1142

TRANSPORTATION RESEARCH RECORD

Urban Signal Systems and Transportation System Management

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
WASHINGTON, D.C. 1987

Transportation Research Record 1142
Price $10.50
Editor: Naomi Kassabian
Typesetting and Layout: Harlow A. Bickford

mode
1 highway transportation

subject area
54 operations and traffic control

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Printed in the United States of America

Library of Congress Cataloging-in-Publication Data
National Research Council. Transportation Research Board.

Urban signal systems and transportation system management.

(Transportation research record, ISSN 0361-1981 ; 1142)
1. Traffic signs and signals. 2. Traffic engineering. 3. City traffic. I. National Research Council (U.S.). Transportation Research Board. II. Series.

TE7.H5 no. 1142	380.5 s	88-600134
[TE228]	[388.3'122]	

ISBN 0-309-04521-5

Sponsorship of Transportation Research Record 1142

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Transportation Research Record 1142

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Contents

Foreword

Urban congestion is a matter of current concern in many metropolitan areas. Where physical increases in capacity are not feasible, or else cannot be accomplished in the short run, other solutions must be found. Improvements in signal systems and applications of transportation system management (TSM) measures are examples of such solutions.

This group of papers, sponsored by the Committee on Traffic Signal Systems and the Committee on Transportation System Management, provides information on improved analytical tools that can be used in developing solutions. The first, by Lee and Machemehl, describes improvements to the TEXAS model. With this user-friendly version, alternatives can be tested not only with greater cost-effectiveness but also with graphic display outputs that facilitate descriptions of the analysis for public meetings. In the next paper, Johnson and Cohen present a method for analyzing the penalties that might accrue on a single arterial when networkwide signal timing is optimized. The third paper, by van Vuren et al., also deals with signal timing, specifically a study of the interaction between signal-timing strategies and traffic assignment in which a new policy shows promising results in a test comparison of stability and delays with Webster method.

The second group of four papers deals with broader issues than traffic signal system analysis. Levinson et al. look at the benefits and impacts of TSM measures, suggesting that the evaluation process for these be conducted on a commensurate scale. Wellander et al. describe a new process drawn from existing models that can be used to assess the traffic effects in a central business district affected by major transportation facility construction. This model was developed for an application in Seattle, and the next paper, on regional traffic analysis, also relates to Seattle. Bernstein focuses on an approach to analyze the impacts of different regional transit-investment alternatives. The approach that was followed was useful for presentations to lay groups and elected officials. In the last paper, Leggett reports on a planning process to develop and evaluate packages of TSM actions that will enable agencies to maximize the combined effects of TSM measures.

The TEXAS Model for Intersection Traffic—A User-Friendly Microcomputer Version with Animated Graphics Screen Display

CLYDE E. LEE AND RANDY B. MACHEMEHL

Two interactive data-entry programs have been incorporated into the TEXAS Model for Intersection Traffic to produce the user-friendly TEXAS model. With these programs, a user, by working through an alphanumeric terminal connected in an interactive time-sharing mode to a mainframe computer or through the keyboard of a microcomputer, can respond to screen-displayed prompts and instructions and enter all the data needed for a simulation run in about one-tenth the time previously required. The actual simulation can then be executed either on a mainframe computer or on an IBM personal computer. During simulation, the progress of each individually characterized vehicle moving through a simulated intersection is recorded and subsequently displayed in real time or in stop action on a microcomputer-driven graphics screen. This animated graphics display allows the user to study the overall traffic performance at an intersection or to examine the behavior of any selected vehicle or vehicles in great detail. It also offers an effective way of describing alternative intersection traffic flow conditions at public meetings and technical work sessions. Tabular summary statistics may be produced for each simulation run if requested by the user. With the user-friendly version of the TEXAS model, alternative intersection designs and traffic-control schemes can be evaluated quickly and accurately in a timely and cost-effective manner.

The Traffic EXperimental, Analytical, Simulation (TEXAS) Model for Intersection Traffic *(1–3)* is a powerful computer simulation tool that allows the user to evaluate in detail the complex interaction among individually characterized driver-vehicle units as they operate in a defined intersection environment under a specified type of traffic control. In the original version of the model, the user was required to input an extensive amount of highly detailed descriptive data in order to characterize a simulated geometric, traffic, and traffic-control situation. A series of data-coding forms was developed to aid the user in the tedious data-input process, but use of the forms proved to be cumbersome at best and coding errors occurred rather frequently. Some potential users of the TEXAS model were discouraged by the amount of effort needed for data entry. Several hours of work were frequently required in preparation for a single run of the model.

Output from the model, which includes several pages of tabular data concerning the behavior of traffic and traffic-control devices during the simulated time period, also lacked user appeal. These data summarize exactly many different measures of intersection performance, but they are rather difficult to interpret, even for an experienced traffic engineer. The need for a more efficient means of communication between the user and the TEXAS model became evident.

The results of a major effort to make the TEXAS model more user-friendly and more accessible are described. In the new version of the model, the user builds compatible data files through alphanumeric terminals networked to the mainframe computer in an interactive time-sharing mode or through microcomputers, which may stand alone or be networked to the mainframe. Two interactive data-entry programs guide the user in entering data via a series of prompts (questions and instructions) displayed on the screen of the terminal or the microcomputer. The results of data entry can be echoed on the screen and also printed on a hard-copy device. The TEXAS model can then utilize these data files when running either on the mainframe or on an IBM personal computer. Running time on the microcomputer is considerably longer than that on the mainframe, but is quite practical in view of the relative availability of time on the two classes of computers. Output data files, which include the instantaneous speed, location, and time relationship for every simulated vehicle, provide the basis for an animated graphics display on a screen driven by the microcomputer. With this display the user can study the overall traffic performance at a particular intersection or examine in great detail the behavior of an individual vehicle in the traffic stream. The tabular summary data from the model are also available for quantitative analysis.

STRUCTURE OF THE TEXAS MODEL FOR INTERSECTION TRAFFIC

The TEXAS Model for Intersection Traffic includes four data processors—GEOPRO (geometry), DVPRO (driver-vehicle), SIMPRO (simulation), and EMPRO (emissions)—for describing, respectively, the geometric configurations, the arriving traffic, the behavior of traffic in response to the applicable traffic controls, and the emissions generated by the traffic. The structural relationship among these data processors is shown in Figure 1.

GEOPRO defines the geometric intersection characteristics needed for simulation. The user specifies the lengths of inbound and outbound lanes on the approaches, and GEOPRO calculates vehicle paths along the approaches and within the intersection. The number of intersection legs together with their associated number of lanes and lane widths define the intersection size and the location of any special lanes. The azimuths of the legs define the intersection shape. The

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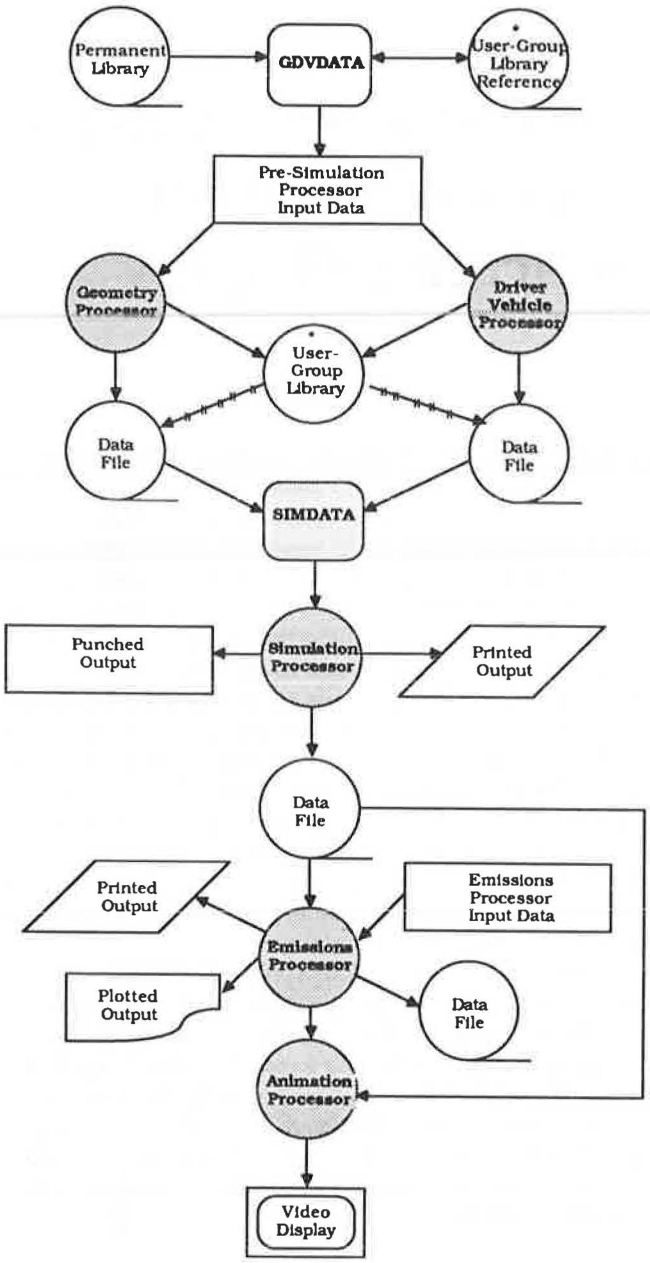

FIGURE 1 Flowchart of structure of user-friendly TEXAS model.

allowed directional movements of traffic on the inbound lanes and the allowed movements on outbound lanes define the directional use of the intersection.

DVPRO utilizes certain assigned characteristics for each class of driver and vehicle and generates attributes for each individual driver-vehicle unit; thus, each unit is characterized by inputs concerning driver class, vehicle class, desired speed, desired outbound intersection leg, and lateral inbound lane position. All these attributes are generated by a uniform probability distribution, except for the desired speed, which is defined by a normal distribution. Units are sequentially ordered by log-in time (the time of entry into an inbound lane) as defined by the input of a selected headway distribution. The total number of driver-vehicle units that must be generated by DVPRO is determined by the product of the input traffic volume and the time to be simulated.

SIMPRO simulates the traffic behavior of each unit according to the surrounding conditions of the moment, including any traffic-control device indications that might be applicable. When the unit enters the inbound approach lane, its entry velocity is set so that the vehicle will neither exceed a selected desired speed nor collide with the unit immediately ahead of it. If the unit ahead is accelerating or is traveling at its desired speed, the entering unit will enter the approach at its own desired speed. If the unit ahead is decelerating, the speed of the entering unit is set to a value that is less than its own desired speed. If there is no leading unit on the inbound lane, the unit enters with its desired speed.

After entry, the unit is checked from moment to moment within SIMPRO to see whether it is in a car-following situation. If it is not, the magnitude of required acceleration or deceleration applicable at any given instant is calculated by linear interpolation between extreme values set for each vehicle class with respect to the desired speed and to zero speed. Maximum required acceleration and deceleration occur at or near zero speed, and zero acceleration occurs at the maximum speed that each type of vehicle can attain. If the unit is in a car-following situation, the speed and acceleration of the unit interact with the speed and position of the unit ahead. A unit is allowed to change into an adjacent lane and follow a path described by a cosine curve if less delay can be expected. Current and relative speeds and positions of all adjacent vehicles are thus utilized in determining the behavior of each driver-vehicle unit in the simulation model.

When car following or traffic control makes it necessary for a unit to accelerate or decelerate, the logic in SIMPRO provides for accelerating to the desired speed, accelerating to the speed of the unit ahead, decelerating to follow the unit ahead, or decelerating to the desired speed within the available distance. As the unit proceeds along the inbound approach lane, the location and the status of traffic-control devices are checked moment by moment. The indication of the traffic-control devices will apply to the unit as soon as it comes into the influence area of the device.

If stop signs control the intersection, SIMPRO lists the units stopped before the sign according to their arrival times and then releases them in a first-arrived–first-served sequence. If there are simultaneous arrivals on adjacent intersection legs, the unit to the right gets priority for earliest release.

If pretimed signals control, each unit responds to the signal indications, which appear in a defined sequence and are of a specified duration for each phase. Each unit will attempt to go on a green indication after checking for intersection conflicts. If the unit is in the leading position and has cleared conflicts, it will enter the intersection. If a leading unit has stopped before the unit being examined, or if the leading unit is decelerating, the unit being examined will begin to stop. When the signal indication is red, each arriving unit will stop; however, a right-turn-on-red option is provided.

If control is by an actuated signal controller, the sequence and duration of each indication are selected in response to the information received by the controller from the detectors. The logic for driver response to signal indications is, of course, the same as that described for the pretimed signal. A detector actuation is defined by the time interval during which the front bumper of a unit has crossed the start of the detector but the rear bumper has not crossed the end of the detector. Actuations may cause the controller to continue the phase or allow the phase to change when a maximum time interval for that phase has elapsed or a sufficiently large gap occurs.

Statistics about delays and queue lengths are also gathered by the TEXAS model for evaluating the performance of traffic at the intersection. Delay statistics include the average of total delay and the average of stop delay incurred by each vehicle processed. Each delay is summarized by left-turn, right-turn, and straight movements and by the total of these three permitted directional movements on each inbound approach. Total delay is the difference between travel time for a vehicle through the system and the time it would have taken the vehicle at its desired speed. Stop delay is the time spent by a vehicle that has a velocity less than 3 ft/sec. Delay statistics show the overall influence of the intersection environment on traffic passing through the intersection. Comparison of the delays expected by traffic making various directional movements indicates the interaction among traffic flows on the intersecting streets. Queue-length statistics include average queue length and maximum queue length. Both are measured in units of vehicles, not feet. Average queue length and maximum queue length are taken for each inbound lane over any selected time interval.

EMPRO, the emissions processor, incorporates models to predict the instantaneous vehicle emissions of carbon monoxide (CO), hydrocarbons (HC), oxides of nitrogen (NOx), and fuel flow (FF) for both light-duty and heavy-duty vehicles. EMPRO utilizes information from SIMPRO about the instantaneous location, speed, and acceleration of each vehicle to compute instantaneous vehicle emissions and fuel consumption at points along the vehicle path.

USER-FRIENDLY TEXAS MODEL

Data Entry

As shown in Figure 1, data that are required for running the TEXAS model are entered by the user through two computer data-entry programs called GDVDATA (geometry, driver, vehicle) and SIMDATA (simulation). These are unique features of the user-friendly version of the model.

A new technique is incorporated into GDVDATA for entering the data needed for defining the geometric features of the intersection area in terms that are acceptable to the geometry processor (GEOPRO) of the TEXAS model. Previously the coordinates of all lines and circular arcs had to be calculated and coded individually, but the new technique uses a modular construction concept to build the intersection geometry from sets of properly configured and oriented lanes, legs, and curb returns. Now all geometric features are specified by lengths and angles, which can be more easily defined by the user. The nomenclature and arrangement of various geometric intersection features are shown in Figure 2.

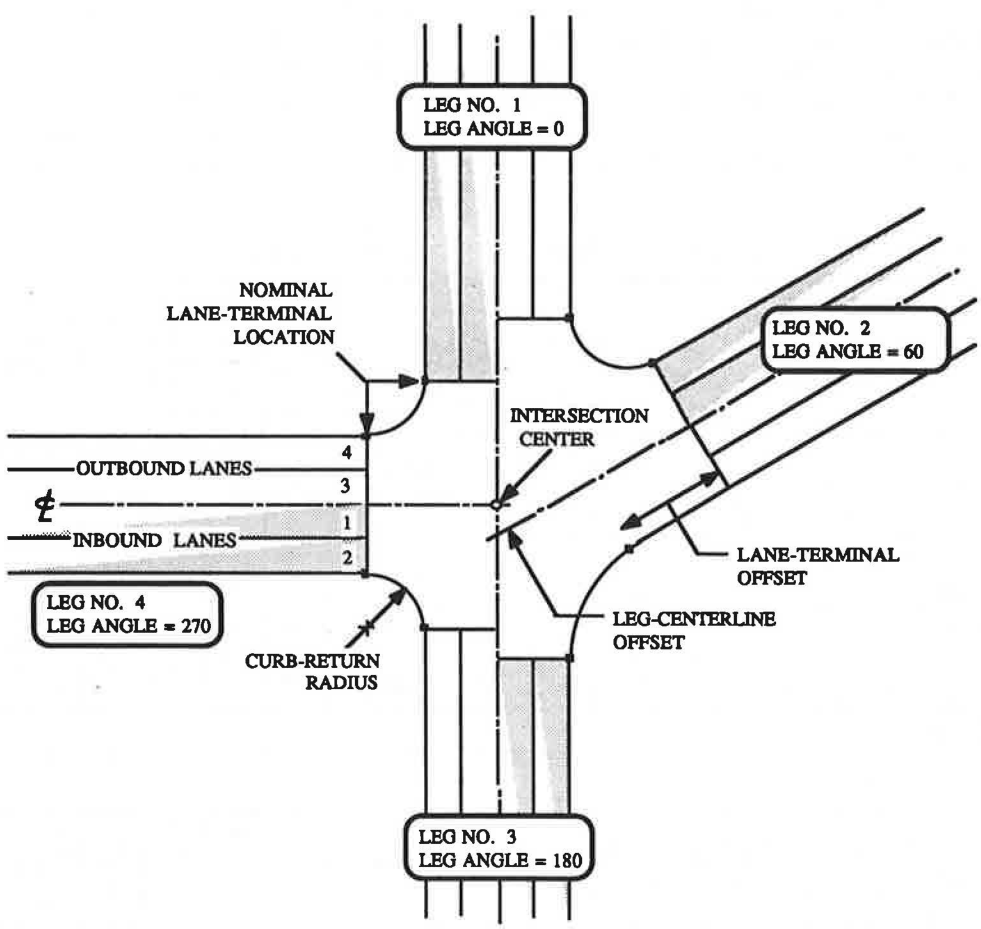

FIGURE 2 Nomenclature and arrangement of intersection geometric features.

In addition to the geometric data needed by the model, the user must enter data to characterize the drivers and vehicles that make up the traffic stream passing through a simulated intersection. GDVDATA includes user aids for entering the data needed by the driver-vehicle processor (DVPRO), which then arranges all data that are needed by the model to characterize driver and vehicle behavior into a format that is suitable for use in the actual simulation process. The driver-vehicle data items that can be defined by the user through GDVDATA are listed in Table 1 along with the default values that will be supplied automatically by the program unless the user requests otherwise.

For efficiency and for the convenience of the user, a series of 20 typical geometric arrangements and traffic patterns have been configured and stored in GDVDATA. These files, which can serve as the basis for many practical cases of interest to the user, are called the "permanent library." Each file in the permanent library contains all the geometric and traffic data that are needed for running the model. The contents of each permanent library file, including a simplified diagram that can be displayed on the screen of an alphanumeric terminal, are described elsewhere *(4)*. The user can study the permanent library files to determine whether one of them contains data that define an intersection situation of interest. If one of the files describes the situation exactly and the user wants to utilize the data contained in the permanent library file without modification, data entry proceeds directly to SIMDATA.

A user-group library is also provided through GDVDATA to allow users to develop, store, index, and retrieve their own data files conveniently for modification or for repeated use without modification. This feature is particularly efficient when the same intersection geometry and traffic are to be used repeatedly in several simulation runs, because it will not be necessary to rerun the geometry and driver-vehicle processors each time. The user-group library consists of the names of up to 16 data files that have been (a) saved on a permanent file and (b) entered into the user-group library. This library serves as a cross reference, or an index, to data files that have been previously prepared and saved by users on the same computer system.

Data that are needed by the simulation processor, SIMPRO, are entered through the data-entry program called SIMDATA. This program pairs the entered data required by SIMPRO with data previously defined by using GDVDATA or with data contained in a permanent library file within GDVDATA. A series of prompts and instructions is utilized in SIMDATA, as in GDVDATA, to guide the user through this part of the data-entry process.

Animated Graphics Display

Output from the TEXAS model includes the instantaneous speed, location, and time relationship for every simulated vehicle. These data are routinely written onto an external file for use by the emissions processor, EMPRO, or for other applications. The user-friendly TEXAS model provides a feature whereby this information can be displayed graphically in real time or in stop action on a screen driven by an IBM PC. Intersection geometry is extracted from the files created by GDVDATA and displayed on the screen first. Then the position of each simulated vehicle with respect to time is represented on the screen by an outline of the vehicle, scaled to size and color coded according to performance capability.

With this animated graphics display the user can study the overall traffic performance at an intersection or examine in great detail the behavior of an individual vehicle in the traffic stream. This is a unique capability that permits the user to

TABLE 1 PROGRAM SUPPLIED (DEFAULT) VALUES FOR DRIVER AND VEHICLE CLASS DATA

VEHICLE CHARACTERISTIC / VEHICLE TYPE	PASSENGER CARS				TRUCKS							
					Single-Unit				Tractor Semi-Trailer			
					Gasoline		Diesel		Gasoline		Diesel	
	Sports	Compact	Medium	Large	PL*	FL#	PL	FL	PL	FL	PL	FL
Class	1	2	3	4	5	6	7	8	9	10	11	12
Operating Characteristics Factor	115	90	100	110	85	80	80	75	70	65	75	70
Maximum Deceleration, ft/sec/sec	14	13	13	8	7	5	7	5	6	4	6	4
Maximum Acceleration, ft/sec/sec	14	8	9	11	7	6	6	5	4	3	5	4
Maximum Velocity, ft/sec	205	120	135	150	100	85	100	85	95	75	100	80
Minimum Turning Radius, ft	20	20	22	24	42	42	42	42	45	45	45	45
Length, ft	14	15	16	18	32	32	32	32	60	60	60	60
Percentage in Traffic Stream, %	1.5	22.5	23.3	44.7	2.6	2.6	0.2	0.2	0.2	0.2	1.0	1.0

DRIVER				PERCENTAGE OF DRIVER CLASS IN EACH VEHICLE TYPE											
Type	Class	P-R Time	Factor	Sports	Compact	Medium	Large	PL	FL	PL	FL	PL	FL	PL	FL
Aggressive	1	0.5	110	50	30	35	25	40	40	40	40	40	40	40	40
Average	2	1.0	100	40	40	35	45	40	40	40	40	40	40	40	40
Slow	3	1.5	85	10	30	30	30	20	20	20	20	20	20	20	20

* Partially-Loaded Truck # Fully-Loaded Truck

examine easily several alternative solutions to a problem by simulation without the time and expense of cut-and-try experimentation in the field. A wide range of conditions can be defined and evaluated visually on the screen as well as in the form of tabular summaries of statistics about traffic and signal-control performance.

Microcomputer Requirements

Development of the microcomputer version of the TEXAS model was significantly aided by grants of hardware and software from IBM through the QUEST Project at the University of Texas. The current microcomputer version of the model is configured to run on IBM PC-XT's or PC-AT's with at least 512K of random-access memory (RAM), a math coprocessor, a color graphics adapter and color graphics monitor, or an enhanced graphics adapter and monochrome, color, or enhanced graphics monitor. DOS 3.1 or the equivalent and a printer are also required by the current configuration.

Because the basic processors are written totally in ANSI Standard FORTRAN 77, implementation on other machines will only require modification of the language used by other operating systems to access and store files. The animation processor contains assembly language routines that enable faster execution of the graphics display. Implementation of these routines on other machines would obviously require additional modification. All other processors, however, can be quite readily implemented on other machines, essentially without modification.

