the development of the algorithm, figure 2 is prepared to show the bottleneck condition on a freeway and the definition of the symbols used in deriving the algorithm.

2.1 Algorithm procedure

The procedure for finding out travel times when there is a lane closure is applicable when the number of lanes downstream of a section of road is less than the number of lanes upstream of that section. Under such circumstances there is not enough room for vehicles to flow freely and a reduced speed section is created upstream due to conflict from merging traffic. This affects the traffic upstream of the bottleneck and consequently a queue builds up on the freeway. The boundary between the queuing traffic and the arriving traffic, known as ‘Shockwave’ will move upstream against the direction of the traffic. The higher the difference between the vehicular flows at upstream and downstream sections, the greater will be the shockwave speed. The lower the difference between densities at these two sections, the higher will also be the shockwave speed.

A description of variables used in the algorithm is provided here first. The detectors are named DT in Figure 2 and there are five of them. These are the detector on link (i-1), detector on link (i+1) and three detectors on link i. The link i is assumed to have three detectors, one at the upstream section, one at the start of the bottleneck and one at the end of the link. The flows are designated in the figure by ‘q’ and the densities by ‘k’. The three links in consideration are link (i-1), link i, and link (i+1) which are respectively the upstream link (i.e. link (i-1)), the link for which travel time is being calculated (link i) and, the downstream link (i+1). The number of lanes is designated by ‘N’. The subscripts to the symbols refer to the link which they represent and the location of the detector on the link. First we determine what data from detectors should be used in the algorithm based on their availability. From Figure 2, assume that
Figure 1: Delay Calculations
upstream detector \( \text{DT}_{iu} \) is available and functioning, and, either of downstream detectors \( \text{DT}_{ib} \) or \( \text{DT}_{id} \) is available and functioning.

Then, if at time \( t_0 \), (assuming \( \bar{q}_{ib} \) is available)

If, \( \bar{q}_{iu} \times N_{iu} \leq \bar{q}_{ib} \times N_{ib} \), i.e., departing flow is less then the arriving flow, then, no congestion is imminent since approaching flow is less than departing flow. We use the normal travel time estimations as for no closure and no incident case. Also, if \( \bar{q}_{ib} \) is not available, then if

\[
\bar{q}_{iu} \times N_{iu} < \bar{q}_{ib} \times N_{ib},
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\]
\[ q_{i-1}, K_{i-1} \quad q_{i_d}, K_{i_d} \quad q_{i_u}, K_{i_u} \quad q_{i_b}, K_{i_b} \quad q_{i+1}, K_{i+1} \]

- DT indicates the detector location on the freeway
- \( \bar{q} \) is the average flow (vehicle per hour per lane, vphpl) at that detector location
- \( \bar{K} \) is the average density (vehicle per mile per lane, vpmpl) at that location. It is calculated as: density (k) = 2.4 Occupancy (OCC). In the QL area it is 120 vpmpl.
- \( N_{ib} \) = number of available traffic lanes downstream from the start of the bottleneck on link (i).
- \( N_{id} \) = number of available traffic lanes downstream of the bottleneck on link (i)
- \( N_{iu} \) = number of available traffic lanes upstream of the bottleneck on link (i)
- \( N_{i+1} \) = number of available traffic lanes upstream of link (i) on link (i+1)
- \( N_{i-1} \) = number of available traffic lanes downstream of link (i) on link (i-1)
- \( l_d \) = the distance in miles from the start of the bottleneck to the downstream node (end node of link (i))
- \( l_u \) = the distance in miles from the start of the bottleneck to the upstream node (end node of link (i))
- \( l_{i-1} \) = the distance in miles from the start of obstruction to the location of the upstream detector on link (i).
- \( l_{id-1} \) = the distance in miles from the detector DT_{i-1} to the downstream node (end node of link (i-1))
- \( l_{i-1} \) = the distance in miles of link (i-1)

Figure 4 Bottleneck Condition
The following are the various steps in the algorithm:

**Step 1**
Determine the backward forming shock-wave velocity upstream of bottleneck in (mph). Shockwave is the interface between the queuing flow and the free flow. It moves against the direction of traffic as long as the queue is building up. Hence, it is always negative in the queue building case.

\[
W_u = \frac{-q_{i_u} \times N_{i_u} - q_{i_b} \times N_{i_b}}{K_{i_u} \times N_{i_u} - K_{i_b} \times N_{i_b}}
\]

\(W_u\) is negative since the denominator is negative indicating the backward movement of the shock wave

then, the Queuing Rate (QR) (veh/hr) is:

\[
QR = \frac{dn}{dt} = \left( \frac{-q_{i_u} \times N_{i_u} - q_{i_b} \times N_{i_b}}{W_u \times K_{i_u} \times N_{i_u}} \right)
\]

**Step 2**
Determine the number of vehicles in queue (Q) at time \((t_0 + n \Delta t)\) where \(\Delta t\) is the time interval for travel time update on this link (i) (considered in this analysis to be 5 minutes), and \(n\) is the number of update cycle from time \(t_0\).

\[Q_n = QR \times \Delta t_n\]

\(Q_m\) is reached when \(-q_{i_u} \times N_{i_u} \leq q_{i_b} \times N_{i_b}\) i.e. when the approaching flow rate becomes less than the departing flow rate. Repeat for every interval and estimate the total number of vehicles in queue using the following expression.

\[Q_m = \sum_{n=1}^{m} Q_n\]

**Step 3**
Now, we determine the average travel time \(\bar{t}_{b_n}\) in hours in the congested region. This is estimated based on the following logic. For \(Q_m\) vehicles to leave the queue at a rate of \(q_{i_b}\) vehicles per hour from \(N_{i_b}\) lanes, the average time required would be

\[\bar{t}_{b_n} = \frac{Q_m}{q_{i_b} \times N_{i_b}} \times \frac{1}{2}\]

hours, for \(n = 1, 2, 3...m\)

This is based on the assumption that the downstream flow remains constant.

