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Roadway Incident Analysis with a Dynamic User-Optimal Route Choice Model[1]

David E. Boyce[2], Der-Horng Lee[2] and Bruce N. Janson[3]

Abstract

Applications and extensions of a dynamic network equilibrium model to the ADVANCE Network are described in this chapter. ADVANCE is a dynamic route guidance field test designed for 300 square miles (770 square kilometers) in the northwestern Chicago area. The proposed dynamic route choice model is formulated as a link-time-based variational inequality (VI) and can be solved efficiently by Janson's algorithm, DYMOD. Realistic traffic engineering-based link delay functions, instead of simplistic BPR (Bureau of Public Roads) functions, are adopted to estimate link travel times and intersection delays of various types of links and intersections. Nearly 23,000 links and 10,000 nodes are modeled in this research. The time-dependent link flow, travel time, speed and queue spillback information are generated for the ADVANCE Network. The ADVANCE Network is divided into 447 zones, originally specified by the CATS (Chicago Area Transportation Study), to assign time-varying travel demand on the basis of CATS estimates for 1990.

Unexpected events that cause nonrecurrent traffic congestion are analyzed with the model. Route choice behavior based on anticipatory and non-anticipatory network conditions are considered in performing the incident analysis. This is the largest dynamic route choice solution which has been obtained thus far, to the knowledge of the authors. The model was solved using the CONVEX–C3880 at the National Center for Supercomputing Applications (NCSA), University of Illinois at Urbana-Champaign. Convergence and computational results are obtained and analyzed.

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1 Introduction

Intelligent Transportation Systems (ITS), also known as Intelligent Vehicle Highway Systems (IVHS), are applying advanced technologies (such as navigation, automobile, computer science, telecommunication, electronic engineering, automatic information collection and processing) in an effort to bring revolutionary improvements in traffic safety, network capacity utilization, vehicle emission reductions, travel time and fuel consumption savings, etc. Within the framework of ITS, Advanced Traffic Management Systems (ATMS) and Advanced Traveler Information Systems (ATIS) both aim to manage and predict traffic congestion and provide historical and real-time network-wide traffic information to support drivers' route choice decisions. To enable ATMS/ATIS to achieve the above described goals, traffic flow prediction models are needed for system operation and evaluation.

This chapter describes a large-scale dynamic route choice model and solution algorithm for predicting traffic flows off-line for ATMS and ATIS; the solution algorithm, DYMOD, was developed by Janson (1991a, 1991b) and Janson and Robles (1995). A variational inequality (VI) formulation provides an improved theoretical basis for the solution algorithm (Lee, 1996). To solve the VI model, a multi-interval, time-varying-demand route choice problem with fixed *node time intervals* (denoted as the diagonalized or *inner* problem) is formulated. This problem is equivalent to a sequence of static route choice problems. Then, the shortest route travel times and *node time intervals* constraints are updated in an *outer* problem with temporally-correct routes and time-continuous traffic flow propagation.

To generate time-dependent traffic characteristics for a large network, realistic traffic engineering-based link delay functions such as Akcelik (1988) functions are applied for better estimation of link delays at various types of links and intersections. Unexpected capacity reducing events causing nonrecurrent traffic congestion are analyzed with the model. Route choice behavior based on anticipatory and non-anticipatory network conditions are considered in performing the incident analysis, extending the capability of this model to contribute to the evaluation of ATMS and ATIS.

Section 2 describes the mathematical formulation and solution algorithm of the model. The link travel time functions are described in Section 3. The ADVANCE Network representation, travel demand and input data preparation are presented in Section 4. Computational results and output analysis are reported in Section 5. Conclusions and a summary are provided in the last section.

2 Model Formulation and Solution Algorithm

The link-time-based variational inequality model of the dynamic user-optimal (DUO) route choice problem solves for the following travel-time-based ideal DUO state (Ran and Boyce, 1996):

> **Travel-Time-Based Ideal DUO State:** *For each O-D pair at each interval of time, if the actual travel times experienced by travelers departing at the same time are equal and minimal, the dynamic traffic flow over the network is in a travel-time-based ideal dynamic user-optimal state.*

The link-time-based variational inequality model can be stated based on the following equilibrium conditions:

$$\Omega_{ra}^{d*} = \pi_{ri}^{d*} + \tau_a[d + \bar{\pi}_{ri}^{d*}] - \pi_{rj}^{d*} \geq 0 \qquad \forall\, a = (i,j), r, d; \tag{1}$$

$$x_a^{rs*}\left[d + \bar{\pi}_{ri}^{d*}\right] \Omega_{ra}^{d*} = 0 \qquad \forall\, a = (i,j), r, s, d; \tag{2}$$

$$x_a^{rs}\left[d + \bar{\pi}_{ri}^{d*}\right] \geq 0 \qquad \forall\, a = (i,j), r, s, d. \tag{3}$$

where Ω^d_{ra} = difference between the minimal travel time from zone r to node i (π^d_{ri}) plus the travel time on link a ($\tau_a[d+\bar{\pi}^d_{ri}]$; $a=(i,j)$), and the minimal travel time from zone r to node j (π^d_{rj}), for vehicles departing from zone r in time interval d

π^d_{ri} = minimal actual travel time from zone r to node i for flow departing in interval d

$\bar{\pi}^d_{ri}$ = number of time intervals traversed in π^d_{ri}

$x_a[t]$ = total flow of vehicles on link a in time interval t

$x^{rs}_a[d]$ = flow of vehicles from zone r to zone s on link a that departed in time interval d

$\tau_a(x_a[t])$ = actual travel time on link a with flow x_a in time interval t

t = a time interval

$\mathcal{N}$ = set of all nodes

$\mathcal{Z}$ = set of all zones (i.e., trip end nodes)

$\mathcal{K}$ = set of all links (directed arcs)

$\mathcal{T}$ = set of all time intervals in the full analysis period (e.g., 18 ten–minute intervals for a three–hour analysis period)

Therefore, the VI formulation is as follows:

$$\text{VI:} \quad \sum_d \sum_{rs} \sum_a \Omega^{d*}_{ra} \left\{ x^{rs}_a[d+\bar{\pi}^{d*}_{ri}] - x^{rs*}_a[d+\bar{\pi}^{d*}_{ri}] \right\} \geq 0 \qquad (4)$$

where $*$ denotes the equilibrium solution, and where the above variables satisfy definitional, flow conservation, nonnegativity and flow propagation constraints (Lee, 1996).

The algorithm for solving the proposed variational inequality model of the DUO route choice model is shown in Figure 1. The *outer* step solves for the *node time intervals* $\{\alpha^d_{ri}[t]\}$, a [0,1] indicator of whether the flow departing zone r in time interval d has crossed node i in time interval t, and shortest route travel times $\{\pi^d_{ri}\}$ with fixed link flows $\{x_a[t]\}$. Then, the algorithm solves the route choice problem using the Frank-Wolfe method with fixed *node time intervals* $\{\alpha^d_{ri}[t]\}$ and shortest route travel times $\{\pi^d_{ri}\}$. The adjustments of link capacities proceed between the *inner* and *outer* steps to account for capacity changes caused by spillback queuing effects, signal timing changes, incidents and other events. The algorithm terminates when the number of changes of *node time intervals* between two consecutive *outer* iterations is less than an acceptable threshold.