Execution time for a normal intersection simulation run on an IBM PC-XT is slightly longer than the real time that is simulated. This includes writing the data file that is required for the subsequent animated graphics display of intersection geometry, traffic signal indications, and vehicles. The same simulation run requires somewhat less than real time for execution on the University of Texas at Austin's CYBER time-shared mainframe.

ACKNOWLEDGMENTS

The research on which this paper is based was sponsored by the Texas State Department of Highways and Public Transportation (SDHPT) and the Federal Highway Administration through the Cooperative Highway Research Program of the Bureau of Engineering Research, Center for Transportation Research (CTR), the University of Texas, at Austin. Blair G. Marsden, who was the principal research study contact for SDHPT, encouraged the development of this practical engineering tool and contributed generously of his talents and energies throughout the project. Charlie R. Copeland, Jr., and Robert F. Inman, engineering research associates with CTR, developed many of the simulation concepts and wrote the extensive computer code that was necessary to implement the traffic simulation and the animated graphics displays. Wiley M. Sanders and other students at the University of Texas at Austin also participated in the development and testing of the model. Most of the microcomputer work was accomplished with hardware and software that was made available to the researchers by IBM through Project QUEST at the University of Texas at Austin. The timely contribution of these resources made the animated graphics and the microcomputer version of the TEXAS model possible. The support of these individuals, agencies, and corporations is gratefully acknowledged.

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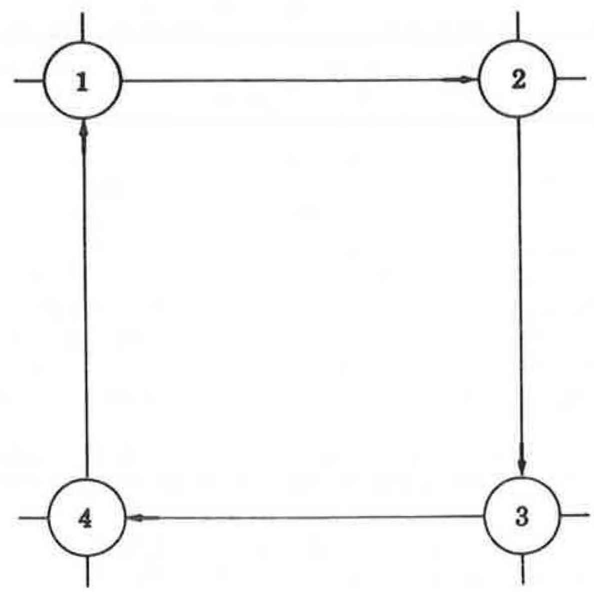
Publication of this paper sponsored by Committee on Traffic Signal Systems.

Single-Arterial Versus Networkwide Optimization in Signal Network Optimization Programs

VERETTA JOHNSON AND STEPHEN L. COHEN

The optimization of signal timing in a traffic network involves finding the timing plan that optimizes the overall performance in the network. In theory, the network closure constraints limit the performance on individual arteries of the network. Thus networkwide optimization has the potential of imposing some cost or penalty, or both, to individual arterials in the network. The objective of this study was to determine how or if the network closure constraint affects or limits arterial performance in the program for maximum-bandwidth, MAXBAND, and in the program for minimum stops, delay, and fuel consumption, TRANSYT-7F. The results of this study show that for small and medium-sized closed networks, optimization of an entire network using MAXBAND or TRANSYT-7F costs very little in terms of stops, delay, and green bandwidth on the arteries within the network. The added cost associated with the additional stops and delays resulting from networkwide optimization can be expected to impose approximately a 5 percent penalty on individual arteries within the network.

The optimization of signal timing in a traffic network requires finding the timing plan that optimizes the overall performance in the network. In a closed network, the timing plan must satisfy a network closure constraint not required for arterials. Thus, it may be the case that individual arteries in the network are sacrificed for the good of the whole. The purpose of this study is to determine the cost or penalty (if any) that networkwide optimization would impose on the individual arteries of the network.



A Network with 1 Loop: 1 → 2 → 3 — 4 → 1

Links of loop: (1,2),(2,3),(3,4),(4,1)

Loop constraint: $O_{12} + O_{23} + O_{34} + O_{41} = nC$
for some natural number n.

FIGURE 1 The network loop constraint.

NETWORK CLOSURE CONSTRAINT

For fixed cycle length and splits at each intersection of a network, the network closure constraint simply requires the sum of the offsets around any loop of the network to be a multiple of the cycle time. This can be stated as follows:

Let C be the cycle length, Oij be the offset between signals i and j, and l be a set of links that form a closed loop.

Define $L = \{l{:}l \text{ is a loop}\}$.

Then the network closure constraint is $Oij = nC$, where n is an integer and the sum is taken over all links (i, j) in the loop l.

For the network shown in Figure 1, the constraint requires that $O12 + O23 + O34 + O41 = nC$ for some integer n.

Closure constraints have the effect, at least in theory, of degrading traffic performance on individual arteries within the network in order to optimize overall performance. For instance, in the case of bandwidth optimization, these constraints would prevent individual arteries from obtaining the maximum bandwidths that they could obtain if optimized separately.

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PROBLEM DISCUSSION

Some computerized signal network optimization programs such as TRANSYT-7F and MAXBAND can be used both on single arteries and on networks *(1, 2)*.

MAXBAND simultaneously optimizes cycle length, phase sequences, and offsets to maximize a weighted sum of all bandwidths on all arteries of the network. Thus for single-artery optimization this reduces to the maximization of the bandwidths in each direction of the artery. Also, for the single-artery case there are no loops, so there are fewer restrictions on the choice of offsets at each intersection. Hence, one objective of the study is to determine how the additional restrictions of the network closure constraint on the offsets limit arterial performance in the maximal bandwidth program MAXBAND.

TRANSYT-7F adjusts offsets and green time separately to minimize a weighted sum of stops and delay. Again the offset selection is limited by the network closure constraint in the networkwide optimization. In this study the objective is to determine how this constraint affects the individual arteries of the network.

STUDY OBJECTIVE

Network closure constraints impose additional restrictions on the arterial settings within a network, which might result in less than absolute optimal settings for individual arteries of a network. The objective of this study is to determine how or whether the additional network constraint affects or limits arterial performance, or both, in the signal optimization programs TRANSYT-7F and MAXBAND, that is, to determine (a) the loss (if any) of bandwidth to individual arteries of a network that results from networkwide optimization rather than individual-artery optimization and (b) the increase in cost (if any) associated with delay and stops to the individual arteries of a network that results from networkwide optimization.

DESCRIPTION OF PROGRAMS

MAXBAND

The MAXBAND programs find traffic signal settings on arteries and general grid networks by using optimization of green bandwidths as the criterion. The optimization problem can be stated as follows: Compute offsets, cycle length, and left-turn phase sequence so as to maximize a weighted sum of all bandwidths on all arteries of the network. This problem is formulated as a mixed-integer linear programming problem. User inputs to the program include the usual volume, capacity, minimum green time, and link length as well as left-turn patterns to be considered and inter- and intraartery weighting.

MAXBAND 86 is the name of the new network version of MAXBAND. For the new program, algorithms were developed that convert network loop characteristics into equivalent mixed-integer linear programming formulations. It was found that large or complex network problems, or both, pushed the optimization technique beyond its capability to produce optimal solutions within reasonable computation time. Networks optimized by using MAXBAND 86 must be completely connected, and no more than two arterials may compose an intersection.

TRANSYT-7F

The TRANSYT program finds the traffic signal settings on networks that minimize a weighted sum of stops and delay. A hill-climbing optimization procedure adjusts offsets and green time separately to minimize a weighted sum of stops and delay called the performance index. User inputs to the program include volume, capacity, minimum green time, link lengths, flow patterns, cycle length, and initial offsets and splits. A TRANSYT optimization run may be of five types: (a) optimization of offsets, cycle length, and splits; (b) optimization of offsets only, (c) simulation only, (d) cycle-length selection only, or (e) optimization of offsets and splits.

EXPERIMENTAL DESIGN

A three-phase experimental plan was used to accomplish the objectives of the study. For the first phase of the study MAXBAND was used to optimize four small closed networks. Each entire network and each artery was optimized individually. A total of 37 MAXBAND optimization runs were made during this phase. A comparison was made between the bandwidths obtained on each artery within the network and those obtained when a single artery was optimized.

The second phase of the study consisted of TRANSYT-7F optimizations of the same four networks used in Phase 1. Each artery of each network was optimized individually and within its network by using TRANSYT-7F. The costs associated with delay and stops were compared for each artery under individual optimization and networkwide optimization.

Phase 3 of the study was essentially a repeat of Phase 2 except that three larger networks were optimized.

TEST DATA SETS

This research concentrated on small closed networks. Pretimed, common-cycle, coordinated traffic signals with primarily two-phase operation were emphasized.

Seven data sets were used. Five of the data sets—Daytona Beach; Washington, D.C., Section 3; Lexington; Chicago; and Washington, D.C., west central business district (CBD)—were obtained from FHWA files. The remaining two—Ann Arbor and Battle Creek—were provided by the University of Florida's Transportation Research Center.

Smaller Networks

Washington, D.C., Section 3

Eight arteries from the Washington, D.C., UTCS-1 network system were used. The network includes three east-west arteries, two of which are one-way streets, and five north-south arteries, two of which are also one-way streets. This network, which is located in the downtown area of the District, is shown in Figure 2.

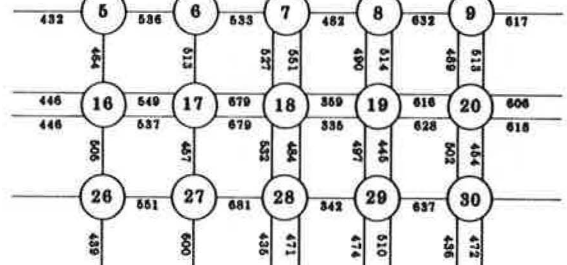

FIGURE 2 Washington, D.C., Section 3 network.

Lexington

The test network is part of the Lexington, Kentucky, downtown signal system. There are five east-west arteries and four north-south arteries. All the east-west arteries and two north-south arteries are one-way streets. The Lexington network is shown in Figure 3.

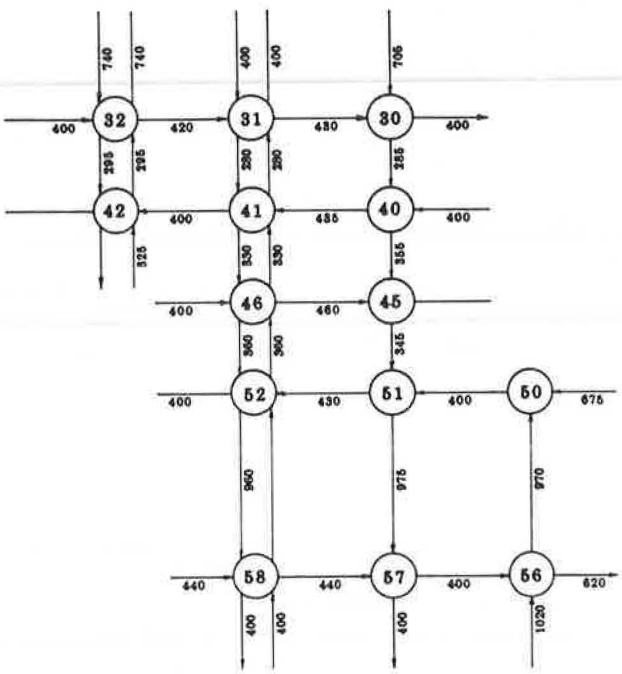

FIGURE 3 Lexington, Kentucky, network.

Daytona Beach

The test network is located in downtown Daytona Beach, Florida. There are three east-west arteries and four north-south arteries, all of which are two-way streets. Included in this network are two major arterials, Ridgewood Avenue and Volusia Avenue. This is the network system example included in the TRANSYT-7F User's Manual. Figure 4 shows this network.

Chicago

The test network is a nine-artery system centered around two major arterials (Michigan Avenue and NS2) in Chicago, Illinois. The two major arterials are two-way, north-south streets and the remaining seven arteries are east-west streets, two of which are two-way streets. Figure 5 shows this network.

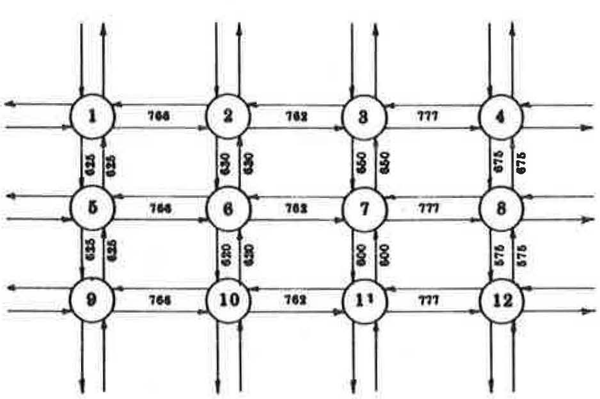

FIGURE 4 Daytona Beach, Florida, network.

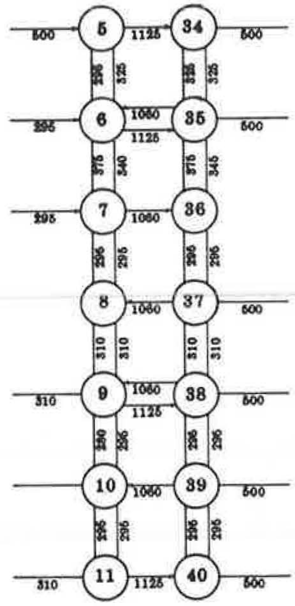

FIGURE 5 Chicago, Illinois, network.

Larger Networks

Ann Arbor

The 15-artery test network is located in the Ann Arbor, Michigan, CBD. Two of the seven east-west streets and one of the nine north-south streets are one way. The Ann Arbor network is shown in Figure 6.

Battle Creek

The test network is part of the Battle Creek, Michigan, CBD. Included are four major north-south arteries and several shorter east-west arteries. This network is shown in Figure 7.

Washington, D.C., West CBD

The largest of the test networks is located in the Washington, D.C., CBD. All six east-west arteries are one-way streets as are all but three of the north-south arteries.

EXPERIMENTAL PLAN

A series of experiments was performed to accomplish the goals of the study.

Experiment 1:

1. Individual arteries of the small networks (Daytona Beach, Florida; Lexington, Kentucky; Chicago, Illinois; and Washington, D.C., Section 3) optimized with MAXBAND; cycle

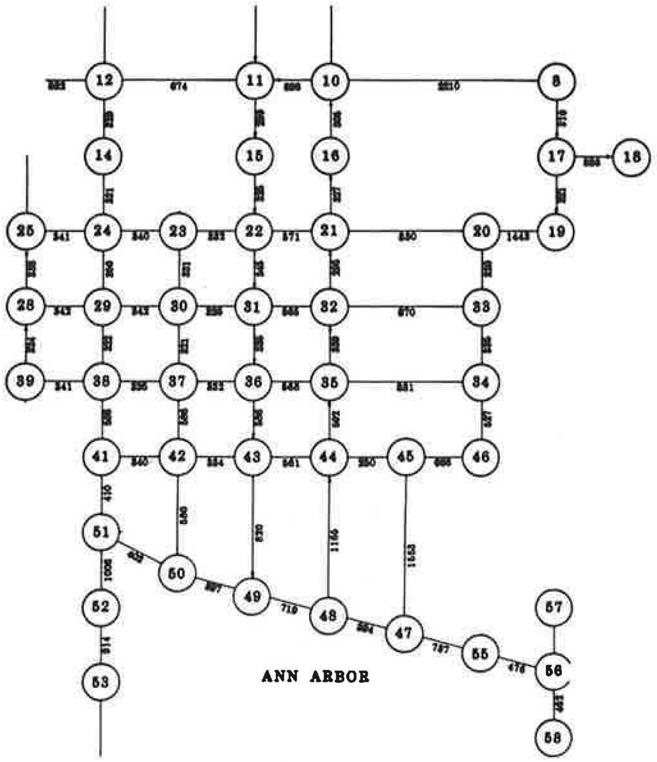

FIGURE 6 Ann Arbor, Michigan, network.

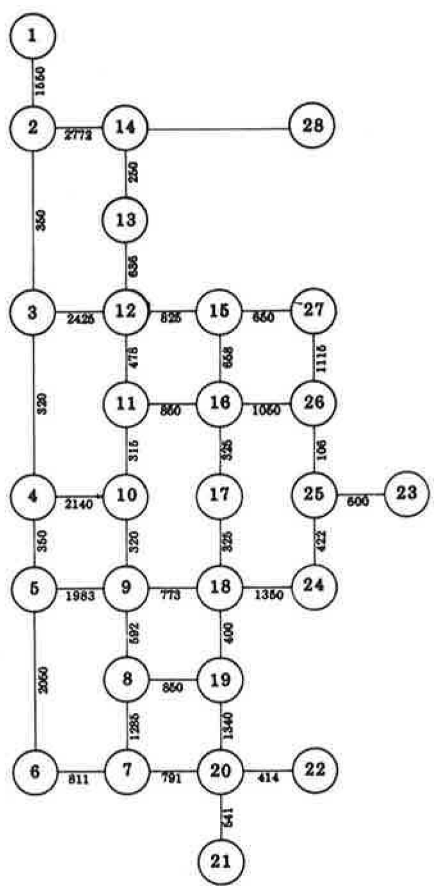

FIGURE 7 Battle Creek, Michigan, network.

lengths and phase sequences fixed for each artery; none of the arteries provided directional weighting.

2. Each of the four small networks network-optimized with MAXBAND; cycle lengths and phase sequences fixed; no directional or between-artery weighting coded.

Experiment 2:

1. Small networks offset-only optimized with TRANSYT-7F; MAXBAND solution of each network used as the starting solution for the TRANSYT-7F runs.

2. Each artery of each small network offset-only optimized with TRANSYT-7F; MAXBAND solution of each artery used as the starting solution for the TRANSYT-7F optimizations.

Experiment 3:

1. Each of the three larger networks (Ann Arbor, Battle Creek, and Washington, D.C., west CBD) optimized normally with TRANSYT-7F; existing conditions of each network used as the starting TRANSYT-7F solution.

2. Larger networks offset-only optimized with TRANSYT-7F; existing timing used as the starting solution; all east-west links of the networks delinked, that is, no nodes connected by east-west links (equivalent to optimizing the north-south arteries separately).

3. Three larger networks offset-only optimized with TRANSYT-7F; existing timing used as the starting solution with all north-south links delinked (equivalent to optimizing the east-west arteries separately).

Comparisons were made between the bandwidths obtained in Parts 1 and 2 of Experiment 1, the cost of uniform stops and of delay obtained in Parts 1 and 2 of Experiment 2, and the cost of stops and delay obtained in Parts 1 and 2 or 1 and 3 (as appropriate) of Experiment 3.

To study the cost of networkwide optimization to the individual arteries of a network, the cost of stops was computed as \$0.04 per stop and the cost of delay as \$0.50 per vehicle hour of delay. These values were taken from the National Signal Timing Optimization Project final report *(3)*.

The detailed comparison of the results was made with a spreadsheet program.

RESULTS

The results of Phase 1 of the study are shown in Tables 1 through 4. The results indicate that, most of the time, under network optimization individual arteries achieve bandwidths that approach or equal the bandwidths that could be achieved if the arteries were optimized separately. Only Connecticut Avenue in the Washington, D.C., Section 3 network and the EW1 artery in the Chicago network showed any substantial degradation in performance. Thus the primary effect of using the network optimization is to provide a means for taking individual arteries, optimized separately, and adjusting the offset of the first intersections of each artery so that the offsets for the network are consistent.

TABLE 1 BANDWIDTH COMPARISONS OF DAYTONA BEACH ARTERIES

ARTERY	SINGLE BANDWIDTH	NETWORKWIDE BANDWIDTH	DIFFERENCE
RIDGEWOOD	28.2	27	1.2
PALMETTO	6.6	6.6	0
BEACH	31	31	0
ORANGE	25.7	25.7	0
MAGNOLIA	10.1	10.1	0
VOLUSIA	26.9	26.9	0
BAY	2.5	2.5	0
TOTAL	131	129.8	1.2

TABLE 2 BANDWIDTH COMPARISONS OF CHICAGO ARTERIES

ARTERY	SINGLE BANDWIDTH	NETWORKWIDE BANDWIDTH	DIFFERENCE
MICHIGAN	26.1	24.4	1.7
NS2	18.6	17.8	0.8
EW1	30.1	20	10.1
EW2	9.6	9.6	0
EW3	28.9	28.9	0
EW4	27.2	27.2	0
EW5	19.7	16	3.7
EW6	17.5	17.5	0
EW7	44.4	44.4	0
TOTAL	222.1	205.8	16.3

TABLE 3 BANDWIDTH COMPARISONS OF LEXINGTON ARTERIES

ARTERY	SINGLE BANDWIDTH	NETWORKWIDE BANDWIDTH	DIFFERENCE
EW1	26.1	23.4	2.7
EW2	23.9	23.9	0
EW3	27	27	0
EW4	30.2	30.2	0
EW5	22.3	22.3	0
NS1	38.5	38.5	0
NS2	3.9	2.8	1.1
NS3	17.5	17.5	0
NS4	28.5	28.5	0
TOTAL	217.9	214.1	3.8

TABLE 4 BANDWIDTH COMPARISONS OF WASHINGTON, D.C., SECTION 3, ARTERIES

ARTERY	SINGLE BANDWIDTH	NETWORKWIDE BANDWIDTH	DIFFERENCE
L STREET	25.3	24.1	1.2
K STREET	12.5	11.5	1
I STREET	2	2	0
19TH ST	35.1	35.1	0
18TH ST	30.2	30.2	0
CONN AV	50.2	31.4	18.8
17TH ST	8.3	7	1.3
16TH ST	24.3	24.3	0
TOTAL	187.9	165.6	22.3

TABLE 5 COST COMPARISONS FOR DAYTONA BEACH

ARTERY	SINGLE DELAY	STOPS	COST	NETWORKWIDE DELAY	STOPS	COST	COST DIFF	% DIFF
RIDGEWOOD	25.63	2651.9	118.891	25.01	2637	117.985	-0.906	-0.00762
PALMETTO	8.97	800.49	36.5046	10	1105.3	49.212	12.7074	0.348104
BEACH	24	2212.6	100.504	23.46	2126.1	96.774	-3.73	-0.037113
ORANGE	14.4	1365.7	61.828	14.32	1339.5	60.74	-1.088	-0.017597
MAGNOLIA	12.88	908.7	42.788	13.44	903.9	42.876	0.088	0.002057
VOLUSIA	31.17	2729.1	124.749	30.94	2724.3	124.442	-0.307	-0.002461
BAY	10.3	835	38.55	9.95	824.2	37.943	-0.607	-0.015746
TOTAL COST			523.8146			529.972		

DIFFERENCE 6.1574

% DIFF 0.0117549

TABLE 6 COST COMPARISONS FOR CHICAGO

ARTERY	SINGLE DELAY	STOPS	COST	NETWORKWIDE DELAY	STOPS	COST	COST DIFF	% DIFF
MICHIGAN	127.32	4912.6	260.164	128.94	5488.9	284.026	23.862	0.091719
NS2	6.69	1109.6	47.729	9.12	1343.9	58.316	10.587	0.221814
EW1	5.22	502.8	22.722	5.27	596.7	26.503	3.781	0.166403
EW2	6.38	712	31.67	5.84	668.6	29.664	-2.006	-0.063341
EW3	4.28	161.1	8.584	4.88	394.3	18.212	9.628	1.121622
EW5	3.06	338.1	15.054	3.71	392.9	17.571	2.517	0.167198
EW6	3.93	475	20.965	3.32	392.9	17.376	-3.589	-0.17119
EW7	104.21	1209.5	100.485	105.12	392.9	68.276	-32.209	-0.320535
TOTAL COST			533.971			551.395		
				DIFFERENCE	17.424			
				% DIFF	0.032631			

TABLE 7 COST COMPARISONS FOR LEXINGTON

ARTERY	SINGLE DELAY	STOPS	COST	NETWORKWIDE DELAY	STOPS	COST	COST DIFF	% DIFF
EW1	3.33	488.5	21.205	4.39	577.9	25.311	4.106	0.1936336
EW2	135.23	1787.8	139.127	162.64	2075.1	164.324	25.197	0.1811079
EW3	3.4	631.2	26.948	3.01	608.4	25.841	-1.107	-0.041079
EW4	2.87	484	20.795	2.79	517.9	22.111	1.316	0.0632844
EW5	3.38	535.4	23.106	4.69	668	29.065	5.959	0.2578984
NS1	122.04	2088.1	144.544	127.1	2390.5	159.17	14.626	0.1011872
NS2	7.69	887.9	39.361	8.52	1065.5	46.88	7.519	0.1910267
NS3	5.33	704.4	30.841	6.13	742.7	32.773	1.932	0.0626439
NS4	4.03	600.1	26.019	5	660.2	28.908	2.889	0.1110342
TOTAL COST			471.946			534.383		
				DIFFERENCE	62.437			
				% DIFF	0.1322969			

Comparisons of the cost of stops and delay for individual arteries of each network under arterial and networkwide optimization were made in Phase 2. Each network contained some arteries for which the cost associated with stops and delay when the artery was optimized separately was greater and some for which it was lower. The Lexington network showed the greatest increase in overall cost under networkwide optimization. The Daytona Beach; Washington, D.C., Section 3; and Chicago networks showed very small increases in cost of 0.4, 1.2, and 3.3 percent, respectively. The results of this phase of the study are shown in Tables 5 through 8.