**Step 4**
Determine the Queue Length (QL) in miles from the start of the bottleneck:

\[QL = \frac{Q_m}{K_{i_b} \times N_{i_u}}\]

The above equation is valid as long as \(QL < s\).
If $QL \geq s$, but $l_u \leq l_u \leq l_u + l_{d_{i-1}}$, then substitute $\bar{q}_{i_u}, \bar{k}_{i_u}$ with $\bar{q}_{i_h}, \bar{k}_{i_h}$ which are the historical flow and densities for the link 'i', if $\bar{q}_{i_u} < \bar{q}_{i-1}$ to determine the flow and density of approaching traffic on link (i) in the next update cycle.

Where, $\bar{q}_{i_h} = \text{The mean historical flow for that day type and time period}$

$\bar{k}_{i_h} = \text{The mean historical density for that day type and time period}$

$\bar{q}_{i-1} = \text{The flow on the immediately upstream link (i-1)}$

However, if $\bar{q}_{i_u} \geq \bar{q}_{i-1}$, then use $\bar{q}_{i-1}$ and $\bar{k}_{i-1}$ as the flow and density of the approaching traffic on link (i) in the next update cycle.

Now, if $l_u \leq QL \leq l_u$, then substitute $\bar{q}_{i_u}, \bar{k}_{i_u}$ with $\bar{q}_{i_h}, \bar{k}_{i_h}$ as the flow and density of the approaching traffic on link (i) in the next update cycle.

If, $QL > l_u + l_{d_{i-1}}$, then use $\bar{q}_{i-1}h$ and $\bar{k}_{i-1}h$ as the historical flow and density values for link (i-1) in the next update cycle.

Repeat (6) and (7) for link $l_{i-2}, l_{i-3}, l_{i-4}$ and so on if necessary.

In general, if $N_{i_u}$ differs upstream use the average length of queue in each link weighted by the different number of lanes on each link.

**Step 5**

Determine the total travel time ($TT_i$) in hours on the link (i);

If equation (5) is valid here, i.e. $QL < s$ then the travel time on the link $i$ is split into three components. First, the travel time upstream of the shockwave, second the travel time in congested region and third the travel time downstream of the congested region. The travel time upstream of the congested region is estimated by dividing the length of the un-congested section by the average speed in the un-congested section. The average speed in the un-congested region is found by dividing the flow by the density. The travel time in the congested region is found as described in equation (4). The travel time downstream of the congested region is again determined by dividing the length by the average speed.

$$TT_i = \left(\frac{l_u - QL}{\bar{q}_{i_u} + \bar{k}_{i_u}}\right) + \frac{l_d}{\bar{q}_{i_d} + \bar{k}_{i_d}}$$

or, if $s \leq QL \leq l_u$,

and if $\bar{q}_{i_u} < \bar{q}_{i-1}$ then;
or, if \( q_{i_u} \geq q_{i-1} \), then:

\[
TT_i = (l_i - QL) \frac{\bar{q}_{i-1}}{K_{i-1}} + \bar{t}_{b_n} + l_d \frac{\bar{q}_{i_d}}{K_{i_d}} + \left[ l_i - (Q - I_u) \right] \frac{\bar{q}_{i-1}}{K_{i-1}}
\]

(10)

if \( QL > I_u \), i.e. equations (6) and (7) hold true, then travel time on link \( i \) has only two components. The travel time in congested region and the travel time downstream of the congested region:

\[
TT_i = \bar{t}_{b_n} \times \frac{l_i}{QL} + l_d \frac{\bar{q}_{i_d}}{K_{i_d}}
\]

(11)

In this case the travel time on link \( i_{-1} \) can be calculated as follows:

\[
TT_{i-1} = \bar{t}_{b_n} \times \frac{QL - I_u}{QL} + \left[ l_{i-1} - (Q - I_u) \right] \frac{\bar{q}_{i-1}}{K_{i-1}}
\]

(12)

otherwise; If \( QL - I_u > l_{i-1} \)

\[
TT_{i-1} = \bar{t}_{b_n} \times \frac{QL - I_u}{QL} + \left[ l_{i-1} - (Q - I_u) \right] \frac{\bar{q}_{i-1}}{K_{i-1}}
\]

(13)

\textbf{Step 6}

Repeat steps 1 to 5 for the next update cycle 2 of \( \Delta t \) (\( \Delta t \) is chosen here to be 5 minutes) and so on until the cycle is reached where; \( q_{i_u} \times N_{i_u} \leq \bar{q}_{i_b} \times N_{i_b} \) then execute the forward shockwave condition which is a repeat of the above steps where the shockwave is positive now.

\section*{3 Conclusions}

The algorithm has been tested on freeway I-66 in Washington, D.C., where several sensors are available on several adjacent links of the freeway. The results were compared to the outputs obtained from the simulation of the same stretch of road using the CORSIM simulation package. The algorithm travel time estimates were in general 5% greater than those obtained from the simulation model. This is attributed to the macro-modeling approach used in this algorithm compared to the micro-simulation approach used in CORSIM. The results were very encouraging and the author is currently pursuing implementation of the algorithm in a real world Traffic Control Center.
References