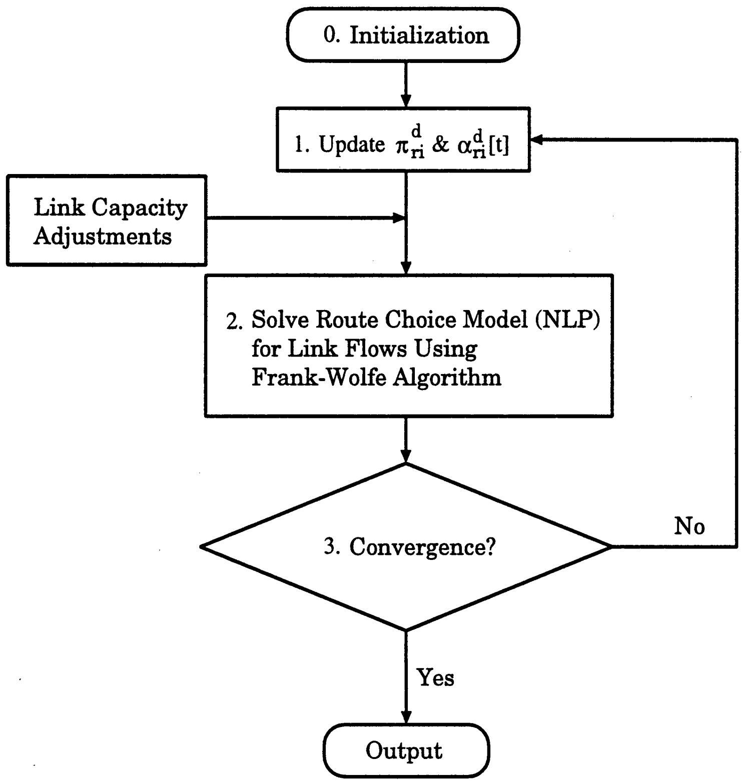

Figure 1: Flowchart of the Solution Algorithm

Steps of the algorithm are described as below:

Step 0: Initialization. Input all network data, temporal trip departure rates and initial link flows. Initial link flows are optional and can be set to zero. However, the static user-optimal flows reduced to the chosen time interval duration may be good initial values. Calculate initial $\{\alpha^d_{ri}[t]\}$ and $\{\pi^d_{ri}\}$ with initial link flows. Set *outer* iteration counter $m = 1$.

Step 1: Update Node Time Intervals and Shortest Route Travel Times. For fixed link flows $\{x_a[t]\}$, update the node time intervals $\{\alpha_{ri}^d[t]\}$ and shortest route travel times $\{\pi_{ri}^d\}$ to maintain temporally-correct, time-continuous routes and flow propagations.

Step 2: Solve Route Choice Problem. Solve the DUO program using the Frank-Wolfe algorithm with the optimal values of *node time intervals* $\{\alpha_{ri}^d[t]\}$ from Step 1, which simplifies to a sequence of static route choice problems.

Step 3: Convergence Test for Updating Iterations. Sum the total number of node time interval differences (NDIFFS) between iterations $m-1$ and m. Compare each $\{\alpha_{ri}^d[t]\}^m$ to $\{\alpha_{ri}^d[t]\}^{m-1}$. If NDIFFS $\leq$ allowable percentage of all *node time intervals*, then stop. Otherwise, return to Step 1.

To determine $\{\pi_{ri}^d\}$ and $\{\alpha_{ri}^d[t]\}$, the following procedure is used in Step 1:

1. find shortest route travel times, $\{\pi_{ri}^d\}$.

2. reset values of $\alpha_{ri}^d[t]$ as follows:

   if $(\bar{\pi}_{ri}^d \leq t\Delta t)$ and $(\bar{\pi}_{ri}^d \geq (t-1)\Delta t)$

   then $\alpha_{ri}^d[t] = 1$; otherwise $\alpha_{ri}^d[t] = 0$

   perform for all $r \in \mathcal{Z},\ i \in \mathcal{N};\ d = 1, \cdots \mathcal{T}$ and $t = d,\ d+1, \cdots \mathcal{T}$

   note: $\sum_{t=d}^{\mathcal{T}} \alpha_{ri}^d[t] = 1$

3. Set $\pi_{rr}^d = d\Delta t,\ \forall\, r \in \mathcal{Z},\ d \in \mathcal{T}$

4. enforce the first-in-first-out (FIFO) conditions:

$$\pi_{ri}^d = \max\left(\beta_{ri}^d\ ,\ \pi_{ri}^{d-1} + h\Delta t\right) \quad \forall\, r, i, d \text{ and } \pi_{ri}^0 = \pi_{ri}^1 - \Delta t \tag{5}$$

$$\theta_{ri}^d[t] = \left[\left(\pi_{ri}^d - (t-1)\Delta t\right)/\Delta t\right] \alpha_{ri}^d[t] \quad \forall\, r, i, d, t \tag{6}$$

$$\mu_{ra}^d[t] = \left[\theta_{ri}^d[t]\tau_a(x_a[t]) + (1-\theta_{ri}^d[t])\tau_a(x_a[s])\right] \alpha_{ri}^d[t] \quad \forall\, r,a,d,t,\; s=t-1 \quad (7)$$

$$\left\{\beta_{rj}^d - \max\left[\pi_{ri}^d\,,\; (t-1)\Delta t + \Delta\tau_a^t[s]\right]\right\} \alpha_{ri}^d[t] \le \mu_{ra}^d[t]\alpha_{ri}^d[t] \quad (8)$$

$$\forall\, r,a,d,t,\; s=t-1; \quad \text{where } \Delta\tau_a^t[s] = \tau_a(x_a[s]) - \mu_{ra}^d[t]$$

where β_{ri}^d = time at which the last flow departing zone r in time interval d crosses node i via its shortest route, less FIFO delay time at node i

$\theta_{ri}^d[t]$ = fraction of a time interval t that the last flow departing zone r in time interval d crosses node i

$\mu_{ra}^d[t]$ = average travel time on link a of the last flow departing zone r in time interval d

h = minimum fraction of time interval separating flows departing in successive time intervals

Equations (5)–(8) impose FIFO trip ordering between all O-D pairs according to their travel times in successive time intervals. Vehicles are assumed to make only one-for-one exchanges of traffic positions along any link, which is acceptable and expected in aggregate traffic flow models. For detailed descriptions of the FIFO conditions used, see Lee (1996).

3 Link Travel Time Functions

Mathematical functions are used within the route choice model to estimate link travel times for given flow rates. The choice of the delay functions involves several criteria: (1) the desired mathematical properties of the function to satisfy the condition for a unique solution of the model; (2) the cost and limited availability of road data; (3) the computational effort required by the model; and (4) the desired accuracy of the travel time estimates generated by the model.

Delay functions selected for this study can be classified by road type and intersection type. First, delay functions for signalized intersections are presented, next for unsignalized intersections, and then for freeway-related facilities.