Phase 3 of the study focused on the larger networks—Ann Arbor, Battle Creek, and Washington, D.C., west CBD. The results were similar to those found in Phase 2, with overall increases in the cost associated with stops and delay of 3.9, 4, and 4.8 percent, respectively. Each network had some arteries for which the cost was lower under individual-artery optimization and some for which it was lower under networkwide optimization. In most cases, as in Phase 2, the arteries that showed increased cost under networkwide optimization were offset by others that showed lower cost. The number of stops, the delay, and the cost associated with stops and delay found in this phase of the study are shown in Tables 9 through 11.

In Phases 2 and 3 it was found that the network that was the least rectangular (Lexington) and the ones that had predominantly one-way streets (Lexington and Washington, D.C., west CBD) showed greater degradation with networkwide optimization than did the other networks of the study.

TABLE 8 COST COMPARISONS FOR WASHINGTON, D.C., SECTION 3

ARTERY	SINGLE DELAY	STOPS	COST	NETWORKWIDE DELAY	STOPS	COST	COST DIF	% DIFF
L STREET	613.58	3723.9	455.746	606.58	2539.3	404.862	-50.884	-0.11165
K STREET	35.63	4107.6	182.119	40.49	5279.3	231.417	49.298	0.2706911
I STREET	4.67	358.7	16.683	9.36	379.6	19.864	3.181	0.1906731
19TH ST	18.3	1389.8	64.742	18.32	1394.2	64.928	0.186	0.0028729
18TH ST	8.29	1029.9	45.341	9.13	1062.5	47.065	1.724	0.038023
CONN AV	170.87	2413.1	181.959	170.81	2480.6	184.629	2.67	0.0146736
17TH ST	64.54	1681.3	99.522	64.62	1655.3	98.522	-1	-0.010048
16TH ST	13.14	1550.7	68.598	13.02	1549.7	68.498	-0.1	-0.001458
TOTAL COST			1114.71			1119.785		

DIFFERENCE 5.075

% DIFF 0.0045528

TABLE 9 COST COMPARISONS FOR ANN ARBOR

ARTERY	SINGLE DELAY	STOPS	COST	NETWORKWIDE DELAY	STOPS	COST	COST DIFF	% DIFF
CATHERINE	10.2	1472.8	64.012	10.07	1458.9	63.391	-0.621	-0.009701
ANN	7.42	918.7	40.458	7.15	960.3	41.987	1.529	0.0377923
HURON	14.32	1280.1	58.364	13.83	1307	59.195	0.831	0.0142382
WASHINGTON	6.83	880.3	38.627	6.97	905.6	39.709	1.082	0.0280115
LIBERTY	3.86	494.4	21.706	5.12	667.6	29.264	7.558	0.3481987
WILLIAM	10.1	1328.7	58.198	11.02	1493.1	65.234	7.036	0.1208976
PACKARD	17.75	2361.1	103.319	17.84	2478.8	108.072	4.753	0.0460032
ASHLEY	4.45	810	34.625	4.41	831.8	35.477	0.852	0.0246065
MAIN	15.46	1736.8	77.202	15.9	1811.8	80.422	3.22	0.0417088
FOURTH	4.83	704.5	30.595	4.08	645.2	27.848	-2.747	-0.089786
FIFTH	4.43	642.8	27.927	5.27	757.3	32.927	5	0.1790382
DIVISION	8.68	1637.9	69.856	8.3	1434.6	61.534	-8.322	-0.119131
NN1	6.46	825.1	36.234	6.53	887.5	38.765	2.531	0.0698515
NN2	6.93	829.7	36.653	7.03	917.2	40.203	3.55	0.0968543
THOMPSON	2.53	621.7	26.133	3.11	667.1	28.239	2.106	0.0805878
TOTAL COST			723.909			752.267		

DIFFERENCE 28.358

% DIFF 0.0391734

TABLE 10 COST COMPARISONS FOR BATTLE CREEK

ARTERY	SINGLE DELAY	SINGLE STOPS	SINGLE COST	NETWORKWIDE DELAY	NETWORKWIDE STOPS	NETWORKWIDE COST	COST DIFF	% DIFF
WASHINGTON	12.65	2279.9	97.521	12.4	2359.8	100.592	3.071	0.0314907
McCAMLY	10.58	1993.5	85.03	10.78	2027	86.47	1.44	0.0169352
CAPITAL	12.7	2291	97.99	12.44	2303	98.34	0.35	0.0035718
CAPITAL2	11.42	1573	68.63	42.31	1343.4	74.891	6.261	0.0912283
CALHOUN	3.92	608.1	26.284	3.95	623.6	26.919	0.635	0.0241592
VAN BUREN	9.84	1452.4	63.016	10.14	1625.8	70.102	7.086	0.1124476
MICHIGAN	3.46	506.7	21.998	3.71	510.8	22.287	0.289	0.0131376
STATE	3.46	591.1	25.374	3.98	640.4	27.606	2.232	0.0879641
JACKSON	6.08	885.2	38.448	6.59	1018.4	44.031	5.583	0.1452091
HAMBLIN	2.28	502.5	21.24	2.53	498.4	21.201	-0.039	-0.001836
DICKMAN	10.46	2009.4	85.606	10.47	1998.6	85.179	-0.427	-0.004988
SPECIAL	3.99	499.7	21.983	3.85	502.7	22.033	0.05	0.0022745
TOTAL COST			653.12			679.651		
				DIFFERENCE	26.531			
				% DIFF	0.0406219			

TABLE 11 COST COMPARISONS FOR WASHINGTON, D.C., WEST CBD

ARTERY	SINGLE DELAY	SINGLE STOPS	SINGLE COST	NETWORKWIDE DELAY	NETWORKWIDE STOPS	NETWORKWIDE COST	COST DIFF	% DIFF
K STREET	8.27	1079.9	47.331	7.03	1340.4	57.131	9.8	0.2070525
L STREET	8.47	1057.5	46.535	7.08	1233	52.86	6.325	0.1359192
M STREET	11.62	2170.4	92.626	11.66	2229	94.99	2.364	0.025522
N STREET	9.57	1946	82.625	11.07	1804.7	77.723	-4.902	-0.059328
O STREET	14.51	1559.5	69.635	11.01	2014.9	86.101	16.466	0.2364615
P STREET	10.53	1751.8	75.337	11.09	1669.4	72.321	-3.016	-0.040033
Q STREET	9.44	1549.5	66.7	9.12	1579.9	67.756	1.056	0.0158321
9TH ST	11.28	1418.9	62.396	8.83	1883.5	79.755	17.359	0.2782069
10TH ST	11.51	1546	67.595	9.53	1582.9	68.081	0.486	0.0071899
11TH ST	10.06	1763.7	75.578	8.09	1910.4	80.461	4.883	0.0646087
12TH ST	12.72	2108	90.68	10.19	2330.1	98.299	7.619	0.0840207
13TH ST	12.66	2043.7	88.078	11.24	2065.4	88.236	0.158	0.0017939
14TH ST	13.67	2806.5	119.095	15.51	2600.4	111.771	-7.324	-0.061497
15TH ST	4.48	1109.2	46.608	4.81	1068.9	45.161	-1.447	-0.031046
TOTAL COST			1030.819			1080.646		
				DIFFERENCE	49.827			
				% DIFF	0.0483373			

CONCLUSIONS

The first phase of the study shows that optimization of arteries within networks using MAXBAND involves no cost other than that of computer time. About the only effect of imposing the network closure constraint is to take the individual timing plans for each artery in the network and put them together in a consistent network timing plan. That is, in small closed networks the bandwidths obtained with networkwide optimization are not significantly different from those obtained with single-artery optimization.

Phases 2 and 3 show that for small and medium-sized closed networks, optimization of the arteries by using TRANSYT-7F results in some cost increases to the networks but not necessarily to individual arteries. In fact, some arteries operate more efficiently when optimized as a part of a network.

RECOMMENDATIONS

Because network MAXBAND is relatively expensive to run, the traffic engineer might be wise to simply optimize each artery of small closed networks individually and adjust offsets manually to achieve near-optimal networkwide performance.

For small and medium-sized closed networks, optimizing an entire network by using TRANSYT-7F results in very little increased cost of stops and delay to the network as a whole. Lower cost associated with stops and delay can be expected for some arteries when optimized as a part of a network. Therefore it is recommended that networkwide optimization rather than individual-artery optimization be done on networks of this sort.

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DISCUSSION

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Several computerized signal timing optimization programs, such as TRANSYT-7F and MAXBAND 86, are currently available to optimize signal timing plans for linear arterials and grid signal networks. The maximal bandwidth program, MAXBAND, was enhanced by the Texas Transportation Institute in 1986 to maximize simultaneously the weighted sum of all progression bandwidths on all arteries of the signalized network. In addition to individual arterial progression constraints, an independent loop identification algorithm and a network closure constraint were added to describe the interconnected network topology in a closed signal network. This added condition requires that the sum of the relative signal offsets around any independent loop of the signal network be equal to a multiple of the common background cycle length. Therefore, it provides a progression-based network approach to optimize the overall traffic system performance of all the arterials within the signal network.

This study was to determine whether the network closure constraint in MAXBAND 86 would limit individual arterial performance in network runs. It essentially evaluated how the network closure constraint put additional restrictions on the coordinated progression offsets. A parallel effort was made to investigate the performance evaluation of minimal stops, delay, and fuel consumption as evaluated by the TRANSYT-7F program. Experimental designs were conducted by running MAXBAND 86 on both small and large closed signal networks. The bandwidths obtained from both the single arterials and signalized networks were compared. Then TRANSYT-7F optimization runs were executed for the same signalized networks. Individual runs of MAXBAND 86, TRANSYT-7F, and combined MAXBAND 86–TRANSYT-7F programs were later made to study the effects of different offset optimization schemes. Finally, the equivalent costs were used to compare the delay and stop measurements as recommended in the National Signal Timing Optimization Project.

When individual arteries were optimized separately, the results indicated that they could achieve approximately the same bandwidths as they would under network optimization. This implies that the signal timing optimization for a small network can be best improved by first optimizing individual arteries separately using MAXBAND 86. Then the offsets can be adjusted for each artery to obtain the needed signal timing plan. The comparisons of stops and delay measurements in small networks demonstrate that network progression can be better optimized when the network contains arteries with greater cost penalties for stops and delay. Large networks with arteries having lower penalty costs would provide better solutions when they are optimized as a network. Overall, the study demonstrated that the progression solution obtained from network optimization in small networks is not significantly different from those obtained with single-artery optimization. In effect, network closure constraints prevent individual arteries from obtaining the maximum bandwidths possible if these arterials were optimized separately. On the other hand, the potential gains in progression optimization are significant for large signal networks.

The major criticism of this study is that the full capacity of MAXBAND 86 network optimization may not have been properly evaluated. Two of the most important network optimization features of MAXBAND 86, phase sequence optimization and bandwidth weighting, were not considered. This was because of the computer resource available in the evaluation. One unique advantage of using MAXBAND 86 for optimizing network signal timing plans is that it provides network phase sequence optimization among all other signal timing programs. Because this particular study used only two-phase settings in all cases, it in fact did not examine the full phase sequence optimization capacity of MAXBAND 86. Furthermore, the study specifically stated that the network MAXBAND 86 runs did not provide more arterial progression bandwidths than the separated MAXBAND 86 arterial runs. Examination of the data sets and the signal timing plans may reveal that most signalized intersections have already reached their available maximum green times for progression optimization under current phase sequences. Therefore, MAXBAND 86 would not provide further improvement in network optimization over the use of single-arterial runs.

The other valuable feature of MAXBAND 86 is its capability of providing both intra- and interartery bandwidth weighting options. The intraartery bandwidth weighting, also called "within-artery" or "directional" bandwidth weighting, provides a method to split progression bandwidths within one artery for inbound and outbound travel. In contrast, the interartery or "cross-artery" bandwidth weighting option provides another technique to supply more weights or emphasis on certain arteries than the others. This new feature in MAXBAND 86 can intentionally constrain or enlarge the progression bandwidths in part of the network. Therefore, priority treatments can be made for a particular part of the overall signal network. In this way, the congested part of the signalized network can be emphasized dynamically in the signal timing optimization process. This new capability in MAXBAND 86 can supply a more flexible progression-based network signal timing optimization scheme for urban traffic management.

In summary, it is relatively easy to modify MAXBAND 86 for handling different network sizes and examining various levels of bandwidths weighting. The optimized timing plan in MAXBAND 86 can later be used as the initial starting solution for TRANSYT-7F after all the possible signal phase sequences for optimized network operations have been investigated. The current deficiency of MAXBAND 86 is neither the capability of problem formulation nor the flexibility of the program to model different traffic signal network configurations. Instead, the deficiency lies mainly in its relatively inefficient execution as a result of using the 1973 version of the Mixed-Integer Linear Programming (MILP) code for solving complicated network optimization problems. Significant improvements have been developed in MILP optimization in the past decade. Therefore, it is highly recommended that

1. Heuristic algorithms be implemented in MAXBAND 86 for developing interartery bandwidth weighting in addition to the available intraartery bandwidth weighting approach in the model, and
2. Significant investigations and revisions replace the existing MILP code with another updated MILP code for more efficient optimization execution in MAXBAND 86 in order to benefit from the unique feature of this progression-based network signal timing model.

AUTHORS' CLOSURE

The discussant points out that the scope of the study reported in this paper was limited by the computational resources, which are consumed by the current MAXBAND 86 optimization algorithms. The authors concur in the recommendation to improve the efficiency of these algorithms. We appreciate the discussant's interest in this paper.

Publication of this paper sponsored by Committee on Traffic Signal Systems.

The Interaction Between Signal-Setting Optimization and Reassignment: Background and Preliminary Results

TOM VAN VUREN, MICHAEL J. SMITH, AND DIRCK VAN VLIET

It is well known that signal-setting policies and traffic assignment mutually influence each other. It is not always certain that an equilibrium can be established between both. With the two most commonly used policies, Webster's and a delay-minimizing one, there may be many such equilibria, some of them unstable. A third policy, P_0, has been designed to have substantially better equilibrium behavior. The characteristics of these three policies are discussed as far as their influence on assignment is concerned. An empirical comparison of the behavior of Webster's policy and that of P_0, especially with regard to stability and delays, is presented for a small network. The results for the P_0-policy appear to be promising.

Traffic signals are useful tools in urban traffic control systems. Over the years several signal-setting policies have been developed for isolated junctions, for example, those of Webster *(1)*, Miller *(2)*, and Allsop *(3)*. Usually these policies try to minimize some measure of delay at each junction for the vehicles in the network.

If it is assumed that drivers choose their routes to minimize their own travel time or cost, so that a Wardrop equilibrium results, this kind of signal-setting policy will obviously influence the assignment of traffic over the network, because changes in green times will change costs for the various routes. On the other hand, changes in assigned flows will influence the delays experienced and thus change the optimal signal settings.

This interaction is the basic theme of this paper. The task is to determine a point at which signal settings and assignment are in equilibrium.

NOTATION

- a_{ij} = 1 if turning movement i runs during state j, 0 otherwise;
- C = cycle time;
- d_i = delay at traffic signals for movement i;
- f_i = flow on movement i;
- g_i = signal green time for movement i;
- l_i = link travel times for movement i;
- s_i = saturation flow for movement i;
- T = time period considered; and
- λ_i = green-time proportion for movement $i = g_i/C$.

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TWO WELL-KNOWN POLICIES

Two commonly used signal-setting policies are introduced, both aiming to minimize some measure of delay explicitly.

Webster's Policy

The essence of Webster's policy is

$$\mathrm{Min}_j \ \mathrm{Max}_i \ a_{ij} f_i/(\lambda_i s_i) \tag{1}$$

or, in words, minimize the maximum flow-to-capacity ratio for a movement over all stages by adjusting green times. Unless a boundary is reached (minimum green time, etc.) this policy will try to equalize the flow-to-capacity ratios for the maximally loaded movements in each stage. The effectiveness of this policy lies in the fact that delays increase more than linearly with increasing $f/\lambda s$-values (flow-to-capacity ratios). Therefore it is beneficial to keep the maximum values as low as possible.

Delay-Minimizing Policy

The objective of the delay-minimizing policy is straightforward: minimize the total delay that is experienced at the observed junction *(3)*:

$$\mathrm{Min} \sum_i f_i d_i \tag{2}$$

PROBLEMS

It has been shown theoretically *(4)* that when Webster's policy is used, there may be many equilibria for assignment and signal settings, some of them unstable. Others [Allsop and Charlesworth *(5)*] found empirically that indeed for a certain network there was no unique equilibrium for the delay-minimizing policy: results strongly depend on the initial settings or assignment.

It can easily be explained why these policies do not behave well. Because the policies try to minimize total delay, the most heavily loaded arms of the junction will be awarded the most green time. This, however, will increase delays on other arms, thus "pulling" more traffic to the already heavily loaded arms that received more green. This in turn requires the signal setting to be changed in favor of these same arms, and so forth. So a self-enforcing process results. In this way the objective of the policy (to minimize delay) will not always be met because, in effect, rerouting may even cause the average delays to increase.

This now is the basic deficiency of the traditional delay-minimizing policies: because they do not take into account changes in assignment as a result of the signal-setting policy, their green-time settings are based on outdated flows and as a result are not optimal.

ANOTHER POLICY

Braess's paradox *(6)* shows that, if possible, roads with high marginal costs should be avoided. In this light the signal-setting policy P_0 that Smith *(7)* proposes is very appealing. In essence the objective is

$$\text{Equalize} \sum_i a_{ij} s_i d_i \quad \forall \text{ stage } j \tag{3}$$

For each stage the sum of the experienced delays for all the movements that run during that stage, weighted by their saturation flows, is equalized. Because of this weighting, green time is assigned to the wider roads, even if currently these roads are little used, so that traffic is pushed toward these wider roads. It is proved that under natural but rather severe conditions there will be a single stable equilibrium *(4, 8)*.

The actual goal of this policy is a maximization of the capacity of the network. It is hoped that as a side effect a decrease in delays and travel times will appear, especially at higher levels of congestion. The advantage of P_0 over the traditional delay-minimizing policies is that instead of adjustment of the signals to the current flow situation (without consideration of rerouting) the aim is a future goal, namely, to maximize network capacity by steering drivers toward the wider roads. So rerouting is actually an explicit objective of the policy.

OBJECTIVES OF THE STUDY

Until now, policy P_0 has only been analyzed theoretically, usually with emphasis on stability characteristics. However, another important feature is its influence on the quality of the network; in other words, will it actually cause a decrease in delays?

The aim of this study is to test the various characteristics of policy P_0 by applying it and comparing its results with those of familiar policies. These tests have been made with the simulation and assignment model SATURN *(9)*. The strength of this model lies in the detailed simulation of junctions, which gives more accurate flow and delay curves, together with a Wardrop equilibrium assignment model, as shown in Figure 1.

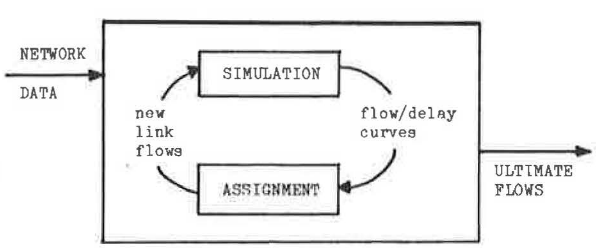

FIGURE 1 Basic structure of SATURN.

The method of testing the various signal-setting policies used here follows naturally from the way in which signals and assigned flows influence each other in reality. After Dickson *(10)* and others, the method is called the iterative optimization reassignment procedure; signal settings and flows are changed alternately until an equilibrium between both is reached (Figure 2).

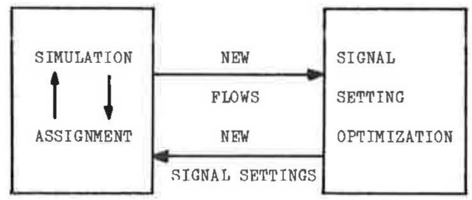

FIGURE 2 Iterative optimization reassignment procedure.

The first tests were carried out on a small network, so the influence of certain network characteristics, such as route lengths, congestion level, and initial signal settings, could be readily distinguished. The effects of policy P_0 compared with the effects of Webster's policy are presented here for the first time, in terms of both the uniqueness and stability of the attained equilibrium and the influence on network delays. The ultimate tests on real-life larger-scale networks are being carried out and will be published later.

RESULTS

The test network (Figure 3) consists of a short, quick route (e.g., through a city center) and a longer but wider route (e.g., a bypass). The main results presented for this network will concern

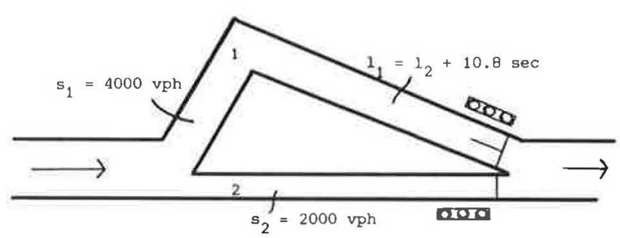

FIGURE 3 Test network.

