INTEGRATION© RELEASE 2.30 FOR WINDOWS:

User's Guide – Volume I:

Fundamental Model Features

Technical Information:
e-mail: hrakha@vt.edu

Sales Information:
Contact: M. Van Aerde & Assoc., Ltd.
18 Kenwoods Circle,
Kingston, Ontario K7K 6Y2
Phone: (613) 549-4646
e-mail: mvanaerde@cogeco.ca

March, 2005

Table of Contents

Preface

This User's Guide can assist traffic engineers, transportation planners, students and researchers in coding and using the INTEGRATION model. It is also intended to provide them with an understanding of the potential and limitations associated with the use of the model.

The User's Guide consists of three volumes, that latter volume is still under development:

Volume I: Fundamental Model Features,

Volume II: Advanced Model Features, and

Volume III: Model Logic and Validation (To be developed).

Volume I provides three types of information:

1. Instructions for installing and executing the actual model and a self-contained demo. The demo includes the same logic as the actual model, but does not require the user to provide any input data. Instead, the demo produces its own input data files for a set of pre-coded scenarios.
2. Descriptions of the main traffic simulation and assignment approaches that have been incorporated into the INTEGRATION model. These descriptions are intended to provide the background information necessary for model users to make informed choices regarding the selection of various model simulation and assignment input data choices.
3. A description of the five fundamental data input files required by the INTEGRATION model, as well as a summary of the fundamental model outputs and the associated master file.

Volume II provides descriptions of various more advanced model inputs, as well as a variety of optional model outputs. These advanced features include modeling different forms of Intelligent Transportation Systems, advanced signal timing options, incidents that block specific lanes, and override some of the basic vehicle dynamics characteristics.

It is strongly recommended that the novice INTEGRATION user first installs and executes the self-contained demo prior to coding a new data set. This initial exposure to the model’s execution will also provide the user with some of the necessary context in which to read the simulation and assignment descriptions provided in Chapter 2 of Volume I. Having viewed the demonstration, and gained an appreciation for the overall simulation and assignment approach of INTEGRATION, the user is then encouraged to explore the basics of network coding and simulation as described in Chapter 3 of Volume I.

Volume III is intended to provide the readers that are interested in understanding the logic of the INTEGRATION model a more in-depth understanding of the underlying logic. In addition, the chapter describes various validation studies that demonstrate the validity of different components of the model. It should be noted that the greatest strength of the INTEGRATION model is its sound basis on traffic flow theory, as would be clearly demonstrated in the manual.

Finally, prior to utilizing the advanced features described in Volume II of this Guide, the model user is encouraged to gain experience with the fundamental model features described in Volume I and understand the underlying logic that is described in Volume III.

In Memory of Michel Van Aerde

The INTEGRATION model was developed by the late Michel Van Aerde as his Ph.D. dissertation (1984-1986) while studying at the University of Waterloo, Canada. The name INTEGRATION stems from the fact that the model integrates a number of unique capabilities. First, it integrates traffic assignment and microscopic simulation. Second, the model integrates freeway and arterial modeling within a single logic.

After completing his Ph.D., Michel Van Aerde moved to Queen's University to serve as a member of the Civil and Environmental Engineering faculty (1986-1997). In 1997 Michel Van Aerde moved to Virginia Tech to serve as a faculty in the Charles Via Department of Civil and Environmental Engineering and the Associate Director of the Center for Transportation Research until his passing on August 17, 1999. Michel Van Aerde will always be remembered as a unique person who advanced traffic flow theory, traffic engineering, and transportation modeling. Most people will remember Michel Van Aerde with the INTEGRATION model and, more importantly, with his unique contributions to the advancement of traffic flow theory as it applies to traffic modeling.

I have been fortunate to work with Michel Van Aerde for 11 years both as a graduate student and as a colleague. The knowledge, friendship, and companionship we shared will be something that I will cherish for the rest of my life. I hope that everyone will remember Michel Van Aerde as a person who always cared for others, a person who advanced traffic flow theory, and a person who had the vision and insight to develop the unique tool INTEGRATION.

H. Rakha, May 2001.

Acknowledgements

INTEGRATION has been heavily influenced during the past decade by many of the people, models, and activities that were present in the environment in which the model was being developed and tested. This note is to acknowledge some of those who have contributed, either directly or indirectly, and sometime unknowingly, to this environment.

The pursuit of an integrated model for dealing with integrated control problems was in part inspired by the writings of D. Robertson, A.D. May, S. Yagar, and S. Teply on this topic during the early 1980's. At that time, E.R. Case and A. Ugge were responsible for encouraging the review of potential approaches to develop an integrated model. In direct contrast to many other approaches, which represented extensions to existing approaches, INTEGRATION was developed as a new approach to solving this problem.

S. Yagar's arguments in favor of simply extending CORQ's macroscopic approach, rather than creating a new microscopic model, were a considerable influence in developing the model's early validation criteria. A similar influence were the comments provided by J. Shortreed on a graduate course project, which explored the requirements and features of medium term planning models, rather than short term traffic models or long term transportation planning models.

The actual model design was subsequently influenced by a review of the main traffic simulation models that existed at that time, such as TRANSYT, SSTOP, PASSER, MAXBAND, TRAFFICQ, NETSIM, FREQ, FRECON, INTRAS, CORQ, TRAFLO, DYNEV, SATURN, TRANSIGN and CONTRAM. While no single line of code from any of these models was ever utilized, their diverse features acted as constant measuring sticks against which the integrated features of INTEGRATION were, and continue to be, compared during any new developments. The extensive use of many of these models by the Ontario Ministry of Transportation (MTO) R&D staff was especially useful in gaining access to both the theoretical, as well as the practical features of these models.

Since that time, the research by FHWA into CORSIM and CORFLO, H. Mahmassani into DYNASMART, MITRE/MITRETEK into THOREAU, M. Ben-Akiva into MITSIM, and Los Alamos into TRANSIMS, have acted as the main competitive sources of inspiration for attempts to make the INTEGRATION model more general, accurate, flexible, and useful. Similarly, the research by F. Wolfe into static traffic assignment and by B. Janson and M. Ben-Akiva into dynamic traffic assignment has provided insights into what types of enhancements to consider for INTEGRATION's traffic assignment. The work by S. Underwood, R. Smith, S. Lafortune and K. Wunderlich into anticipatory route guidance has also been helpful in this regard. The insights gained into dynamic traffic assignment, in particular through the early work by Whithing in CONTRAM and by S. Yagar in CORQ, is acknowledged because of their unique blend of practical and theoretical considerations.

The structure and style of the INTEGRATION program was heavily influenced initially by the GPSS language and subsequently by developments in the area of

objected-oriented programming. The actual code writing followed a style taught by J. Moore. The implementation of the model was made possible through the use of FORTRAN compilers created by WATCOM, LAHEY and IBM, but early prototypes were also developed in C and BASIC.

A version of Dijkstra's minimum path algorithm, as coded by J. Shortreed, served as a starting point for the shortest path algorithm that was initially in INTEGRATION. However, research and testing by L. Rilett resulted in significant enhancements to this code several years ago. This code has been reworked again more recently in order to achieve more integrated blend of minimum path algorithms and assignment algorithms. The initial method of coding and handling links also follows approaches taught by J. Shortreed. The impact of complementary research into macroscopic approaches to dynamic traffic assignment by L. Yu, as well as his work into microscopic platoon dispersion, is also acknowledged.

The research into multipath loading methods in INTEGRATION was inspired by work done by Dial and Sheffi, and comments by L. Rilett. The research of H. Fu, D. MacKay, G. MacKinnon and K. Toni, into the application of these multipath-loading methods is also acknowledged. The theoretical aspects of the development of synthetic O-D demands for use in INTEGRATION owe much to earlier research efforts by Willumsen and Van Zuylen. Contributions of J. Voss, G. Noxon and G. MacKinnon to a static predecessor version of QUEENSOD, and of B. Hellinga to the present static and dynamic version of QUEENSOD, have been instrumental to its practical success. L. Bloomberg has also made considerable contributions to the calibration of synthetic O-D's for large networks.

The current speed-flow relationship owes much to early calibration efforts by G. MacKinnon, B. Hellinga, Y. Gardes, and L. Aultman-Hall, who identified, on the basis of Ontario and California data, weaknesses of the earlier approaches within INTEGRATION. This work has also benefited from the general research on this topic by A.D. May, S. Yagar, F. Hall, W. Brillon and J. Banks. The current relationship owes much to the earlier work by Greenshields and the car-following theories of Gazis and the General Motors Research Laboratory. Reviews by Y. Gardes, and the concise description of subsequent enhancements to this earlier work in a textbook by A. D. May were also of benefit. H. Rakha, B. Hellinga and A. Stewart have more recently made important contributions to the automated calibration of the current form based on actual loop detector data from Ontario, Orlando, Paris and Amsterdam.

The modeling of traffic signals in INTEGRATION owes much to the direct contributions of H. Rakha and to the efforts of V. Bacon and Y. Gardes, as well as the indirect contributions through their research of S. Teply, D. Robertson and A. Stewart. The modeling philosophy, with respect to traffic signals, was heavily influenced by the approaches contained within TRANSYT, SCOOT and the Canadian Capacity Guide, as well as the approaches contained within TRANSIGN, SATURN, CONTRAM, RTOP, UTCS 1-3, SCATS, PRODYN and OPAC. Recently, A. Stewart has contributed to the testing of INTEGRATION's signal coordination logic.

The model has also benefited considerably from its applications by the FTMS section of MTO, IBI and McCormick-Rankin, on behalf of MTO, the PATH program at UC Berkeley, on behalf of the Research Laboratories of General Motors with respect to TravTek, on behalf of the Evaluation Working Group of the TravTek system, FHWA's IVHS architecture study teams, and by its application as part of the Central

Artery/Tunnel project in Boston. The model has also benefited from applications by JHK in Salt Lake City, Kittleson and Associates in Portland, and Goudappel-Coffeng in the Netherlands. The on-going diverse contributions of J. Wang, G. Hetti, P. Masters, D. Tsui, C. Rayman and A. Ugge at MTO should not go unmentioned, as should the comments provided by the various undergraduates and graduate students who have utilized INTEGRATION at Queen's as part of their course work.

The IVHS/ITS specific routines have benefited considerably from the comments by E.R. Case and M. Krage, and the research by G. MacKinnon and L. Rilett, while the modeling of vehicle probes was influenced by research by L. Yu, B. Hellinga and H. Rakha. The fuel consumption and vehicle emission models were developed by M. Baker based on data collected using a TravTek vehicle and based on parallel research in EPA's MOBILE model. INTEGRATION's safety routines have benefited significantly from research by B. Perez, the camera car work of T. Dingus, the literature reviews of M. Robinson and A. Stewart, and the comments provided by A. Burgett.

Finally, during the past decade, J. Voss, G. MacKinnon, J. Kelly, H. Rakha, L. Rilett, B. Hellinga and M. Baker have made various direct contributions to the checking of code for certain new routines while at Queen's University. This user's guide, in specific, has also evolved through contributions and additions by J. Voss, J. Kelly, H. Rakha, L. Aultman-Hall and B. Hellinga. Editorial comments by G. MacKinnon and K. Wunderlich are equally acknowledged.

Recently, the work by S. Velan on gap acceptance behaviour, M. Carter and M. Kirkpatrick on plug-and-play modules for INTEGRATION, and M. Robinson in the area of modelling toll plazas should be acknowledged, as should the efforts of C. Robbins of Kimley-Horn into ramp metering and K. Quackenbush and C. Pagitsas of CTPS into HOV facilities. Finally, M. Baker and A. Stewart have contributed to the testing and validation of INTEGRATION's weaving and merging behaviour.

During the past year, effort by F. Hebrard and F. Giorgi in the area of bus modeling, and A. Abdulkarim in the area of weaving should be noted. In addition, the applications of INTEGRATION for MITRETEK and JPO have served to expose many of INTEGRATION's strengths and limitations.

All Fortran code is compiled and linked using Lahey FORTRAN 4.5. Graphics and menus are generated using Winteracter subroutines.

Finally, it should be noted that the development of INTEGRATION has taken place while I was at 3 different universities, namely the University of Waterloo, Queen's University and Virginia Tech. None of these universities directly provided financial support for the development of INTEGRATION. However, the opportunities they have created for me to do this work are much appreciated.

M. Van Aerde, July 1997.

Chapter 1: Introduction

1.1 New Features of INTEGRATION 2.30

The new release of INTEGRATION, INTEGRATION 2.30 offers a number of unique features that were not available in previous commercial versions of INTEGRATION. These features include the following:

a. Modeling of adaptive traffic signal optimization including cycle length, phase split, and constrained offset signal optimization. The offset optimization uses a hillclimb search technique to find the optimum offset timings that minimize a localized performance index that accounts for vehicle delay and stops.

b. Modeling of actuated signal control by vehicle class, which allows for the modeling of transit signal priority.

c. Modeling of pre-trip and on-trip provision of real-time traffic information to drivers.

d. Modeling of toll roads for different driver populations with different values of time.

e. Modeling of vehicle dynamics using a constant power vehicle dynamics model that accounts for the vehicle’s tractive effort, aerodynamic, rolling, and grade resistance forces. The model is capable of modeling up to 25 different vehicle types including light duty cars, light duty trucks, heavy duty trucks, and buses.

f. Modeling normal light duty vehicle and light duty truck fuel consumption, HC, CO, CO_2, and NO_x emissions.

g. Modeling of 4 high emitter light duty vehicle categories.

h. Modeling of emission dispersion using a simplified plume dispersion model.

i. Modeling of 14 different crash types with their associated levels of severity using average crash rates for the US.

j. Estimating freeway, signalized, and stop sign Level of Service (LOS) using the Highway Capacity Manual Procedures.

k. Running the program in batch mode and altering the random number seed.

1.2 Typographical Conventions

The typographical conventions listed in Table 1 are used throughout this guide. These conventions should assist the user when reading the manual, but especially when first setting up the INTEGRATION model.

Table 1: Typographical conventions used in this Guide

Type Style	Used for
`UPPERCASE COURIER FONT`	Filenames appear in uppercase courier font.
`Small mixed case courier font`	Example input and output data file contents are illustrated in small mixed case courier font.

1.3 INTEGRATION Release 2.30: Network Size Restrictions

At present, four different versions of INTEGRATION are commercially available. All of these versions incorporate the same core model logic. However, the size of the network that each can model is different, as summarized in Table 2. Many special model features are also available in the research version of the model. However, these features are at present not commercially available.

Table 2: Maximum program/network dimensions

Model Size:	Small	Medium	Large	Huge
Highest link number	250	1,250	3,500	10,000
Highest node number	250	1,250	3,500	10,000
Max. network size (lane-kilometers)	250	1,250	3,500	10,000
Max. number of lanes	1750	8,750	24,500	50,000
Max. total individual vehicles in simulation	25,000	125,000	350,000	500,000
Max. vehicles concurrently on the network	5,000	25,500	70,000	150,000
Highest macro zone cluster number	10	25	100	150
Highest origin/destination zone number	25	125	350	500
Max. number of OD demand loading	250	1,250	35,000	100,000
Highest traffic signal /ramp meter number	25	125	350	1,000
Max. concurrent routing trees/vehicle class	5	5	5	5
Max. number of incidents/blockages	5	12	35	100
Highest detector station number	250	1,250	3,500	10,000

1.4 Computer Hardware and Software Requirements

The computer hardware that is recommended for executing the various versions of the INTEGRATION model is described in Table 3. It can be noted that all versions run in Windows 95/98/2000 or Windows NT. A DOS executable for INTEGRATION is no longer available.

Allocating Virtual Memory in Windows 98

For Windows 98 users make sure that the computer settings allow Windows to manage your virtual memory settings. This can be set by double clicking on the "System" icon in the "Control Panel." Click on the Performance tab then the "Virtual Memory" button. Make sure that the "Let Windows manage my virtual memory settings" item is enabled.

Setting Paging File in Windows 2000

For Windows 2000 make sure that the memory paging file is of sufficient size to run the program. If the paging file is not large enough you will need to logon as the system administrator and double click on the "System" icon in the "Control Panel." On the "Advanced" tab, click "Performance Options," and under "Virtual Memory," click the “Change” button. In the “Drive” list, click the drive that contains the paging file you want to change. Under “Paging file size for selected drive,” type the new paging file size in megabytes in the “Initial size (MB)” or “Maximum size (MB)” box, and then click the “Set” button.

Setting Paging File in Windows NT

For Windows NT users make sure that the paging is set to allow the program to use the recommended RAM for the version of INTEGRATION that is being used. For example, a huge version of INTEGRATION requires that the paging be set for a maximum of 4 GBytes. To change the paging file size in Windows NT the user will have to logon as the system administrator. The changing of the paging file requires going to the “Control Panel” and double clicking on the “System” icon. Click on the “Performance” tab and then click on the “Change” button in the “Virtual Memory” section. Set the initial and maximum size to the desired memory allocation.

Table 3: Recommended computer hardware

Model Size:	Small	Medium	Large	Huge
Recommended CPU	Pentium	Pentium	Pentium	Pentium
Recommended RAM[1]	32-64 Mb	320 Mb	1 Gb	4 Gb
Required Operating System[2]	Windows 95/98 or Windows NT			
Min. Disk Space[3]	100 Mb			

[1]If insufficient RAM is available; the model will attempt to use any available hard disk space as virtual memory. However, this option may significantly increase execution time.

[2] All graphics are implemented in Windows (16 colors)

[3] When virtual memory is used, additional disk space may be required per note 1.

1.5 Executing the INTEGRATION Demo

In order to accelerate a novice modeler’s progress along the INTEGRATION learning curve, a self-contained demo has been created.

Summary

The INTEGRATION 2.30 Windows DEMO contains a full-featured version of the latest release of the model. This demo will automatically be executed for a number of sample networks. These networks, which are selected from the main demo menu, illustrate a number of small, but interesting, traffic scenario vignettes.