The specific delay function for links at signalized intersections applied in the study has the following form (Akcelik, 1988):

$$d = \frac{0.5C(1-u)^2}{1-ux} + 900T\gamma \left[x - 1 + \sqrt{(x-1)^2 + \frac{8(x-0.5)}{cT}} \right] \tag{9}$$

where d = average delay per vehicle (second/vehicle)

C = signal cycle length (second)

$u = g/C$ = green split

g = green time (second)

$x = v/c$ = flow-to-capacity ratio

T = duration of the flow (hour)

γ = 1, for $x > 0.5$; 0, otherwise

The first term, called the *uniform delay*, was originally developed by Webster (1958). It reflects the average delay experienced by drivers in *undersaturation conditions*, that is when the arrival flow does not exceed capacity. In oversaturation conditions, $x = 1$ is used in the uniform delay term. The second term of Equation (9) is called the *overflow delay*. It reflects the delay experienced by the vehicles when the flow rate is close to or exceeds capacity. Temporary overflow at an intersection may also occur when the average arrival rate is lower than the capacity, due to a random character of the arrival pattern. The earliest delay functions (e.g., Webster, 1958) were based on the steady-state model and were defined only for undersaturation conditions. Figure 2 shows an example of an Akcelik function.

Delay functions for unsignalized intersections, whether major/minor priority intersections or all-way-stop controlled intersections, are discussed next. For major/minor priority inter-

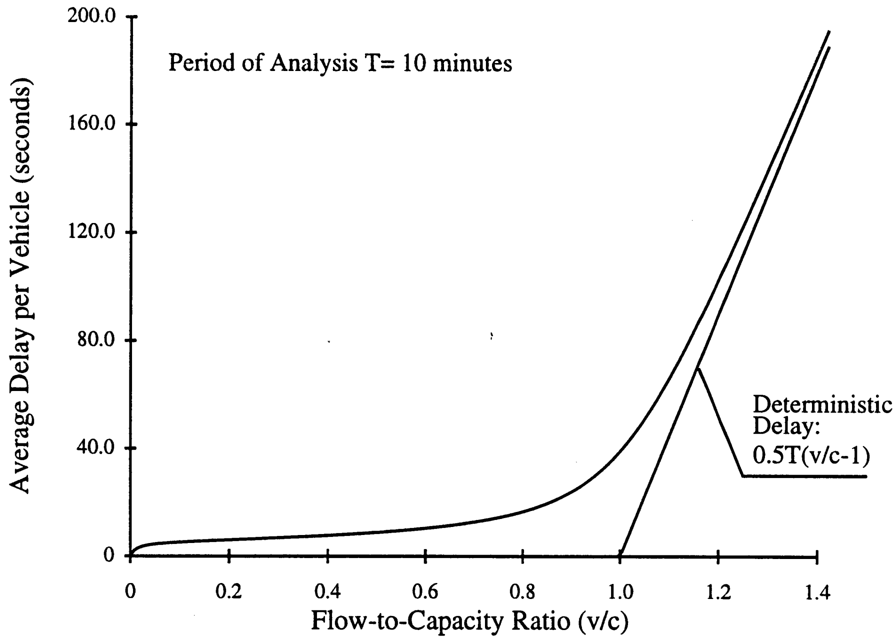

Figure 2: Steady-State Delay Model vs. Time Dependent Formulae

sections, formulae developed by Kimber and Hollis (1979) are adopted by a related study of an asymmetric route choice model. While the delay function developed by Kimber and Hollis is being tested by that study in the large-scale network, the BPR function is temporarily used for estimating delays at major/minor priority intersections.

As for the delay function for all-way-stop intersections, the following exponential delay model is used (Meneguzzer et al., 1990):

$$d = \exp\left[3.802(v/c)\right] \tag{10}$$

where d is the average approach delay, v is the total approach flow, and c is the approach capacity. Note that the form of this exponential function, which is relatively flat at low flow-to-capacity ratio but becomes very steep as the degree of saturation increases, reflects the operational characteristics of all-way-stop intersections well. Kyte and Marek (1989) found that approach delay is approximately constant and in a range of five to ten seconds per vehicle for approach flows up to 300 to 400 vehicles/hour, but increases exponentially beyond this threshold. An increase in conflicting and opposing flows has the effect of reducing this threshold. In addition, it should be noted that Equation (10) is suitable for use in a network equilibrium model, since it is defined for any flow-to-capacity ratio.

Several types of freeway-related facilities occur within the test area: basic freeway segments, ramps and ramp-freeway junctions, weaving sections and toll plazas. A sophisticated scheme of delay functions has been developed for individual freeway-related facilities (Berka et al., 1994). Those functions are expected to be incorporated into the solution procedure in the future. The BPR function is adopted for freeway-related facilities.

4 The ADVANCE Network

ADVANCE (Advanced Driver and Vehicle Advisory Navigation Concept), a field test of ATIS was recently concluded by the Illinois Department of Transportation (IDOT) and the Federal Highway Administration (FHWA), in collaboration with the University of Illinois at Chicago, Northwestern University, and the IVHS Strategic Business Unit of Motorola, Inc.

The ADVANCE Test Area is depicted in Figure 3. It is located in the northwestern suburbs of the Chicago area and covers about 800 square kilometers. Dense residential communities, office centers, regional shopping centers, subregional government centers, and the O'Hare International Airport are located in the ADVANCE Test Area. The network topology of the test area is almost a regular grid with a few diagonal major arterials directed towards the Chicago CBD. The freeway system includes I–90, I–94, I–190, I–290, I–294, IL–20 and IL–53. Except for the remote northwest corner, the freeways serve nearly all parts of the Test Area. The southwest quadrant is characterized by modern, multi-lane arterials designed for high volumes. In Figure 3, collectors, arterials and freeways are drawn with lines of different widths, freeways being the widest line. The heavy black line indicates the boundary of the ADVANCE Test Area.

The ADVANCE Test Area is divided into 447 zones, originally specified by the Chicago