- Green times at equilibrium,
- Assigned flows at equilibrium, and
- Delays and travel times at equilibrium.

The major assumptions that were made are as follows:

- Cycle time of 60 sec;
- No intergreen times, so the green times add up to the cycle time;
- Two stages, one for each road;
- Minimum and maximum green times of 0.5 and 59.5 sec, respectively;

- Observed time period T of 30 min or 1,800 sec;
- Delays calculated by a three-term so-called sheared delay formula consisting of geometrical delays, random delays, and queueing delays (above capacity).

Note that for this simple network the two policies tested reduce to

$$f_1/(\lambda_1 s_1) = f_2/(\lambda_2 s_2) \quad \text{(Webster)}$$

$$s_1 d_1 = s_2 d_2 \quad (P_0)$$

Green Times at Equilibrium

Figure 4 shows the resulting green times at equilibrium for the two policies tested. It is obvious that with the Webster policy the iterative optimization reassignment procedure always causes the signal settings to reach one of the two extremes (i.e., minimum or maximum green times), but more important is the fact that the actual boundary reached is determined by the initial signal settings. There turn out to be three equilibria for the signal settings when Webster's signal-setting policy is applied—the two extremes and an intermediate, which is unstable. Figure 5 shows these equilibria for the various flow levels. Evidently the two equilibria at the boundaries will lead to totally different delays and flows. The two boundary signal settings will be called Upper Webster and Lower Webster.

The P_0-policy gives rise to an equilibrium at a 35/25 setting for low total flows, changing to a 24/36 setting at a flow of 1,073 vph. At that point, green times for the wider route increase until at a 3,910-vph flow level all green time is assigned to this route.

Flows at Equilibrium

The distribution of green times is strongly related to the distribution of flows over the two routes. Figure 6 shows this distribution of flows and again the Webster policy reaches a boundary, which depends on the initial signal settings.

The flows tend naturally to follow the green times: Upper Webster distributes all traffic to the long, wide route until capacity is reached; then some traffic (about 1 percent) is also distributed to the shorter route according to the Wardrop assignment. Lower Webster distributes all traffic to the shorter route until capacity is reached, which in this case is about 2,000 vph; then some redistribution to the longer route also takes place.

Up to 1,073 vph the P_0-policy distributes all traffic to the narrower and shorter route, although at least 40 percent of the green time is given to the other route. This of course causes nonoptimal travel times, as will be seen subsequently.

Beyond 1,073 vph a redistribution to the longer route takes place and as soon as this occurs, the amount of green time for this route also increases (see Figure 4).

From 1,073 to 3,190 vph, traffic uses both routes, following the assignment of green times; at 3,190 vph all traffic is assigned to the wider route. However, not all the green time is shifted to this route until capacity is nearly reached. This is because of the equality condition for the $s \cdot d$ values and so in this range the P_0-policy is inefficient. Above 4,000 vph (the maximum capacity of the network) a small amount of traffic is again assigned to the shorter route to satisfy the Wardrop conditions.

Summarizing, it is seen that although the P_0-policy does not behave efficiently at all flow levels, at least the structure of green time and flow changes is correct. At low levels all traffic

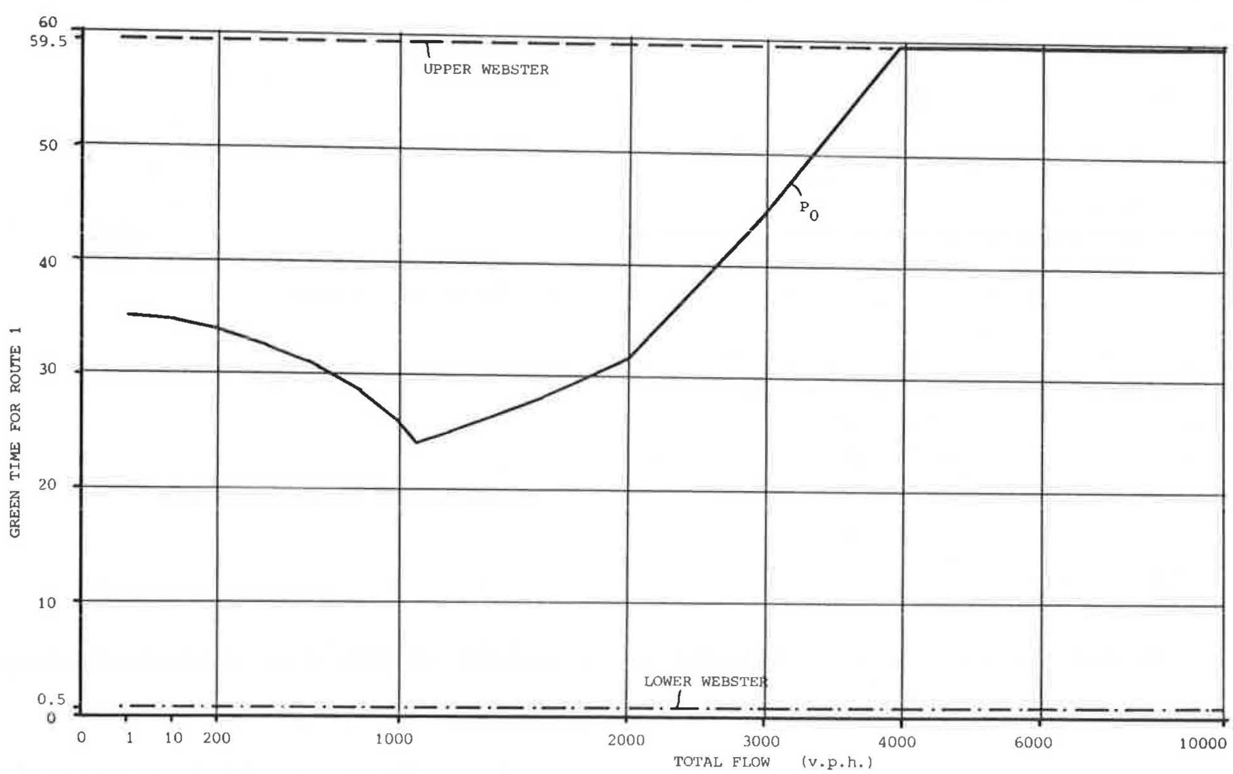

FIGURE 4 Green times at equilibrium.

is assigned to the shorter route, but as flow levels increase toward capacity, a redistribution to the wider route takes place together with a corresponding shift of green time. This is exactly the behavior one would expect from a sound signal-setting policy.

Average Delay and Average Travel Times at Equilibrium

The ultimate test for the performance of the policies is by comparison of their influence on delays experienced and total travel times (the sum of link travel times l_i and delays d_i). It can

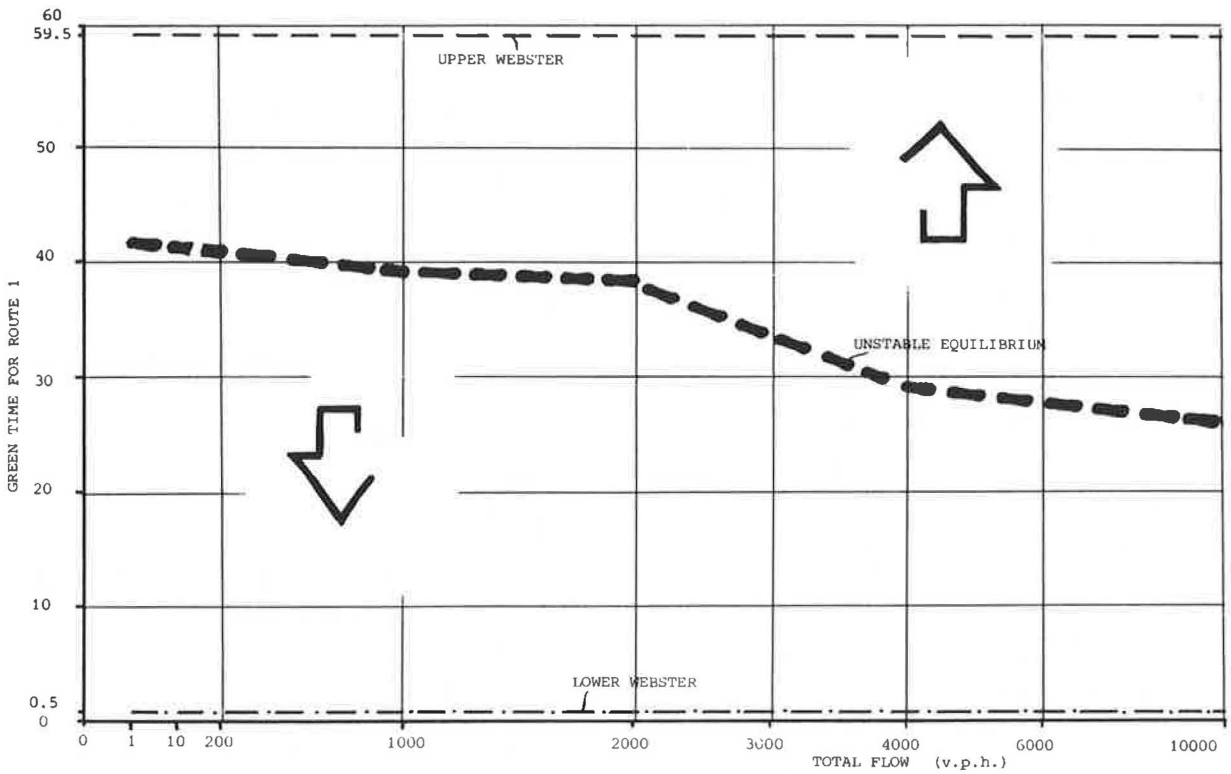

FIGURE 5 Stable and unstable equilibria for the Webster policy.

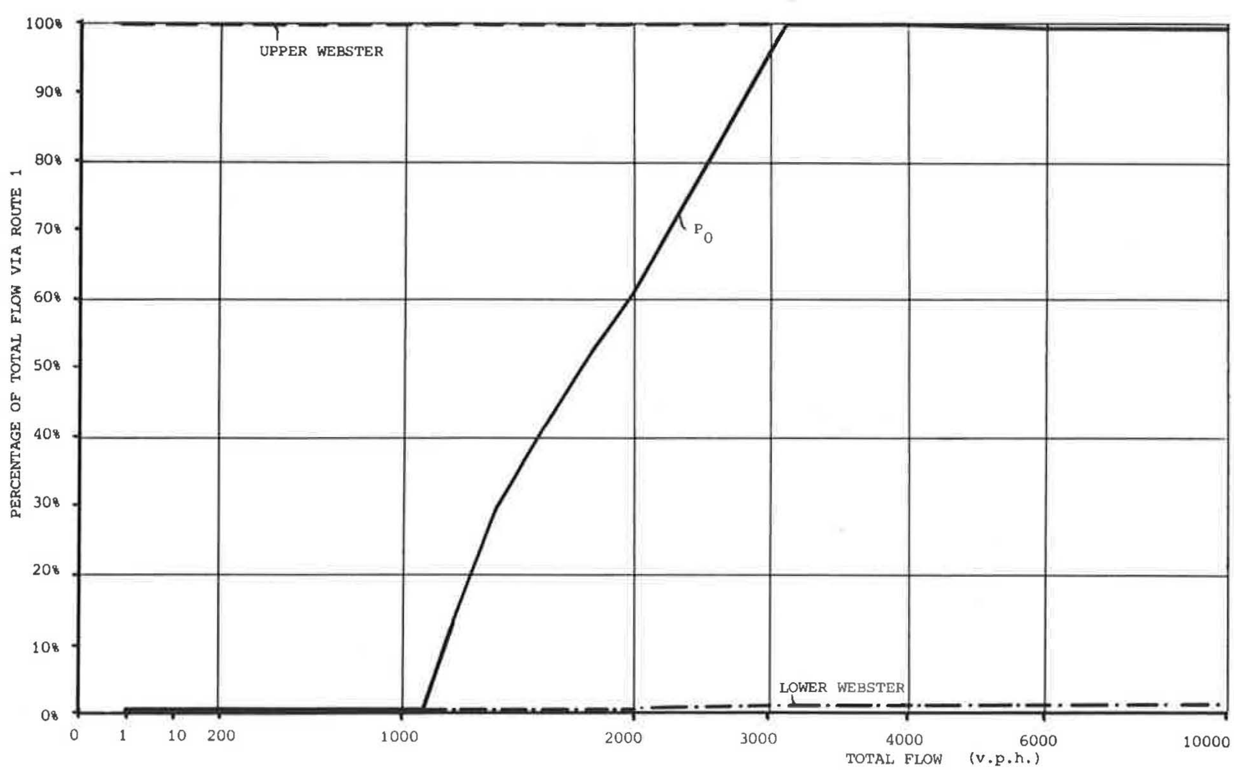

FIGURE 6 Assigned flows at equilibrium.

be seen that the Lower Webster settings give both minimum delays and minimum travel times up to near capacity of the short route (2,000 vph). However, above this (when the flow-to-capacity ratio exceeds 1), queueing increases delays substantially. From a flow of

$$f = \lambda_{max} \cdot s = (59.5/60) \times 2{,}000 = 1{,}983 \text{ vph}$$

The addition of one extra vehicle per hour will cause an increase in delays of

$$0.5T/f = 0.5 \times 1{,}800/1{,}983 = 0.45 \text{ sec}$$

Extra average delay equals half the observed time period (which is the average time a vehicle has to wait if queueing) divided by the total flow. So this lower branch of Webster's policy becomes rapidly worse than either the upper branch or P_0. The high-initiated Upper Webster setting gives all traffic to the wider and longer route and causes minimal delays up to $f =$ 3,967 vph. Delays then also increase rapidly but only at about 0.23 sec per extra vehicle, which is half the rate calculated earlier for the lower branch. Because the wide route is 10.8 sec longer, average travel times will be l_2 + 10.8 sec higher than average delays, as shown in Figures 7 and 8.

Finally, the P_0-policy again shows the most interesting graph. It is no surprise that break points appear at the same places as they do in Figures 4 and 6. Average travel time and average delay are increasing (via the same curve) up to 1,073 vph. Up to about 200 vph average travel times are lower than for the Upper Webster settings because all traffic is assigned to the shorter route (l_2). Above 200 vph delays for the P_0-policy (induced by the "unfavorable" signal settings) are higher than 10.8 sec (which is the extra travel time via the longer route) so that average travel times for the P_0-policy are higher than those for the Webster policy.

At 1,073 vph a redistribution of traffic over both routes takes place, thus decreasing average delays, but because of a redistribution to the longer route, average travel times keep increasing. The gap between delays and travel times keeps widening until at 3,190 vph all traffic is assigned to the longer route (l_1), and the gap is l_2 + 10.8 sec.

To describe the behavior of the P_0-policy at varying flow levels, it can be said that at low flow levels (below 2,000 vph) the policy does not behave efficiently because of a nonoptimal combination of flows and green times. However, the differences with the other policy are limited to some 10 to 20 sec. Above about 2,000 vph the policy performs better than the Lower Webster settings, although still average travel times are some 10 sec higher than those for the Upper Webster settings.

Above capacity (about 4,000 vph) delays increase rapidly. At this stage both the Upper Webster and the P_0-policy perform alike and optimally.

DISCUSSION AND CONCLUSIONS

Conclusions that can be deduced from the test runs on this simple network are as follows:

1. The P_0-policy indeed gives a unique and stable equilibrium for the combined signal-setting optimization and reassignment process.
2. The Webster policy has more than one equilibrium solution; final signal settings and the corresponding flows and delays depend strongly on initial settings.
3. The P_0-policy performs tolerably well with respect to delays and travel times at low flow levels. With increasing flow

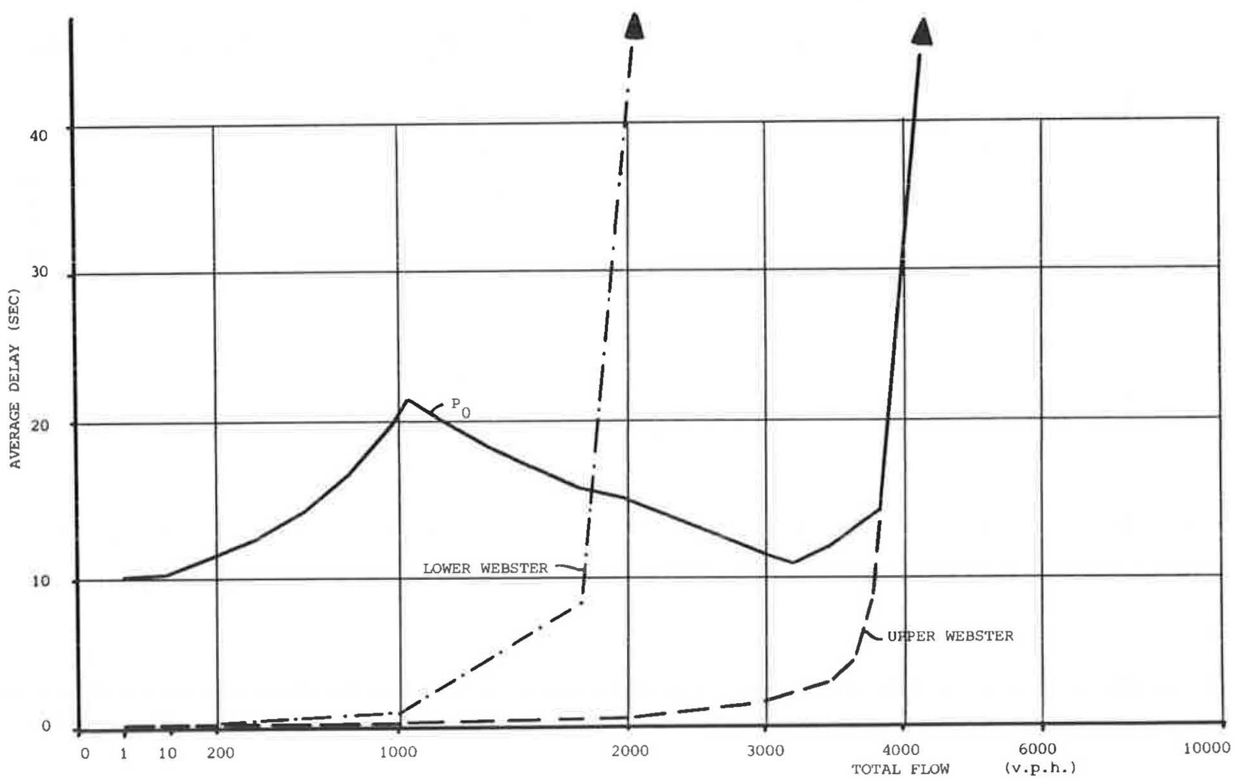

FIGURE 7 Average delays at equilibrium.

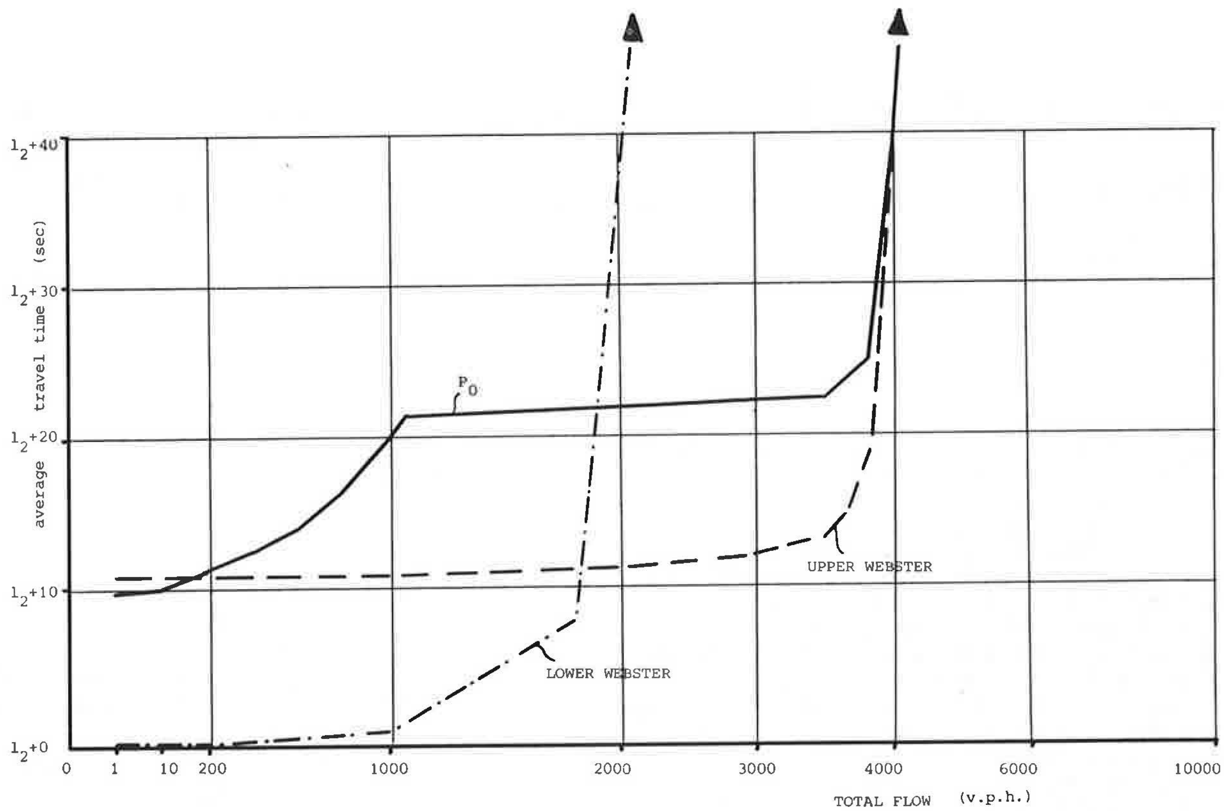

FIGURE 8 Average travel times at equilibrium.

levels, the policy performs better. The policy performs very well, especially above capacity, a confirmation of expectations.

The less-than-efficient performance of the policy at low flow levels is not that disastrous, because delays are small then. Good performance at high flow levels is more important, together with a unique and stable equilibrium. The multiple equilibria that arise with the Webster policy mean that unfavorable initial settings can give very poor results.

The promising results for this simple network may not appear in general. Further tests on larger and more complex networks, to show all the characteristics of the P_0-policy, are being carried out. Some of the first results of these tests were detailed by Smith et al. *(11)*. They appear to show that also on larger networks P_0 performs favorably in comparison with more traditional policies at higher congestion levels. More information can be obtained from the authors.

ACKNOWLEDGMENT

This research project was financed by the Science and Engineering Research Council of Great Britain.

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Publication of this paper sponsored by Committee on Traffic Signal Systems.