The demo can be executed an unlimited number of times and permits full on-screen interaction with the simulation model. However, the user cannot modify the input data or utilize the demo version of the INTEGRATION model to create or run other scenarios.

The particular scenarios available within the demo have been selected to illustrate the most common elements of integrated transportation networks. These scenarios, which are presented as a series of isolated vignettes, were designed to clearly illustrate the underlying and fundamental logic and the traffic dynamics associated with each component of an integrated transportation network.

Executing the Demo

To execute the demo, the user is required to copy the file called `DEMO.EXE` into its own subdirectory on a Pentium-type computer (or better) with at least 32 Mb of RAM. The subdirectory into which the file is placed should provide at least 10 Mb of free disk space in order to store the executable code and any temporary files.

Once the demo program is copied into the chosen directory, the user may then access a file browser and double-click the icon associated with the `DEMO` program.

Alternatively, the user may use the RUN option in Windows to execute the `DEMO.EXE` from within the sub-directory in which the `DEMO` program has been placed. Finally, the user may also execute the `DEMO` program by opening a DOS Window in Windows 98 or NT and typing DEMO <return> at the DOS prompt in the appropriate subdirectory. In Windows 2000 the user will have to go to the "Command Prompt" in the "Accessories" menu and then type DEMO <return> at the prompt.

Control Menu

One of the unique features of the INTEGRATION traffic model is that the on-screen animation is provided concurrent to the execution of the actual model logic. One consequence of this feature is that even with the demo, the user is experiencing at all times the actual logic in progress, rather than simply a graphical post-processing of an earlier simulation.

The second consequence is that in any simulation, the animation can only be paused or advanced, but it can never be turned back. However, once the model has started, the execution of a particular demonstration scenario can be terminated at any time by accessing the INTEGRATION control menu. This menu allows the modeler to terminate the current scenario and immediately proceed to execute the same, the next, or an alternative scenario. At the end of the final scenario, the INTEGRATION menu can then be used to return control of the computer to MS-DOS or Windows.

1.6 User Feedback

Inquiries or Comments

We appreciate receiving constructive criticisms on the demo, the model and/or this guide. Please direct any inquiries/comments to:

maureenv@primus.ca

or call/ us at (613) 549-5807.

Chapter 2: Overview of Simulation Features

2.1 Introduction

The INTEGRATION model is a trip-based microscopic traffic simulation model. The model is designed to trace individual vehicle movements from a vehicle's origin to its destination at a level of resolution of one status update every 1/10th of a second, or deci-second. In order to allow the model user to gain a certain understanding of the basic features of the ITEGRATION simulation model, this chapter describes the most important simulation features and attributes of the model.

2.2 Domain of Application

In order to appreciate the intended domain of application of the INTEGRATION model, it is useful to view travel activities within urban areas as an interrelated sequence of six decisions that travelers must typically make in order to complete a particular trip. Three of these decisions are made prior to the time when drivers leave their driveway, and usually cannot be revisited during that same trip. The three others, however, need to be revisited repeatedly, even once a particular trip has been initiated.

Pre-trip Decisions

At the highest level of the trip making process are decisions related to where a particular trip maker lives, works, and shops. At this level of decision, the trip maker must determine how many trips to make towards each potential destination during each particular departure time window. Once the decision to make a particular trip to a given destination has been made, the traveler must then decide whether to utilize a mode of public transit (if available) or a private vehicle, either as a single vehicle occupant or as a car pool participant. The third set of pre-trip decisions relates to the particular time at which the trip maker may elect to start its intended trip within a given time window. Each of these first three types of decisions may be interdependent, but are usually not made more than once for a particular trip.

On-Route Decisions

In contrast, the next three types of trip decisions are made once a trip has started, and usually need to be revisited several times as the actual trip progresses. Specifically, when initiating the trip, the trip maker must select what route to take. This decision is usually not fixed, even after the trip has commenced, as a driver usually may still elect to change any remaining portion of his/her intended trip. Once a vehicle has entered a particular link along the selected route, the driver must also determine the speed at which to drive at and which lane to utilize while on the link. Again, a driver's speed and lane choice are likely to change. Changes could occur at least from one link to the next, or several times along the same link as a result of interactions with other vehicles. Finally, when a driver arrives at the end of a link, he/she may be required to cross an opposing traffic stream and must then decide whether to accept or reject any available gaps and/or how to merge with a converging traffic stream.

Domain of Application

The current domain of application of the commercial version of INTEGRATION includes the latter set of en-route driver decisions, starting from the time a driver has elected to depart from a particular origin with the objective of reaching a particular destination, at a particular time, and by means of a specific vehicle class. This implies that, at present, the commercial version of INTEGRATION does not directly model the impact of drivers electing to depart at a different time, by means of a different mode, or to an alternate destination.

However, in order to reflect increasing interest in being able to explore the potential traffic impacts on these latter decisions, an outer loop is being developed around a research version of the INTEGRATION model. This outer loop will permit estimates of the expected changes in trip mode, departure time and/or destination to be made through systematic iterative applications of the model. This outer loop is beyond the scope of this manual.

2.3 Microscopic Modeling Approach

INTEGRATION is a fully microscopic simulation model, as it tracks both the lateral and longitudinal movements of individual vehicles at a resolution of up to one deci-second.

This microscopic approach permits the analysis of many dynamic traffic phenomena, such as shock waves, gap acceptance, and weaving. These attributes are usually very difficult, or infeasible, to capture under non-steady state conditions using a macroscopic rate-based model, but become emergent behavior with the INTEGRATION model. For example, in a dynamic network, average gap acceptance curves typically cannot be utilized at permissive left turns if the opposing flow rate varies from cycle to cycle and/or within a particular cycle. These curves also cannot be used if the size of the acceptable gap varies as a function of the length of time for which a vehicle has been waiting to find an acceptable gap. Similarly, most macroscopic models cannot model platoon progression between adjacent traffic signals that have cycle lengths that are not multiples of each other.

The INTEGRATION model can consider virtually continuous time varying traffic demands, routings, link capacities and traffic controls without the need to pre-define explicit time-slice duration between these processes. This implies that the model is not restricted to hold departure rates, signal timings, incident severity, or even traffic routings, at a constant setting for any particular common period of time. Consequently, instead of treating each of the above model attributes as a sequence of steady-state conditions, as needs to be done in most rate-based models, all of these attributes can be changed on virtually a continuous basis over time.

The microscopic approach also permits considerable flexibility in terms of representing spatial variations in traffic conditions. For example, while most rate-based models consider traffic conditions to be uniform along a given link, INTEGRATION permits the density of traffic to vary continuously along the link. In particular, such dynamic density variation permits, along an arterial link, the representation of platoons departing from traffic signals and the associated propagation of shock waves in an upstream or downstream direction, or both.

Finally, it is important to note that the model is primarily microscopic. However, these microscopic rules have been carefully calibrated in order to capture concurrently most of the target macroscopic traffic features that traffic engineers are most familiar with. Examples of these features are link speed-flow relationships, multi-path equilibrium traffic assignment, and uniform, random or over-saturation delay, as well as weaving and ramp capacities. The main challenge in the design of INTEGRATION has been to ensure that these important macroscopic features automatically remain emergent behavior arising from the more fundamental microscopic model rules that are needed to represent the system dynamics using a single integrated approach.

2.4 General Traffic Flow Fundamentals

The manner, in which INTEGRATION represents traffic flows, can be best presented by discussing how a typical vehicle initiates its trip, selects its speed, changes lanes, transitions from link to link, and selects its route.

Initiation of Vehicle Trips

Prior to initiating the actual simulation logic, the individual vehicles that are to be loaded onto the network need to be generated. As most available O-D (Origin-Destination) information is macroscopic in nature, INTEGRATION permits the traffic demand to be specified as a time series histogram of O-D departure rates for each possible O-D pair within the entire network. Each histogram cell within this time series can vary in duration from 1 second to 24 hours. The duration of each cell is also independent for one O-D pair versus the next, or for one time period versus the next. Furthermore, when the same O-D is repeated within the departure list for an overlapping time window, the resulting vehicle departures are considered to be cumulative.

The actual generation of individual vehicles occurs in such a fashion as to satisfy the time-varying macroscopic departure rates that were specified by the modeler within the model's input data files. Specifically, the model simply disaggregates an externally specified time varying O-D demand matrix into a series of individual vehicle departures prior to the start of the simulation. For example, if the aggregate O-D input data requests departures at a uniform rate of 600 vehicles per hour between 8:00 and 8:15 AM, a total of 150 vehicles will be generated with headway averaging 6 seconds.

It should be noted that, as the externally specified O-D Traffic Demand File is disaggregated, each of the individual vehicle departures is tagged with its desired departure time, trip origin and trip destination as well as with a unique vehicle number. This unique vehicle number can subsequently be utilized to trace a particular vehicle along the entire path towards its ultimate destination. It can also be utilized to verify that subsequent turning movements of vehicles at, for example, network diverges are assigned in accordance to the actual vehicle destination, rather than some arbitrary turning movement probabilities, as is the case in many other microscopic models that are not assignment based.

When the simulation clock reaches a particular vehicle's scheduled departure time, an attempt is made to enter that vehicle into the network at its origin zone. From this point the vehicle will begin to proceed, in a link-by-link fashion, towards its final destination. Upon entering this first link, the vehicle will then select the particular lane in which to enter. This is usually the lane with the greatest available distance headway at the point of entry.

Steady-State Car-Following Behavior

Once the vehicle has selected which lane to enter, the vehicle computes its desired speed on the basis of the distance headway between it and the vehicle immediately downstream of it, but within the same lane. This computation is based on a link specific microscopic car following relationship that is calibrated macroscopically to yield the appropriate target aggregate speed-flow attributes for that particular link (Van Aerde, 1995; Van Aerde and Rakha, 1995). A more detailed description of the car-following logic and the calibration of the car-following parameters is provided in Volume III of the User's Guide. Having computed the vehicle's speed, the vehicle's position is updated every deci-second to reflect the distance that it can travel at this speed during each subsequent deci-second. The updated positions, which are derived during each given deci-second, then become the basis upon which the new headways and speeds will be computed during the next deci-second.

The macroscopic calibration of the microscopic car-following relationship ensures that vehicles will traverse each link in a manner that is consistent with the link's desired free-speed, speed-at-capacity, capacity and jam density. Figure 1 illustrates the direct correspondence between the more familiar macroscopic speed-flow and speed-density relationships, and the less familiar car-following relationship that is plotted in terms of speed-headway. This correspondence is illustrated for three different traffic conditions, which are identified as points *a*, *b* and *c*.

From the speed-flow relationship illustrated in Figure 1, it can be noted that point *a* represents uncongested traffic conditions, point *b* capacity flow, and point *c* congested conditions. It can also be noted that both speed *a* and speed *c* are occurring at the same flow rate. It is further observed that the attributes of points *a*, *b* and *c* are more difficult to discern from the speed-density and speed-headway relationships, which simply represent mathematical transformations of the same relationship, than from the speed-flow relationship. In this case, however, speed *a* and speed *c* have unique densities and headways associated with them, which allow the specific traffic conditions associated with each speed to be identified.

Qualitatively, it can be noted from the speed-headway relationship that vehicles will only attain their desired free speeds when the distance headway in front of them is very large. In contrast, when the distance headway becomes sufficiently small, as to approach the link's jam density headway, vehicles will decelerate until they eventually comes to a complete stop.

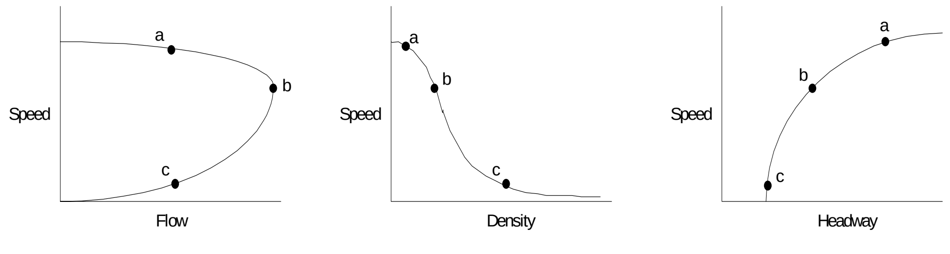

Figure 1: Determination of microscopic speed from corresponding macroscopic relationships

Non-Steady State Car-Following Behavior

The above built-in relationship, between macroscopic and microscopic traffic flow attributes, ensures that vehicles will travel at appropriate steady-state speeds upstream, within and downstream of bottlenecks that may or may not involve queues. The relationship does, however, not address how vehicles transition from one steady-state to another. In other words, while this relationship ensures that vehicles travel at the correct speed prior and subsequent to joining a queue, it does not describe the speed transitions of individual vehicles at the interface of different traffic densities, as will be discussed next.

Prior to discussing these speed transitions of individual vehicles at the interface of different density regimes, it should be noted that these are different from the shock-wave speeds that are commonly computed to denote the movement of the interface. This movement of the interface between different traffic flow densities will determine where individual vehicle speed transitions will take place, but it does not describe how. It should be noted that, in traditional shock-wave analyses, individual vehicle speeds are considered to change instantaneously, implying infinite accelerations and decelerations.

INTEGRATION provides separate deceleration and acceleration logic. The deceleration logic recognizes speed differentials between the vehicle that is making desired speed decisions, and the vehicle ahead of it. It is simplest to first describe how this logic applies to a vehicle approaching a stationary object. In this case, a vehicle will first estimate the excess headway between itself and the vehicle ahead of it. This excess headway is defined as the residual that remains when the currently available headway is reduced by the minimum headway. Based on this residual headway, the vehicle will next compute the time it has to comfortably decelerate from its current speed to the speed of the object/vehicle in front of it. This time is, for constant deceleration rates, equal to the residual headway divided by the average speed of the leading vehicle and the following vehicle. Subsequently, the following vehicle computes the required deceleration rate as the speed differential divided by the deceleration time. These calculations are best illustrated using a simple example, as is provided next.

Consider a vehicle at a position of 30 meters approaching a stationary object at location 100 meters. Consider further that the jam density headway is 10 meters, and that the approaching vehicle is presently traveling at 20 meters per second. Given a difference in relative positions of 70 meters, and a jam density headway of 10 meters, the following vehicle has a stopping distance of 60 meters. Given a current speed of 20 m/s and a final speed of 0 m/s, the vehicle should be covering the 60 meters at an average speed of 10 m/s assuming a linear decay in speed. This distance would require 6 seconds to travel at the average speed of 10 m/s, which given the initial speed of 20 m/s, would involve a constant deceleration rate of –3.33 m/s^2 ((0-20)/6).

The above calculations ensure that a following vehicle will select a constant deceleration rate that allows it to just come to a stop at a jam density headway behind a following vehicle. If the leading vehicle is actually moving, the following vehicle would attempt to decelerate at a constant rate in such a manner as to attain the speed of the leading vehicle when it reached the location that is a single jam density headway behind its current location. Of course, by the time the following vehicle would reach this location, the lead vehicle would already have moved ahead on the highway, resulting in an asymptotic deceleration of the follower vehicle to the lead vehicle’s speed, rather than a constant deceleration. Similarly, if the lead vehicle were accelerating, the follower vehicle would only continue to decelerate until it was traveling at the same speed as the lead vehicle. From that point onwards, the follower vehicle would likely begin to accelerate again, as the increasing gap to the lead vehicle would cause the follower vehicle to perceive increasing desired speeds.

Of interest here is to note how the above logic is utilized to determine how a vehicle determines whether or not to stop at an amber light, or whether to keep going through at steady speed. When a vehicle approaches a traffic light, it only considers the vehicle in front of it when the traffic light is either red or green. In the case of a green light, the vehicle perceives its headway past the traffic light should the lead vehicle have already passed across the stopline. In the case of a red light, the vehicle perceives its headway to be the shorter of the distance to the lead vehicle and the distance to the stop line. Consequently, when a vehicle faces a red light, it will treat it as a stationary object and slow down as indicated in the previous paragraph. Similarly, the next vehicle in the platoon will simply slow down in order to not collide with the first vehicle in the platoon, which is slowing down because of the red light indication.

However, when a vehicle approaches a traffic light that has just changed from green to amber, the vehicle is faced with a dilemma. On one hand, it can continue to proceed at constant speed, and risk crossing the stop line after the traffic light has transitioned from amber to red. On the other hand, it can elect to slow down, and decelerate at a rate that allows it to come to a stop prior to reaching the stop line. Within INTEGRATION, a vehicle that approaches an amber traffic light indication will compute the deceleration rate required to come to a safe stop prior to crossing the stop line. When this deceleration rate is greater than a comfortable maximum, the vehicle in question will not slow down and will proceed through across the stop line at its current speed. Alternatively, if the required computed deceleration rate is less than the maximum comfortable rate, the vehicle will slow down at a constant deceleration rate.

While deceleration is governed primarily by kinematics, the acceleration rate of vehicles is governed by vehicle dynamics. Specifically, a vehicle with a desired speed that is greater than its current speed will attempt to accelerate at the maximum possible rate. This maximum possible rate is constrained by the maximum acceleration rate that is derived from a constant power vehicle dynamics model. This model considers the resultant force between the vehicle's tractive effort and three resistance forces including the aerodynamic resistance, the rolling resistance, and the grade resistance (Rakha et al., 2001a).

Vehicle Queuing

A natural by-product of the steady state and non-steady state car-following logic is that INTEGRATION represents all queues as horizontal rather than vertical entities. The representation of horizontal queues ensures that queues of vehicles spill back upstream, along a given link or potentially across multiple links. Furthermore, the representation of horizontal queues also ensures that the number of vehicles in the queue will be greater than the net difference between the arrival and departure rate, as the tail of the queue grows upstream towards the on-coming traffic. Finally, the use of the above speed-headway relationship also enables these horizontal queues to exhibit a variable density, depending upon the associated speeds of vehicles within the queue.