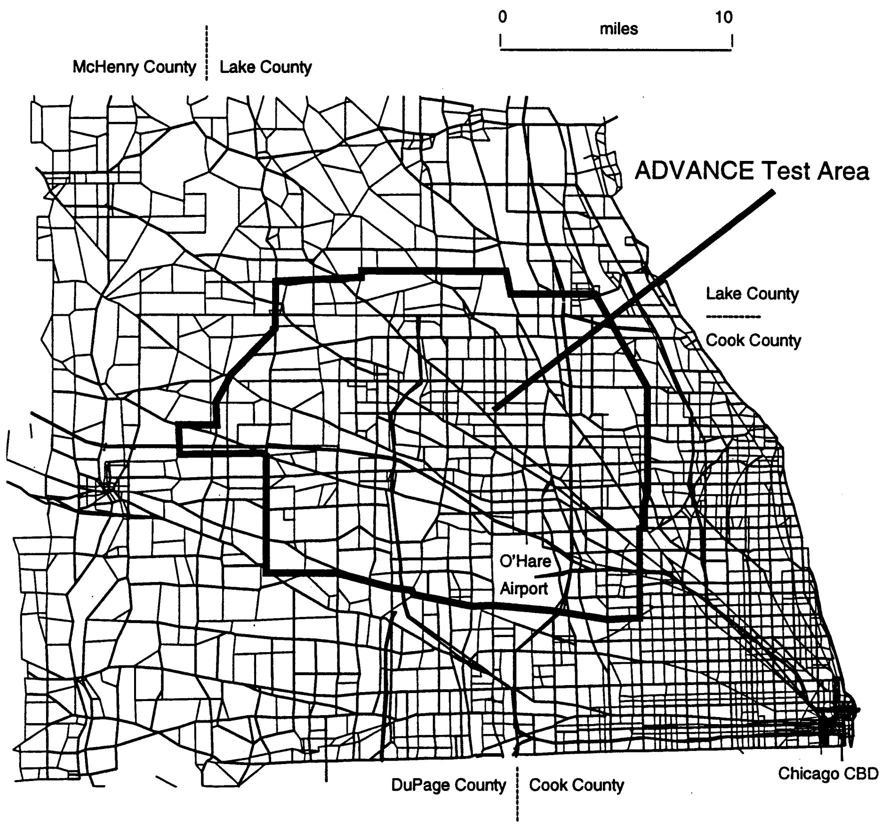

Figure 3: Test Area and Network in the Northwestern Suburbs of Chicago

Area Transportation Study (CATS), to assign time-dependent travel demand. Daily trip tables based on CATS estimates for 1990 were factored to represent travel demand for five time-of-day periods (night, 12 am to 6 am; morning peak, 6 am to 9 am; mid-day, 9 am to 4 pm; afternoon peak, 4 pm to 6 pm; and evening, 6 pm to 12 am). Each time-of-day period is divided into 10-minute intervals used for solving the proposed dynamic route choice model. The 10-minute departure rates for each origin zone are derived from the time-of-day half-hour departure rates obtained from CATS. Trips originating and/or terminating outside the test area are represented by zones on the boundary. An auxiliary analysis of these flows is based on route flows generated by a static route choice model for a larger region encompassing the ADVANCE Test Area (Zhang et al., 1994).

In a conventional route choice model, the network is coded so each intersection is represented as a single node and each approach is represented as a single link. With conventional network representation, 7,850 approach links and 2,552 nodes are included in the ADVANCE Test Area (Table 1). Table 2 lists the frequency of links by the facility type. Table 3 presents the breakdown of arterial/collector intersections by the number of the legs and control type. To account for better link flow/delay relationships and potential queue spillback effects, each turning movement is coded as a separate link called an intersection link. For example, a typical four-leg intersection with two-way approaches without any turning restrictions (U-turn excluded), four approach nodes, four exit nodes and twelve intersection links are required in this expanded network representation (see Figure 4).

The expanded intersection representation procedure is applied only to nodes representing an intersection of arterials or collectors. Nodes that do not need to be expanded are freeway nodes, no-delay intersections, and other nodes not representing intersections. Because of the expanded network representation, the network size increases about three times in comparison with conventional network representation. To that end, 22,918 links and 9,700 nodes are

Table 1: Size of the Test Network

Number of Links	Number of Nodes	Number of Zones
7,850	2,552	447

Table 2: Frequency of Links by Facility Type

Type of Facility	No. of Links
Arterial/Collector	4,061
Tollway/Freeway	197
Freeway Ramp	202
Toll Plaza	14
Freeway Weaving Section	11
Centroid Connector Links	2,491
Approach Link	874
Total	7,850

Table 3: Intersection Frequency by Number of Legs and Control Type

	Signalized	Priority	All-way-stop	Total
Three-leg	257	174	51	482
Four-leg	558	52	60	670
Five-leg and more	7	0	0	7
Total	822	226	111	1,159

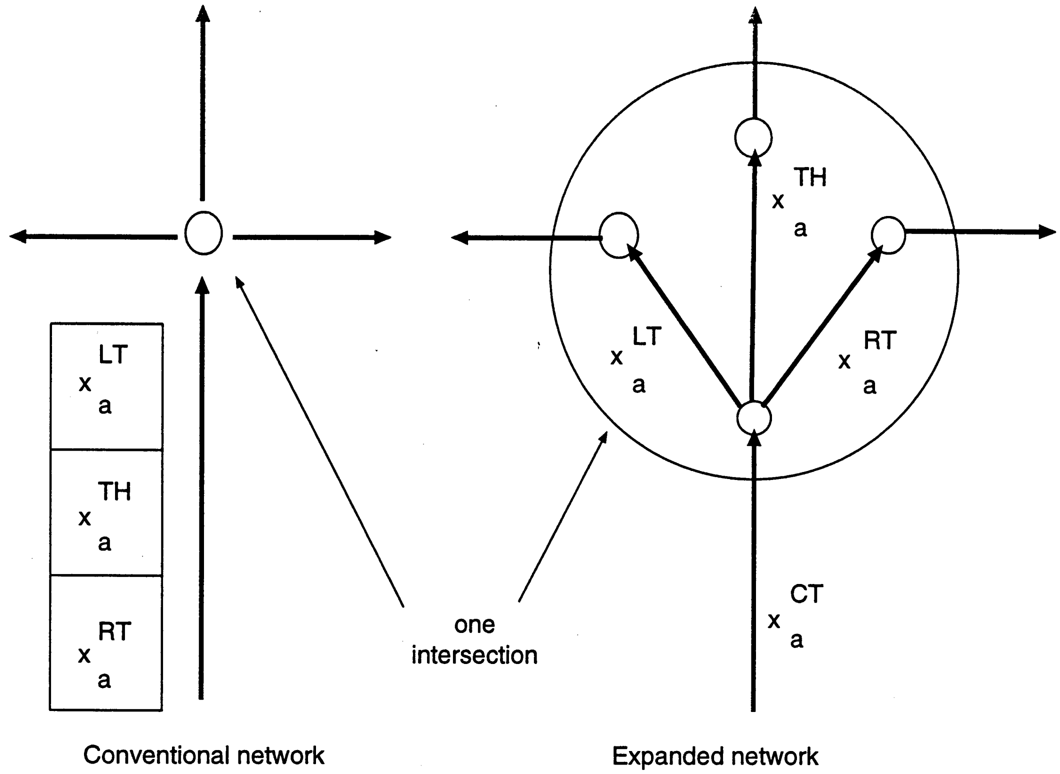

Figure 4: Expanded Intersection Representation

actually modeled for solving the proposed dynamic user-optimal route choice model. Note that the detailed delay functions by turning movements are applied only within the actual ADVANCE Test Area.

In order to utilize the traffic engineering-based link delay functions,the following data need to be available. Besides the typical input data such as link capacity and free flow travel time, types of turning movements (left, through, right), link facility types (e.g., centroid connector, freeway, tollway, arterial), intersection categories, types of traffic control at intersections (e.g., signalized, priority, all-way-stop), cycle length, saturation flow, and signal timing split are required with this regard.

A static asymmetric user-optimal route choice model for ADVANCE Network was used to generate the above information (Berka et al., 1994). That static asymmetric user-optimal route choice model is the basis for generating link travel times for the static link travel time profiles of the ADVANCE Project.

5 Computational Results and Analysis

The model is implemented on the CONVEX–C3880 at the National Center for Supercomputing Applications (NCSA), University of Illinois at Urbana-Champaign. The Convex–C3880 is a vector shared memory machine consisting of 8 processors (240 MFLOPS per processor peak) and 4 Gbytes of memory and 60 Gbytes disk space. For the ADVANCE Network with morning peak (18 ten–minute intervals) travel demand, 512 Mbytes of memory and nearly 60 CPU hours are needed to reach a very fine convergence (see definition in the next subsection) from a zero flow initial solution. A modified version of DYMOD can be executed with much less memory (only 55 Mbytes are needed for the same problem size as described above) and thus can be implemented on a workstation; however, it requires higher CPU times and more disk space.