Transportation System Management—How Effective? Some Perspectives on Benefits and Impacts

HERBERT S. LEVINSON, MARVIN GOLENBERG, AND KONSTANTINOS ZOGRAFOS

The process of transportation system management (TSM), the nature of its impacts, impact measures, and analysis techniques are described. The use of basic measures such as capacity, travel time, vehicle occupancy, accidents, transit ridership, and costs is emphasized, and it is shown how each can be estimated on the basis of analogy, published relationships, or analytical models. Impact measures are relatively few for any project, not universally required, and have specific interrelationships. Once the primary measures are computed, the secondary ones can be derived as necessary. Most TSM actions deal with localized improvements whose impacts are small in scale and difficult to estimate. Therefore impact assessment techniques should be direct, simple, and in scale with the problems involved, degree of accuracy required, and resources of the community. Impact assessment is a means, not an end. The main goal of TSM is improvement, not analysis.

Transportation system management (TSM) is in transition. Conceived in the mid-1970s as a way of making better use of existing transportation resources, its initial focus was on managing demand—more specifically, reducing automobile trips. Many analytical models were developed to estimate the likely reductions in travel due to demand management, and a broad range of performance measures was identified.

As TSM became more pragmatically oriented in ensuing years, the need to simplify analysis procedures and impact assessments became more apparent. This led to a "problem" focus of TSM, with solutions keyed to problems and use of simple, direct approaches to impact assessment *(1)*. Impact assessment became part of an iterative process that deals with problems, analysis, proposals, and programs.

The nature and scale of TSM impacts are reviewed, impact (performance) measures are suggested, and impact analysis techniques that can be used to assess potential problem solutions are described. The suggested approaches generally are easy to use, produce reasonable results, and focus on specific problems. They are consistent with the scale and needs of short-range actions and the resources of most transportation agencies.

THE TSM PROCESS

The key steps in the TSM planning process flow out of the problems and objectives for any given situation. They include analyzing the problem, identifying likely solutions, screening candidate actions, assessing performance (benefits and impacts), refining or combining actions or both, and developing improvement programs.

Analyze the Problem and Its Setting

The first step is to clearly identify the specific transportation or environmental problems, or both, to be addressed. Is it arterial street congestion along the main artery leading to the city center? Is it inadequate transit service within a growing residential area? Is it ineffective control of driveways along a suburban highway?

A field reconnaissance or "base conditions analysis" will prove useful in answering these questions and in pinpointing problems.

Identify Likely Solutions

Once the problems have been defined, possible solutions should be identified. The solutions should be consistent with the size and nature of the problems, for example, single intersection, entire street, major employment center, or entire region. This also makes it possible to bring appropriate agencies into the planning process and to assess the likely range of impacts.

Screen Actions

The candidate actions should be screened to see whether they are realistic in terms of actual land use, transportation system characteristics, and transportation needs. This may call for reviewing similar situations in the same town or in other communities to screen out obviously inappropriate measures. For example, a bus lane is not appropriate along a section of road that has neither buses nor congestion.

Assess Performance

Actions that survive the screening should be further analyzed in terms of how well they solve the problems. Analysis should focus on primary performance measures that influence transportation service and in turn affect energy consumption and air quality. The choice of primary measures will vary according to specific circumstances and actions, but normally will include

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- System use: number of vehicle and person trips by mode of travel [i.e., transit ridership, car occupancy, traffic volumes, vehicle miles of travel (VMT)]
- System capacity (vehicle and person)
- Service quality (travel times, delays, level of service or VMT)
- Accidents
- Costs (capital, operating, and maintenance)

These measures usually are computed directly. Fuel consumption and emissions can then be derived. Costs should be compared with benefits to see how effective the measures are.

Other relevant factors should be analyzed. Is the solution really workable? Does it reflect community preferences? Will it benefit or adversely affect surrounding shops and activities? What are its political implications?

Combine Actions

In many cases it will be necessary to combine related actions into groups to avoid transferring problems or to attain perceptible time and safety savings. The various impacts of these groups of actions should be reassessed as necessary.

Develop Improvement Program

The last step is to develop a staged improvement program that brings together recommended actions for each time period in a coordinated manner. This program should include schedules for implementation, including costs, responsibilities, and recommendations for supportive actions by various agencies. Assigning priorities should reflect

- Degree of problem and need
- Likely benefits
- Geographic equity
- Coordination with other projects
- Costs

THE NATURE OF TSM IMPACTS

There are important differences between the impact analysis for short-range low-cost improvements and that for long-range transportation improvements. The costs, extent of benefits, and likelihood of generating secondary impacts usually are less for TSM actions.

Impact Scale

Differences in travel time savings illustrate how TSM measures usually vary from major new construction. A new rail transit line might save 2 to 3 min of travel time per mile when it traverses an area that was previously without service. Thus, if it extends for 3 or 4 mi, the total time savings might exceed 10 min. (Chicago's Milwaukee Avenue subway, a diagonal line replacing two legs of a triangle, cut travel times from 22 to 10 min over a 3.5-mi run, a saving of more than 3 min/mi.) But TSM actions normally generate smaller unit time savings and extend for shorter distances. Thus, their total impacts are less.

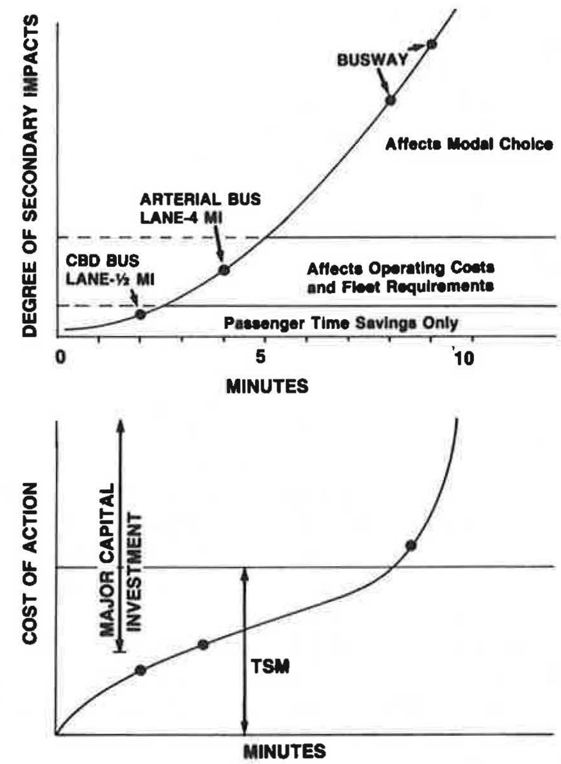

FIGURE 1 Example of impacts and costs.

A traffic signal system improvement that raises speeds from 20 to 30 mph saves 1 min/mi; if it extends for 2.5 mi, the aggregate saving is 2.5 min.

The differences between short- and long-range improvements are apparent from the conceptual relationships shown in Figure 1.

- A 1-mi central business district (CBD) bus lane may save up to 2 min. But this passenger time savings would be too small to modify fleet requirements or to induce changes in travel mode.
- A 4-mi arterial bus lane may save up to 4 min (e.g., 1 min/mi). This time savings might reduce fleet requirements and operating costs. But it is not likely to be perceived as significant on a 30-min trip, and therefore it would not affect ridership or mode-choice decisions.
- A new busway may save 8 min per trip. This time savings generally is sufficient to affect choice of mode. But such a facility normally lies outside the domain of low-cost TSM actions.

Thus, the impact analysis can be simplified once the scale of the primary impact is quantified. This is readily identified from the arterial street bus-lane analysis shown in Figure 2.

- A bus lane will have the primary effects of reducing bus passenger delay and possibly increasing automobile passenger delay. The primary measure becomes net reduction of person delay. This delay reduction is achieved for a certain cost, a second primary measure. (These primary measures are represented by solid lines.)
- If the bus lanes are implemented over an extended distance and the time savings are increased, bus fleet requirements

and operating costs would reduce. Ridership may or may not increase.

- Introducing service changes along with an extended bus-lane operation might increase ridership. The increased bus ridership conceivable could lead to reduced VMT and energy consumption, but in most cases it would not create measurable impacts in these areas.

Impact-Chain Concept

The choice of specific performance or impact assessment measures to use is simplified when the relationships between the primary measures and auxiliary measures are clarified. This is because any given action produces a sequence or chain of impacts. A few of these impacts are basic ones from which the other impacts can readily be calculated.

Consequently, most TSM analysis requires that only the few primary impacts on which the others depend be considered. Table 1 gives examples of impact chains. The numbers in the table denote, in ascending order, the sequence and relative dependency of impacts for each action.

For example, in assessing the effectiveness of widening an intersection (i.e., adding a left-turn storage lane), the basic impacts are increasing capacity and reducing accidents. Reduced delay (or better level of service) and hence reduced vehicle hours of travel (VHT) are a direct consequence of increasing capacity (and lowering the volume-to-capacity ratio). Finally, air quality and energy gains can be computed from the basic impacts.

The impact estimation chain provides a useful guide in planning and analysis. It enables the evaluation procedure to focus on measuring the one or two basic impacts for any given problem solution. This will vastly simplify the analysis, especially when resources are limited. The other measures in the chain can be derived where relevant, treated qualitatively, or otherwise ignored.

SELECTING PERFORMANCE MEASURES

Specific measures of impacts were selected from a review of existing TSM classification schemes, an analysis of candidate actions, a look at how measures relate to commonly encountered problems, and an appraisal of the capabilities of local transit, traffic, and planning staff. Emphasis was placed on the few significant performance measures that address goal achievement or problem solution with respect to the key issues of congestion, mobility, environment, energy, and safety.

A further simplification of the choice of measures is possible when the distinction is made among the three types of measures:

1. Basic measures can be directly estimated or obtained through data collection. These include such measures as capacity, travel time, number of accidents, car occupancy, and cost.
2. Derived measures depend on a basic measure for their calculation. Air quality and energy impacts are commonly derived from values for VMT or VHT. Level of service is derived from traffic signal timing and volume-to-capacity ratios.
3. Intermeasures show relationships between measures, that is, cost per person or minute saved or cost per VMT reduced. The intermeasures are useful in comparing the relative merits of different types of actions.

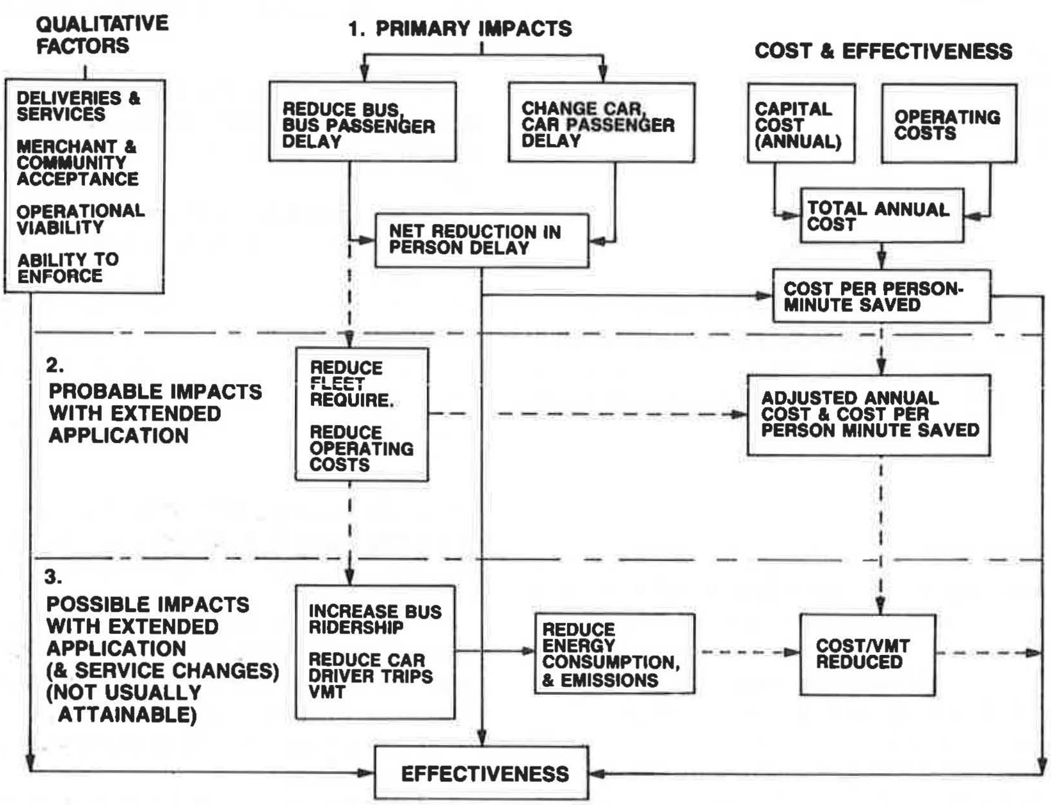

FIGURE 2 Bus-lane impact analysis.

TABLE 1 IMPACT-CHAIN CONCEPT: EXAMPLES

Goal or Impact	Carpool Program	Staggered Work Hours	Reduced CBD Parking Supply	Park-and-Ride Lot	Widened Intersection	Coordinated Traffic Signals	Metered Freeway Ramp	Arterial Bus Lane	Bus-Only Street	Expanded Busway Service
Increase capacity	–	–	–	–	1	–	1[a]	–	–	–
Reduce delay (save time)	–	3	2?	–	2	1	1	1[b]	1[c]	–
Reduce VHT	–	3[d]	3	2	2	2	2	3?	–	–
Reduce car trips	2	2	1	1	–	–	–	3?	3?	3?
Reduce VMT	3	2[d]	3	2	–	–	–	3?	3?	3?
Increase vehicle occupancy	1	–	–	–	–	–	–	3?	–	–
Reduce accidents	–	–	–	–	1	2	2	–	–	–
Improve transit access/service quality	–	2	–	–	–	–	–	2	–	1
Increase transit ridership	–	–	2	2	–	–	–	2?	2?	2
Reduce emissions	4	4[d]	4	3	3	3	3	4?	4?	4?
Reduce energy used	4	4[d]	4	3	3	3	3	4?	4?	4?
Change operating/ maintenance costs	–	–	–	3	–	–	–	3	3	2
Change net subsidy	–	–	–	3	–	–	–	4	4	3
Other										
Reduce peak demand	2	1	–	–	–	–	–	–	–	–
Reduce transit equipment needs	–	2	–	–	–	–	–	–	–	–
Reduce equipment requirements	–	–	–	–	–	–	–	2	–	–
Improve CBD environment	–	–	–	–	–	–	–	–	?	–

NOTE: Numbers denote sequence of impacts. Impact 1 is basic. Impact 2 depends on 1, 3 on 2, and so on. Question mark denotes possible impact. Dashes indicate data not applicable.

[a]Through-lane capacity.
[b]Person.
[c]Bus.
[d]Peak.

SOURCE: H. S. Levinson, unpublished data.

Measures in each category are listed in Table 2. These measures are generally applicable, easily understood, readily quantified, and adaptable to statistical analysis.

The basic measures require data collection or direct estimation. They include the following:

Traffic volume or person flow, from which VMT or person-miles of travel (PMT) can be derived.

Capacity, expressed as persons or vehicles per hour or vehicles per mile (freeway), from which level of service can be derived.

Travel time, expressed as minutes per mile or average speed, from which vehicle or person-hours of travel (PHT) can be derived. Vehicle-hours of delay is a related measure.

Average vehicle occupancy (persons/vehicle).

Safety, expressed as total accidents, from which accident rates can be derived (i.e., accidents per 100 million VMT).

Transit service quality, expressed in terms of service provided and load factors.

Transit ridership, total daily or annual riders by line or system, which can be correlated with the transit hours or miles provided or with the population in the service area.

Capital cost, total and annualized.

Operating and maintenance costs (cost per bus hour or bus mile).

The derived measures depend on the basic measures, such as traffic volumes and speeds:

Level of service is derived from volume-to-capacity ratios, traffic flow densities, or traffic signal timing, or from all three.

Air quality, expressed in terms of the amount of pollutants emitted, depends on traffic volumes and speeds.

Energy consumption, expressed in gallons of gasoline or British thermal units (BTUs) per person or vehicle mile, also depends on traffic flow conditions.

The intermeasures reflect the cost per unit of attainment:

Annual cost per person-minute saved or per VMT reduced.
Gallons of fuel saved per dollar spent or per VMT reduced.

Qualitative factors should also be considered in assessing improvement effectiveness. Will the improvement work? Will it enhance the environment? Will the community accept it? Can it be maintained and enforced? Is it politically feasible? These qualitative factors are commonly viewed as secondary measures, but sometimes they may dominate the decision. They underlie TSM actions such as pedestrian malls or residential street enhancement.

Finally it should be realized that these measures will not apply to every specific problem. The relevance of each will depend on the nature of the problem, goal, or action. A pedestrian mall may improve retail sales, but it will not improve on-time bus performance. Reducing overcrowding on transit vehicles will have little impact on VMT or VHT. A carpool program probably will not affect existing road capacity. The average vehicle occupancy is not meaningful in assessing impact of traffic signal timing changes or intersection improvements. It is necessary to choose the appropriate primary and secondary measures and to discard those that do not apply.

SELECTING IMPACT ASSESSMENT TECHNIQUES

Discussions with public agencies and reviews of the literature produced a broad range of impact assessment techniques. The following criteria should influence selecting and evaluating techniques:

Does the technique provide accurate, reliable, and, above all, reasonable estimates?

Are the estimates consistent with the definition and level of detail needed for the desired impact?

Is the technique sensitive to the scale of the TSM action?

Does the technique account for interactions among different TSM actions that might be implemented as a group or package?

Can the estimates be used directly to assess the effectiveness of TSM actions, or must the estimates be transformed?

Are the data requirements of the technique within the existing resources of identified classes of users, or are special collection efforts required?

Does application of the technique by many users require the assistance of other agencies?

Does the staff of most public agencies have the time and skills necessary to learn and understand the technique?

Is an application of the technique easy to document, allowing the quick assessment by other staff of changes and refinements of proposed TSM actions?

Can the technique be applied (including any necessary calibration steps) within the time limitations imposed by meeting, hearing, and documentation schedules?

In sum, estimation methods should be easy to use, produce

TABLE 2 PRINCIPAL PERFORMANCE MEASURES

Measure	Parameter	Remarks
Basic		
Capacity	Persons/hour, vehicles/hour or passengers/car unit/hour, vehicles/mile (freeways)	Base on peak 15-min flow rate
Travel time	Minutes/mile, vehicle hours of travel (VHT), person-hours of travel (PHT), delay (sec/person or vehicle), average speed	Applies to cars and transit
Vehicle miles of travel (VMT)	Volume (i.e., car trips), volume times distance	Volume is a basic input or surrogate
Average vehicle occupancy	Persons/car, persons/transit vehicle	–
Safety	Accidents/year, accidents/100 million VMT, accidents/vehicle entering, intersection or volume product	May refine by type or severity of accident or both
Transit service quality	Coverage (percentage of population within 1/4 or 1/2 mi), passengers/seat or ft^2/passenger, peak and off-peak; bus miles/1,000 residents	Transit travel time is a complementary measure
Transit ridership	Daily or annual riders (annual rides/capita in service area, daily riders/bus mile or bus hour)	–
Capital cost	Annualized capital cost in dollars	–
Operating and maintenance cost	Annual cost in dollars (cost/bus or car mile, cost/bus or car hour)	Employees/transit vehicle is surrogate
Net cost of service	Annual transit subsidy in dollars, percentage of operating costs covered by fares (subsidy per passenger in cents)	Similar measures apply for parking facilities; key factor is coverage ratio: net annual income to annual debt service
Derived		
Air quality	Emissions in grams of HC, CO, NO_2 (emissions/mile)	Volume/speed or volume × (min/mi) is a good surrogate
Energy	Gallons of gasoline, BTUs (megajoules), BTUs/vehicle mile (BTUs/person mile)	–
Level of service	Avg stopped delay or vehicles per mile	–
Intermeasures		
Annual cost/unit of attainment	Cost/increase in vehicle or person capacity, cost/person or vehicle minute saved, cost/increase in transit ridership (i.e., cost per additional rider), cost/accident reduced, cost/VMT reduced, cost/gallon saved	–
Benefit-cost ratio	Discounted ratio of benefits to costs	–

SOURCE: H. S. Levinson, unpublished data.

TABLE 3 PRINCIPAL IMPACT TECHNIQUES KEYED TO PERFORMANCE MEASURES

Performance Measure	Impact Techniques
Capacity	2, 3
Travel time	1, 2, 3, 4, 11
Vehicle volume/VMT	5, 6
Avg vehicle occupancy	1, 11
Safety	1, 2, 11
Transit service quality	2, 3, 6, 7
Transit ridership (mode share)	1, 5, 6, 11
Air quality (emissions)	9
Energy	10
Capital cost	1, 12
Operating and maintenance cost	8
Net cost of service	2, 6, 8
Level of service	3, 4

NOTE: Impact techniques are as follows: (1) analogy and experience, (2) design specification, (3) capacity analysis, (4) speed-flow analysis, (5) mode-choice models, (6) elasticity factors, (7) transit performance analysis, (8) transit operating and maintenance cost analysis, (9) speed versus emissions, (10) speed versus fuel consumption, (11) before-and-after statistical comparison, and (12) engineering cost estimates.

reasonable results, and provide reliable answers (estimates) to specific problems.

The major impact assessment techniques can be grouped into three overall categories that reflect the amount of information available.

1. For situations in which detailed local data are available, equations or analytical models can be applied to predict impacts directly. Procedures in this category include modal-choice analysis, pivot-point procedures, and selective disaggregate behavioral demand modeling. These techniques are most accurate where they directly relate impacts to system characteristics or to changes in these characteristics. Yet, for many TSM actions, the cost of application is not justified by the low-cost nature of the action itself.

2. Where less local information is available but statistically valid information on observed results has been synthesized, tabular values or graphs showing a range of experience can be applied. Care must be taken in using these techniques to be sure that the local conditions are comparable with those reported.

3. For TSM actions that have not been extensively applied (as is often the case), the existing data base is insufficient for the calibration of models or relationships to directly predict their impacts. For such actions or impacts, therefore, an "analogy" approach can be used, transferring data from a limited number of case studies to illustrate general impacts. The analogy method is useful in many cases either to predict general impacts or to verify the impacts obtained from analytical methods.

The principal impact techniques can be grouped into the following categories: analogy and experience, design specification (i.e., specifying future performance), capacity analysis, speed-flow relationships, mode-choice models, elasticity factors, transit performance analysis, transit operating and maintenance cost analysis, speed-emission-energy relationships, statistical tests that compare before-and-after conditions, and engineering cost estimates. Table 3 shows how these techniques relate to the various performance measures.

Analogy and Experience

Available experience provides a powerful tool for assessing impacts of most improvements. This method includes a broad array of look-up tables and charts that summarize and synthesize the state of the art. Site-specific parameters can transfer one community's impacts to an analogous situation. Analogy is the most practical method for assessing changes in accident rates, that is, accident reduction factors. It is also valuable in providing first-order estimates of installation costs. Typical examples include reported time savings for a one-way street system, likely market penetration of a staggered-hours program, and the increased vehicle occupancy resulting from a high-occupancy-vehicle (HOV) lane. An example is given in Table 4, which shows costs of freeway priority-lane projects.

Design Specification

In the design approach, the impacts are inherent in the solution; that is, standards desired for a particular improvement are based on design or simulation. Net benefit or change is then estimated by comparison with existing conditions. This approach is commonly applied to actions that involve transit or traffic improvements.

For example, average travel times along an arterial street might approximate 3.5 min/mi. A time-space diagram analysis of a coordinated traffic signal system would yield progressive speeds of 30 mph, or 2 min/mi. The anticipated savings would amount to 1.5 min/mi.