Lane Changing Logic

When a vehicle travels down a particular link, it may make discretionary lane changes, mandatory lane changes, or both. Discretionary lane changes are a function of the prevailing traffic conditions, while mandatory lane changes are usually a function of the prevailing network geometry.

In order to determine if a discretionary lane change should be made, each vehicle computes three speed alternatives each deci-second. The first alternative represents the potential speed at which the vehicle could continue to travel in its current lane, while the second and third choices represent the potential speeds that this vehicle could travel in the lanes immediately to the left and to the right of its current lane. These speed comparisons are made on the basis of the available headway in each lane, and pre-specified biases for the vehicle to remain in the lane in which it is already traveling or to move to the shoulder lane.

A vehicle will then elect to try to change into that lane which will permit it to travel at the highest of these three potential speeds. For example, a vehicle may elect to leave the shoulder lane for the center lane in order to increase its headway, and therefore also the speed at which it can comfortably travel. Such lane changing, while discretionary, is still subject to the availability of an adequate gap in the lane into which the vehicle wishes to move.

While discretionary lane changes are made by vehicles in order to maximize their speed, mandatory lane changes arise primarily from a need for vehicles to maintain lane connectivity at the end of each link. For example, a vehicle may ideally desire to remain in the median lane, in order to maintain a higher speed. However, if this vehicle must access the next off-ramp, it must first enter the deceleration lane prior to exiting the link on which it is currently traveling. Consequently, the vehicle will be compelled to move towards the shoulder lanes as it approaches the end of the link and the off-ramp.

In general, lane connectivity requires that eventually every vehicle must be in one of the lanes that are directly connected to the relevant downstream link onto which the vehicle anticipates turning. A unique feature of INTEGRATION's lane changing model is that the lane connectivity at any diverge or merge is computed internally. This saves the model user the extensive amount of hand coding that would be necessary to code link connectivity in networks comprising thousands of links.

Once a lane-changing maneuver has been initiated, a subsequent lane change is not permitted until a pre-specified minimum amount of time has elapsed. In the first instance, this minimum interval ensures that lane changes usually involve a finite length of time to materialize, and that two consecutive lane changes cannot be executed one immediately after the other. Furthermore, while an actual lane-changing maneuver is in progress, the vehicle is modeled as if it partially restricts the headway in both the lane it is moving from and the lane it is changing into. This concurrent presence in two lanes will result in an effective capacity reduction beyond that which would be observed if the vehicle had not made any lane change. The relationship of this impact to the speed and capacity of weaving sections is discussed later.

Link-to-Link Lane Transitions

Upon approaching the end of a link, the above mandatory lane changing logic will ensure that vehicles automatically migrate into those lanes that provide direct access to the next desired downstream link. When the end of the first link is eventually reached, the vehicle is automatically considered for entry into the appropriate lane onto the next downstream link.

The entry onto this downstream link is subject to the availability of an adequate minimum distance headway that is required in order to absorb the new vehicle without violating the specified jam density for the downstream link. In addition, any available headway beyond this minimum is also utilized to set the link entry speed of the vehicle in question. If the maximum headway in the downstream link is insufficient to accommodate the vehicle in question, the vehicle will be retained on its original link until an acceptable headway becomes available. Consequently, congestion in one link can constrain the outflow rate of one or more upstream links, such that queues can spill back across multiple links as required.

Any available downstream capacity is also implicitly allocated proportionally to the number of inbound lanes to the merge. For example, consider a diverge where all lanes have a saturation flow rate of 2000 vehicles/hour/lane. If a pair of two-lane sections merge into a single three-lane section, the combined inflow from the two inbound links will be limited to 6000 vehicles/hour when the downstream link is not congested. However, if an incident were to have reduced the capacity of the three-lane section to only, say, 4000 vehicles/hour, the two inbound approaches would then only have a combined outflow capacity of 4000 vehicles/hour available to them.

The exit privileges of a particular link may also be constrained by a conflicting opposing flow. In this case, the vehicles attempting to cross the opposing flow would need to delay their entry into their next downstream link until a sufficient gap appears in the opposing traffic stream. On a single lane approach, such gap seeking would also delay any subsequent vehicles, even if these subsequent vehicles were to perform a movement that would not be opposed. However, on a multi-lane approach, unopposed vehicles may be able to change lanes in order to utilize the residual capacity in one of the remaining lanes. When discharges in multiple directions occur from the same link, shared lane capacity calculations are performed automatically.

On the basis of the above logic, vehicles proceed toward their destination in a link-by-link fashion, where their speeds, as well as longitudinal and lateral positions, are updated each deci-second until the vehicle's final link is reached. When the vehicle eventually reaches the end of this final link, the vehicle is removed from the simulation network, any trip statistics are tabulated, and any temporary variables assigned to that vehicle are released.

Route Selection and Traffic Assignment

The selection of the next link to be taken by a vehicle is determined by the model's internal routing logic (Rilett and Van Aerde, 1991a and b). There exist many different variations to the model's basic assignment technique, but these variations are beyond the scope of this simulation-oriented discussion. Therefore, only a quick overview of their common procedures is provided.

In general, there exist many different ways within the model in which the next downstream link can be determined. Some of these techniques are static and deterministic, while others are stochastic and dynamic. However, regardless of the particular technique that is utilized to determine these routings, the selection of the next link that a vehicle should take is done using a vehicle-specific array that lists for the vehicle the entire sequence of links from its origin to its destination. Upon the completion of any link, a vehicle simply queries this array to determine which link it should utilize next to reach its ultimate destination in the most efficient manner. When travel across this next link is in turn completed, the selection process is then repeated until a link whose downstream node is the vehicle's ultimate trip destination is reached.

Within this traffic assignment process, the key simulation feature to be noted is that turning movements, and therefore all mandatory lane changes, are vehicle-specific and a function of path-based turning movements, rather than based on more arbitrary turning percentages.

Link Use and Turning Movement Restrictions

One of the features that allows the model to better represent the operational characteristics of many actual networks is the ability to restrict vehicles from using either specific link lanes, and/or making specific turning movements.

Restrictions of links can be implemented for a specific subset of vehicle classes. Therefore, it may be utilized to represent either the restricted availability of a certain link to only HOV vehicles, or the availability of a certain toll booth to a vehicle that possesses a specific toll collection technology (Robinson and Van Aerde, 1995). Alternatively, this feature can also be utilized to model the impact of a defining truck sub-network within a more general road network.

It is also possible to restrict certain individual lanes to specific vehicle classes in order to model, for example, an HOV lane that is exclusive to one vehicle class. Alternatively, a given vehicle may be constrained to utilize only a given lane, for example a truck lane, by restricting this vehicle from utilizing all other lanes. In either case, this restriction is sufficiently flexible to permit vehicles that are turning onto or off of the link to pass through these restricted lanes in order to complete their turning movement.

A third type of restriction is that vehicles can be confined to only make certain turning movements from certain lanes. This ability permits the modeling of exclusive versus shared lanes, and is critical to properly model the impact of advanced/leading phases and/or estimating the number of vehicles that may be able to make a right turn on red before a through vehicle blocks a shared lane.

The final type of restriction can be applied to prohibit specific turning movements. It is typically utilized to represent banned turning movements at intersections for certain periods of time. However, the same feature can also be utilized to represent time dependent access restrictions to the use of a particular reversible lane or on-ramp.

Simulation of Incidents and Diversions

The continuous nature of the simulation model permits incidents to start at any time (to within one minute), be of any duration, and be of any severity (blocking from 0 to 99% of the available capacity). In addition, any specific group of lanes can be blocked at any point along the link, and this blockage can be of any feasible length. Incidents may be modeled concurrently at different locations, or different incidents may be modeled at the same location at different instances of time. The net effect of the incident is that it will reduce the saturation flow, the maximum speed or the availability of each targeted lane on the given link.

At present, the routing logic of INTEGRATION does not directly respond to the occurrence of an incident. Instead, it responds indirectly to the incident delay that arises from the flow or speed restrictions associated with the incident. This indirect response has the effect that diversions do not occur until the delay experienced by vehicles becomes sufficiently large as to make an alternative route more desirable. Similarly, the model may sustain diversions, even after the actual blockage at an incident site has been cleared, but when some residual queues remain to produce on-going delays. Finally, diversions may occur away from links which are incident free, but which are experiencing congestion arising from the incident.

2.5 Freeway Sections

The INTEGRATION simulation logic, while facility independent, is designed to deal with a number of conditions or situations that are usually perceived to be unique to freeway sections, such as merges, diverges and weaving sections. Many of these elements also appear on surface streets, as will be indicated next.

Modeling Merges

In general, when two traffic streams merge, all available merge capacity is allocated using entitlements in proportion to the non-queued capacities of the two merging links. However, since one of the merge lanes may not be able to fully utilize its entitlement, or as the total merge capacity may be further reduced by queues spilling back into the merge, the actual merge capacity always needs to be allocated dynamically.

Therefore, at an on-ramp merge, queues may form downstream of the ramp, upstream of the ramp on the freeway, on the on-ramp, or on both depending upon the prevailing demands. For example, when an acceleration lane is present, following the ramp merge, the queue will automatically be modeled as occurring immediately upstream of the lane drop. When the queue subsequently grows to then fill the entire merge area, the queue may spill back further onto the on-ramp or onto the upstream section of the mainline freeway, depending upon the split in the vehicle arrival rates. However, if there is no acceleration lane provided, the queue will form upstream of the on-ramp merge. The split of the queues on the on-ramp is again a function of the relative vehicle arrival rates on the mainline and the on-ramp.

Once the above merge flow rates are determined, INTEGRATION automatically computes the appropriate shock waves upstream of the merge, on either the mainline or the on-ramp as may be appropriate. Furthermore, the absence of an explicit time slice in the model’s analysis permits the formation of such queues to be analyzed over both very short and/or relatively long time intervals. For example, the formation of merge queues over time periods from, say 15 minutes to several hours, can be modeled if typical peak period demand overloads are to be considered. However, if a traffic signal is present upstream of the particular on-ramp, the model can also consider short-term merge over-saturation for 30 to 60 seconds at a time. Such an event could occur each time that the upstream traffic signal discharges its queues.

It is important to note that the allocation of queues to different upstream arms at a freeway merge is critical to estimating the relative capacities and travel times on each of these links. Errors in estimating these relative travel times not only affect the resulting Measures of Effectiveness (MOEs) in isolation, but also have a significant impact on any dynamic traffic assignments or diversions that consider these MOEs within their routing objective functions.

Modeling Diverges

At diverges, queues may also form when one of the discharge arms receives more demand than there is available capacity. Such limitations of off-ramp capacity may result from poor off-ramp geometry or from limitations arising from the net capacity of a traffic signal that is located at the downstream end of the off-ramp. Alternatively, the mainline section may spill back at diverges due to downstream mainline congestion, even though there is sufficient capacity available both upstream of the diverge and on the off-ramp.

When one of the diverge arms, say an off-ramp, possesses insufficient capacity, vehicles destined for this bottleneck will eventually queue on the upstream section of the diverge. Consequently, this queue may ultimately constrain the flow of through vehicles, where these through vehicles may not even be utilizing the off-ramp in question. The INTEGRATION model computes the resulting queue spill-back as a function of the prevailing off-ramp over-saturation, as well as the extent to which vehicles utilizing the off-ramp tend to congregate in the lane feeding the off-ramp. Research to-date has shown that assumptions, with respect to the extent to which vehicles voluntarily queue in only the shoulder lanes, have a pronounced impact on the ultimate delay to the mainline flow.

An important impact of such queue spill back at diverges is that the link as a whole no longer complies with standard FIFO (First-In-First-Out) queuing logic. Instead, considerable differences in link travel times may arise, depending upon the destinations of the vehicles. Such differences complicate any routing logic, which may need to consider that different link users will arrive at the next downstream link following a different lag time, even though they may have entered the link concurrently. In addition, while the two traffic streams do not share the same travel times, they do interact considerably, as the split in off-ramp versus through vehicles will significantly alter the travel times experienced by both groups. This is the case even if the total arrival flow on the link remains relatively constant.

Modeling Weaving

Within INTEGRATION the final impact of a weaving section is a direct function of the interaction between the prevailing car-following and lane-changing behavior (Van Aerde *et. al.*, 1996; Stewart *et. al.*, 1996).

Specifically, vehicles that engage in a lane-changing maneuver briefly occupy space in both the lane they are leaving from and the lane into which they are changing. The fact that a single lane-changing vehicle consumes capacity in both lanes makes a single weaving vehicle temporarily have the equivalent impact of two vehicles. This effect, which is proportional to the duration of the lane change and the fraction of vehicles making lane change maneuvers, automatically results in a dynamic calculation of weaving capacity in the model. The reduced effective capacity of weaving sections in which a large number of vehicles are making lane changes is more pronounced at the onset of queuing. Lane changing also reduces the prevailing speeds in the weaving section prior to the onset of queuing. Total throughput capacity is usually also reduced through the fact that the availability of lane changing gaps is not uniform, but is instead random. This randomness has further impacts on weaving flows, in the form of speed and capacity reductions that are beyond what would be considered by a deterministic analysis of the above weaving logic.

The weaving logic is also sensitive to the type of weave that takes place, as different weave types require different numbers of lane changes per vehicle. The model is also sensitive to the length of the weave, as a longer weaving section permits the impact of the lane changes to be spread out over a longer length of road segment. It is important to note that weaving logic and impacts are emergent features of the default model logic, and therefore do not require the modeler to tag specific sections as being weaving sections. Therefore, any area in which a large number of mandatory lane changes are necessary will automatically experience weaving impacts. Furthermore, the magnitude of the capacity reduction will dynamically depend upon the mix of weaving versus non-weaving flows. This effect is also automatically captured on arterials, where a rapid succession of alternate turning movements may create surface street weaving section bottlenecks.

2.6 Traffic Signals

The second main roadway element within most urban areas is the presence of signalized intersections. While the highway sections between signalized intersections often operate in a fashion analogous to lower speed freeways, the behavior of traffic at signals is quite unique, as indicated below.

Modeling of Signal Cycles

Within INTEGRATION, a signalized link is identical in virtually all respects to a freeway link. The only exception is that the exit privileges of this link may periodically be suspended, and that the free-speed and saturation flow rates usually take on slightly lower values (Rakha, *et. al.*, 1993).

The suspension of exit privileges is set to occur when the traffic light indicates an effective red. When the traffic light is red, vehicles must still obey the link's car-following logic, except that a red traffic signal is now considered as an additional vehicle positioned just beyond the stop line at the end of each lane on the link. This *virtual* vehicle creates a reduction in the vehicle's perceived headway, and causes vehicles that approach a red signal to slow down as their headway to the traffic signal decreases. Eventually, the first vehicle to approach the red signal comes to a complete stop upstream of the stop line. Subsequent vehicles then automatically queue upstream of the first vehicle in a horizontal queue, where the minimum spacing of vehicles in this horizontal queue is governed by the user-specified jam density.

As shock wave theory applies to both freeways and arterials, the rate at which the tail of this queue moves upstream along the link can be determined in a standard fashion. This rate is equal to the ratio of the "arrival rate at the tail of the queue", divided by the "net difference between the density of queued vehicles and the density of arriving traffic". The dynamic nature of the model's car-following logic also permits the rate at which this queue grows to vary dynamically when the arrival rate varies as a function of time during the cycle. Furthermore the vehicle constrained deceleration and acceleration behavior results in a smooth transition between traffic regimes.

Shock waves at Traffic Signals

When the effective green indication commences, the *virtual* object at the signal stop line of each lane is removed and the first vehicle in queue faces an uninterrupted headway down the next downstream link. The initial acceleration of the first vehicle in the queue, together with the subsequent impact of the model's car-following logic on any additional vehicles, causes two shock waves to form concurrently, as illustrated in Figure 1.

The first shock wave moves downstream from the traffic signal stop-line. It consists of the front of the surge of traffic that crosses the stop line at saturation flow. The second shock wave moves upstream from the traffic signal stop-line. It develops when queued vehicles start to accelerate as the vehicles ahead of them accelerate up to the speed-at-capacity. This backward moving shock wave consists of the dividing lines between those vehicles that are still stationary, and those vehicles that already have begun to accelerate to a speed associated with the saturation flow rate. The speed of this second shock wave is again a function of the speed-flow characteristics of the link.

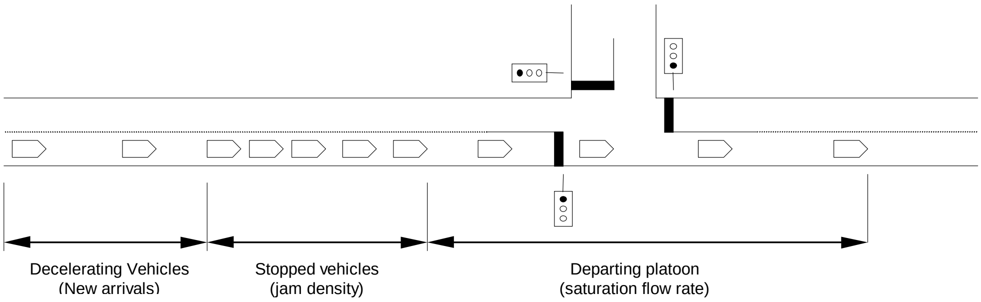

Figure 2: Traffic flow dynamics at a traffic signal several seconds into a green phase.

Uniform, Random and Over-saturation Delay

If the link is under-saturated, the queue that is formed during the red indication will be completely served each cycle. Such a situation occurs during the first cycle, as illustrated in Figure 3. In this situation, the discharge at saturation flow rate will cease prior to the end of the effective green, so that any subsequent vehicles will pass the stop-line at their arrival rate without being required to stop. However, if the signal is over-saturated, vehicles will continue to move at saturation flow rate until the end of the effective green. This will result in a residual queue of unserved demand to be served in subsequent cycles. This scenario occurs during cycles 4, 5 and 6 in Figure 3.