Five global network performance measures are adopted to monitor the solving process of the Convergent Dynamic Algorithm and to assess the traffic condition over the ADVANCE Network. Definitions of these global network performance measures are listed as below:

1. *Average travel time*

$$\bar{c} = \frac{1}{R}\sum_a \sum_t c_a[t]x_a[t]$$

where	$\bar{c}$	=	average travel time (minutes)
	R	=	total flow during the analysis period (trips/period)
	$c_a[t]$	=	travel time on link a at time interval t (minutes)
	$x_a[t]$	=	flow on link a at time interval t

2. *Average travel distance*

$$\bar{\ell} = \frac{1}{R}\sum_a \sum_t \ell_a x_a[t]$$

where $\bar{\ell}$ = average travel distance (miles)

ℓ_a = length of the link a (miles)

3. *Network space mean speed*

$$\bar{S} = \bar{\ell}/(\bar{c}/60)$$

where $\bar{S}$ = network space men speed (mph)

4. *Average flow-to-capacity ratio*

$$\bar{x} = \frac{1}{K}\frac{1}{T}\sum_a \sum_t \frac{x_a[t]}{C_a[t]} x_a[t]$$

where $\bar{x}$ = average flow-to-capacity ratio

K = number of links in the network

T = number of time intervals

$C_a[t]$ = capacity of link a at time interval t

5. *Convergence Index*

The convergence index monitors the change in node time intervals between two consecutive *outer* iterations. The solution algorithm terminates if the change in node time intervals (NDIFFS) is less than $nodes \times zones \times intervals \times x$. This index indicates that only x changes of the total node time intervals are allowed for the last flow that departed from each zone over the total analysis period. With the perfect convergence, the changes of node time intervals equal to zero.

Table 4 shows the selected characteristics of the final solution for morning peak period of the ADVANCE Network and separated road classes (collector, arterial and freeway related facility). Figures 5, 6, 7 and 8 show the variations of the network performance measures among the *outer* iterations.

Table 4: Solution Characteristics for Morning Peak Period by Road Class

Link Class	Travel Distance (miles)	Travel Time (minutes)	Mean Speed (mph)
Collector	1.18 (1.34)	4.20 (5.97)	16.86 (13.47)
Arterial	6.03 (6.83)	17.42 (21.65)	20.77 (18.92)
Freeway	2.82 (3.01)	3.62 (3.95)	46.74 (45.73)
All Classes	10.02 (11.18)	25.24 (31.61)	23.81 (21.22)

(·) results from Berka et al. (1994)