Capacity Analysis

Values, relationships, and adjustment factors for highways, transit and pedestrian capacities, and service levels are set forth in the 1985 *Highway Capacity Manual (3)*. Techniques for signalized intersections include both capacity computations and level-of-service analysis.

- The capacity of any lane group at a signalized intersection depends on the number of effective moving lanes, traffic signal timing, and saturation flows (or vehicle headways).
- The level of service is defined by the average stopped delay in seconds per vehicle. The delay depends on the volume-to-capacity ratio, traffic signal cycle length, green time, and the quality of the traffic signal progression.
- Changes in intersection capacity can be approximated by comparing the lane-seconds of green available before and after an improvement.

Speed-Flow Relationships

Speed-flow relationships based on the 1985 *Highway Capacity Manual* and earlier editions of this manual show how speeds decrease as the volume-to-capacity ratios increase. They can be

TABLE 4 COSTS OF FREEWAY PRIORITY-LANE PROJECTS (2, p. 45)

Project	Capital Cost ($)	Cost per Mile ($)	Annual Operations and Maintenance Cost ($)
With-flow lanes			
Boston, Southeast Expressway	91,500	11,400	194,000
Los Angeles, Santa Monica Freeway	163,000,[a]	13.000	
	358,000[b]		Unknown
San Francisco, US-101	25,000[a]	7,000	Negligible
Miami, I-95	18,500,000[c]	2,500,000	88,000
Honolulu, Moanalua Freeway	10,000[a]	3,700	Negligible
San Francisco, Oakland Bay Bridge	50,000[a]		
	350,000[d]		28,000
Portland, Banfield Freeway	2,100,000[e]	780,000	Unknown
Contraflow lanes			
Boston, Southeast Expressway	40,000	5,000	137,500
New York, I-495 Lincoln Tunnel Approach	700,000	280,000	200,000
New York, Long Island Expressway	44,000	22,000	150,000
San Francisco, US-101	180,000	45,000	60,000
Separated HOV express lanes			
Washington, D.C., Shirley Highway	28,000,000–43,000,000[f]	2,500,000–4,000,000	Unknown
San Bernardino busway	56,000,000[g]	5,000,000	Unknown

[a]Signing and marking.
[b]Marketing.
[c]Including freeway widening but excluding park-and-ride lot.
[d]Special signal system.
[e]Includes freeway widening and other roadway improvements.
[f]Depending on assumptions.
[g]Including park-and-ride lot.

used to estimate the changes in travel time (minutes per mile) resulting from expanding capacity or reducing demand. They also provide input for energy and air quality impact analysis.

Table 5 shows how the travel time on freeways increases as the volume (or volume-to-capacity ratio) increases for 50, 60, and 70 mph average design speeds. An example is as follows: For a design speed of 70 mph and a V/C ratio of 0.60, the average travel time is 1.05 min/mi. If the V/C ratio increases to 0.80, the average travel time rises to 1.15 min/mi.

Mode-Choice Estimates

The choice of travel mode can be estimated by a variety of methods. These include full mode-choice models, direct-demand estimates, pivot-point methods, and elasticity factors.

Mode-Choice Models

The mode-choice models normally require detailed origin-destination information and detailed descriptions of travel times, costs, and utilities for each trip interchange. They are best suited for long-range demand forecasting, although they may be useful in testing areawide transportation system policies. However, from the perspective of obtaining quick, meaningful, and realistic assessments of localized, fine-grained changes, they do not appear practical. The many assumptions and weights associated with estimating disutilities, as well as the cost and complexity of their application, further limit their usefulness for early action, low-cost service changes. Thus, the use of full mode-choice models is warranted only when actions are expected to produce major changes in existing services or when major new services are introduced.

Direct-Demand Estimates

Direct demand is estimated when new service is introduced to a corridor or area and when transit ridership is expected to have minimum impact on automobile trips. The method calls for

TABLE 5 FREEWAY SPEED-FLOW RELATIONSHIPS: TRAVEL TIME VERSUS VOLUME–CAPACITY RATIO (3, Table 2-5)

Passenger Cars/Lane/ Hour	Volume-to-Capacity Ratio	Estimated Minutes per Mile by Design Speed		
		70 mph	60 mph	50 mph
800	0.40	0.97	1.16	1.29
900	0.45	0.99	1.18	1.31
1,000	0.50	1.07	1.20	1.33
1,100	0.55	1.03	1.22	1.35
1,200	0.60	1.05	1.24	1.37
1,300	0.65	1.07	1.26	1.39
1,400	0.70	1.09	1.29	1.42
1,500	0.75	1.11	1.34	1.45
1,600	0.80	1.15	1.39	1.48
1,700	0.85	1.20	1.45	1.56
1,800	0.90	1.27	1.62	1.71
1,900	0.95	1.42	1.79	1.90
2,000	1.00	2.00	2.00	2.14
2,000+[a]	>1.00	3.00	–	–

[a]Assumed for breakdown conditions or future demand conditions.

estimating the number of people in the proposed service area and their likelihood of riding transit *(4)*. Market and employer surveys and analogy methods will prove useful in estimating the market penetration of the new transit service.

Elasticity Factors

Elasticity factors can be used to assess the impact of changes in transit service, fares, or parking costs. The factors are easy to understand and use and provide a quick response to particular transportation changes in which minor to moderate impacts are expected. Care should be exercised in their use because of the wide range of particular factors from place to place and, in some cases, the limited data base. A 100 percent increase in fares, headways, population coverage, or bus miles is likely to produce the following changes in transit ridership based on current experience:

Type of Increase	*Change in Ridership (%)*
Fares	–40
Headway	–40 to –60
Coverage	+60 to +90
Bus miles	+70 to +100

Transit Performance Analysis

Existing transit performance can be based on field observations of speeds and delays, running-time checks, and passenger counts at maximum load points. Future performance can be estimated by assuming changes in key variables. The values shown in Table 6 can be used to estimate the effects of reduced traffic congestion or frequency of stops.

Transit Operating and Maintenance Cost Analysis

Operating and maintenance costs are specific to a given community at a given point in time. Transit operating costs, in particular, must be kept current to reflect changes in wage rates and fuel prices. Transit costs can be estimated by two basic methods or models:

1. Costs can be allocated to bus (or rail car) hours, bus or car miles, or peak vehicles, or all three. One-, two-, or three-variable equations can be derived of the form Cost = A (bus hours) + B (bus miles) + C (peak vehicles). This is the most common method, although it may not accurately estimate the costs of small-scale system changes.
2. Costs can be allocated to drivers (trainmen) and bus or car miles. This approach provides relatively precise cost estimates whenever service changes require extra drivers and vehicles:

Cost = (drivers) × (wage rate/driver) + bus miles × [nondriver costs/bus (car) mile]

Operating and maintenance costs for bus priority facilities, reversible-lane operations, carpooling programs, and other actions can be estimated from current experience.

Speed-Emission-Energy Relationships

Air quality and energy benefits are realized whenever the amount of travel, travel times, or traffic densities decrease. This calls for estimating the travel-time savings of specific improvements. Such estimates can be based on (a) direct before-and-after studies of actual conditions, (b) expected benefits of specific actions, or (c) V/C–travel-time relationships.

Illustrative relationships among average speed, fuel consumption, and emissions are shown in Table 7. Tables such as this can be used to estimate the energy and air quality savings from improvements in street system efficiency. Table 7 shows that an increase in speed from 15 to 20 mph would

- Save 1.0 min/mi.
- Reduce fuel consumption from 0.0825 to 0.0725 gal/mi, a savings of 0.0100 gal/mi.

TABLE 6 BUS TRAVEL TIMES AND SPEEDS AS A FUNCTION OF STOP SPACING AND TRAFFIC CONGESTION

				With Traffic Delays (peak conditions)					
		Without Traffic Delays		Central Business District: 3.0 min/mi delay		Central City: 0.9 min/mi delay		Suburban: 0.7 min/mi delay	
Time per Stop (sec)	Stops per Mile	Travel Time (min/mi)	Speed (mph)	Travel Time (min/mi)	Speed (mph)	Travel Time (min/mi)	Speed (mph)	Travel Time (min/mi)	Speed (mph)
10	2	2.40	25.0	5.40	11.1	33.30	18.2	3.10	19.4
	4	3.27	18.3	6.27	9.6	4.17	14.4	3.97	15.1
	6	4.30	14.0	7.30	8.2	5.20	11.5	5.00	12.0
	8	5.33	11.3	8.33	7.2	6.23	9.6	6.03	10.0
	10	7.00	8.6	10.00	6.0	7.90	7.6	7.70	7.8
20	2	2.73	22.0	5.73	10.5	3.63	16.5	3.43	17.5
	4	3.93	15.3	6.93	8.8	4.83	12.4	4.63	13.0
	6	5.30	11.3	8.30	7.2	6.20	9.7	6.00	10.0
	8	6.67	9.0	9.97	6.0	7.57	7.9	7.37	8.1
	10	8.67	6.9	11.67	5.1	9.57	6.3	9.37	6.4
30	2	3.07	19.5	6.07	9.9	3.97	15.1	3.77	15.9
	4	4.60	13.0	7.60	7.9	5.50	10.9	5.30	11.3
	6	6.30	4.5	9.30	6.5	7.20	8.3	7.00	8.6
	8	8.00	7.5	11.00	5.5	8.90	6.7	8.70	6.9
	10	10.33	5.8	13.33	4.5	11.23	5.3	11.03	5.4

SOURCE: H. S. Levinson, unpublished data.

TABLE 7 EFFECT OF SPEED ON ENERGY AND AIR QUALITY *(5, 6)*

Avg Speed (mph)	Avg Travel-Time Rate (min/mi)	Fuel[a] Economy (mpg)	Fuel Consumption Rate (gal/mi)	1977 Emissions (g/mi)		
				NMHC	CO	NOx
10	6	9.76	0.1025	6.8	95.6	20.5
12	5	10.8	0.0925	5.8	80.0	2.5
15	4	12.1	0.0825	4.7	63.9	2.6
20	3	13.8	0.0725	3.8	49.4	2.8
25	2.4	15.0	0.0665	3.2	40.3	3.0
30	2	16.0	0.0625	2.7	33.4	3.2
35	1.7	16.7	0.0060	2.4	28.3	3.3
40	1.5	17.4	0.0575	2.1	24.8	3.4

[a]Based on composite VMT-weighted mix of automobile weights in the 1976 U.S. fleet. Not corrected for cold starts.

- Reduce HC emissions from 4.7 to 3.8 g/mi, a savings of 0.9 g/mi.
- Reduce CO emissions from 63.9 to 49.4 g/mi, a savings of 14.5 g/mi.
- Increase NOx from 2.6 to 2.8 g/mi, a gain of 0.2 g/mi.

This table is straightforward to use and provides a good order-of-magnitude assessment of impacts. Detailed emission and fuel consumption factors by vehicle type, speed, and temperature are available and should be used when greater accuracy is desired. Methods are also available for estimation of impacts of starts and stops. In assessing impacts, it is important to use the most recent data on the highway and bus fleets.

Before-and-After Statistical Comparisons

Before-and-after comparisons are important to show community leaders and the general public the benefits of improvements and thereby attain support for improvement programs and to assess the statistical significance of specific improvements or improvement programs. Published before-and-after studies, such as those distributed through UMTA's Service and Methods Demonstration Program, provide a good basis for analogy models.

Engineering Cost Estimates

Initial estimates of capital and operating costs can be obtained from previous estimates for similar projects or from information contained in *Characteristics of Urban Transportation Systems (7)* or similar documents. However, because costs vary with each specific project, care should be exercised in transferring cost data. Ideally, cost estimates should be site specific.

APPLYING ANALYSIS TECHNIQUES

The choice of methods and application procedures will depend on the intended use of the results and on the information base and other resources available for estimating the performance and impact measures required for design and evaluation. Limited data, planning budgets, time, staff availability, skills and experience, and access to computers all place restrictions on the methods and procedures that can be applied. The restrictions are usually apparent, although the best approaches to dealing with them may not be.

A general guide is to quantify as few impacts as necessary. However, relevant qualitative factors should be carefully considered.

The level of detail and desired accuracy will be influenced by factors such as these:

1. Size of likely impact: Small changes in performance and other measures are difficult to predict with confidence; they are often smaller than the errors inherent in both the estimation procedure and the observed data.
2. Sensitivity of design features: Capacity measures vary in sensitivity to estimated values as a result of their nature. A crude estimate of patronage, for example, might indicate that two buses were required for a suburban feeder service. If that service design does not change with a 40 percent lower or higher estimate of patronage, the crude estimate is adequate for the analysis.
3. Scale of action: More accurate estimates are generally required for expensive actions or actions with relatively long service lives, because mistakes in changing these actions are likely to be costly or difficult to remedy.
4. Ability to fine tune: Many TSM actions can be modified after implementation when direct measurements of performance can be made.
5. Trade-offs among impacts: Changes in transportation performance may create adverse impacts that should also be identified. One example is removing curb parking to create a bus lane along a street that has many small shops and no off-street parking. Conversely, a pedestrian street will improve the amenity, but it may affect goods delivery and parking garage access.

EFFECTIVENESS OF TSM

The effectiveness of TSM actions has varied widely. The potential time savings generally depends on the amount of congestion experienced before an improvement has been implemented. The greater the congestion, the greater the benefits. Coordinating traffic signals for a 30-mph progression will save 4 min/mi if the initial speed was 10 mph, but only 1 min/mi when the initial speed was 20 mph.

Examples of impacts estimated from a literature review, ongoing studies, and actual experience are the following:

1. Person and vehicle capacity gains
   On-street parking controls, 50 to 100 percent;
   General traffic improvements (typical), 10 to 20 percent; and
   Express transit service, 0 to 20 percent.
2. Travel-time savings
   Bus malls, 2 to 5 min/mi;
   Bus lanes on city streets, 1 to 5 min/mi;
   On-street parking controls, 0.2 to 2.4 min/mi;
   Traffic signal improvements, 0.4 to 1.6 min/mi;
   Bus lanes on freeways, 0 to 1.2 min/mi;
   General traffic improvements, 10 to 20 percent;
   Bus lane around major queue, 3 to 5 min;
   One-way toll collection, 2 to 3 min/car;
   HOV ramp bypass, 1 to 3 min/vehicle;
   Transit service coordination, 0 to 12 min/trip; and
   Express transit service, 2 to 5 min/trip.
3. VMT reductions (estimates)
   Automobile-free zone, up to 20 percent reduction across screenline;
   Bridge tunnel tolls, 2 to 5 percent reduction per affected crossing;
   Gas tax (+$0.10), 2 percent areawide reduction; and
   Areawide surcharge of $0.50 on licenses, 0.7 to 1.3 percent reduction (Manhattan).
4. Cost-effectiveness
   Carpools, $20 to $51/pool;
   Traffic signals, 2¢/VHT reduced;
   Staggered work periods, 25¢/VHT reduced (suburbs);
   Ramp metering, $1.00/VHT reduced; and
   Park-and-ride, 2 to 3.5¢/VMT reduced.

These examples provide a guide for making initial estimates and checking detailed calculations for reasonableness. Significant findings are as follows:

- Many actions have major impacts over a very localized area. It is hard to derive areawide impacts from the application of these actions, although site-specific impacts can be readily quantified.
- Traffic engineering improvements can increase capacity up to 100 percent, with 10 to 20 percent gains common. Travel-time reductions of 20 percent can translate into energy and air quality benefits.
- Demand management measures can achieve reductions in VMT up to 5 percent at specific locations on the basis of theoretical studies of travel elasticities and carpool formation. An effective ridesharing program, for example, would reduce VMT an estimated 0.2 percent in suburban areas and 0.1 percent in a large city like New York or Chicago; costs would average about 2¢/VMT reduced and about $20 to $50 per capita.
- Bus lanes can save bus passengers from 1 to 5 min/mi, depending on the amount of congestion.
- Bus bypass lanes at multilane freeway ramps will save bus passengers from 1 to 3 min per ramp, depending on the amount of congestion.
- Transit improvements will increase ridership, but at a rate less than the amount of additional service provided. A 2 percent gain in bus mileage would result in a 1 to 1.5 percent gain in riders, of which up to about one-half might be former motorists. Express transit extensions could increase corridor passenger capacity up to 20 percent and save passengers 2 to 5 min per trip.

IMPACTS IN PERSPECTIVE

In the preceding sections key impacts to be assessed have been identified and the commonly used methods for assessing benefits and impacts have been reviewed. The approaches provide a realistic basis for screening and evaluating options and, in a broader sense, formulating coordinated improvement programs.

The suggested impacts focus on basic factors such as capacity, travel time, accidents, transit ridership, and costs. The use of as few measures as possible is desirable to simplify rather than to complicate the evaluation process. The impact-chain concept supports this approach and provides one means to identify the few primary impacts that should be measured.

Impact measures are relatively few in number for any project, are not universally required for all problems, have a sequence of importance that varies according to the problem, and have specific interactions that enable a large subset to be derived from a few basic measures.

The effects of traffic engineering actions on speed, delay, and accidents are well documented in terms of both experience and analytical approaches. Transit ridership estimates can be derived from elasticity data, although there may be variations in the results. Actions that involve restraining or reducing motor vehicle travel have not been implemented in most cities, and the models used to predict their impacts give widely varying results. The data base for assessing impacts by analogy or by comparison with similar situations is limited.

There is need to expand the existing data base in three important ways: (a) better compilation of before-and-after experience of various improvements, (b) improved stratification of accidents by type and road or traffic condition, and (c) good capital cost data. More information of this type is needed to promote the benefits of specific actions.

Most TSM actions deal with localized improvements that involve fine-grained changes to the transportation system. Their impacts are small in scale and may be difficult to estimate in practice, and their statistical significance cannot be detected.

Impact assessment techniques, therefore, should be in scale with both the problems and the resources of the community. Simplicity and responsiveness are the underlying themes. Impact assessment is a means, not an end.

This implies adopting pragmatic approaches to identifying and assessing actions and formulating coordinated improvement programs. It calls for translating concepts and analysis into productive improvements, for viewing TSM as an action program, not merely as a planning process. It calls for streamlining the impact analysis by using methods that are consistent with the degree of accuracy required and the capabilities of communities.

ACKNOWLEDGMENTS

This paper builds on the technical memoranda and final report for NCHRP Project 7-11, *NCHRP Report 263: Simplified Procedures for Evaluating Low-Cost TSM Projects—User's Manual (1)*, and *TSM in Connecticut (8)*.

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Publication of this paper sponsored by Committee on Transportation System Management.

Development of a Traffic Modeling System for Detour Planning on the Downtown Seattle Transit Project

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A major transit subway project is being constructed in downtown Seattle, Washington. A 1.3-mi electric-bus tunnel and associated surface-street improvements are in the final design phases and initial tunnel construction has begun; the expected completion date is 1990. The Downtown Seattle Transit Project (DSTP) was initiated by Metro Transit, the city of Seattle, and UMTA to help relieve existing traffic congestion in downtown Seattle and to provide capacity for growth. The tunnel will have three underground stations as well as combined station and staging areas at each end of the alignment. Both cut-and-cover and tunnel boring construction techniques will be utilized on the project. One of the greatest consequences of such a major construction project in a central business district (CBD) can be the adverse impacts on CBD traffic. An important task for project planners has thus been to assess the likely impacts of construction on traffic and to develop traffic maintenance plans that will best facilitate the tunnel construction and keep traffic impacts to a minimum. An innovative and complex traffic modeling system has been developed to aid in this task. Based on three existing traffic planning software programs (LINKOD, MINUTP, and TRANSYT-7F), a modeling chain has been developed that provides a systematic means for assessing the impacts of street closures, detours, and other traffic restrictions; identifies potential "hot spots"; and facilitates the development of traffic control plans to mitigate these impacts. The development and calibration of this modeling system, which has several innovative features likely to be of interest to other traffic modelers, are described. The modeling system organizes the analysis of traffic maintenance schemes as well as provides an ongoing tool for helping to design the longer-range (design-years) traffic improvements. Also included in the paper is a discussion of the effort involved in developing the modeling system and some suggestions for further research to improve the system for future applications.

In an effort to relieve existing traffic congestion in downtown Seattle, to stimulate and meet projected transit ridership demand, and to ensure that the transportation system will have the capacity to accommodate future growth, Metro, the city of Seattle, and UMTA initiated the Downtown Seattle Transit Project (DSTP). DSTP consists primarily of a 1.3-mi electric-bus tunnel and associated surface-street improvements. The tunnel alignment and station locations are shown in Figure 1. The tunnel route generally follows Pine Street west from Interstate 5 in a cut-and-cover structure to Westlake Station in the retail district. The route then runs under Third Avenue in twin-bore tunnels through two additional stations to its southern terminus at the old Union Railroad Station. At either end of the alignment, combined station and staging areas will connect with surface streets and the Interstate highway system via exclusive ramps.

An inevitable consequence of a central business district (CBD) construction project as large as DSTP is the adverse impact on CBD traffic. An important task of DSTP is to assess these expected impacts and to develop traffic maintenance plans that would facilitate the tunnel construction and minimize traffic impacts. A benefit of the project is to provide the city of Seattle with a microcomputer-based assignment package of the downtown area.

OVERVIEW OF MODELING SYSTEM

In order to identify a.m. and p.m. peak-hour traffic management plans, three different transportation and traffic

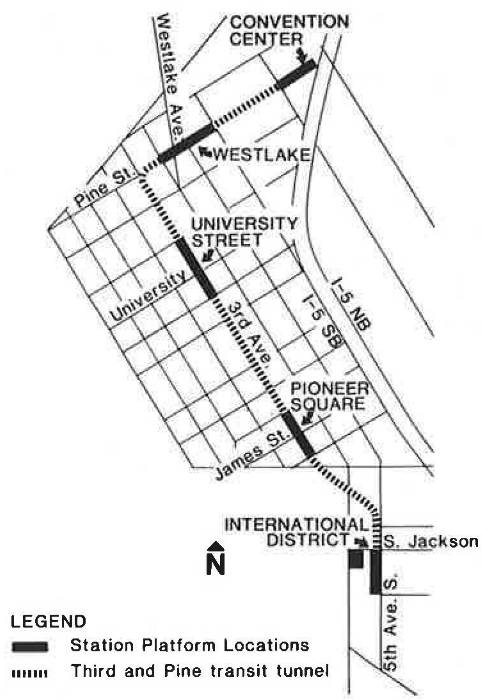

FIGURE 1 DSTP tunnel alignment and station location.

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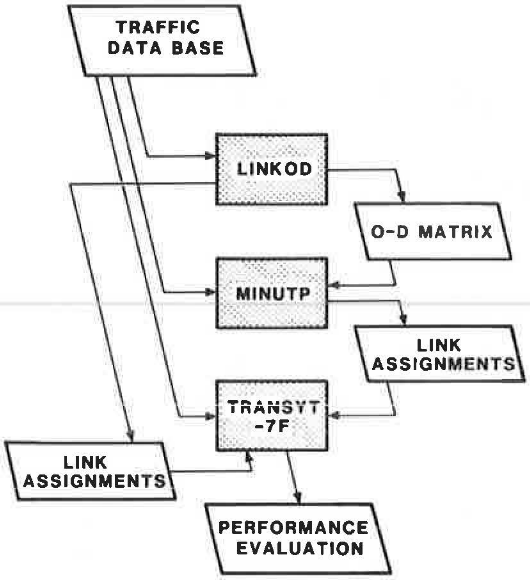


FIGURE 2 DSTP traffic modeling system.