Cycles 2 and 3 exhibit an intermediate state. In this case, over-saturation occurs during one cycle due to randomness in arrival rates, but this over-saturation queue is completely cleared during the subsequent cycle. The complex problem of analytically estimating the expected delay associated with the above situation is effectively circumvented in INTEGRATION. Such estimation is possible as the above uniform, random and over-saturation delays become emergent behavior, instead of being computed based on an explicit formula. Specifically, the vehicle delay is measured each deci-second as the difference in travel time between the speed of the vehicle and travel at free-speed, as will be discussed in more detail in Volume III.

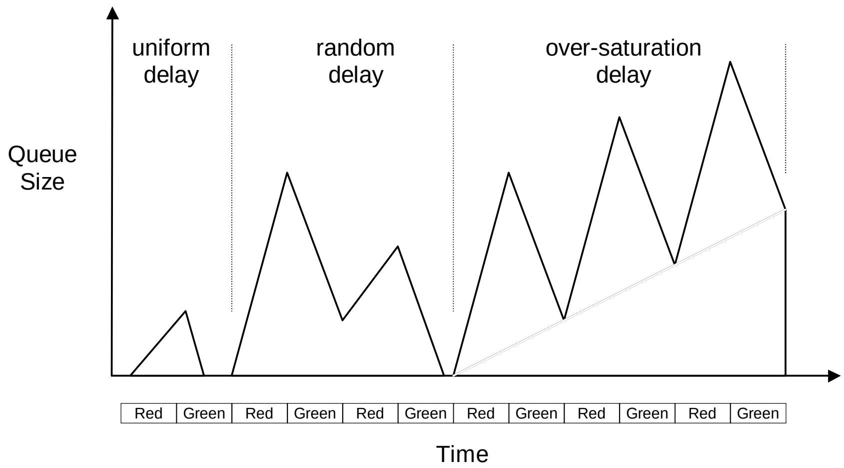

Figure 3: Cycles exhibiting uniform, random, and over-saturation delay.

Gap Acceptance Modeling at Traffic Signals

One of the most complex modeling tasks in estimating the capacity of both isolated and coordinated traffic signals is the treatment of permissive left turns and/or right turns on red. Within INTEGRATION, a microscopic gap acceptance model is utilized to reflect the impact of opposing flows on opposed left turners and right turners on red (Velan and Van Aerde, 1996). This opposition is automatically customized by the model at each intersection by means of built-in logic. This logic specifies which opposing movements are in conflict with the movement of interest. The internal logic also determines which of the turning movements are opposed within a shared lane or shared link. Given the above data, the model can automatically provide opposition to left turners when the opposing flow link discharges concurrently. However, it can also automatically allow the discharge rate to revert back to the unopposed saturation flow rate whenever the opposed movement is given a protected phase.

The incorporation of this logic within INTEGRATION permits the model to evaluate the impact of protected versus permissive left turns. It can also consider the impact of leading versus lagging greens, and the impact of the duration of the left turn phase in great detail. In addition, the gap acceptance logic can work concurrently with the queue spill-back model to determine when, or if, vehicles in a left turn bay spill back into the through lanes, or conversely, when the through lanes spill back to cut off entry into the left turn bays. The combination of lane striping, to change certain lanes from being exclusive to being shared, and the selective opposition of vehicles (depending upon the direction in which they are turning), permits the implicit computation of shared lane saturation flow rates within the model.

Stop and Yield Signs

Exactly the same mechanism as described above for the simulation of permissive left turns at traffic signals can be utilized to model the impact of stop or yield signs. In this case, different critical gap sizes may need to be identified, and several links may concurrently oppose a given turning movement. The simulation logic within INTEGRATION also automatically models the hierarchy in gap acceptance priority of one movement over a lower priority movement. The above hierarchy not only permits the modeling of all-way stop sign controlled intersections, but also allows for a consideration of impedance effects when multiple traffic streams seek gaps in the same opposing traffic flow.

Signal Coordination

The model is also capable of evaluating the impact of alternative signal coordination effects. In a simulation, the discharge pattern downstream of any upstream intersection is preserved as it travels down the link to become the inflow pattern of the next traffic signal. However, the preservation of this pattern is subject to dispersion resulting from variability in vehicle speeds, and from the inflow and outflow of vehicles at mid-block intersections that are not signalized.

The INTEGRATION model is also not constrained to operate all traffic signals on a common cycle length. Consequently, it is possible to evaluate explicitly the impact of a lack of coordination at the boundary of two coordinated sub-networks, and/or to evaluate the impact of removing a single traffic signal from a coordinated network. The latter step might be taken in order to explore the relative benefits of placing one intersection under some form of critical intersection control.

Finally, signal timing plans are optimized using the methods of Webster and Cobbe (l996), as implemented in the Canadian Capacity Guide (ITE, 1984, 1995). This method is similar, but not identical to, the method described in the Highway Capacity Manual (TRB, 2000).

2.7 Measures of Effectiveness

It is implicit, in the earlier discussion of the use of speed-flow and car-following relationships, that the INTEGRATION model does not contain an explicit link travel time function in a fashion similar to most macroscopic or planning-oriented traffic assignment models. Instead, link travel time emerge as the weighted sum of the speeds that vehicles experience as they traverse each link segment. This distinction introduces both a level of complexity and level of accuracy not present in most other models.

Specifically, the dynamic temporal and spatial interactions of shock waves, which form upstream of a traffic signal or along a freeway link that is congested, are such that the final link travel time is neither a simple function of the inflow nor the outflow of the link. Instead, the travel time is a complex product of the traffic flow time series and associated dynamics along the entire link. In addition, the temporal interactions of this flow with the signal timings and flow oppositions at the end of these links are also considered. The strength of a microscopic approach is that, beyond the basic car-following/lane-changing/gap-acceptance logic, there is no need for any further analytical expressions to estimate either uniform, over-saturation, coordination, random, left-turn or queue spill-back delay. While such complexity precludes the simplicity of a functional relationship, such as the Bureau of Public Roads relationship, it does permit two distinct travel times to be properly considered for the same flow level, depending on whether forced or free-flow conditions prevail. It also allows the model to deal much more readily with the concurrent presence of multiple vehicle/driver types on the same link.

Estimation of Link Travel Time

The model determines the link travel time for any given vehicle by providing that vehicle with a *time card* upon its entry to any link. Subsequently, this *time card* is retrieved when the vehicle leaves the link. The difference between these entry and exit times provides a direct measure of the link travel time experienced by each vehicle.

Estimation of Vehicle Delay

As was mentioned earlier the INTEGRATION model estimates vehicle delay every deci-second as the difference in travel time between travel at the vehicle's instantaneous speed and travel at free-speed, as indicated in Equation 1. This model has been validated against analytical time-dependent queuing models, shockwave analysis, and the Canadian Capacity Guide, Highway Capacity Manual, and Australian Capacity Guide procedures, as will be discussed in detail in Volume III of the guide.

$$d_i = \Delta t\left(1 - \frac{u_i}{u_f}\right) \quad \forall\, i \qquad [1]$$

Where:

Δt : The time increment of data processing (1 deci-second).

Estimation of Vehicle Stops

Each time a vehicle decelerates, the drop in speed is recorded as a partial stop, as demonstrated in Equation 2 (Rakha et al., 2001b). The sum of these partial stops is also recorded. This sum, in turn, provides a very accurate explicit estimate of the total number of stops that were encountered along that particular link.

It is noteworthy that INTEGRATION will often report that a vehicle has experienced more than one complete stop along a link. Multiple stops arise from the fact that a vehicle may have to stop several times before ultimately clearing the link stop line. This finding, while seldom recorded by or even permitted within macroscopic models, is a common observation within actual field data for links on which considerable over-saturation queues exist. The details of estimating vehicle stops and the validation of the procedure are presented in Volume III of the manual.

$$S(t_i) = \frac{u(t_i) - u(t_{i-1})}{u_f} \quad \forall\ i \quad \ni u(t_i) < u(t_{i-1}) \qquad [2]$$

Where:

$S(t_i)$	Instantaneous partial stop estimate at time t_i
$u(t_i)$	Speed at instant t_i
$u(t_{i-1})$	Speed at instant t_{i-1}
u_f	Roadway free-speed

Estimation of Vehicle Fuel Consumption

The INTEGRATION model computes the speed of vehicles each deci-second. This permits the steady state fuel consumption rate for each vehicle to be computed each second on the basis of its current instantaneous speed and acceleration (Rakha et al., 2000 and Ahn et al., 2001). These models were developed using data that were collected on a chasis dynamometer at the Oak Ridge National Labs (ORNL). The models use instantaneous speed and acceleration levels as independent variables, as demonstrated in Equation 3. The derivation of the model parameters is described in the literature (Ahn et al., 2001) and also provided in Volume III of the User's Guide.

$$\ln(MOE_e) = \begin{cases} \sum_{i=0}^{3}\sum_{j=0}^{3}(L_{i,j}^{e} \times s^i \times a^j) & \text{for } a \geq 0 \\ \sum_{i=0}^{3}\sum_{j=0}^{3}(M_{i,j}^{e} \times s^i \times a^j) & \text{for } a < 0 \end{cases} \qquad [3]$$

where:

MOE_e	Instantaneous fuel consumption or emission rate (ml/s or mg/s)
$L^e_{i,j}$	Model regression coefficient for MOE "e" at speed power "*i*" and acceleration power "*j*" for positive accelerations
$M^e_{i,j}$	Model regression coefficient for MOE "e" at speed power "*i*" and acceleration power "*j*" for negative accelerations
s	Instantaneous Speed (km/h)
a	Instantaneous acceleration (km/h/s)

Figure 4 illustrates a good fit between the instantaneous fuel consumption models (lines) and the ORNL data (symbols) for an average composite vehicle. The figure clearly demonstrates that vehicle accelerations have significant impacts of vehicle fuel consumption rates especially at high speeds with the resulting high engine loads. A more detailed description of the model development and validation is presented in Volume III.

The above fuel consumption analysis features are built into the model and are executed every second for every vehicle in the network. They are applied in a fashion that is consistent across all facility types, operating regimes, and control strategies. This consistent internal use permits a very objective assessment of the fuel consumption implications across a wide range of potential traffic or demand management strategies. The use of vehicle speed and acceleration levels for estimating vehicle fuel consumption requires an accurate modeling of vehicle acceleration behavior as will be described in more detail in Volume III.

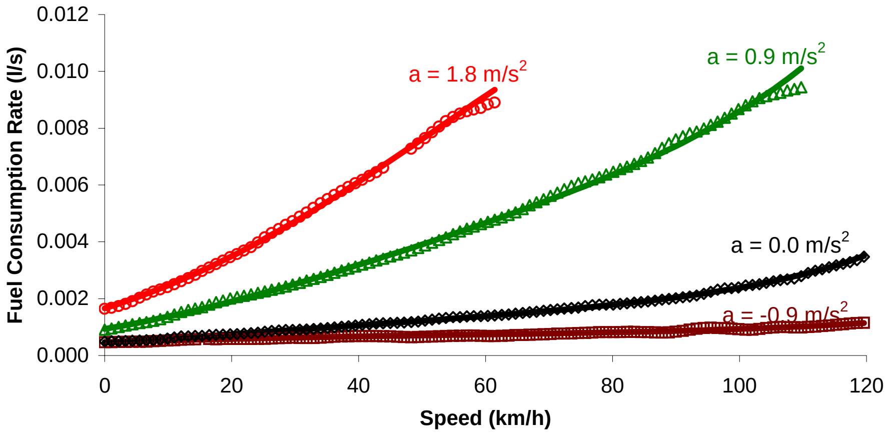

Figure 4: Instantaneous fuel consumption model based on ORNL Data (Rakha et al., 2000 and Ahn et al., 2001)

Estimation of Vehicle Emissions

A series of compatible vehicle emission models have been developed using the same ORNL data (Rakha et al., 2000 and Ahn et al., 2001). These models, which estimate hot-stabilized tail-pipe hydrocarbon (HC), carbon monoxide (CO) and nitrous oxide emissions (NO_x) also operate on a second-by-second basis. As was the case with the fuel consumption models, the emission models are sensitive to the instantaneous vehicle speed and acceleration levels, as illustrated in Figure 5. Further enhancements are being conducted to account for the ambient temperature, the extent to which a particular vehicle's catalytic converter has already been warmed up during an earlier portion of the trip, and high emitting vehicles.

Applications of these models have shown that the emission of these three compounds is related to vehicle travel time, distance, speed, and fuel consumption in an often highly nonlinear fashion. Consequently, traffic management strategies that may have a significant positive impact on one measure are not always guaranteed to have an impact of the same magnitude, or even sign, on any of the other measures. The types of analyses that can be performed with these built-in models therefore extend far beyond the capabilities of EPA's MOBILE5 model (EPA, 1993). This model considers only a single fixed speed profile for any given average speed. In addition, it considers primarily the number of vehicle miles traveled as the main predictor variable. INTEGRATION, however, does not explicitly consider vehicle age or maintenance level, as in the case of MOBILE. A more detailed description of the models is provided in Volume III together with a description how these models were validated.

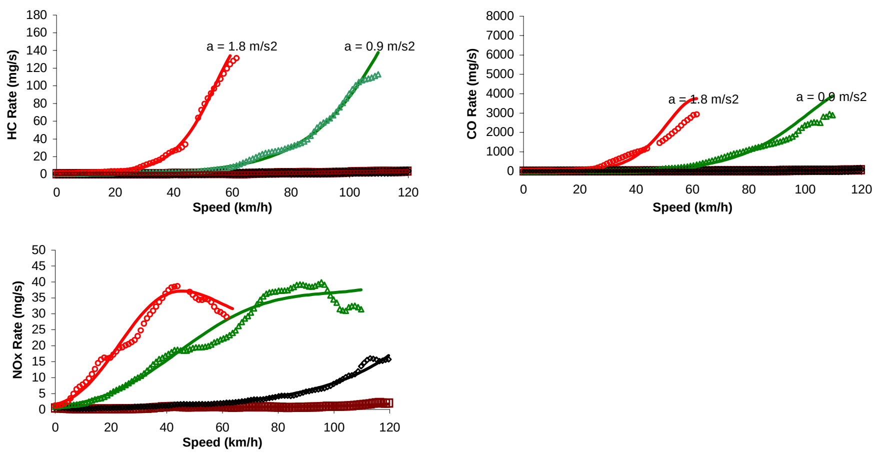

Figure 5: Instantaneous emission models based on ORNL data

Estimation of Crash Rate and Severity

The safety model that is embedded in the INTEGRATION software is based on US national crash statistics. The model computes the crash risk for 14 different crash types as a function of the facility speed limit and a time-dependent measure of exposure. The use of a time-dependent measure of exposure allows the model to capture differences in the crash risk that result from differences in the network efficiency. The model also computes the vehicle damage and level of injury to the passengers involved in the crash based on the vehicle's instantaneous speed. The use of the instantaneous speed means that the crash damage and injury level is responsive to the level of congestion. Consequently, the model can capture the safety impacts of operational-level alternatives including Intelligent Transportation Systems.

Aggregation of Statistics by Link and O-D Pair

The same *time card* concept used for recording a vehicle’s travel time and number of stops on a particular link is also utilized to track the fuel consumption and emissions for each vehicle on each link. Internal to the model, these statistics are further aggregated, both for all links traversed by a particular vehicle and for all the vehicles that have traversed a particular link. The former statistics can be aggregated at the O-D level by time period or vehicle class. Alternatively, they can be aggregated by time period for each link, or by cell within a latitude/longitude grid. When emission data are tracked by latitude and longitude as a time series, these data can in turn be provided as input to an external air quality emission model of the atmospheric conditions for an entire urban area.

In addition to tracking the number of lane changes occurring within the network and counting the number of vehicle passes, the model also provides an estimate of cumulative crash risk. This crash risk is again estimated on a second-by- second basis by summing up a time-dependent crash rate that is dependent on the roadway free-speed. The crash risk model was developed using the General Estimates System (GES) database, which reflects average crash rates in the US. The use of a time-dependent crash risk allows the model to account for the higher crash risk that is associated with spending more time on the network. While a more detailed description of the crash model is presented in Volume III, it is sufficient to note at this point that the use of the model in this capacity permits the estimation of the safety of operational level projects including Intelligent Transportation Systems.

Loop Detectors and Vehicle Probes

The microscopic approach used by INTEGRATION also permits the generation of surveillance data that is representative of the data obtained from loop detectors and/or vehicle probes.

Specifically, when a vehicle traverses any location that is considered to be a loop detector site, the model records a vehicle count, the vehicle's speed, and the estimated vehicle detector occupancy. These data are then accumulated into 20-, 30- or 60-second readings, which can be provided as an additional model output. The ability to locate vehicle detectors anywhere on a link, and/or to locate multiple vehicle detectors on a given link, permits considerable flexibility in the collection of additional model statistics. In addition, this ability permits the evaluation of alternative surveillance levels in support of real-time control and/or various incident detection schemes. Detectors can also be coded as trapping only a certain vehicle class. This could represent, for example, a special bus or truck sensor.

Certain vehicles can also be flagged as being vehicle probes. In the most basic setup case, a separate record is generated each time a probe vehicle starts or ends a trip. In a more intermediate level of analysis, a separate record can be generated each time a probe vehicle completes a link. Finally, in its most detailed form, a separate record can be recorded for each probe vehicle at a user-specified time frequency. The latter detailed statistics are most useful in tracking the speed profiles of vehicles along a targeted set of links.

Chapter 3: Fundamental Model Input Requirements

3.1 Introduction

The input data that are required to run the model are divided into fundamental data and advanced data. The fundamental data are essential to run the model, while the advanced data allow optional model features to be activated. It should be noted, at this time, that the input data for INTEGRATION 2.30 for Windows in general remain unchanged from the input data for INTEGRATION 2.10 for DOS.

This chapter provides an introductory discussion on the structure and the use of the fundamental input data files that are used within the INTEGRATION model. More advanced model capabilities, and the associated input data files/fields required to invoke them, are described in Volume II of this Guide. It is recommended that the model user become familiar with the fundamental model capabilities prior to attempting to use the more advanced model features described in Volume II.