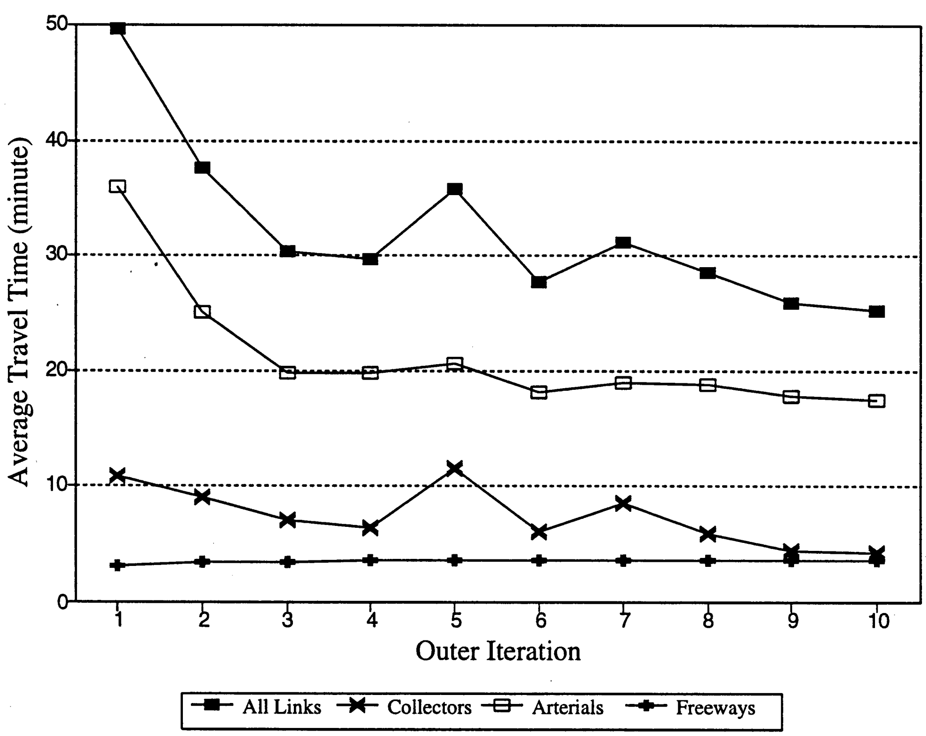

Figure 5: Predicted Average Travel Time

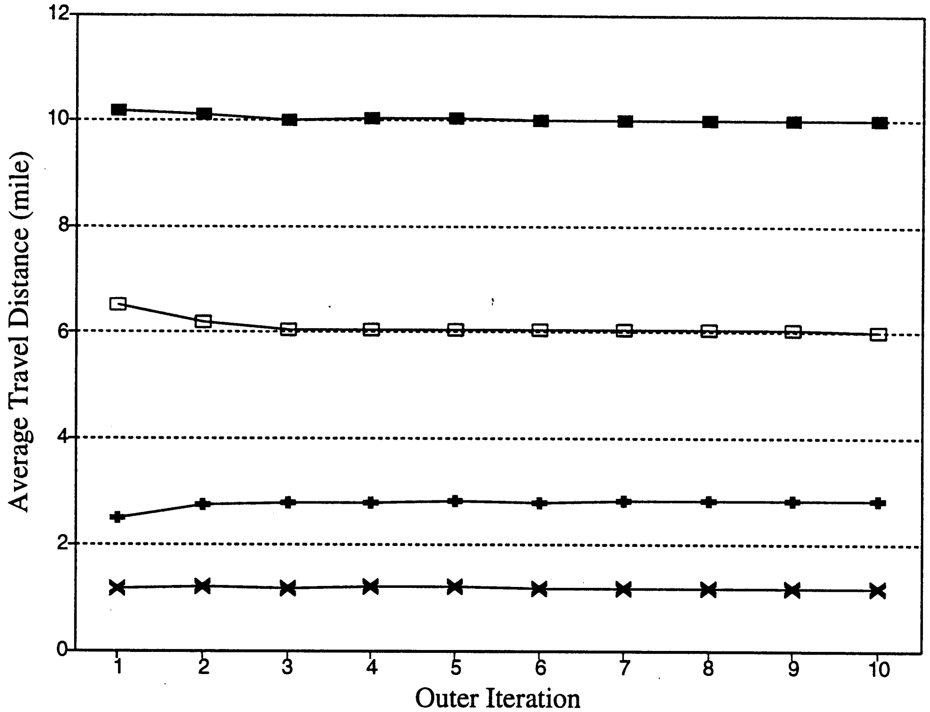

Figure 6: Predicted Average Travel Distance

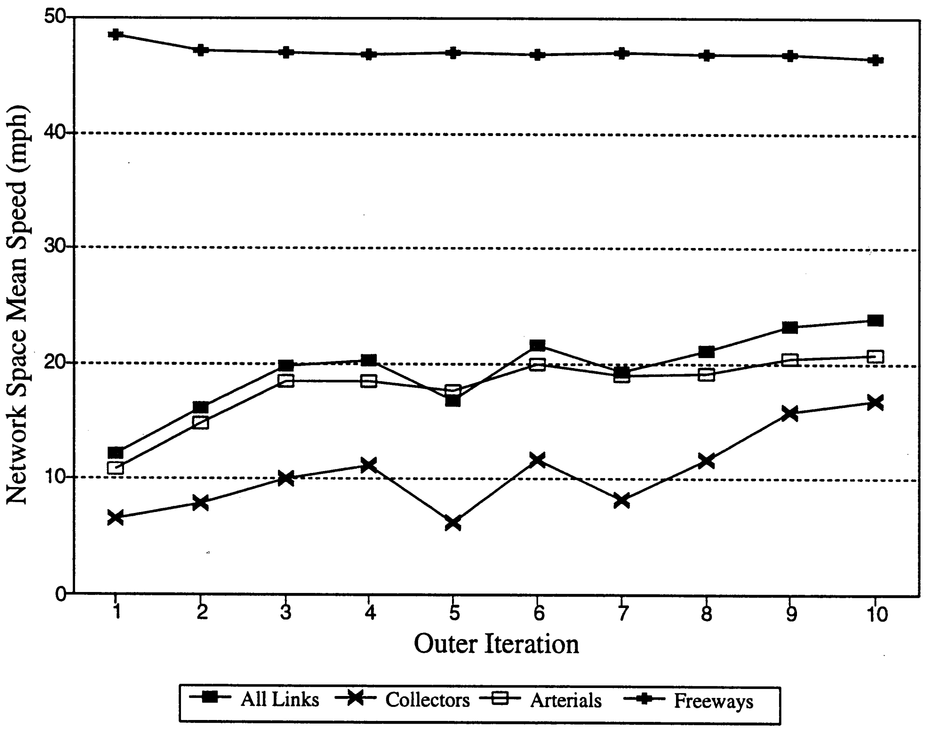

Figure 7: Predicted Average Space Mean Speed

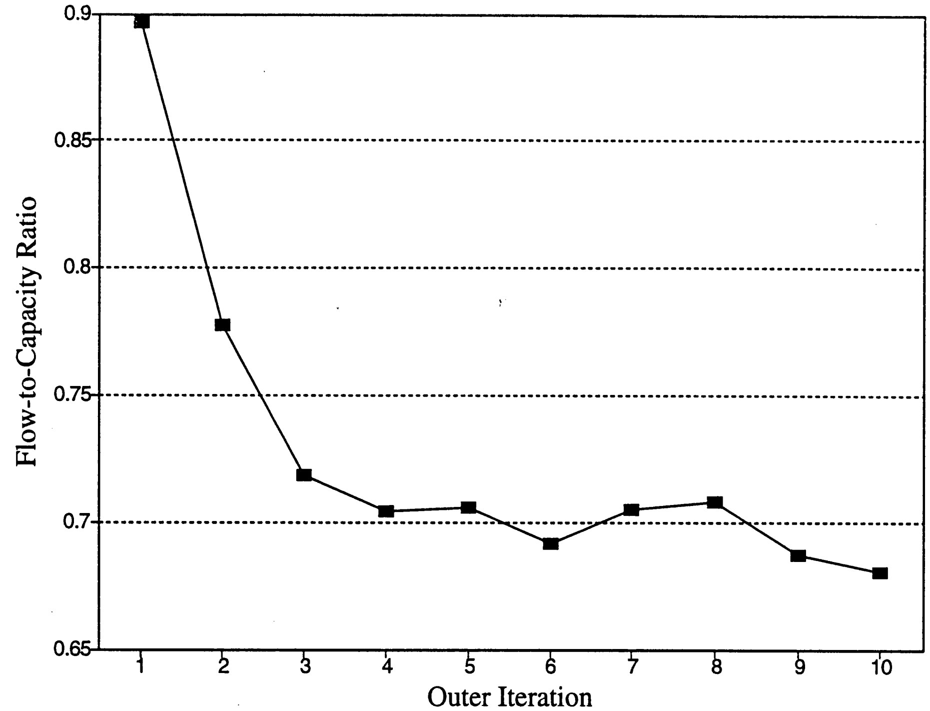

Figure 8: Predicted Flow-to-Capacity Ratio

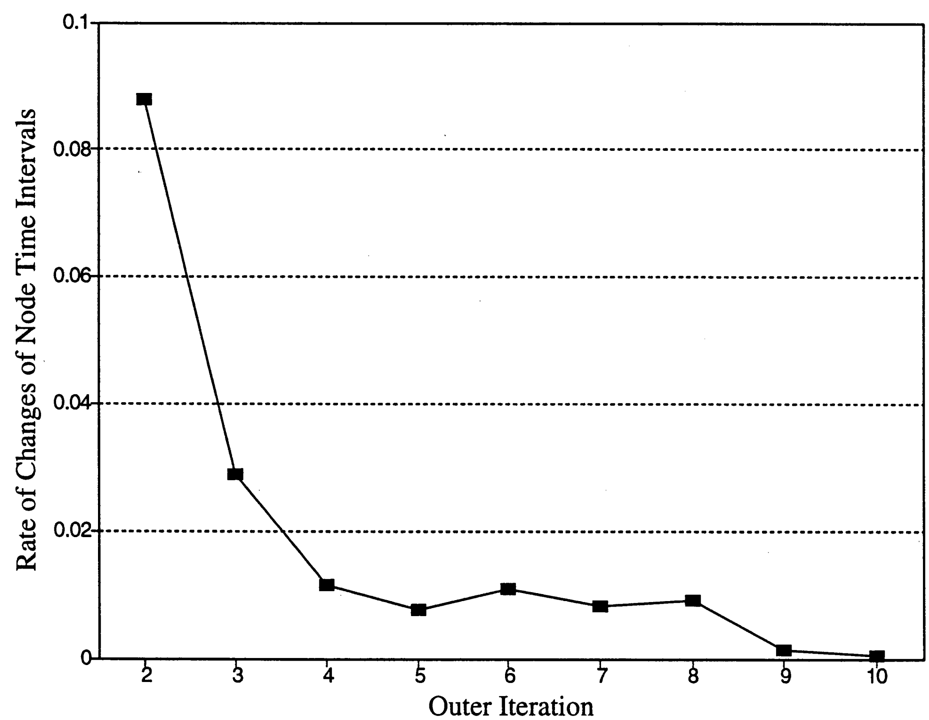

Figure 9: Rate of Change of Node Time Intervals

For the convergence measure, $nodes = 9,700$, $zones = 447$, $intervals = 18$ and $x = 0.001$ were used in this solution. Note a rather small value was chosen for x indicating a fine convergence of the algorithm is desired. After 10 *outer* iterations of Step 1, NDIFFS equals to 20,537 showing the algorithm has converged at the level of $x \approx 0.00026$. The rate of change of *node time intervals* is displayed in Figure 9 which indicates that this model was solved quite smoothly by the solution algorithm for the ADVANCE Network. The flow-to-capacity ratio was 0.68 using this dynamic model versus 0.78 using the asymmetric static model (Berka et al., 1994).

Unfortunately, link flows and link travel time data are not available for the ADVANCE Network, either in general, or more specifically for the O-D matrix used in this solution. These data, as well as route flow data, are urgently needed to advance the state of the art of network modeling for ITS.