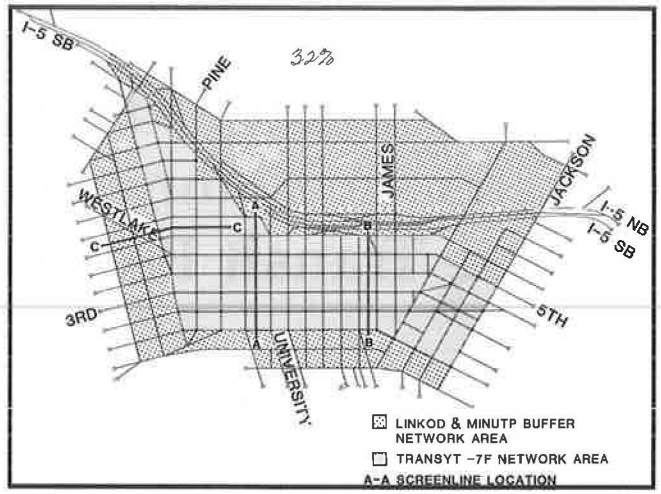


FIGURE 3 DSTP traffic model network area and screenline locations.

engineering software packages were used sequentially to assess the traffic impacts of construction-related street closures: LINKOD, MINUTP, and TRANSYT-7F. The overall modeling system structure is outlined in the flowchart in Figure 2. The first program, LINKOD, was used to synthesize a trip table for downtown Seattle by using an initial estimate of trips generated and attracted to 253 centers of activity (e.g., parking facilities), along with traffic count information and street network characteristics. The LINKOD/MINUTP network area (Figure 3) consists of the primary CBD plus a fringe area to act as a buffer zone between points of traffic loading and the CBD streets of interest.

To validate the LINKOD trip table and the a.m. and p.m. base-case assignments, the trip tables created by LINKOD were assigned to the downtown street network and the assigned volumes were compared with a.m. and p.m. ground counts. Adjustments to link travel times were made in order to reduce differences between estimated and observed volumes.

Once the trip tables for downtown had been created and validated, evaluation of the impact of street closures on traffic patterns could be done with either LINKOD or MINUTP. Both programs were tested for this step. MINUTP is a general transportation planning package that includes subprograms for trip generation, trip distribution, modal choice, and traffic assignment. Only the traffic assignment routines were used for DSTP. Both LINKOD and MINUTP contain traffic assignment subroutines that take into account capacity restraints, and although LINKOD's assignments were found to be somewhat more accurate, both models were deemed capable of producing acceptable assignment results. MINUTP operates locally on a microcomputer, whereas LINKOD operates on a mainframe computer.

With either LINKOD or MINUTP, the process of assessing the impacts of street closures on traffic volumes was the same. The network was modified to reflect street closures or restrictions during various phases of construction, and the trip table was assigned to the modified network. The resulting volumes were then compared with the base assignment (with no closures), which thus provided an estimate of traffic diverted to other streets.

The last step in the modeling process was the use of the TRANSYT-7F simulation model, which allows analysis of traffic flow through a street network by producing output and comparing results with measures of effectiveness (MOEs). These measures-of-effectiveness are used to summarize intersection and network performance in terms of delay, travel time, queue length, and fuel consumption. Inputs to the model include volumes, intersection and street geometry, signal timing, and saturation flow rates. Additional parameters such as free-flow speed, platoon dispersion factors (PDFs), bus dwell times, stop penalties, and delay weighting allow manipulation and calibration of a given network. Included in the TRANSYT-7F program are optimization routines for refinement of existing signal timing plans. The volumes that are input into TRANSYT-7F are taken from the traffic assignment step.

The TRANSYT-7F model area (see Figure 3) comprises 119 signalized intersections located in Seattle's CBD. The network is bounded on the north by Stewart Street, on the south by Jackson Street, on the east by Interstate 5, and on the west by First Avenue.

METHODOLOGY

The two most common methods for developing a trip table involve either conducting an origin-destination (O-D) survey or using the transportation planning process of trip generation (based on population and employment figures), trip distribution, modal split, and model calibration. Both of these methods are time consuming and expensive. An alternative to these methods is the use of LINKOD, which utilizes traffic counts to synthesize a trip table, thereby obviating the need for an extensive O-D survey.

Trip-Table Creation Using LINKOD

LINKOD is a FORTRAN program written to run on a mainframe computer and is the result of a 1980 FHWA study *(1)*. To create a trip table, LINKOD requires traffic count information, including (passenger-car) volumes on streets and turning volumes at intersections; parking information, including location and characteristics (numbers of productions and attractions); and physical and geometric information about streets and intersections, including length, direction, number of lanes, and type of facility.

The LINKOD software package program logic is shown in Figure 4. LINKOD includes program modules to develop and edit a network, build paths and skims, distribute trips for small areas on micronetworks, and make assignments by using an equilibrium assignment process. This last step is iterative, assigning and correcting the trip table to best replicate observed traffic flows.

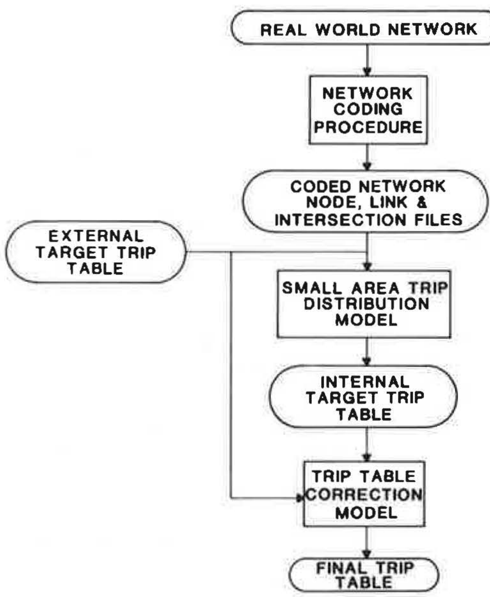


FIGURE 4 LINKOD model logic flow.

Traffic Assignment Using LINKOD or MINUTP

To simulate traffic disruptions caused by construction activities in downtown Seattle, a capacity-restrained equilibrium-based assignment algorithm was used to assign the trip tables generated by LINKOD to networks representing various construction scenarios. Each scenario consisted of sets of street closures, capacity restrictions, and other disruptions, representing effects of ongoing, although disparate, construction activities during specific phases of DSTP.

The two computer programs employed to perform the assignments were LINKOD and MINUTP. The process by which the LINKOD trip table is corrected so as to produce volumes similar to input volumes is bypassed in this case. This standard equilibrium assignment aspect of LINKOD was used several times and produced highly satisfactory results.

MINUTP is a privately developed library of microcomputer programs that performs the usual functions of traditional transportation planning with regard to trip generation, distribution, and network assignment *(2)*. For the case described here, only the network assignment module was used. MINUTP has three different assignment methods: all or nothing, all shortest paths, and stochastic. Any one or a combination of these methods can be used in the iterative assignment process. MINUTP was designed primarily for regional transportation forecasting. The input data describing a MINUTP network are less detailed than those describing a LINKOD network—the primary difference is the detailed intersection description file, which is an input to LINKOD but has no counterpart in MINUTP. However, with creative use of the turn-penalty capabilities of MINUTP, a limited level of intersection control is possible. The updated version (May 1986) of MINUTP allows application of turn penalties for individual intersections, thereby providing some degree of fine-tuning of traffic assignments. In addition, preloading of traffic volumes assists in defining through-travel patterns more definitely.

Assessment of Impacts Using TRANSYT-7F

TRANSYT-7F is the most recent version of the computer program TRANSYT (Traffic Network Study Tool), a traffic signal optimization model originally written in England by the Transport and Road Research Laboratory. TRANSYT-7F was developed under FHWA's National Signal Timing Optimization Project. Modifications to the program developed in England included reorganized inputs, U.S. signal timing conventions, improved output formats, estimates of fuel consumption, and the provision of time-space diagrams *(3, 4)*.

Results of the traffic assignments, representing estimates of the magnitude and extent of traffic disruptions, were input to TRANSYT-7F, which has two computational modes: simulation and optimization. For the assignments, the simulation mode was used, allowing the comparison of MOEs to a base case. The base case was a carefully calibrated simulation of conditions existing before major construction activities. Results of TRANSYT-7F simulation were expected to provide a logical and consistent framework for comparing the degree of traffic disruption between phases of construction. In addition, the effects of modifications (to signal timing, for instance) on a local basis could be estimated.

MODEL CALIBRATION AND VALIDATION

Data Collection and Organization

To the extent possible, required data were obtained from existing documents. The Seattle Engineering Department (SED) provided traffic counts, signal timing, and on- and off-street parking information. Printed bus schedules were used for bus volumes, and maps provided network information. On-street surveys by project personnel were needed for travel-time studies, special intersection approach and geometric data, turn restrictions, bus zones, and missing or inconsistent traffic counts. Data were stored and manipulated with a microcomputer spreadsheet program. In all, the data base contained more than 15,000 input entries and 11,000 calculated values.

Network Creation

Because of data input requirements unique to each computer program and because of differences in purpose between programs, a total of six networks (three for each of the a.m. and p.m. peak hours) were created.

The LINKOD networks were very detailed abstractions of the existing street and highway system in the Seattle downtown area; the level of accuracy generally decreased with distance from the CBD. A total of 255 intersections (nodes) and 650 links were included in the network. Productions and attractions (such as parking information) were similarly detailed reflections of existing conditions, because individual parking lots and garages were included in the network.

The network for MINUTP was similar to the network developed for LINKOD, within the constraints imposed by program differences. The primary difference between the networks for the two models was that LINKOD enabled a very detailed description of intersections and associated delays, whereas MINUTP allowed only the input of turning penalties at intersections.

A total of 119 intersections were included in the TRANSYT-7F network. The network also included both automobile and transit links because of the congestion caused by CBD bus operations. However, because of limitations in the network size allowed by the microcomputer version of TRANSYT-7F, two subnetworks were created with enough overlap to allow the combination of the two in a consistent manner. The TRANSYT-7F network differed from the LINKOD network in that a fringe area was not required and intersections having more than four legs required special treatment.

Data Input and Output Considerations

LINKOD Input

LINKOD has three input files: the link file, the node file, and the intersection file. Although the intersection file is optional, its inclusion leads not only to a more accurate trip table, but also to a better matching of existing turning volumes, which is very important because turning-movement volumes are input into TRANSYT-7F. All the data were entered by using a data preprocessing program written in dBASE III, which allowed data to be input by technicians and performed the sorting and file-writing routines required by LINKOD.

MINUTP Input and Output

MINUTP requires only one input file, which is basically a link file. The input requirements for this file are similar to those for the LINKOD link file but are in an entirely different format. MINUTP is capable of modeling networks of up to 8,190 links and 4,095 two-way links. Because the LINKOD network was created before MINUTP, it was possible to manipulate LINKOD data and match the format requirements of MINUTP by using programs developed by the project team. PREMUTP, MUTPT, and CARDIT are utility programs for transfer of data among LINKOD, MINUTP, and TRANSYT-7F that were developed for DSTP (May 1986). MUTPT also extracts MINUTP assigned volumes and inputs them into the TRANSYT-7F preprocessor programs.

TRANSYT-7F Input

Whereas LINKOD and MINUTP require data in a link format, TRANSYT-7F requires data in a node format. Because of the extensiveness of TRANSYT-7F input data, two data preprocessors were used, SIGNAL *(5)* and PRETRANSYT *(4)*. Both assist in the creation of TRANSYT-7F input data. SIGNAL requires geometric and physical information and performs individual intersection analyses that are used subsequently in PRETRANSYT. PRETRANSYT uses the information base established by SIGNAL and, with additional signal timing connectivity and control card information, generates the input files required for TRANSYT-7F.

LINKOD Calibration

Methodology

The ultimate calibration objective was to generate a trip table that, when assigned to the network, would replicate the input (or ground count) volumes within reasonable limits. The initial aim was to generate a trip table that produced assigned volumes of which 80 percent were within ±20 percent of the input volumes. A secondary calibration objective was to generate a trip table that generally appeared to reflect known trip patterns. In the initial runs the resulting trip table indicated a large number of trips from one internal load node to another (e.g., trips from one parking garage to another, which is an unlikely pattern for peak-hour traffic). The assignment of this "initial run" trip table also resulted in accurately assigned link volumes but relatively inaccurate turning-movement volumes. The calibration effort focused primarily on rectifying the pattern of trips between internal load nodes and increasing the assigned volume accuracy for turning movements. Manipulation of the intersection input data file helped increase the accuracy of turning-movement volumes, but additional efforts were necessary to refine the model's accuracy.

The two basic methods of calibrating LINKOD involved manipulating either the program's control-card parameters or its input data. Each of the program modules required control-card input. Every control-card parameter had default values; however, several values were changed in an effort to encourage more trips between internal and external zones. Details of control card parameters used are contained in related documentation.

Further calibration involved the manipulation of the input data, particularly link and intersection impedances and input link volumes. Initial model runs produced a trip table with an unusually large number of trips from one internal load node to another. The assignment of the resulting trip table also tended to underassign link volumes. After some trial runs, the following steps were taken in an attempt to resolve these problems:

1. In the few instances in which the model had calculated abnormally high intersection delay, a more reasonable intersection delay based on field studies and typical delays for other intersections was input.
2. The original coded link impedances included impedance for the link as well as the intersection delay. For links connected to coded intersections (i.e., intersections coded in the intersection file) link impedances were reduced by an amount equivalent to the estimated intersection delay to avoid double-counting of intersection impedance. Reducing these and unusually high intersection impedances encouraged less underassignment on network links.

3. To promote fewer trips between internal load nodes, the impedances on the internal load-node approach links were adjusted. In the p.m. period, trips were encouraged to flow from internal to external load nodes by placing a high impedance (5 min) on approach links coming into internal load nodes and a lower penalty (2 min) on approach links out of the internal load nodes. The reverse of this was done for the a.m. period.

4. Another significant calibration effort involved a "smoothing" of the original input (i.e., observed) link volumes. LINKOD requires volumes in and out of an intersection to be balanced within certain limits. The modelers accomplished this originally by manually adjusting the incoming and outgoing observed aggregate link volumes until they balanced. In this initial effort, however, turning movements were not taken into consideration and hence the balancing process was not as accurate as it could have been. After initial runs of the model, it was determined that rebalancing—or smoothing—the link volumes by taking into account turning-movement volumes was necessary.

Results

In general, it was found that changes in the input data caused more dramatic changes in the model results than did changes in the control-card parameters. However, it was also determined that an appropriate combination of control-card parameters was necessary as a base from which to further calibrate with input data changes. To check the accuracy of the calibrated LINKOD model results, a comparison of input smoothed (observed) volumes with the output assigned volumes was made. The comparison is shown for both macro and micro links. Macro links represent the total directional link volume between two intersections, whereas micro links represent the individual turning movements within each intersection. In general, the a.m. base-case run of LINKOD produced results that were generally superior to those obtained from the p.m. base case.

Macro-Link Comparison Table 1 is a summary comparison of the LINKOD assigned volumes with the macro-link input volumes for both a.m. and p.m. cases. The percentage of links within a given volume range for which assigned volumes fell within 10 and 20 percent of input volumes is shown.

For both the a.m. and p.m. cases, the larger volumes had less error than the smaller volumes. In general, for the macro links, the a.m. LINKOD trip table and assignment process produced better results than did the p.m. table.

TABLE 1 LINKOD MACRO-LINK ERROR SUMMARY BY VOLUME RANGE

	Percentage of Assigned Volumes			
	Within 10 percent of Input Volumes		Within 20 Percent of Input Volumes	
Input Volume Range	A.M. ($N = 51$)	P.M. ($N = 40$)	A.M. ($N = 74$)	P.M. ($N = 55$)
Less than 100	31	19	52	32
100–250	45	18	70	43
250–500	56	36	78	65
500–750	76	54	92	81
Greater than 750	63	51	90	78

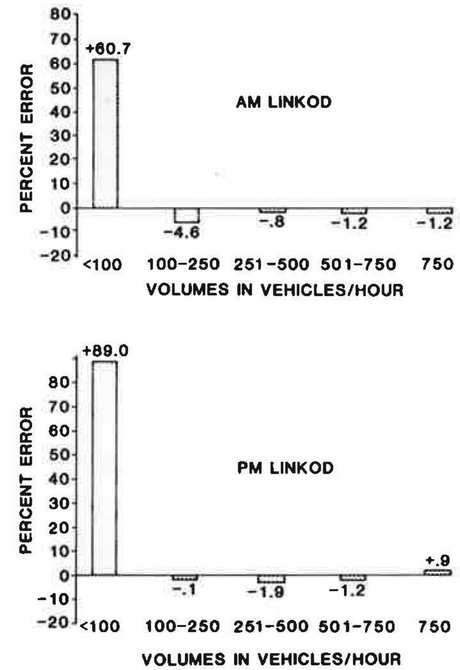
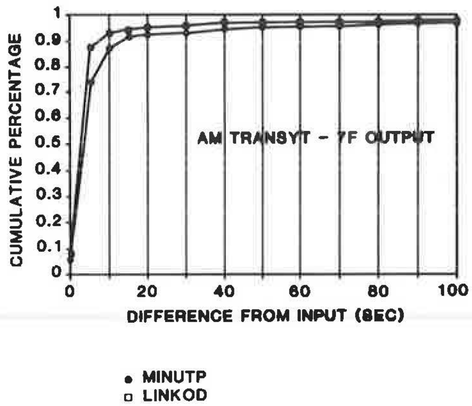


FIGURE 5 LINKOD calibration: percent error by volume for micro links.

Micro-Link Comparison Figure 5 shows the average percent error by volume for all the micro links (individual turning and through movements at intersections) in the network. Accuracy in turning-movement volumes is important because these volumes are used in TRANSYT-7F. The percent error in Figure 5 represents the percent difference between the volumes assigned using the calibrated LINKOD trip table and the base input (observed) volumes. The results show that the model has a high degree of accuracy in replicating volumes or links that had more than 100 vehicles per hour (vph). Although the low-volume links were assigned with a high percentage error, the absolute errors were typically small. For instance, a particular turning movement may have an observed volume of 25 vph and the model may have assigned it 45 vph. In this case even though the percentage error (+80 percent) is high, the absolute error (20 vph difference) is relatively low.

Another finding was that the accuracy of the micro links within any given intersection was significantly increased when that intersection was coded in the LINKOD intersection file. This was tested and confirmed by comparing the results from model runs both without and with the intersection file as part of the input. In the final LINKOD run, the intersections coded in the intersection file included only those in the core CBD network area. Intersections in the fringe or buffer area were not included. It was assumed that the model's level of accuracy was higher in the core area—the area of interest. The results in Figure 5, however, are aggregated across both the core and fringe areas and hence are less accurate than if they included the core area only.

In comparing the p.m. results with the a.m. results in Figure 5, a significant improvement is seen in the a.m. results. One reason for this may be that the p.m. network was calibrated first and the lessons learned and experience gained in the process enabled calibration of the a.m. network in half the time and



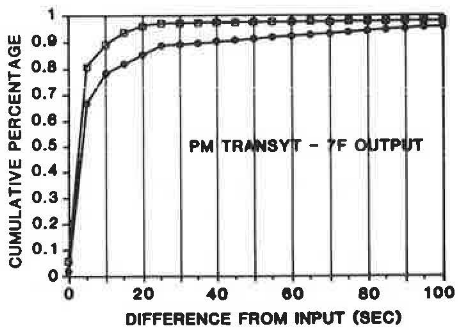


FIGURE 6 DSTP model calibration: difference in TRANSYT-7F calculated average delay using LINKOD and MINUTP output volumes compared with smoothed ground counts.

with better results. Another contributing factor may be that the p.m. case is simply more difficult to model because of the higher degree of diverse trip patterns as compared with the a.m. case.

The purpose of using LINKOD was to generate a trip table that, when assigned, would produce turning volumes with a level of accuracy that, when input into TRANSYT-7F, would produce results similar to TRANSYT-7F results produced when observed volumes were input. A key output from the TRANSYT-7F model is average vehicle delay for each intersection. To test the accuracy of the calibrated LINKOD assigned (output) volumes, they were input to TRANSYT-7F and the resulting average vehicle delays for all the intersections in the core network area were compared with the corresponding TRANSYT-7F average delays generated with observed (LINKOD input) volumes. The results of this comparison are shown in Figure 6, in which the frequency of coded intersections is plotted against the difference in TRANSYT-7F calculated average delay. The plots indicate that for the a.m. case, 95 percent of the intersections had less than a 10-sec difference in calculated average delay using the LINKOD assigned (output) volumes versus that using the LINKOD input (observed) volumes. The results for the p.m. case were slightly more disparate in that 90 percent of the intersections had less than a 10-sec difference in calculated average delay. However, this still represented a relatively high degree of accuracy.

Screenline Comparison One other basic measure used in the calibration process was screenline comparisons of assigned versus observed volumes. In all, volumes across nine screenlines and for each of the downtown freeway ramps were compared. In general, the LINKOD assignment volumes were consistently less than the observed volumes. The percentage differences between observed and assigned volumes ranged from 0.2 to 27.0 percent, but fell primarily between 1.0 and 16.0 percent. Table 2 gives a more detailed breakdown of

TABLE 2 LINKOD AND MINUTP SCREENLINE COMPARISONS

Screenline	Street	Observed Volume, 1985 Smoothed Ground Counts	Percentage Difference: LINKOD Assigned over Ground Counts	Percentage Difference: MINUTP over Ground Counts
A.M. Network				
A–A northbound	University	5	1.20	1.20
	6th	1,260	0.80	0.90
	4th	835	0.94	1.03
	3rd	225	0.68	1.17
	1st	690	0.91	0.61
Total		3,015	0.86	0.89
A–A southbound	1st	215	0.86	0.73
	2nd	890	1.00	1.08
	3rd	230	1.16	1.13
	5th	1,230	0.98	1.01
	University	135	0.63	0.73
Total		2,700	0.98	1.01
B–B northbound	1st	210	1.00	1.70
	3rd	190	0.93	1.53
	4th	755	0.99	1.12
Total		1,155	0.98	1.29
B–B southbound	1st	165	1.02	0.92
	2nd	570	1.00	1.01
	3rd	185	0.90	0.81
	5th	485	1.04	0.88
	6th	1,280	1.00	0.91
Total		2,685	1.00	0.92
C–C eastbound	Virginia	375	0.83	0.77
	Westlake	185	0.77	0.95
	Olive	330	0.92	1.47
	Pike	475	0.75	0.60
Total		1,365	0.82	0.90
C–C westbound	Union	965	0.93	0.99
	Pine	775	1.03	0.62
	Stewart	990	0.75	1.19
	Westlake	155	0.96	0.94
	Lenora	165	1.02	0.73
Total		3,050	0.90	0.94
P.M. Network				
A–A northbound	University	20	1.05	0.75
	6th	1,080	0.84	0.83
	4th	1,250	1.02	0.95
	3rd	330	0.84	0.80
	1st	852	0.93	1.09
Total		3,532	0.93	0.83

TABLE 2 *continued*

Screenline	Street	Observed Volume, 1985 Smoothed Ground Counts	Percentage Difference: LINKOD Assigned over Ground Counts	Percentage Difference: MINUTP over Ground Counts
P.M. Network				
A–A southbound	1st	450	0.96	0.83
	2nd	1,530	0.96	0.94
	3rd	385	0.79	0.75
	5th	1,260	0.98	1.01
	University	150	0.51	0.79
Total		3,775	0.93	0.92
B–B northbound	1st	450	0.94	0.96
	3rd	209	1.06	1.06
	4th	1,200	0.83	0.74
Total		1,940	0.89	0.84
B–B southbound	1st	400	0.87	0.89
	2nd	1,225	0.94	0.97
	3rd	360	0.85	0.86
	5th	950	0.92	0.87
	6th	880	0.81	0.81
Total		3,815	0.89	0.89
C–C eastbound	Virginia	925	0.77	0.72
	Westlake	100	1.53	1.89
	Olive	925	0.78	0.77
	Pike	1,105	0.79	0.99
Total		3,055	0.81	0.87
C–C westbound	Union	750	0.91	0.85
	Pine	690	0.90	0.87
	Stewart	645	0.83	0.85
	Westlake	230	0.86	0.84
	Lenora	200	0.89	1.43
Total		2,515	0.88	0.90

screenline comparisons for three selected screenlines by showing a.m. and p.m. peak-hour volumes and percentage differences for each screenline link.