For each of the fundamental input files, an example is provided, together with a description of each of the fields within the file. Table 4 provides a listing of all of the model's fundamental input data files and a brief description of their content. Each of the fundamental input data files must be provided in order for the simulation model to be able to execute. If one of these fundamental input files is either not present or not found, the model will issue an appropriate error message and terminate the simulation.

Table 4: Fundamental input data files

File Name	Description
Master File	Master control file which specifies the global simulation parameters, and the location as well as the names of any input and output files
File 1	Node coordinates, characteristics and attributes
File 2	Link structure, characteristics and signal phasing discharge
File 3	Signal timing plans
File 4	Origin-Destination traffic demands
File 5	Incident descriptions

3.2 General Input Data File Format and Generation

The names of all input data files are to be entered as ASCII characters in a tabular format. The specifics of each file are described in the subsequent sections to this chapter. Spaces, commas, or tabs can separate numeric fields. However, the use of any other special visible or hidden characters must be avoided. It may be useful to note that all files are generally read in using standard free-format style FORTRAN READ statements.

The input data files can be generated/modified using any standard editor, spreadsheet or word processor (in non-document mode), given that the above guidelines are complied with. However, the inclusion of special formatting characters and the insertion of blank lines must be avoided. The input data files may also be generated through the use of special purpose translation programs that convert the input data files that were initially generated for another traffic simulation or transportation planning model into an INTEGRATION format.

In the following sections, the data structure of each of the input data files is provided. In each case, an example data file and a description of each of the fields comprising the file records are provided. The data type (integer, real, alphanumeric) of each input data field is also identified. Finally, where appropriate, the range of acceptable values for each field is provided.

The tables presented in this first volume of the User's Guide only provide descriptions of the basic model features. Volume II of the Guide, which describes the model's advanced features, repeats the same field description tables, but adds to the basic values for each field all valid parameter values allowed in a simulation. The tables presented in this volume are therefore a sub-set of those presented in Volume II. Despite this fact, the model user is strongly encouraged to first become familiar with the model's fundamental features presented in this volume, prior to exploring the advanced features described in Volume II

3.3 Executing the Sample Simulation on the Distribution Disk

It is instructive at this time to discuss the contents and use of the INTEGRATION DISTRIBUTION DISK and the sample networks provided on it. The operation of the INTEGRATION model for the sample networks is similar for all commercial versions of the program, except that the larger executable modules will have different names and require additional memory to be available.

Installation of QNET Sample Network

In order to install the software, it is necessary to create a subdirectory on the hard drive into which all the files on the INTEGRATION DISTRIBUTION DISK can be copied. Subsequently, the files on the disk need to be unpacked.

Running the SETUP.BAT batch file will extract the executable and the sample input and output directories. Run the setup batch file by typing its name at the DOS prompt or by double clicking on the file.

In order to run the INTEGRATION software you will have to type the name of the executable followed by the name of the master control file with a space between the executable and the master control files.

The **INTEGRATION** model will then start to execute a simulation run based on the various traffic network and traffic flow parameters that are specified in the file `QNET.INT`. As a reference, it should be noted that the simulation of a 45-minute scenario of the QNET network on a 400 MHz Pentium with 16 Mb of RAM requires approximately 10 minutes to complete.

In order to execute variants of the sample network, the existing `QNET.INT` file can be modified. It is recommended that a backup copy of the original files be made prior to making modifications. The user may add trips, links, traffic signals or nodes to the network, as long as none of the network size limitations for the particular version of the model are exceeded. The network size limitations, that are associated with a particular version of the INTEGRATION model, are listed at the start of the `RUNERR.OUT` diagnostic file. This file is created automatically by the program each time it attempts to execute.

3.4 Master Control File

The Master Control File provides general simulation parameters to the model and defines which input data files are to be used. It also indicates where these inputs are located, defines which output files are to be produced, and where these output files are to be stored.

The user may freely select the name of the Master Control File. By convention, however, master files are typically assigned the file type extension "`.INT`." This file extension convention is not necessary for the model to execute correctly, but it has been found helpful to assist the user in maintaining multiple data sets.

The master file is usually located on the current disk drive and within the current "logged-on" sub-directory. In this case, only the name and type of the master file need to be specified as command line options (e.g. `INTGRATS TEST1.INT`). Alternatively, if the master file is located in a different sub-directory or on a different disk drive, the complete path to this alternate location of the master file must be specified (e.g. `INTGRATS D:\TRAFFIC\TEST1.INT`).

An example of a Master Control File for the QNET network is provided in Table 5. A description of the fields in this file is provided in Table 6.

Structure of Master File

The master control file contains three main types of data. The first of these three data sections is contained on Lines 1 and 2. On Line 1, a title for the Master Control File is provided. On Line 2, several parameters defining the length of the simulation, the frequency at which various outputs are generated, and the routing characteristics of the drivers, are provided. The second data section is contained on Lines 3 and 4. This section describes the input and output subdirectories to be utilized. All input data files are considered to be present in the subdirectory listed on Line 3, while all output will be routed to the subdirectory listed on Line 4. The third data section is contained on the remaining lines. It provides the names of the various input and output files that are to be utilized by the simulation model.

Simulation duration

The simulation duration on Line 2 is specified in seconds. This value should be sufficiently long to permit all generated vehicles to complete their trips prior to the end of the simulation. This will ensure that complete trip statistics are recorded for all vehicles. Typical simulation duration may range from several minutes to an entire 24-hour day.

Frequency of output statistics

The generation of output files may occasionally become a time-intensive computational process for large networks. Furthermore, the frequent generation of outputs may also cause these files to become very large. Care should therefore be exercised in choosing practical values of these output interval times (i.e. typically greater than 900 seconds or 15 minutes). In the event that all available hard drive space is used during a simulation run, a system run time error (error 125) will occur.

Note that the amount of available hard drive space is the total free space, minus any amount that is used as virtual memory.

Route choice selection

Field 4 on Line 2 of the Master Control File specifies the routing mechanism for all vehicles. A value of "1" results in a Dynamic Traffic Assignment being applied to all vehicles. This method is recommended for novice model users. A value of "2" permits a backward compatibility to the old routing format prior to INTEGRATION 2.30. Finally, a value of "3" provides for a new expanded routing format specification. Routing based on options 2 and 3 are described in Volume II.

Simulation Termination

Field 5 on Line 2 determines if the traffic simulation will be paused at the end of the simulation horizon or not. Specifically, a value of "0" will simply result in the simulation being terminated with the end of the simulation horizon is reached. In contrast, a value of "1" will result in the simulation being paused just prior to the last second of the simulation. This option may permit to review the on-screen animation or any on-line simulation statistics prior to ending the simulation.

Location of data files

Lines 3 and 4 indicate the location of the simulation input and output subdirectories. All input data files must be placed in a common input sub-directory, as specified on Line 3. Likewise, all requested output files will automatically be placed in a common output sub-directory, as specified in Line 4.

If input and output files are to be located in the same directory as the Master Control File, blank lines can be left in each of these fields. In this case no sub-directory paths will be added to the subsequent file names, when these files are opened.

Both absolute (`C:\TEST_RUN\CASE2a\INPUT\`) and relative (`INPUT\`) sub-directory references are permitted. The only requirement is that they are valid for the version of the operating system that is being utilized. Also, please note that the name of the subdirectory should be concluded with a backward slash. Also, it should be noted that the output directory should be created prior to executing the simulation model.

Names of data files

Lines 5 through 27 of the master file allow the user to specify the names of the 5 required fundamental input data files, as well as the names of any optional input and output files. When any optional files are not required, the character string "`none`" or "`NONE`" should be entered on the appropriate line.

All of the input files should be given the file type extension "`.DAT`". Similarly, all output files should be given the file type extension "`.OUT`". The use of these files type extensions is not required, however, this convention is recommended in order to assist the user to quickly identify different file types. It is also recommended that each input/output filename be followed by the corresponding file number (1 through 22). In this case, `TEST2.DAT` could then be quickly identified as the link file for the test run, while `TEST10.OUT` could be easily recognized as Output File 10.

Table 5: Sample control file with fundamental input and output data files.

```
QNet master file
2700 900 300 1 0
qnet\
qnet\output\
qnet_01.dat
qnet_02.dat
qnet_03.dat
qnet_04.dat
qnet_05.dat
none
none
none
none
qnet_10.out
none
none
none
none
none
none
none
none
none
none
none
none
```

Table 6: Description of basic fields in Master Control File

Line	Field	Description
1	1	File title (up to 300 characters) The title will appear in formatted output and can serve to clearly identify the results.
2	1	Total simulation time (seconds) [real] ($x \geq 1$)
	2	Frequency of outputs (seconds) to Output File 10 [integer] ($0 \leq x \leq$ *simulation time*) If $x = 0$ then no output is written to File 10. If $0 < x \leq$ *simulation time* then output is written to File 10 at time zero and every *x* seconds.
	3	Frequency of outputs (seconds) to Output Files 12 - 14 [integer] ($0 \leq x \leq$ *simulation time*) Recommend initial value of $x = 0$ (See *Volume II* for more details)
	4	Routing Option [integer] (1, 2 or 3) if x=1, the routing will based on a Dynamic Traffic Assignment if x=2, the routing will be based on the pre-2.30 routing format. If x=3, the routing will be based on an expanded format. (recommend setting x=1)
	5	End of simulation PAUSE flag [integer] if x=0 simulation will not be paused at the end of the simulation horizon. If x=1, simulation will be paused at the end of the simulation. (Recommend setting x=1)
3	1	Directory path for directory in which the input files are located. If the line is left blank, then the model assumes that the files are in the current directory.
4	1	Directory path for directory in which output files are to be placed. If the line is left blank, then the model assumes that the files are in the current directory. Note that path must be different that the directory of the input files.
5-9	1	Names of required input data files (Files 1 - 5)
10-13	1	The name `none` or `NONE` should appear in these fields.
14	1	Name of the required standard output file (File 10) If `none` or `NONE` is entered, a default file called `default.u10` will be created.
15-27	1	Names of the optional input/output files (Files 11-22) The name `none` or `NONE` should appear if the optional file is not desired.

3.5 File1: Node Characteristics

The Node Characteristic File specifies the X and Y coordinates of all the zones and nodes in the network. In addition, it also lists which nodes can only serve as intermediate points along a given vehicle's trip path. A sample node characteristic file for the QNET network is provided in Table 7 together with a description of the fields contained within the Node Characteristic File in Table 8.

The first line of the file, as in the case of all files in INTEGRATION, allows the user to input a file title that is descriptive of the data in the file. The second line includes the number of entries in the file and two scaling factors.

Scaling factors

The scaling factors are utilized to modify the X and Y coordinates of the nodes that follow. These multiplicative scale factors facilitate the transformation of the coordinates from one coordinate system into another one, such as the transportation of a mile-based system into a kilometer-based system. For instance, the use of a scale factor of 1.0 or 0.0 leaves the X and Y coordinates unchanged. However, a scale factor of 1.609 could be used to convert coordinates from miles to kilometers.

Node numbers

Each node must be assigned a unique identification number. Node numbers need not be consecutive and can have gaps in their sequence. However, restrictions exist on the highest destination number, the highest origin number, the highest macro zone number and the highest node number that is permitted. Therefore, in order to run the largest possible network, it is desirable to adhere to a convention of assigning the lowest node numbers to origins/destinations first, leaving the higher numbers to nodes which are neither origins nor destinations.

Node X-Y coordinates

The values of the X coordinates should increase from West to East, while the values of the Y coordinates should increase from South to North. No two nodes may be assigned identical X and Y coordinates.

Node coordinate values are primarily utilized for graphical display purposes and therefore need not be exact. They may therefore at times be deliberately distorted to provide a better visual representation of the network on the screen. However, the coordinates should ensure that the relative direction of all turning movements is maintained. Link lengths are explicitly defined in the link file and are not determined from the graphical node locations. The X-Y coordinates do affect at what level of zooming the link, node and zone labels will be listed on the screen. They also control the displayed lane width and vehicle size. To reduce the displayed lane width and vehicle size, relative to the size of the network, the scaling factors should be made smaller. To increase the displayed relative lane width and vehicle size, one may increase the scaling factors.

When displaying the network on the screen, INTEGRATION attempts to maximize the amount of the screen that is used, while still adhering to a 1:1 aspect ratio. A long but thin network may therefore be automatically stretched to become nearly rectangular (in order to fill the graphics window) by changing the Y scale factor. While such a change will cause the Y dimension to become somewhat exaggerated, the resulting distortion has no impact on the quantitative simulation results.

Macro zones

Macro zones provide users with a mechanism for modeling large traffic networks with reduced computational and memory requirements. In this volume of the Guide, it is sufficient to note that the model will run with the least level of complexity when each destination zone is assigned a unique macro zone number. A unique macro zone number is assigned by inputting a negative zone number in the macro zone field, as is the case in the sample data file in Table 7. In general, a macro zone number need not be equal to the node number associated with the destination zone, however, it must be unique and less than the maximum macro zone number that is permitted within the version of INTEGRATION being used.

Every destination zone (i.e. node type 1 or 2) must be associated with a macro zone. Every macro zone must have one and no more than one destination zone coded as the macro zone centroid. **Nodes that do not serve as trip destinations should have their macro zone number set to zero**.

Optional node labels

Optional node labels are not utilized directly by the model, but may assist in any error checking of the data file. This label can be any mixture of numeric codes and characters, where the label is considered to start at the first alphabetical character following Field 6. In the `QNET1.DAT` sample file, optional labels have only been used to identify the origin and destination zones, as demonstrated in Table 7.

Table 7: Subset of a Sample Node Characteristic File (File 1)

```
QNET Node Coordinate File
  32     1.0     1.0
   1     0.25    1.5    1 -1  0   zone 1
   2     1.0     1.75   1 -2  0   zone 2
   3     3.0     1.75   1 -3  0   zone 3
   4     3.75    1.5    1 -4  0   zone 4
   5     3.75    1.0    1 -5  0   zone 5
   6     3.75    0.5    1 -6  0   zone 6
   7     3.0     0.25   1 -7  0   zone 7
   8     1.0     0.25   1 -8  0   zone 8
   9     0.25    0.5    1 -9  0   zone 9
  10     0.25    1.0    1 -10 0   zone 10
  11     0.5     1.5    4  0  0
  12     1.0     1.5    4  0  0
  13     2.0     1.5    4  0  0
  14     3.0     1.5    4  0  0
  15     3.5     1.5    4  0  0
```

Table 8: Description of fields in Node Characteristic File (File 1)

Line	Field	Description
1	1	File title (up to 300 characters)
2	1	Number of nodes for which information is provided in the file [integer] ($2 \leq x \leq$ *max. number of nodes*)
	2	Scale factor applied to all x-coordinate values listed in the file [real] ($x > 0.0$) This factor can be used to convert coordinates from any unit system into another system, such as converting from mile to kilometers.
	3	Scale factor applied to all y-coordinate values listed in the file [real] ($x > 0.0$) This factor can be used to convert coordinates from any unit system into another system, such as converting from mile to kilometers.
3+	1	Unique node identification number [integer] ($1 \leq x \leq$ *max. node number*) Node numbers cannot be duplicated and should be consistent with those in Files 2 and 4.
	2	X coordinate of node location [real] ($x > 0.0$)
	3	Y coordinate of node location [real] ($x > 0.0$)
	4	Node type identifier [integer] ($1 \leq x \leq 4$) 1 = both origin and destination zone 2 = destination zone only 3 = origin zone only 4 = intermediate node only
	5	Macro-zone cluster number [integer] ($\|x\| \leq$ *max. macro-zone number*) $x = 0$ for node type 3 or 4. $x < 0$ when zone is the centroid of macro-zone cluster x (node type 1 or 2 only).
	6	Information availability indicator [real] Recommend $x = 0$ when no information providing device exists at that node. (See *Volume II* for more details).
	7	Optional alpha-numeric descriptor of this node (maximum 40 characters)

3.6 File 2: Link Characteristics

The Link Characteristic File defines the upstream and downstream node of each directional link or arc in the network. It also specifies the spatial characteristics of the link, as measured by its length and the associated number of lanes. The traffic flow characteristics are described in terms of the saturation flow rate, free-speed, speed-at-capacity, and jam density. In addition, the presence and type of any traffic control devices (e.g. turn prohibitions, signals, stop signs, and yield signs) is indicated. A portion of a sample file is presented in Table 9 together with a brief description of the fields contained in File 2 in Table 10.

Scaling Factors

In File 2, the scaling factors listed on the second line operate in a fashion similar to the scaling factors of File 1. Specifically, they permit the most important link variables to be scaled uniformly for all the links in the network. Such scaling may be appropriate to perform network-wide sensitivity analyses related to the impact of, say, rain, snow, or errors in estimating capacity or free-speed estimates.

Link Numbers

In a network setup, each link must be assigned a unique link identification number. Link numbers need not be consecutive and can have gaps in their sequence. However, restrictions exist on the maximum link number. Within this constraint, it may be useful to group link numbers by facility type, geographic sub-area or interchange number.

Upstream and Downstream Nodes

The orientation and direction of each link within the network is defined by the nodes that the link connects. Vehicles travel along a link from the upstream node to the downstream node. No two links may share both the same upstream and downstream nodes.

Link Length

The length of a link is explicitly specified in File 2, rather than derived from the coordinates of the link's upstream and downstream nodes as indicated in File 1. This permits the deliberate distortion of a network in order to provide clearer graphics without altering simulation results. However, if a consistent scale is not used when coding a network, the graphical representation of link lengths will not be consistent with the specified link lengths and vehicles appear to move faster on some links than others. Again, this effect does not affect the underlying model logic. However, it may at times lead the model user to draw incorrect conclusions from the animation.

In order for the model to yield realistic lane changing behavior, the use of very short link lengths should be avoided. The minimum link length permitted is a function of the number of lanes on the link and is set to 0.010 kilometers × *number of lanes*. Thus, the length of a link with 3 lanes must be equal to, or longer than, 30 meters. This value represents an absolute minimum. However, it is recommended that links not be made shorter than required when possible.