The problematic dynamic user-optimal assumption that complicates the modeling of many route choice options is that route choice decisions must be based on travel costs which are temporally-consistent with future link flows at time of link use. The assumption is appropriate for recurrent trips and traffic conditions and is also acceptable for scheduled events (e.g., ballgames, concerts, detours, road constructions) and for predicted weather conditions. However, this behavioral assumption is inconsistent with *unexpected* events (e.g., stalled vehicles, dropped objects, accidents) at future times because drivers have very limited capability to be informed about the times and locations of such events before they encounter unusual queuing delays caused by those incidents. Enroute diversions are thus expected to happen when incidents are encountered by drivers.

The solution algorithm DYMOD was modified to model incident-related enroute diversion. First, base-level link flows of all trips assuming usual (incident-free) conditions are generated. Then, the portion of trips assumed to choose a diversion strategy is removed from those base-level flows and reassigned onto the network according to then-current travel times. Through this approach, drivers who choose non-diverting strategy are actually based on fully anticipatory travel times for route choice. This approach might violate temporal continuity of trips to some extent in those time intervals in which the non-diverting base-level flows use the links are affected by the enroute diversion flows. However, it becomes less severe as the non-diverting portion of drivers increases. Further, this approach also provides an estimate when enroute diversions are guided by in-vehicle information.

A key advantage of DYMOD is that link-specific capacity adjustments can be input exogenously or generated endogenously over time intervals when they occur. Exogenous link capacity adjustments specified to the program when detected or expected to occur might be caused by accidents, weather conditions, time-of-day restrictions for special events, road construction or signal timing changes (if signalization attributes are given outside from the program). Link capacity changes endogenously because spillback queues reduce capacities of upstream links, ramp metering or signal timing changes (if actuated signal timing mechanism is presented and integrated). Capacity adjustments are made between the *inner* and *outer* iterations.

To demonstrate the effects of enroute diversions that resulted from incidents, we placed an incident on a through-movement link of a major intersection which caused a 50% capacity reduction for one hour (through 7 AM to 8 AM) and 50% of drivers were assumed to choose the diversion strategy. Note that the scale of capacity reduction can also be specified over intervals to account for the gradually capacity loss caused by the incident.

Figure 10 shows the layout of the incident intersection. It is a four-leg signalized intersection without any turning restrictions (U-turn excluded). Figure 11 exhibits the flow profile of the incident link along time intervals. Obviously, the flows on the incident link are lower than the non-incident flows during the incident periods. Due to the incident removal, the