Link Capacity

The base capacity of a link is derived from the product of the specified number of link lanes and the saturation flow rate per lane. This base capacity may be subsequently reduced to reflect the impact of traffic signal controls, ramp meters, incidents, or weaving traffic streams. Since the occurrence of incidents and the dynamic impact of weaving and signal timing plans are not constant and not known *a priori*, the final capacity is arrived dynamically by the model during the course of the simulation.

Link Speed-Flow Relationship

The speeds of vehicles are continuously updated during the simulation based on a continuous speed-flow relationship. This relationship is derived from the user-specified free-speed, capacity, speed-at-capacity, and jam density. The choice of different values of these four parameters permits the derivation of virtually any speed-flow relationship that may be desired. For example Table 9 demonstrates that all of the ten links that are listed have a free-speed of 60 km/h, a base capacity of 2000 veh/h/lane, a speed-at-capacity of 40 km/h, and a jam density of 100 veh/km/lane.

The speeds/travel times during congested conditions are automatically calculated by the model using the lower portion of the speed-flow relationship, and are updated as a function of the sizes of any queues that may be present on each link. The use of an instantaneous speed-flow relationship also permits very accurate tracing of shock waves at freeway bottlenecks, traffic signals and incident sites.

Opposing Links

An opposing traffic stream may restrict the movements of traffic flows exiting a link. INTEGRATION permits up to two potential opposing traffic streams to be specified for each link discharge. To identify these streams, the user need only identify the link(s) containing the conflicting traffic streams, and not the specific turning movements to which conflicting vehicles on these links are associated. In all conflicting scenarios, INTEGRATION's internal logic determines if an opposing vehicle poses an actual conflict or not. For example Table 9 demonstrates that link 2 is opposed by a single link, namely link 12.

Traffic Signals, Stop and Yield Signs

The extent of any signalization (or ramp metering) on a link is specified to the model with reference to the number of a traffic signal. The traffic signal number should correspond to the timing plans that are provided in File 3. Similarly, the phase numbers, which allow the appropriate phase timings to be picked up within each plan, should also be consistent with File 3. Any non-signalized links should have the traffic signal number set to 0 in File 2. It should be noted that several links may be coded as being controlled by the same traffic signal, and subsets of these links may also be coded to discharge during the same phases. For example Table 9 demonstrates that link 3 is controlled by traffic signal 1 and discharges during phase 1.

Stop signs can be coded as traffic signal 10001, while yield signs can be coded as traffic signal 10002. Links controlled by yield signs will simply invoke the gap acceptance logic for any vehicle that discharges from that link, but vehicles will not be required to slow down. However, links controlled by stop signs will first force vehicles to come to a complete stop before the gap acceptance logic is invoked. In each case, the actual link that opposes the traffic flow of interest must be identified explicitly, such that any opposition effects will be computed on-line as a function of the traffic flows that are observed on the opposing links. If a link is coded as being controlled by a yield sign, in the absence of any opposing link specification, it will have no impact on vehicles that discharge from that link. When a link is coded as being controlled by a stop sign, in the absence of any opposing link specification, it will simply cause vehicles to come to a stop, before they are permitted to proceed.

INTEGRATION Version 2.30 does not require a separate link to be used for each separate turning movement. Instead a single link may be configured for an intersection such that all turning movements are still considered explicitly.

Fixed-Time Ramp Meters

Ramp metering controls can best be modeled by coding a single on-ramp as two separate sequential links. In this case, the first link can be coded such that it is controlled by a ramp metering traffic signal. This link would store any queues. The second link would then allow for the acceleration by the vehicles from the ramp meter stop line to the actual freeway-ramp merge area.

The timings for the ramp-metering signal should be specified in File 3, using an appropriate ramp metering cycle length and green phase duration. However, unlike regular traffic signals, no links should be coded as discharging in the second phase of the ramp signal. Ramp metering signals should be coded as *not* participating in any signal timing optimization, as isolated intersection capacity and optimization procedures clearly do not apply in this case.

Surveillance Coding

The final field in the link characteristics file indicates if the link in question is being monitored for real-time travel time data (x = 11111), or if only the link's free-speed travel time is available (x = 00000). If a link is under surveillance, then real-time link travel time data are considered to be available for this link for the purpose of any feedback routing, as will be discussed later.

Table 9: Subset of a Sample Link Characteristic File (File 2)

```
QNET Link File
68   1.0  1.0  1.0  1.0  1.0
 1   11  1  0.250  60  2000 1  0 40 100  0 0 0  0   0     0  0  0  00000 11111
 2   12 11  0.500  60  2000 1  0 40 100  0 0 0 12   0     0  0  0  00000 11111
 3   13 12  1.000  60  2000 1  0 40 100  0 0 0 11   0     1  1  0  00000 11111
 4   14 13  1.000  60  2000 1  0 40 100  0 0 0 10   0     0  0  0  00000 11111
 5   15 14  0.500  60  2000 1  0 40 100  0 0 0  9   0     2  1  0  00000 11111
 6    4 15  0.250  60  2000 1  0 40 100  0 0 0  8   0     0  0  0  00000 11111
 7   15  4  0.250  60  2000 1  0 40 100  0 0 0  0   0     0  0  0  00000 11111
 8   14 15  0.500  60  2000 1  0 40 100  0 0 0  0   0     0  0  0  00000 11111
 9   13 14  1.000  60  2000 1  0 40 100  0 0 0  5   0     2  1  0  00000 11111
10   12 13  1.000  60  2000 1  0 40 100  0 0 0  0   0     0  0  0  00000 11111
```

Table 10: Description of fields in Link Characteristics File (File 2)

Line	Field	Description
1	1	File title (maximum 300 characters)
2	1	Number of links listed in the file [integer]. ($2 \le x \le$ *max. number of links*)
	2-6	Scale factors for link length, free-speed, saturation flow rate, speed-at-capacity, and jam density, respectively [real]. ($x > 0.0$)
3+	1	Unique link identification number [integer]. ($1 \le x \le$ *max. link number*)
	2	Identification number of the node at the upstream end of the link [integer]. The node number must be contained within File 1. ($1 \le x \le$ *max. node number*)
	3	Identification number of the node at the downstream end of the link [integer]. The node number must be contained within File 1. ($1 \le x \le$ *max. node number*)
	4	Link length (km) [real] (*min. length* $\le x \le 10.000$), where *min. length* = 0.010×number of lanes on the link
	5	Free-speed on link (km/h) [real]. ($10.0 \le x \le 200.0$)
	6	Basic saturation flow rate per lane (veh/h) [real]. ($100.0 \le x \le 10000.0$)
	7	Number of lanes [real]. ($1.0 \le x \le 7$)
	8	Vehicle speed coefficient of variation (mean/standard deviation) [real]. Recommend $x = 0.0$. (See *Volume II* for more details).
	9	Speed-at-capacity (km/h) [real]. ($10.0 \le x \le$ *free speed*)
	10	Jam density (veh/km/lane) [real]. (*density-at-capacity* $\le x \le$ 5**density-at-capacity*), where *density-at-capacity* = saturation flow/speed-at-capacity.
	11	Turn prohibition indicator [integer]. ($0 \le x \le$ *max. link number*) If $x = 0$ then no turns are prohibited. If $x > 0$ then x is interpreted as the identification number of the link to which turns from the present link are not permitted.
	12	Simulation time at which the prohibition of Field 11 is activated [integer]
	13	Simulation time at which the prohibition of Field 11 is deactivated [integer]
	14	Link number of first link opposing current link [integer]. ($0 \le x \le$ *max. link number*) If $x = 0$ then no links oppose the current link. If $x > 0$ then x is interpreted as the number of the first link that opposes the flow of the current link.
	15	Link number of second link opposing current link [integer]. ($0 \le x \le$ *max. link number*) If $x = 0$ then no second link opposes the current link. If $x > 0$ then x is interpreted as the number of the second link that opposes the flow of the current link.
	16	Number of the traffic signal which controls exit privileges of this link [integer]. ($x \le$ *max. signal number* or $x = 10001$ or 10002) If $x = 0$ then no signal controls exit privileges. If $x > 0$ then x is the traffic signal number controlling exit from this link. If $x = 10001$ then a stop sign is modeled. If $x = 10002$ than a yield sign is modeled.
	17	Number of the first phase of the above signal in which this link discharges [integer] ($1 \le x \le$ *max. number of phases*) If $x = 0$ then signal does not exist.
	18	Number of the second phase in which this link discharges [integer] ($0 \le x \le$ *max. number of phases*) If $x = 0$ then signal does not exist or link does not discharge during a second phase.
	19	Vehicle class link prohibition indicator [integer]. ($00000 \le x < 11111$) Recommend $x = 00000$ indicating no prohibitions exist
	20	Surveillance level indicator [integer]. ($00000 \le x < 11111$) Recommend $x = 11111$ to indicate that link is under surveillance
	21	Optional descriptive name of link [alpha-numeric]

3.7 File 3: Signal Timing Plans

Traffic signals are specified within the INTEGRATION model using a two-step process:

1. In the first step, the traffic signal number that controls each network link is identified within File 2. As default, up to two phases of discharge can then be specified for this link.
2. In the second step, each signal's phasing scheme and interval duration is specified in File 3.

This coding scheme requires the model user to first identify which signals control which links. Subsequently, one can identify the cycle length, phase split and offset of each traffic signal.

Table 11 provides a sample traffic signal timing file with the description of the data fields contained within the file provided in Table 10. All plan durations, as well as the durations of each element of each signal-timing plan, are expressed in seconds, where any inputs are truncated to the nearest deci-second. Specifically, the second line in the program permits the user to indicate how many different fixed-time signal plans are to be considered, and what the duration of each such timing plan is in seconds.

Typically, several time-of-day signal timing plans are identified in File 3, where each such plan is implemented for a given user-specified period of time. If the duration of the simulation exceeds the end time of the last signal-timing plan that is coded, the last timing plan will be retained.

Within each signal timing plan, the signal timings are specified for each signal in terms of the cycle length, the offset of the start of phase 1, and the number of phases within the signal cycle. The durations of the green intervals and the lost times associated with each phase are also provided. The model structure allows up to eight separate phases to be identified for each traffic signal.

Although the sample signal file specifies a single timing plan for the entire simulation, with all signals having the same cycle length, there is no requirement within the model to retain a constant cycle length for all signals within a given time period's plan. There is also no need to have the same signal maintain a constant cycle length from one control time period to the next. Consequently, during a given control time period or timing plan, the timings of all signals can be coordinated using a common cycle length, or one or more signals can be operated in isolation using one or more different cycle lengths. The choice between these alternatives, or any combinations thereof, need not be stated explicitly. Instead, it is implicit in the signal timing plan specification.

Automatic Internal Signal Re-timing

When the automatic cycle and phase split optimization option is utilized, only the offsets and the lost times specified in the original signal timing plan file are kept constant. The other timing plan parameters are optimized every x seconds, where x is equal to the value specified in the last field, which must be a multiple of 60 seconds. The procedure utilized for the signal timing plan optimization allocates green time on the basis of the approach's volume/saturation flow ratios, according to the procedures specified in the Canadian Capacity Guide and the Highway Capacity Manual. Approach volumes are computed in this case as the running exponential average of the traffic flow entering each approach leg.

While signal coordination is always *simulated* within INTEGRATION, signal coordination offset *optimization* is only considered as an option in the standard version of INTEGRATION. Without invoking this option, signal coordination settings are held constant, while each signal is being optimized as being isolated. Therefore, if signals in a network are not truly isolated, it may be best to specify a time-series of fixed-time plans that have been optimized externally, rather than to permit internal optimization.

Not all signals are required to participate in an automatic signal re-timing. For example, some signals may be optimized, while other signals maintain their timings at constant settings throughout the simulation period. Furthermore, not all traffic signals that are optimized need to be optimized at the same frequency.

Modeling of Alternate Signal Timing Strategies

Simple fixed-time control can be modeled within INTEGRATION by providing a single signal-timing plan for the entire simulation period. Time-of-day fixed-time control can be modeled by providing several different signal timing plans, each plan being in effect for the same user-specified duration. A model user could then implement a different plan every hour, every 15 minutes (900 seconds), or at any other desired time interval that is a multiple of 60 seconds. A strategy where plan *A* is to be in effect for 1 hour, plan *B* is in effect for 15 minutes, and plan *C* is in effect for the remaining 45 minutes, would need to be coded as 8 consecutive timing plans. The first 4 plans would be identical and correspond to plan *A*, while the 5th plan would be plan *B*. The final 3 plans, namely 6, 7 and 8 would all correspond to plan *C*. Fixed-time time-of-day control plans for ramp meters can be modeled in essentially the same manner.

Independent of the type of control, there is no provision at present within the INTEGRATION model for an offset transition algorithm between either consecutive externally specified plans, or between consecutive internally generated plans. The impact of this lack of an offset transition algorithm becomes acute when frequent plan changes are invoked in rapid succession.

Constrained Real-time Control

A mixture of the above schemes could be implemented by providing a sequence of signal timing plans, where each new plan would provide different minimum and/or maximum cycle length constraints. Alternatively, a real-time optimization of only phase splits could be performed by requesting real-time optimization, but specifying a maximum cycle length that is identical to the minimum cycle length. In this case the offsets of the first phases in each signal timing plan would be maintained at the values that were first coded.

While multiple signal timing plans may be used, the number of phases at a given signal must remain constant for all time periods. This requirement exists because, while File 3 is dynamic, File 2 is static throughout the entire simulation period. Consequently, a link will automatically always be controlled by the same signal and by the same phase numbers.

Fixed-time Ramp Meters

Fixed-time ramp meters are currently modeled in INTEGRATION as traffic signals. The cycle length should be based on the desired ramp metering rate, while the green time should reflect the number of vehicles to be discharged per cycle and the saturation flow rate. A second signal phase must be specified in File 3, but no links in File 2 should be coded to discharge during this second phase. Lost times may be set to zero.

Internal real-time ramp metering control is available as an option in this version of INTEGRATION, but its features are not detailed here. Any signals, which act as ramp meters, should in this case, not be coded as being optimized (i.e. the last field should be equal to zero). However, time-of-day fixed-time ramp metering plans can be specified in the same manner as for arterial traffic signals.

Table 11: Sample Signal Timing Plan File (File 3)

```
QNET Signal File
   6    1  3600
   1
   1   60   30    70    0   2    26   4  26   4   0
   2   60   30    70   10   2    26   4  26   4   0
   3   60   30    70   20   2    26   4  26   4   0
   4   60   30    70   30   2    26   4  26   4   0
   5   60   30    70    0   2    26   4  26   4   300
   6   60   30    70    0   2    26   4  26   4   300
```

Table 12: Description of fields in Signal Timing Plan File (File 3)

Line	Field	Description
1	1	File title (maximum 300 characters)
2	1	Number of traffic signals within the traffic network [integer]. ($0 \le x \le$ *max. number of signals permitted*)
	2	Number of traffic signal plans provided in this file for each signal [integer]. ($0 \le x \le 1000$)
	3	Duration of each signal timing plan (seconds) [integer]. ($x \ge 60$, *where x is a multiple of 60*)
3	1	Signal plan number [integer]. Signal plans must be numbered sequentially in increasing order. ($1 \le x \le$ *number of traffic signal plans specified*)
4+	1	The traffic signal number for which the remaining fields on this line apply [real] ($1 \le x \le$ *max. signal number*) Decimal portion indicates the vehicle classes that receive actuation (x.abcde). Where the five digit number *abcde* specifies the vehicle actuation flag (0 no actuation; 1 actuation), where: *a*: vehicle class 1 actuation flag [binary] *b*: vehicle class 2 actuation flag [binary] *c*: vehicle class 3 actuation flag [binary] *d*: vehicle class 4 actuation flag [binary] *e*: vehicle class 5 actuation flag [binary] Example 5.01001 results in traffic signal 5 being actuated by vehicle classes 2 and 5.
	2	Initial cycle length (seconds) that will be utilized in the simulation [real]
	3	Minimum cycle length (seconds) that will be permitted in the event that signal timing optimization is performed [real]. ($0.0 \le x \le$ *max. cycle length*)
	4	Maximum cycle length (seconds) that will be permitted in the event that signal timing optimization is performed [real]. ($x \ge$ *min. cycle length)*
	5	Time in seconds, after the start of the current time plan, that the first phase of this signal begins. This time in normally referred to as the signal offset [integer]. ($0 \le x \le$ *cycle length*)
	6	Number of phases at this traffic signal [integer]. ($2 \le x \le 8$)
	For each of the phases specified in Field 6 the following two fields must be provided	
		Displayed green time (seconds) of this phase [real]. ($1.0 \le x <$ *cycle time specified*)
		Displayed inter-green time (seconds) of this phase [real]. ($0.0 \le x \le 10.0$)
	last	Frequency (seconds) at which the phase splits and cycle lengths of this signal will be optimized [integer]. ($x \ge 0$, *where x is a multiple of 60*)

3.8 File 4: Origin-Destination Traffic Demands

Traffic demands are specified to the INTEGRATION model as a time series of aggregate origin-destination departure rates. During the simulation, the requested time series of departures is decomposed into a time series of individual vehicle departures, where the timing of these individual departures is set to be the closest discrete equivalent to the continuous input.

A subset of a sample O-D file is presented in Table 13 for illustrative purposes together with a description of the various fields in Table 14.

O-D Number

Each O-D demand specified in the file must be assigned a unique identification number. Usually, these numbers are continuous and sequential. However, this convention, while convenient, is not strictly required.

Demand Rates

Traffic demands are specified to the model in terms of O-D traffic departure rates (Field 4) between a specific set of origin-destination zones (Fields 2 and 3). In addition, the time period for which these rates are assumed to prevail must be indicated (Fields 6 and 7). For example, the demand at a rate of 2500 veh/h for the first O-D pair is coded to depart from time 0 to time 1800 seconds, which results in a total of 1250 vehicles (= $(1800\text{-}0)/3600\times2500$).

Several O-D rates can be specified for the same origin-destination pair within either unique or overlapping time periods. Any such vehicle departures are treated as being cumulative. An O-D rate of 0.0 is permitted but does not result in the creation of any vehicles.

If a vehicle departure rate results in a fractional number of vehicles needing to be generated within the prescribed time interval (e.g. a demand of 250.4), then there exists a probability of 40 percent, that 251 vehicles will be generated, and a probability of 60 percent that only 250 vehicles will be generated.

Vehicle Headways

The user may specify the degree of randomness in the vehicle departure headways. When completely random headways are requested, they will follow a negative exponential distribution. Less randomness is obtained by utilizing a shifted negative exponential distribution. The above randomness is only imparted into the traffic stream by modifying the inter-vehicle trip departure headways. However, once the vehicles enter the network, subsequent en-route headways become modified as a function of vehicle speeds, lane-changing activities, merges and diverges, and traffic controls.

The randomness value that is specified indicates the fraction of headway that will be random. For example, if a value of 0.6 is entered in Field 6, then the headway of a vehicle will consist of a constant component, equal to 40% of the average headway (derived from the departure rate), plus an exponential component with a mean of 60 percent of the average headway.

Traffic Stream Composition

The user may specify the driver class composition of the associated demand by indicating the fraction of the demand that is to be of each of the five possible vehicle classes. Upon generation, vehicles are randomly assigned a vehicle class using the probabilities indicated in these 5 fields. As a result of this stochastic element, the actual fraction of vehicles of each class that are generated may vary somewhat about the fraction specified in the control file. In addition to variance due to the stochastic vehicle class assignment process, variance may also result from the fact that only an integer number of vehicles can be generated. Thus, it is impossible to have 20 percent of the vehicles be of each of the 5 classes if, say, 4 or 7 trips are to be generated. In this case, each of the 4 or 7 vehicles would have a 20 percent chance of being assigned a specific vehicle class.

Vehicle Probes

Field 13 specifies the fraction of the demand that will be considered to act as vehicle probes. In addition, Field 14 indicates the applicable passenger car unit vehicle equivalency factor that can be added to represent the capacity impact of non-standard vehicles.

The effect of Field 14 is to scale the following headway by the specified amount. An equivalency of 2.0 will therefore cause the vehicle that follows this vehicle to leave twice the normal headway. Such vehicle equivalencies must be greater than 50 percent of a standard passenger car and less than 500 percent of the standard.

Table 13: Subset Sample O-D Traffic Demand File (File 4)

```
QNET O-D Demand File
46   0   0   1.0
 1  10   5 2500   1.0    0  1800    1.0  0.0  0.0  0.0  0.0  0.0  1.0
 2   5  10 2500   1.0    0  1800    1.0  0.0  0.0  0.0  0.0  0.0  1.0
 3   4   1  100   1.0    0  1800    1.0  0.0  0.0  0.0  0.0  0.0  1.0
 4   4   2  100   1.0    0  1800    1.0  0.0  0.0  0.0  0.0  0.0  1.0
 5   4   3  100   1.0    0  1800    1.0  0.0  0.0  0.0  0.0  0.0  1.0
 7   4   5  100   1.0    0  1800    1.0  0.0  0.0  0.0  0.0  0.0  1.0
 8   4   6  100   1.0    0  1800    1.0  0.0  0.0  0.0  0.0  0.0  1.0
 9   4   7  100   1.0    0  1800    1.0  0.0  0.0  0.0  0.0  0.0  1.0
10   4   8  100   1.0    0  1800    1.0  0.0  0.0  0.0  0.0  0.0  1.0
```

Table 14: Description of fields in O-D Traffic Demand File (File 4)

Line	Field	Description
1	1	File title (maximum 300 characters)
2	1	Number of O-D demands to be read [integer]. ($0 \le x \le$ *max. number of O-D pairs*)
	2	The number of the first O-D pair loading to be utilized by the model [integer]. If $x = 0$ then x is set to be 1. ($0 \le x \le$ *max. O-D number specified*)
	3	The number of the last O-D pair loading to be utilized by the model [integer] ($0 \le x \le$ *max. O-D number specified*) If $x = 0$ then x is set to be *max. O-D number specified*.
	4	Global O-D demand scaling factor [real]. ($x > 0.0$) For example, if x equals 2.0 then all listed departure rates are assumed to be double their listed values.
3+	1	O-D line identification number [integer]. ($1 \le x \le$ *number of O-D pairs specified*)
	2	Origin zone identification number [integer]. ($1 \le x \le$ *max. zone number*)
	3	Destination zone identification number [integer]. ($1 \le x \le$ *max. zone number*)
	4	Departure rate (vph) for given O-D [real].
	5	The fraction of the vehicle headway that is random [real]. ($0.0 \le x \le 1.0$)
	6	Time (seconds) at which specified O-D demand begins [integer]. ($x \ge 0$)
	7	Time (seconds) at which specified O-D demand ends [integer]. ($0 \le x \ge$ start of O-D demand)
	8-12	The fraction of the demand that is to be of driver class i [real]. ($0.0 \le x_i \le 1.0$) $\sum_{i=1}^{5} x_i = 1.0$
	13	The fraction of the total demand that will act as vehicle probes for the purposes of generating output for optional Output Files 15 and/or 16 [real]. ($x \le 1.0$) Recommend an initial value of $x = 0.0$.
	14	The passenger car length equivalency for all vehicles of this O-D demand [real] ($0.5 \le x \le 10.0$)

3.8 File 5: Incidents or Lane Blockages

Incidents are modeled as temporal reductions in capacity, where such capacity reductions are specified as an effective number of lanes that is lost.

General Characteristics

Table 15 provides a sample incident characteristic file, while Table 16 provides a brief description of the fields that comprise the default incident data file. The default incident data file indicates the incident number (Field 1), the link impacted (Field 2), the number of lanes affected (Field 3), the start time (Field 4), and the end time (Field 5). Several incidents are allowed for consideration at the same time on different links, or at different times on the same link.

DDWhen an incident takes place, it is modeled as occurring at the link's terminal end. The reduction in capacity is calculated in view of the effective number of lanes of traffic that are expected to be lost. This number can be an integer value, to reflect a complete lane blockage, or a fraction, in order to indicate a partial lane blockage. The latter may be due, for example, to a vehicle parked on the shoulder on a freeway link.

It is possible to have multiple simultaneous incidents on the network on different links. It is also possible to have consecutive incidents on the same link. However, multiple incidents should not be coded to occur on the same link either at the same time, or during overlapping time periods. In this case, only the most severe capacity reduction will be considered to be in effect, and no cumulative effects will be modeled.

The type of diversion, which occurs due to an incident, is a function of the mix of driver classes, the travel times on any competing routes, and the level of access that drivers have to travel time information updates. There is, therefore no explicit diversion in response to the occurrence of an incident. Instead, there is only an implicit one, when incidents become the cause of increases in travel time.

The `QNET5.DAT` example file describes an incident that blocks 1.5 lanes for 5 minutes (900 seconds) on link 25, starting 10 minutes (600 seconds) into the simulation. Subsequently, 15 minutes (900 seconds) into the simulation, the vehicle is considered moved to the shoulder, where it blocks 0.75 lanes, and remains for another 10 minutes (from time 900 seconds to time 1500 seconds).

Table 15: Sample Incident Descriptor File (File 5)

```
Incident Data File - Dec. 1995
 2
 1   25   1.5    600    900
 2   25   0.75   900   1500
```

Table 16: Description of the fields in Incident File (File 5)

Line	Field	Description
1	1	File title (maximum 300 characters)
2	1	Number of incident descriptor records to be read [integer]. (0 $\leq x \leq$ *max. number of incidents permitted*)
3+	1	Incident record number [integer]. (1 $\leq x \leq$ *number of incident descriptor records specified*)
	2	Identification number of the link in which the incident will occur [integer]. (1 $\leq x \leq$ *max. link number*) Note that the incident is assumed to occur at the downstream end of this link and occupy a length of 250 m.
	3	Effective number of lanes blocked by the incident [real] (0.0 $\leq x \leq$ *number of lanes existing on the link*)
	4	Simulation time (seconds) at which incident is to begin [integer]
	5	Simulation time (seconds) at which the incident is to end [integer]

Chapter 4: Fundamental Model Outputs

The INTEGRATION model provides four types of fundamental simulation outputs for interpretation by the model user, namely:

1. On-Screen Graphics/Text (which are Displayed on the Video Monitor)
2. Simulation Run time Errors (which are logged in the `RUNERR.OUT` file)
3. Concise Summary Statistics (which are logged in the Summary File)
4. Labeled Output Statistics (which are provided in File 10)

These outputs, their definitions, and their purpose are discussed in this chapter of the manual. It should be noted that the additional outputs provided by the model are described in Volume II of the manual.

4.1 On-Screen Graphics/Text (Displayed on Video Monitor)

The Windows version of the INTEGRATION model provides on-screen graphics that continuously reflect the current network status. This output is highly valuable to ensuring the simulation is executing as intended. The user should also check the screen output to ensure the network is configured properly.

In a large network it may be desirable to code a network in stages, ensuring that after each stage the network operates and displays properly. The zoom feature, in particular, may be very useful to confirm the correct network operation. The location, severity and duration of congestion usually can be more quickly determined visually from the screen than quantitatively from the numeric output files. Finally, the cause of any excessive queuing, such as poorly timed signals, an incorrect network configuration, or inappropriate link/lane prohibitions, may be identified more readily visually from the model's animation than from tabular output statistics.

The graphics screen consists of a number of different windows that can be accessed by running the model. A number of key features of the on-screen display are described in the following sections.

Vehicles

The movements of vehicles traversing the network are displayed by default in main graphics window in one of four possible colors. Drivers traveling at near free-speed conditions, are illustrated as green vehicles on the screen. The color of these boxes changes to blue when the speed drops to approach the link's speed-at-capacity. Once the speed of a vehicle is below the speed at capacity, the color of the vehicle becomes yellow. Finally, if the vehicle slows down further, the vehicle eventually becomes red. Stopped vehicles should always appear on the screen as being red.

The exact transitions in color are defined by the half way points between the free-speed and the speed-at-capacity (for a transition from green to blue), and by half the speed-at-capacity (for the transition from yellow to red). For example, vehicles on a link with a free-speed of 100 km/h and a speed-at-capacity of 60 km/h would be green when traveling from 100 to 80 km/h and be blue for a speed from 80 to 60 km/h. They would become yellow when traveling from 60 to 30 km/h, and be red for speeds of less than 30 km/h.

Traffic Controls

Traffic signals are indicated as being either in green, amber or red mode. Separate arrows are provided to show each turning movement. Stop signs are illustrated as red octagons, while yield signs are shown as red triangles. Ramp meters are indicated as solid traffic signals. The status of traffic signals is updated each deci-second.

Incidents

Any incidents are shown as dashed yellow lines in the affected lanes. The occurrence and location of any incidents is only updated on the screen on a minute-by-minute basis.

Departure/ Arrival Statistics

A floating window indicates the number of vehicles that are scheduled to have started their trips. It also indicates the number of vehicles that are currently en-route, and the number that already have arrived at their final destinations. At the end of the simulation period all vehicles should have arrived. If this is not the case, the simulation time horizon may need to be increased to provide complete simulation run statistics on all completed trips.

Screen Zooming & Pausing

During the course of the simulation the model can be paused at any time by pressing any key or clicking the mouse on any part of the screen. This will cause a graphical user interface menu to pop up. This menu will allow the user to zoom and pan the network view, and to obtain a number of statistics describing the network, the links, the controls, and the vehicles on them.

4.2 Simulation Run Error File (`RUNERR.OUT`)

Whenever the INTEGRATION model is initiated, the `RUNERR.OUT` file is automatically created in the current subdirectory. This serves three main functions as described below:

Network Size Constraints

First, the file lists the network size constraints that have been built into the particular version of the model that has been run. This includes the array dimensioning limits for the maximum node number, the maximum link number, as demonstrated in Table 17. This list will assist the model user in determining if the particular model version will be suitable for running the size of simulation run that is desired. In view of the increased ability to access more memory, differences in these constraints are the main distinguishing feature between the various model versions.

Listing of Detected Errors

Second, the `RUNERR.OUT` file provides a listing of any errors that are detected by the model during the course of initiating or running the model. These errors may either be detected during the course of the input data processing and/or during the actual execution of the simulation portion of the model. The error descriptions, while brief, often provide an adequate guide to assist in correcting the error. In addition, error codes usually provide a direct mechanism for model developers to identify precisely the source and the nature of the problem. Some of the errors that are trapped are deemed FATAL, which will cause the model logic to terminate the program. Other errors are simply listed as warnings and allow the model to continue execution.

Execution Time Statistics

Third, the `RUNERR.OUT` file provides a number of statistics related to the execution time of the model as a whole and some of its component subroutines. The first of these outputs is the production of a statement that indicates every 600 seconds or 10 minutes of simulated time, how long it took the computer to simulate this series of events. This tabulation permits an assessment of how the simulation to clock time ratio varies as a function of network loading throughout the simulation run. In addition, a second tabulation is provided which provides a summary of the percentage breakdown of the total execution time between the various model subroutines. These data are primarily important to the model developer, and the details of the codes are not provided here.

Table 17: Summary Run Error File

```
     ==================================================================
         INTEGRATION Release 2.30d: TRAFFIC NETWORK SIMULATION MODEL

                   Copyright 1984-2001  M. Van Aerde & Assoc., Ltd.

                         Small      Version - Feb.  2001
     ==================================================================

A. Array dimensions :

   - max number of od pairs           =       2500
   - max number of vehicles           =      25000
   - max number of vehicle types      =          5
   - max number of links              =        250
   - max node number                  =        250
   - max links into/out of node       =          7
   - max number of vehicles on network=      50000
   - max zone number                  =         25
   - max number of future time steps  =        120
   - max signal number                =         25
   - max number of phases per signal  =          8
   - max incident number              =          5
   - max number of files              =         50
   - max number random number seeds   =          6
   - max number equilibrium paths vt1 =         25
   - max number of forward tree nodes =          1
   - max number of forward minutes    =         15
   - max number of macro zone clusters=         25
   - max number of network lanes      =       1750
   - max veh concurrent on network    =       5000
   - max detector station number      =        250


    - Opening Master file: qnet.int
    - Master file title: QNET master file

    - Simulation Time (sec):     2700
    - Output Rate 1 (sec)  :      900
    - Output Rate 2 (sec)  :      300
    - Output Rate 3 (sec)  :        0
    - Output Rate 4 (sec)  :        0

    - Master File Format   :    0.000000


    - Input  Subdirectory  : qnet\
    - Output Subdirectory  : qnet\output\

    - Summary output   : summary.out
```

4.3 Summary Output File (SUMMARY.OUT)

The fourth output provided by the INTEGRATION model is a summary of a number of important aggregate network level measures of performance. This file is automatically created whenever a simulation run is completed.

The first group of lines of the `SUMMARY.OUT` file provide totals for all vehicles of class *i* combined. The following group of lines provide average measures of performance on the basis of the total divided by the total number of vehicles of type *i*. The first 5 columns following the MOP identification number, correspond to the 5 vehicle classes. The last column provides cumulative totals across all vehicle classes.

It is important at this point to provide a description of how these variables are computed. This section provides a brief description of the output statistics.

Vehicle & Person Trips

The vehicle trips are computed as the total number of vehicles that complete their trips within the simulation period. Consequently, vehicles en-route at the conclusion of the simulation are not accounted for in estimating total vehicle trips. The person trips are similar to the vehicle trips except that each vehicle trip is weighted by the number of passengers in the vehicle.

Vehicle & Person Kilometers

The total vehicle kilometers is computed as the sum of the trip lengths for all vehicle trips that are completed within the simulation period. Again, any vehicles that do not reach their destination prior to the conclusion of the simulation are not included in the estimation of the total vehicle kilometers traveled. The total person kilometers traveled is computed by weighting each vehicle trip length by the number of passengers in the vehicle and summing up across all trips completed within the simulation period. The average vehicle trip length and person trip length is computed by dividing the total vehicle and person trip length by the total number of vehicle and passenger trips, respectively.

Vehicle Stops

The total vehicle stops are computed as the summation of the number of vehicle stops per trip across all the vehicle trips that were completed within the simulation period. The computation of vehicle stops is based on Equation 2, as was described earlier. The average vehicle stops are computed as the total vehicle stops divided by the total vehicle trips.

Vehicle & Passenger Travel Time

Total vehicle travel time is computed as the summation of trip times across all vehicle trips completed within the simulation period. Alternatively, the total passenger trip time is computed as the sum of the passenger weighted trip times across all vehicle trips completed within the simulation period. The average vehicle and passenger trip times are computed as the total vehicle and passenger trip times divided by the total vehicle and person trips, respectively.

Total, Stopped, Deceleration & Acceleration Delay

The total delay is computed as the sum of delay across all vehicle trips that were completed within the simulation period. The estimation of vehicle delay is computed each deci-second based on Equation 1, as was described earlier. The stopped delay is computed for all instances that a vehicle speed is less than or equal to 1 km/h. Alternatively, the deceleration delay is computed for all instances that the vehicle is decelerating while the acceleration delay is computed for all instances that the vehicle is accelerating.

Acceleration Noise

The acceleration noise is a less common variable that was developed in the late 1950's in order to quantify the smoothness of flow in a traffic stream. Specifically, Drew (1968) mentions, "the term *noise* is used to indicate the disturbance of the flow, comparable to the coined phrase *video noise*, which is used to describe the fluttering of the video signal on a television set." Drew mentions that acceleration noise received considerable attention as a possible measurement of traffic flow quality for two basic reasons. First, it is dependent on the three basic elements of the traffic stream, namely, (1) the driver, (2) the road, and (3) the traffic condition. Second, it is in effect, a measurement of the smoothness of flow in a traffic stream. Specifically, the acceleration noise (standard deviation of accelerations) can be considered as the disturbance of the vehicle's speed from a uniform speed.