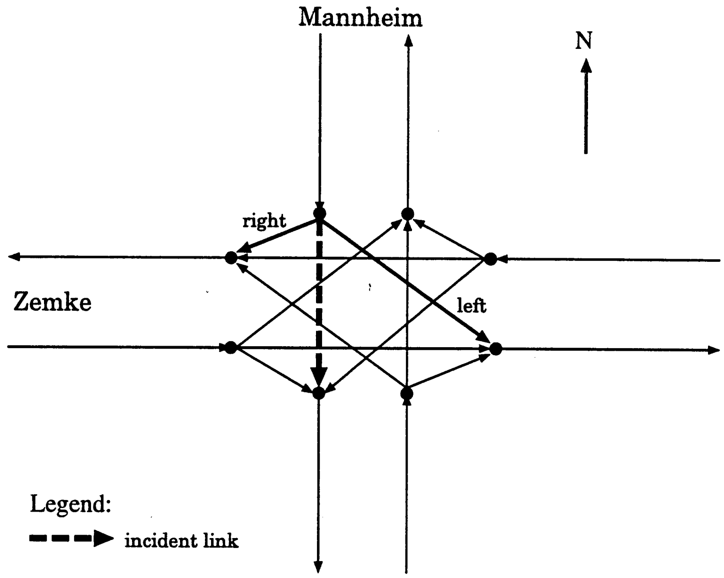

Figure 10: Layout of the Incident Analysis Area

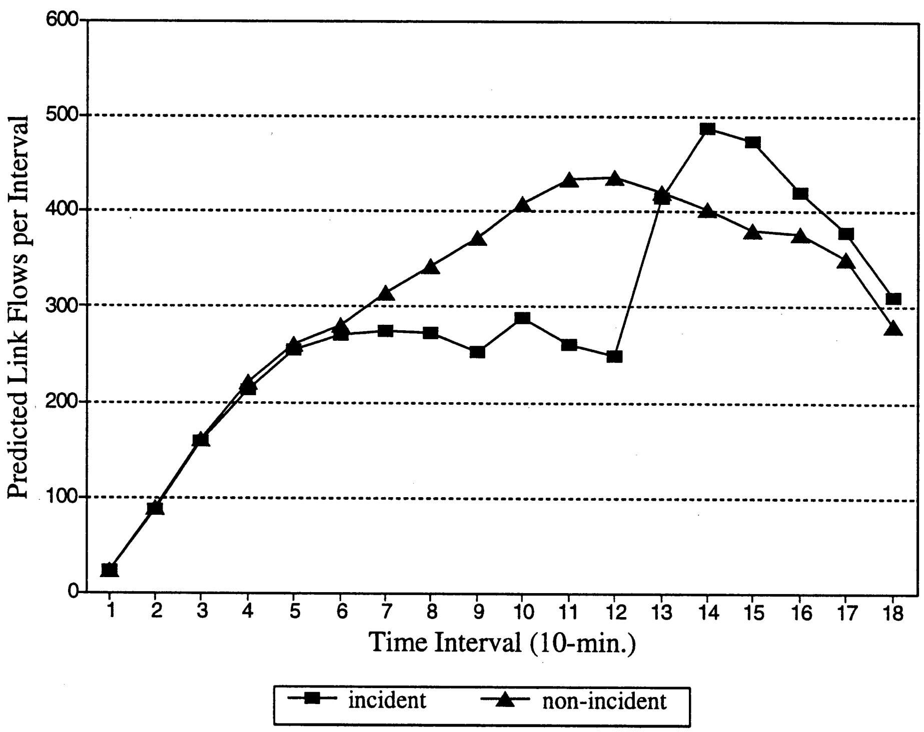

Figure 11: Predicted Flows of the Incident Link

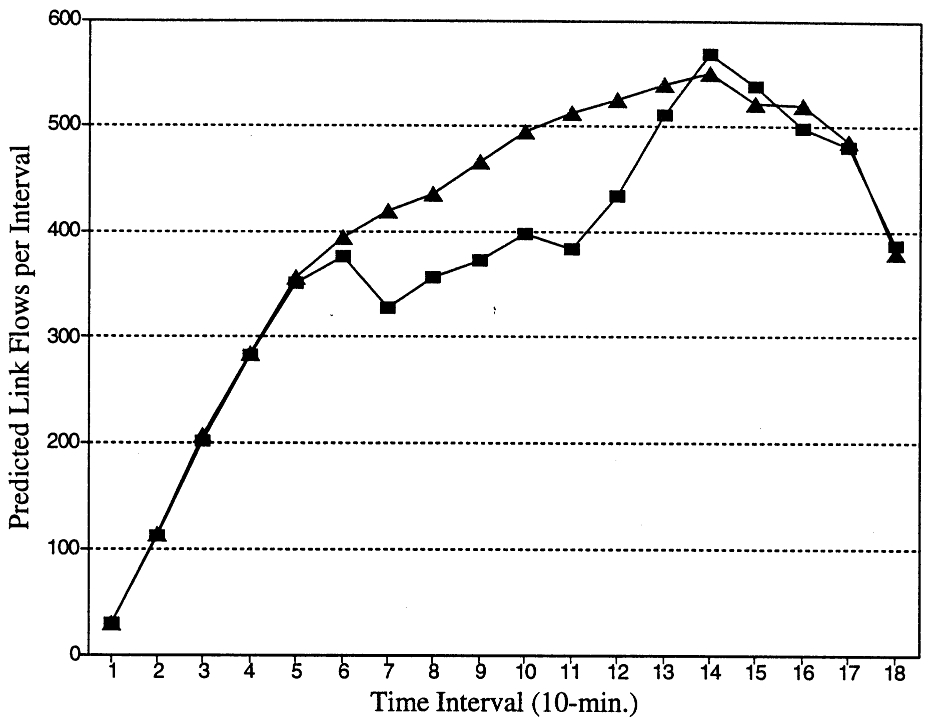

Figure 12: Predicted Flows of the Upstream Link

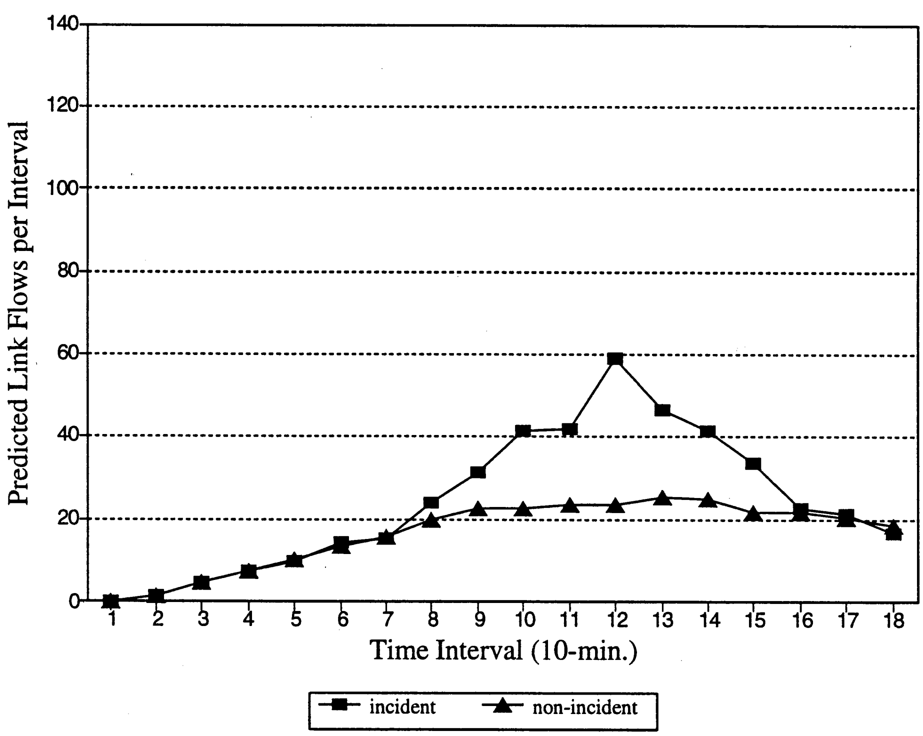

Figure 13: Predicted Flows of the Right-Turn Movement

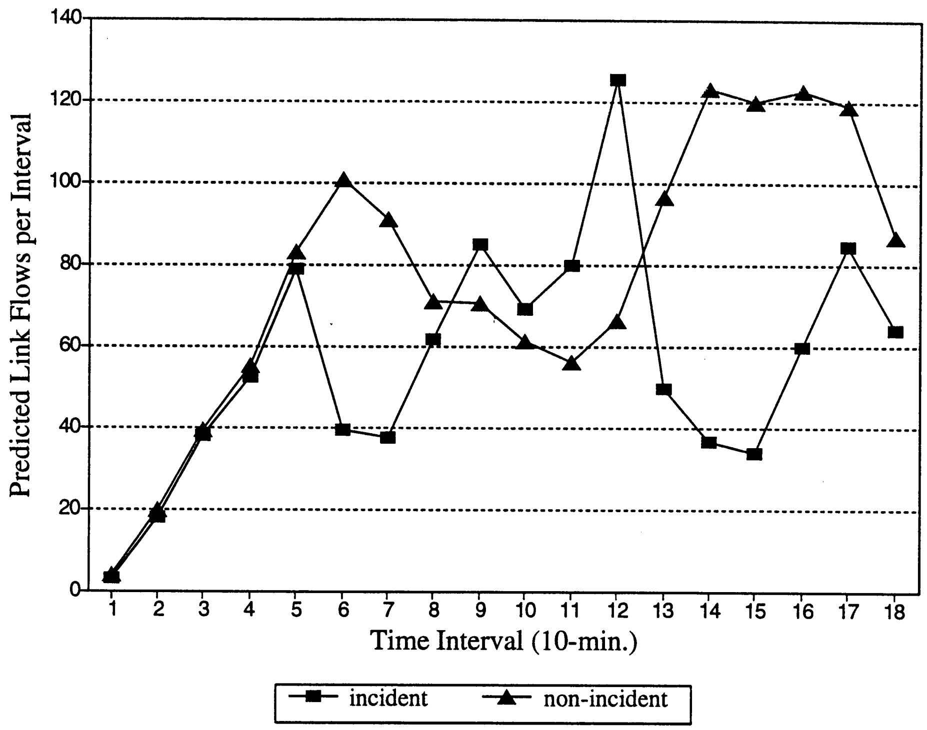

Figure 14: Predicted Flows of the Left-Turn Movement

incident flows become higher than the non-incident flows again following the removal of the incident. The upstream link has a similar result as shown in Figure 12. However, significant flow diversions appear in the right-turn and left-turn links during the incident periods (see Figure 13 and 14). The diversions on the right-turn link are quite obvious; the incident flows are higher than the non-incident flows during the incident period and a few subsequent intervals. The flow profile of the left-turn link is somewhat complex but still reasonable. The incident flows begin to exceed the non-incident flows at the second incident interval because of the normal left-turn delay. The incident flows drop soon and return to the usual used links following the clearance of the incident because of the capacity restoration on the incident link. Based on this modeling capability, this model can be utilized as a tool to assess an area-wide incident management strategy and resulting flow pattern.

6 Conclusions

This chapter describes a VI formulation of a dynamic user-optimal route choice model for predicting real-time traffic flows for Advanced Traffic Management Systems (ATMS) and Advanced Traveler Information Systems (ATIS). The model formulation and solution algorithm are described. Model refinements, extensions and computational results from the ADVANCE Network are also presented. Realistic traffic engineering-based link delay functions are utilized for better estimation of traffic dynamics.

The proposed model is capable of modeling the effects of enroute diversions caused by highway incidents such as stalled vehicles, dropped objects and traffic accidents, etc. Using Janson's DYMOD algorithm, various time-dependent traffic characteristics are obtained and analyzed. We believe this is the largest dynamic route choice model implementation which has been solved thus far.

To date, very few traffic flow prediction models are suitable for ATMS and ATIS applications. Although not yet fully validated, DYMOD is able to predict time-dependent traffic characteristics for a large-scale traffic network which are reasonable and internally consistent. Eventually, dynamic route choice models should be integrated into a traffic control and management center to support the decisions on the adjustments of arterial signal timing, ramp metering, incident management and future route guidance strategies, etc.

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