The acceleration noise that is present on a road in the absence of traffic is termed the *natural noise* of the driver on the road (Drew, 1968). Several factors affect acceleration noise, such as the roadway geometry, the type of control on the roadway, and the level of congestion on the roadway. Specifically, a field study in the mid 1950's indicated that the acceleration noise increased with an increase in congestion (Jones and Potts, 1955).

Jones and Potts (1955) developed a mathematical equation for approximating the acceleration noise. Specifically, using an acceleration profile Jones and Potts computed the average acceleration and the acceleration noise as the standard deviation of the acceleration noise. The details of the derivation are beyond the scope of this User's Guide, however, it is worthwhile mentioning that the formulation only computes the acceleration noise when the vehicle is in motion (speed is greater than zero). A modified acceleration noise formulation is considered in the INTEGRATION model. The first modification is that for long trips (*T* large) the average acceleration that is computed typically tends to zero. Consequently, it is assumed that the average acceleration is zero. The second modification to the Jones and Potts formulation is that each acceleration observation is weighted by the vehicle speed because acceleration levels at higher speeds result in higher fuel consumption and emission estimates than equivalent acceleration levels at lower speeds.

It should be noted that Drew (1968) demonstrated that the kinetic energy of a traffic stream can be computed using Equation 4 where β is a unitless constant. Furthermore, Drew demonstrated that there is an internal energy or lost energy associated with the traffic stream, which manifests itself in erratic motion and is nothing but the acceleration noise. Consequently, Capelle (1966) hypothesized that the internal energy or acceleration noise measured over a segment of roadway is equal to the total fuel consumed.

$$E_k = \beta \sum_{i=1}^{n} u(t_i)^2 \qquad [4]$$

Fuel Consumption and Emissions

As was mentioned earlier the model estimates hot-stabilized vehicle fuel consumption and emissions using instantaneous models that were developed using field data collected at the Oak Ridge National Laboratory. While a more detailed description of these models is provided in Volume III of the User's Guide, it is important to note at this point that the model only estimates tail-pipe emissions of hydrocarbons, carbon monoxide, and oxides of nitrogen for gasoline vehicles under hot-stabilized conditions.

Crash Rate & Severity

The model estimates the number of vehicle crashes using crash rate models that were developed using the General Estimates System (GES) US national crash database. Using the database crash rates were estimated as a function of a roadway speed-limit. These crash rates are utilized within the model to estimate a network-wide crash risk that is computed as the summation of the crash risk of each individual vehicle trip. In addition, the severity and injuries involved in the crash are computed based on US national crash rates. The details of the crash model development are presented in Volume III of the User's Guide.

Table 18: Sample Summary Output File

```
QNET master file
 Total Statistics:
  1        4700      0     0     0     0      4700     - vehicle trips
  2        4700      0     0     0     0      4700     - person trips
  3       15039      0     0     0     0     15039     - vehicle-km
  4       15039      0     0     0     0     15039     - person-km
  5        6921      0     0     0     0      6921     - vehicle-stops
  6      810348      0     0     0     0    810348     - vehicle-secs
  7      810348      0     0     0     0    810348     - person-secs
  8      186755      0     0     0     0    186755     - total delay
  9       66527      0     0     0     0     66527     - stopped delay
 10      120227      0     0     0     0    120227     - accel/decel delay
 11   157669024      0     0     0     0 157669024     - accel-noise
 12        1700      0     0     0     0      1700     - fuel (l)
 13        4150      0     0     0     0      4150     - HC (g)
 14       90009      0     0     0     0     90009     - CO (g)
 15        5411      0     0     0     0      5411     - NOx (g)
 16       26816      0     0     0     0     26816     - crashes*10e-6
 17       12650      0     0     0     0     12650     - injury crashes
 18         131      0     0     0     0       131     - fatal crashes
 19        1323      0     0     0     0      1323     - no damage
 20       12107      0     0     0     0     12107     - minor damage
 21        7852      0     0     0     0      7852     - moderate damage
 22           0      0     0     0     0         0     - dollars of toll

 Average Statistics:
  1       1.000  0.000 0.000 0.000 0.000       1.000   - vehicle trips
  2       1.000  0.000 0.000 0.000 0.000       1.000   - person trips
  3       3.200  0.000 0.000 0.000 0.000       3.200   - vehicle-km
  4       3.200  0.000 0.000 0.000 0.000       3.200   - person-km
  5       1.473  0.000 0.000 0.000 0.000       1.473   - vehicle-stops
  6     172.415  0.000 0.000 0.000 0.000     172.415   - vehicle-secs
  7     172.415  0.000 0.000 0.000 0.000     172.415   - person-secs
  8      39.735  0.000 0.000 0.000 0.000      39.735   - total delay
  9      14.155  0.000 0.000 0.000 0.000      14.155   - stopped delay
 10      25.580  0.000 0.000 0.000 0.000      25.580   - accel/decel delay
 11   33546.602  0.000 0.000 0.000 0.000   33546.602   - accel-noise
 12       0.362  0.000 0.000 0.000 0.000       0.362   - fuel (l)
 13       0.883  0.000 0.000 0.000 0.000       0.883   - HC (g)
 14      19.151  0.000 0.000 0.000 0.000      19.151   - CO (g)
 15       1.151  0.000 0.000 0.000 0.000       1.151   - NOx (g)
 16       5.706  0.000 0.000 0.000 0.000       5.706   - crashes*10e-6
 17       2.692  0.000 0.000 0.000 0.000       2.692   - injury crashes
 18       0.028  0.000 0.000 0.000 0.000       0.028   - fatal crashes
 19       0.282  0.000 0.000 0.000 0.000       0.282   - no damage
 20       2.576  0.000 0.000 0.000 0.000       2.576   - minor damage
 21       1.671  0.000 0.000 0.000 0.000       1.671   - moderate damage
 22       0.000  0.000 0.000 0.000 0.000       0.000   - dollars of toll
```

4.4 Labeled Output Statistics (File 10)

Labeled output statistics are generated in File 10. This output is ideal when INTEGRATION output is required for simple visual inspection. However, these data are not well suited for subsequent direct input into a spreadsheet, or subsequent re-use as a program input. Output Files 11 through 19, as described in Volume II, are better suited for this purpose.

File 10 contains 8 types of output that are described below in the order in which they appear in File 10.

Data Input File Echo

The initial output of the INTEGRATION provides an echo of the data input into the model. The echo indicates which file names have been specified in the master file for both the input and the output data files.

Analysis of Input Data

The next section provides an analysis of each of the input data files, as it is read in. This information is provided for input data verification and diagnostics, and will subsequently clearly identify the model run.

Decomposing O-Ds

The next section provides information describing the number of vehicles that were generated for each O-D demand. It also lists the total number of individual vehicles that were represented by the aggregate demands specified in File 4.

Signal Timing Plan Summary

Interspersed between the link-oriented statistics are listings of the optimized signal timing settings for each signal in the network. These signal optimizations occur every x seconds, where x is the value specified by the user in the last field for each signal plan entry in File 5. If signals are not optimized no signal plans are output. It should be noted that the signal timings that are printed do not include changes in the signal timings that result from signal actuation or from transit priority.

System Oriented Link Statistics

File 10 provides statistics for each link in the network describing its current status. This table is generated at the frequency specified in Field 2 of Line 2 of the Master Control File. The output file indicates the flows, total travel time, average travel time, volume/capacity ratio, number of stops, and number of vehicles that are currently on each link.

Summary Statistics of all Trips

At the end of the simulation run, two further summary tables are generated. The first one summarizes the number of vehicles that completed their journey (as specified by O-D pair), the average journey time per arrival and the accumulation of the total trip times for all vehicles. A summary of the vehicle demand on the network, the number of vehicles that entered and left the network, and the number of vehicles left on the network after the simulation horizon is completed is also provided. Furthermore, such statistics are generated not only for all driver classes combined, but also for each driver class by itself.

The next section illustrates average and total O-D statistics for each vehicle class on its own. This output is followed by a table that illustrates average and total O-D statistics for all vehicle classes. The next section provides aggregate network statistics for all vehicle classes combined.

Incident Summary

The final entry, into the `QNET10.OUT` file, is a summary of any incidents that occurred in the network. This entry lists the location, severity, start time and duration of each incidents.

SELECTED REFERENCES

Ahn K., Rakha H., Trani A., and Van Aerde M. (2001), *Estimating Vehicle Fuel Consumption and Emissions Based on Instantaneous Speed and Acceleration Levels*, Accepted for publication in the ASCE Journal of Transportation Engineering.

Capelle D.G. (1966), "*An Investigation of Acceleration Noise as a Measure of Freeway Level of Service*", Doctoral dissertation, Texas A&M University, College Station.

Drew D. (1968), "*Traffic Flow Theory and Control*", McGraw Hill, 68-13626.

EPA (1993). *User's Guide to MOBILE5A: Mobile Source Emissions Factor Model.* U.S. Environmental Protection Agency, Ann Arbor, MI.

Gardes, Y., A.D. May, and M. Van Aerde (1993). *Evaluation of IVHS Strategies on the Smart Corridor*, ITE Annual Meeting, The Hague, The Netherlands.

Hellinga, B., and M. Van Aerde (1994). *An Overview of a Simulation Study of the Highway 401 Freeway Traffic Management System*. Canadian Journal of Civil Engineering, Vol. 21, pp. 439-454.

ITE (1984). *Canadian Capacity Guide for Signalized Intersections*. First Edition, S. Teply Editor, Institute of Transportation Engineers, District 7, Canada.

ITE (1995). *Canadian Capacity Guide for Signalized Intersections*. Second Edition, S. Teply Editor, Institute of Transportation Engineers, District 7, Canada.

Jones T. and Potts R. (1955), "*The Measurement of Acceleration Noise: A Traffic Parameter*", Operations Research, Vol. 10, pp. 755-763.

Rakha, H., M. Van Aerde, E.R. Case, and A. Ugge (1989). *Evaluating the Benefits and Interactions of Route Guidance and Traffic Control Strategies Using Simulation*. Proceedings of the Vehicle Navigation and Information Systems (VNIS) Conference, IEEE, New York, NY, pp. 296-303.

Rakha, H., M. Van Aerde, and J. Wang (1993). *A Comparison of Two Alternative Traffic Simulation Models: TRANSYT vs. INTEGRATION*. Proceedings of the Canadian Institute of Transportation Engineers Annual Meeting, Edmonton, Alberta.

Rakha H., Van Aerde M., Ahn K. and Trani A. (2000), *Requirements for Evaluation of Environmental Impacts of Intelligent Transportation Systems using Speed and Acceleration Data*, Transportation Research Record, No. 1738, pp. 56-67.

Rakha H., Lucic I., Demarchi S., Setti J., and Van Aerde M. (2001a), *Vehicle Dynamics Model for Predicting Maximum Truck Accelerations*, Accepted for publication in the September issue of the ASCE Journal of Transportation Engineering.

Rakha H., Kang Y., and Dion F. (2001b), "Estimating Vehicle Stops at Under-Saturated and Over-Saturated Fixed-Time Signalized Intersections," Presented at the Transportation Research Board 80th Annual Meeting, January, [Paper # 01-2353].

Rilett, L. R., and M.W. Van Aerde (1991a). *Modeling Distributed Real-Time Route Guidance Strategies in a Traffic Network that Exhibits the Braess Paradox*. Proceedings of the 1991 Vehicle Navigation and Information Systems (VNIS) Conference, Dearborn, MI, SAE Conference Proceedings No. 253, Part 2, Society of Automotive Engineers, Warrendale, PA, pp. 577-587.

Rilett, L., and M. Van Aerde (1991b). *Routing Based On Anticipated Travel Times*. Applications of Advanced Technologies in Transportation Engineering, Proceedings of the Second International Conference, American Society for Civil Engineering, pp. 183-187.

Rilett, L. R., M. Van Aerde, G. MacKinnon, and M. Krage (1991). *Simulating the TravTek Route Guidance Logic using the INTEGRATION Traffic Model*. Proceedings of the 1991 Vehicle Navigation and Information Systems (VNIS) Conference, Dearborn, MI, SAE Conference Proceedings No. 253, Part 2, Society of Automotive Engineers, Warrendale, PA, pp. 775-787.

Robinson, M., and M. Van Aerde (1995). *Examining the Delay, Safety Impacts, and Environmental Impacts of Toll Plaza Configurations*. Proceedings of the 6th Vehicle Navigation and Information System Conference (VNIS), Seattle, WA, published by IEEE, Piscataway, NJ, pp.259-266.

Stewart, J., M. Baker, and M. Van Aerde (1996). *Analysis of Weaving Section Designs using INTEGRATION*. Presented at the 75th Annual Meeting of the Transportation Research Board, Washington, D.C.

Van Aerde, M. (1995). *A Single Regime Speed-Flow-Density Relationship for Congested and Uncongested Highways*. Presented at the 74th Annual Meeting of the Transportation Research Board, Washington, D.C.

Van Aerde, M. (1985). *Modeling of Traffic Flows, Assignment and Queuing in Integrated Freeway/Traffic Signal Networks*. PhD. Thesis, Department of Civil Engineering, University of Waterloo, Waterloo, Canada.

Van Aerde, M., and M. Baker (1993). *Modeling Fuel Consumption and Vehicle Emissions for the TravTek System*. Proceedings of the IEE-IEEE Vehicle Navigation of Information Systems (VNIS) Conference, Ottawa, Canada, , published by IEEE, Piscataway, NJ, pp. 126-129.

Van Aerde, M., M. Baker, and J. Stewart (1996). *Weaving Capacity Sensitivity Analysis using INTEGRATION*. Presented at the 75th Annual Meeting of the Transportation Research Board, Washington, D.C.

Van Aerde, M., M. Krage, and E.R. Case (1991). *Supporting Routines for Modeling the Traffic Responsive features of the TravTek System Using INTEGRATION*. Proceedings of the 1991 Vehicle Navigation and Information Systems (VNIS) Conference, Dearborn, MI, SAE Conference Proceedings No. 253, Part 2, Society of Automotive Engineers, Warrendale, PA, pp. 681-691.

Van Aerde, M., and H. Rakha (1995). *Multivariate Calibration of Single Regime Speed-Flow-Density Relationships*. Proceedings of the 6th Vehicle Navigation and Information System Conference (VNIS), Seattle, WA, published by IEEE, Piscataway, NJ, pp.334-341.

Van Aerde, M., J. Voss, and Y. Blum (1989). *Modeling the Burlington Skyway FTMS During Recurring and Non-Recurring Traffic Congestion, Traffic Control Methods*. Proceedings of the 1989 Engineering Foundation Conference, pp. 247-272.

Van Aerde, M., J. Voss, A, Ugge, and E.R. Case (1989). *Managing Traffic Congestion in Combined Freeway and Traffic Signal Networks.* ITE Journal, Vol. 59, No. 2, pp. 36-40.

Van Aerde, M., and S. Yagar (1988a). *Dynamic Integrated Freeway/Traffic Signal Networks: A Routing-Based Modeling Approach*. Transportation Research A, Vol. 22A, No. 6, pp. 445-453.

Van Aerde, M., and S. Yagar (1988b). *Dynamic Integrated Freeway/Traffic Signal Networks: Problems and Proposed Solutions*. Transportation Research A, Vol. 22A, No. 6, pp. 435-443.

Van Aerde, M., and S. Yagar (1990). *Combining Traffic Management and Driver Information in Integrated Traffic Networks*. Third International Conference on Road Traffic Control, IEE Conference Publication Number 320, Institution of Electrical Engineers, Michael Faraday House, Stevenage, England, p 11-15.

Van Aerde, M., S. Yagar, A. Ugge, and E.R. Case (1987). *A Review of Candidate Freeway-Arterial Corridor Traffic Models*. Transportation Research Record 1132, TRB, National Research Council, Washington, D.C., pp. 53-65.

Van Aerde, M., B. Hellinga, L. Yu, and R. Rakha (1993). *Vehicle Probes as Real-Time Sources of Dynamic O-D and Travel Time Data.* Advanced Traffic Management Systems Conference, Engineering Foundation, St.-Petersburg, Florida.

Velan, S., and M. Van Aerde (1996). *Relative Effects of Opposing Flow and gap Acceptance on Approach Capaicty at Uncontrolled Intersections*. Presented at the 75th Annual Meeting of the Transportation Research Board, Washington, D.C.

Yagar, S. (1983). *The Future for Integrated Control, Traffic Monitoring and Control Systems*. Proceedings of the 1983 Engineering Foundation Conference.

Webster, F.V., and B.M. Cobbe (1966). *Traffic Signals*. Road Research Technical Paper No. 56, Her Majesty's Stationery Office, London, England.

GLOSSARY

Release	Releases represent a chronological listing of model enhancements. The current release is referred to as 2.30.
Version	Versions refer to the size of problem that can be handled. INTEGRATION versions range from small to huge.
Simulation	Simulation refers to the tracking of individual vehicle movements. Strict simulation only relies on externally specified controls and routes.
Traffic assignment	Traffic assignment refers to the computation of routes for vehicles to follow. Traffic assignment results in link flows, but this is only a secondary output.
Microscopic	Microscopic refers to the representation of individual vehicles as discrete entities. INTEGRATION provides for microscopic simulation of traffic flow.
Macroscopic	Macroscopic refers to the representation of traffic as a continuous fluid. INTEGRATION provides for macroscopic traffic assignment as an option.
Optimization	Optimization refers to the selection of parameters to improve some objective. INTEGRATION includes signal timing and ramp metering rate optimization.
Simulation clock	The simulation clock tracks the time elapsed with respect to the modeled vehicles. The simulation proceeds at a rate that is dependent on the computer.
CPU clock	The CPU clock tracks the time elapsed with respect to the computer's CPU. The CPU clock advances at the same rate as the modeler's wristwatch.