Summary and Conclusions

LINKOD was found to be a complex, data-intensive, expensive program to run. Becoming familiar with and calibrating the program was very time consuming. However, once the critical lessons had been learned and calibration was complete, the model produced highly satisfactory results.

MINUTP Calibration

In calibrating the MINUTP base-case network, adjustments were made in order to minimize the difference between the assigned link volumes and the original input volumes to LINKOD (smoothed volumes). Different assignment combinations were tested and various calibration techniques experimented with.

Assignment Methodology

MINUTP provides three methods of assignment, as mentioned previously. Based on the experience with various assignment method combinations and discussions with the developer of the model, the combination of one stochastic iteration followed by two all-or-nothing iterations was found to yield the best results. This is also the combination used most frequently by the developer of the software.

Calibration Techniques

Five calibration techniques were identified: changing speed (which changes travel time of a link), changing capacity in vehicles per hour per lane, changing the number of lanes, using turning penalties, and using turning prohibitions. The most effective methods found for modifying the assignment were to modify the link speeds and to impose turning penalties and prohibitions.

Initial calibration work consisted of changing lane capacities and link travel speeds. Initial runs showed that the MINUTP assignments were very close to the input smoothed volumes on the screenline level but varied significantly at the individual street or link level.

The p.m. network was calibrated first. The assignments did not vary significantly with gradual changes in lane capacities. They were excessively sensitive, however, to gradual changes in link speed. This excessiveness was curbed in two ways, first, to make fewer and more gradual changes in speed and second, to establish turning penalties at the locations where traffic was diverting to the parallel route. Turning prohibitions were also set at those locations in the network where turns are actually prohibited during the peak hours.

Generally, fewer than five changes to speeds or turning penalties, or both, were modified from run to run. Many runs were made with only one or two changes. After each run, volumes were posted on a screenline spreadsheet. New assigned volumes were checked to see whether they more closely matched the smoothed input volumes that the previous MINUTP run had assigned. If most screenline link volumes were worse than before, the changes were undone and a new approach was tried.

The MINUTP assigned volumes were then run through TRANSYT-7F to compare the delay time in seconds for all intersection movements (including left, right, and through) with the delay times that had resulted from inputting both the smoothed volumes (LINKOD input) and the LINKOD base-case assigned volumes (LINKOD output). The TRANSYT-7F run for the p.m. case showed that there were more instances of excessive movement delays from the MINUTP assignment than from LINKOD's base assignment. This can be seen in Figure 6, where for the p.m. case, 90 percent of the average delays calculated using the LINKOD assigned volumes fell within 10 sec of those calculated using the LINKOD input (smoothed) volumes, whereas only 78 percent of the calculated delays using the MINUTP assigned volumes fell within 10 sec of those calculated by using the smoothed volumes. Further calibration of the p.m. case focused on the intersections for which the TRANSYT-7F run calculated unreasonably high delays because of excessive turning-movement volumes. These turning volumes were reduced by placing turn penalties on the movements in question.

Calibrating the a.m. network began by using two different base networks. One was the same as the LINKOD a.m. network. The other was the p.m. calibrated network modified to account for the a.m. changes in the reversible freeway express lanes. The reason for two base networks was to see whether the changes made to the p.m. network would benefit the a.m. network also. As it turned out, the assignment using the original a.m. LINKOD network provided better results at the screenline level than that using the modified p.m. MINUTP network.

As seen in Figure 6, the a.m. network when calibrated provided better results than the p.m. TRANSYT-7F run. This was due in part to having a better trip table (produced by the a.m. LINKOD runs), to being more experienced in calibration techniques, and to generally having fewer vehicle trips in the network.

Summary and Conclusions

In general, it was found that the advantages of using MINUTP instead of LINKOD for assignment purposes outweighed the lower level of accuracy that MINUTP provides. MINUTP provided the necessary accuracy in identifying "hot spots" when run through TRANSYT-7F and allowed application on a microcomputer to proceed easily and inexpensively when compared with the mainframe utilization of LINKOD.

MINUTP does not provide quite as good an assignment as does LINKOD; however, it provides an assignment that is acceptable in terms of identifying hot spots when run through TRANSYT-7F, especially when the updated version of MINUTP, which allowed more detailed control of intersection turning movements and preloading of through trips, is used.

The primary advantages of MINUTP are that it is an easy microcomputer program to use, the turnaround time between runs is short, and the computer costs are relatively low (especially when compared with those using LINKOD on a mainframe computer in a remote office).

In addition, MINUTP has several features that enable a more thorough analysis of the traffic assignment, such as select link analysis capabilities, path tracing, convenient trip table manipulation, and preloading of volumes onto certain links.

TRANSYT-7F Calibration

The TRANSYT-7F network included both transit and automobile links. These links are modeled as either shared stoplines (automobile and transit share lanes) or exclusive links (transit-only lanes). In general, it was difficult to model transit operations accurately in the networks. TRANSYT-7F is limited in its ability to account for the delays caused by passenger loading, skip-stop operation, and other elements of transit operations. Despite these problems, it was decided that including transit in the model was necessary because congestion due to transit operations would be a controlling factor in the development of mitigating measures via use of the TRANSYT-7F model.

The process of calibrating the TRANSYT-7F model to match existing conditions required extensive data collection, including travel-time studies, flow-profile analyses, maximum queue data, manual counts, and spot speed studies. Five north–south avenues and nine east–west streets were selected for these data collection activities on the basis of their classification as major CBD surface routes. The five data collection activities were performed concurrently on a given route to obtain a peak-hour "snapshot" of traffic flow on the route. Thirty-five intersections were included during both a.m. and p.m. peak hours.

Sensitivity Analysis

Before use of the calibration data, a sensitivity analysis was performed on the coded TRANSYT-7F network. Four coding variables were tested for sensitivity: the platoon dispersion factor (PDF), which adjusts the rate at which platoons of vehicles disperse as they leave a queue; bus dwell time, which places an impedance on a bus link to simulate passenger loading activities; saturation flow, which quantifies the maximum number of vehicles that can travel on a link during a 1-hr period (continuous green time); and speed, which represents free flow or the unconstrained travel speed along a link. In the test for sensitivity, input values were varied incrementally. The sensitivity of the model to these changes was evaluated by posting average delay on the link to which the changes were made. In general it was found that the average delay calculated for a given link was not sensitive to changes in PDF and bus dwell time. Average delay was found to be sensitive to changes in free-flow speed and saturation flow. Spot speed studies were performed under uncongested conditions to approximate the initial free-flow speed estimate. Though changes in the input speed would assist in replicating existing delay, it was decided not to change input developed from field studies. For saturation flow, however, initial estimates were made by using the 1985 *Highway Capacity Manual (6)*. Though this provided a sound initial estimate, it was not able to fully account for the complexity of congested urban traffic conditions. TRANSYT-7F is reasonably sensitive to changes in the saturation flow input. Because it was believed that the initial estimate of saturation flow did not fully account for existing conditions, manipulation of this input was selected for calibration of the networks.

Travel-Time Comparisons

Calibration was performed by comparing travel times output by the model with those observed in the field. An iterative process of changing the saturation flow on individual links was performed until comparable travel times were achieved. The extent of change to the initial saturation-flow value was limited to maintain reasonable estimates of this input. In addition, an attempt was made to maintain comparable saturation-flow input along streets and avenues that had similar or identical characteristics.

A comparison of TRANSYT-7F travel-time output versus observed travel time along major arterials for the final calibrated versions of the a.m. and p.m. networks showed that travel times for both networks were matched within a range of ±18 percent, with many of these within 5 percent. Figure 7 shows this comparison for both the a.m. and the p.m. case. In general, comparative travel times on the longer north–south routes matched better than those on the shorter east–west routes. This is due largely to the difficulty in performing a random travel-time study on the shorter routes. Third Avenue was difficult to calibrate because of large peak-hour transit

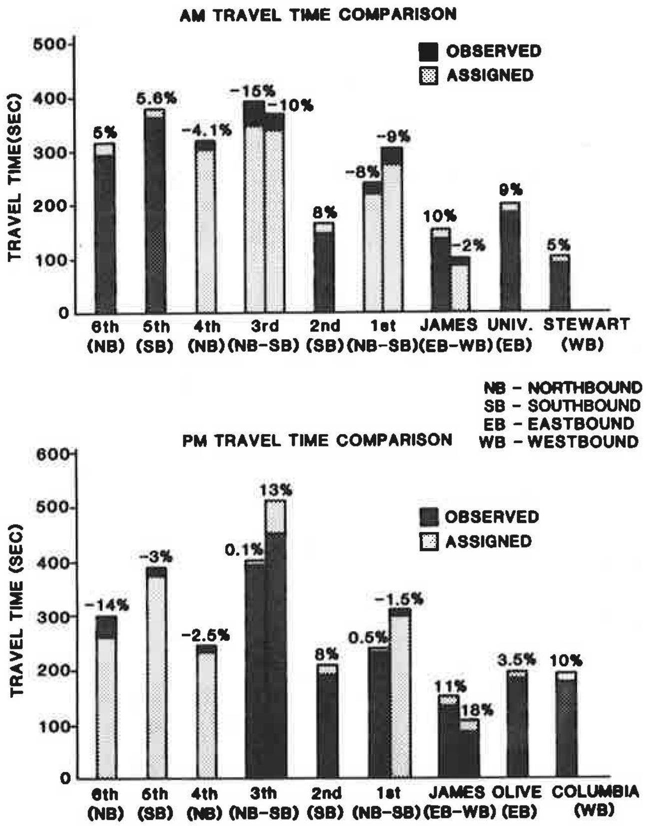


FIGURE 7 TRANSYT-7F calibration: peak-hour travel-time comparisons on major CBD streets.

volumes. Third Avenue was modeled with separate transit and automobile links. In order to simulate the extensive congestion on Third Avenue, automobile links were given a low saturation flow. In many cases, even these low saturation flows did not result in delays comparable with those experienced in the field. Calibration was also difficult where sizable pedestrian volumes significantly inhibited traffic flow and with the steeply graded east–west streets. Overall, though, a very reasonable representation of existing traffic conditions in the CBD was attained. Delay and saturation-flow analyses performed since calibration have correlated closely with the input and output of the original calibrated models.

PROJECT APPLICATIONS AND RESOURCE CONSIDERATIONS

The modeling chain described in this paper can best be applied in large construction projects in complex urban settings, such as major downtown freeways or transit systems, or in the evaluation of complex signal networks. The utilization of LINKOD, a very detailed network-based model, was found to be moderately time consuming but necessary as opposed to developing a detailed micro trip table from a traditional regional model base. Once the project team became familiar with the intricacies of the LINKOD model, development of the data base input file and application became more straightforward and less time consuming for the MINUTP portion.

The models were developed by two teams of analysts working in parallel over a 6-month period. During that span, a.m. and p.m. networks were developed for each of the three separate models.

The LINKOD and MINUTP models were developed by one team of analysts for 255 intersections, an area that included major CBD approach arterials and freeways (Figure 3). This team used LINKOD to formulate a.m. and p.m. trip tables and, using the MINUTP assignment package, applied that trip table to the 255-intersection network. The effort required to do this is estimated at approximately 7 hr per intersection for developing the a.m. and p.m. trip tables using LINKOD and another 4 hr per intersection to extract, code, and calibrate the two MINUTP networks using LINKOD as a basis.

The TRANSYT-7F model was developed for a CBD core study area of 119 intersections by the second team of traffic analysts (Figure 3). The level of effort per intersection for the TRANSYT-7F model is estimated at approximately 20 hr per intersection for combined a.m. and p.m. conditions.

The total effort is estimated at about 2.4 person-years. The distribution of time, for planning purposes, that was needed to define, develop, calibrate, and validate the models is estimated as follows:

Project Activity	*Percent of Total*
Data collection	9
Model definition and software development	14
P.M. model development and calibration	43
A.M. model development and calibration	30
Model documentation	4

The teams developed the p.m. model configuration initially and modified the coded p.m. network to replicate the a.m. network, thereby reducing effort and time considerably in completing the second network.

Model definition efforts included the research and evaluation of available software for application to a microcomputer environment. The development of software to link the three models together represents a major element of this task. The mainframe LINKOD network files were reformatted into MINUTP microcomputer files to develop a microcomputer-based network. A more efficient translation of MINUTP assignment output to TRANSYT-7F input files was also provided. Following this major investment of time and effort, about 40 hr is required to define and code network changes to MINUTP and produce TRANSYT-7F output for evaluation of street closures in the core CBD area during given construction phases.

CONCLUSIONS

The DSTP traffic modeling system represents a systematic approach for detour planning during major construction projects. The modeling chain is appropriate when the transportation engineer needs to develop a trip table independent of regional models for a complex signal network. Because of the level of effort entailed, the most cost-effective application for the model chain is for large projects or those with complex signal networks. It has several advantages over the traditional, more ad hoc detour-planning procedures in that it has the capability to test numerous "what if" situations and to objectively quantify associated impacts. On the basis of the projected impacts, traffic control plans can be developed and then tested with this approach. The investment in the modeling system during planning phases of the project has benefits in later phases in that it is a tool that can be used for preliminary engineering and final design as well as for input into traffic operations both during and after construction.

Calibration results of the modeling system, as described in this paper, have proven highly satisfactory. Application of the system for facilitating detour planning during the DSTP construction is still in the initial stages. As application of the system and the DSTP construction progress, several opportunities will exist to compare actual field results with predictions by the modeling system. The city of Seattle has proposed a traffic counting program scheduled to run throughout the DSTP construction that will provide valuable input toward this end. Throughout its use, the system will be evaluated as it is currently structured in order to refine and improve any steps that appear to reduce its effectiveness in practical applications. Further research will focus on streamlining and documenting the procedure so as to generalize the process for application to other projects. Areas for improvement will become clearer as current application of the system progresses; however, some specific areas have already been identified:

- A lack of consistency of level of detail exists among the three software packages. Although LINKOD and TRANSYT-7F are extremely detailed in their coding conventions and output content, MINUTP is not as suitable for detailed analysis. Other software packages with similar functions to MINUTP should be examined for their potential use as the intermediate package between LINKOD and TRANSYT-7F. Alternatively, a microcomputer version of LINKOD would be beneficial.
- The data acquisition and model calibration procedures need to be streamlined, well defined, and documented in order to facilitate a more cost-effective model development phase on future projects.
- In order to facilitate more effective analysis and presentation of the modeling system results, an analysis and priority ranking of the system's various outputs needs to be conducted and templates summarizing the desired outputs need to be developed.

Research on these and other areas identified will be ongoing during 1988.

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Publication of this paper sponsored by Committee on Transportation System Management.

Multicorridor Project Traffic Analysis

ROBERT BERNSTEIN

An approach used by the Puget Sound Council of Governments to analyze the regional traffic impacts of alternative major regional transit investments is described and evaluated. Because the transit system capacity increases resulting from each alternative were roughly equivalent and the proportion of overall regional travel carried by transit was relatively small, the traffic impacts of the alternatives did not differ significantly on a regional scale. The technical analysis was of a general nature and was aimed at elected officials and the public, who often do not have a comfortable grasp of the meaning and implications of *v/c* data as they relate to traffic congestion. The problem is more easily understood when presented in terms of length of the peak period or the number of hours of congestion. One of the key elements of the regional traffic analysis was to determine the length of the peak period on various segments of the highway system. For the purposes of this analysis, length of peak was defined to be the number of hours during which level-of-service E conditions exist (*v/c* greater than 0.90). Peak-period length was estimated empirically on the basis of the average *v/c* for a longer time period. A linear regression equation was developed to represent the relationship between 12-hr average *v/c* and the number of hours of *v/c* greater than 0.90 for a set of actual freeway counts. Traffic assignments were plugged into the regression equation to generate estimates of future congestion. The analysis results provided a good sense, not only of relative congestion problems, but also of the magnitude of those problems in absolute terms. The analysis approach proved to be useful educationally as well as simple and straightforward computationally.

An approach used by the Puget Sound Council of Governments (PSCOG) to analyze the regional traffic impacts of alternative major regional transit investments is described and evaluated. First, however, it is important to understand the context in which it was applied.

The bottom line to the various growth and travel forecasts for the central Puget Sound region (Seattle-Tacoma-Everett) is much the same as that in other expanding urban-suburban areas: growth in regional travel demand resulting from continuing increases in population and employment will lead to increasingly severe congestion in major transportation corridors unless (or even if) additional capacity is provided.

Policies adopted in the Regional Transportation Plan (RTP) recognize that additional capacity will be required in the major transportation corridors to implement urban development and activity center policies. The RTP policies further state that most new capacity in major corridors should be provided by investment in transit and high-occupancy-vehicle (HOV) facilities and services.

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MULTICORRIDOR PROJECT

In response to these policies, the PSCOG Executive Board and the Municipality of Metropolitan Seattle (Metro) Council in 1984 initiated a 2-year effort called the Multicorridor Project to analyze alternative major transit and HOV investments in the region's three highest-priority corridors (see Figure 1).

The purpose of the Multicorridor Project was to identify the best long-range transit or HOV alternatives for the three corridors in terms of (a) corridor utilization trade-offs (among transit, HOVs, and other users), (b) cost-effectiveness, (c) support for regional and local land use plans, (d) user benefits, and (e) impacts. The main purpose of the traffic analysis conducted for the Multicorridor Project, then, was to compare the traffic impacts of the major transit investment alternatives in order to identify impacts that would make a difference in the selection of a preferred alternative.

After a screening process and preliminary cost, ridership, and impact analyses, three basic regional transit system alternatives were selected for detailed analysis. These included the baseline-bus alternative, the trunk–feeder-bus alternative, and the bus-LRT alternative. The baseline-bus alternative included a major expansion of the existing transit-HOV system, including more local bus service, more express bus service, more

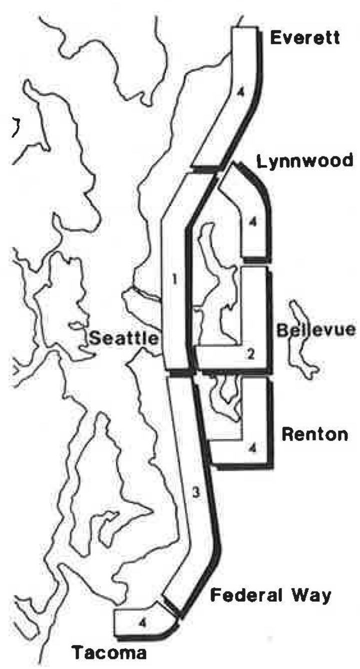

FIGURE 1 Transit corridors.

park-and-ride lots, more HOV lanes, more transit centers, and so on. The trunk–feeder-bus alternative was based on a fundamental change in the route structure of the existing bus system that would introduce an extensive system of line-haul bus service on major highway facilities (trunks), with local bus service feeding it (feeders). Finally, the bus-LRT alternative included an LRT line in each of the three highest-priority corridors. The rail lines were to be fed by local bus service, with supplementary bus service connecting activity centers not served by LRT. Two variations of the LRT alignments in each corridor were assessed.

Given the rough equivalence of the transit system capacity increases included in each alternative and the relatively small proportion of overall regional travel that is carried by transit, it was recognized from the outset that the traffic impacts of the alternatives would not be fundamentally different on a regional scale. Nevertheless, differences worth noting or crucial flaws might exist at specific locations or along specific highway segments. The traffic analysis was designed on these premises. A general assessment of the overall regional highway system was made, with more detailed analyses focusing on specific screenlines and small areas of interest.

CONCEPTUAL APPROACH FOR REGIONAL TRAFFIC ANALYSIS

Despite the expected similarity among future regional traffic conditions under the various alternatives, the regional portion of the traffic analysis was not considered to be an unnecessary exercise. The regional traffic analysis was useful in that it painted a general picture of future conditions on the regional highway system. This provided an important context for the Multicorridor Project decisionmakers (local elected officials) and for the public.

Rather than a more technical analysis geared solely toward technical staff, then, the regional traffic analysis was of a general nature and was aimed at a lay audience. For this audience, the typical means of presenting traffic congestion information—peak-hour or peak-period volume-to-capacity ratios (*v/c*)—was deemed inappropriate. Elected officials and the public often do not have a comfortable grasp of the meaning and implications of *v/c* data, and transportation planning and engineering professionals themselves have argued over how to interpret traffic forecasts that result in computed peak *v/c*'s greater than 1.0. In addition, the accuracy of the future *v/c*'s was suspect, because the ratios were computed by simply taking assignments of daily traffic and applying a rule-of-thumb 8, 9, or 10 percent factor to compute the peak-hour traffic volumes (peak-period assignments were not used for lack of a good peak-period trip table). Finally—and most important—peak *v/c* information does not adequately describe future congestion in a way that elected officials and the public can easily comprehend. It would be more understandable if expressed in terms of the length of the peak period or the number of hours of congestion.

For the foregoing reasons, one of the key elements of the regional traffic analysis was to determine the length of the peak on various segments of the highway system. For the purposes of this analysis, length of peak was defined as the number of hours during which level-of-service E conditions exist (*v/c* greater than 0.90). The analysis focused on the afternoon and evening hours, because in most cases the p.m. peak is longer than the a.m. peak.

PEAK-PERIOD LENGTH

Peak-period length was estimated empirically by using an approach based on the thesis that the number of hours of congestion on a given freeway segment varies with the average *v/c* for a longer time period (e.g., 12 or 24 hr, the time period for which traffic assignments are available). In other words, the higher the daily or 12-hr traffic volume relative to capacity, the longer the peak period. (A corollary to this thesis suggests that the peaking characteristics on currently congested freeway segments elsewhere in the country provide a more realistic model of future local peaking characteristics than would an extrapolation of current, less-congested local conditions.)

In order to test the thesis as well as to actually estimate peak-period length at various points on the regional highway system, traffic count data were obtained from a number of U.S. cities. Hourly counts were obtained for 50 directional freeway segments in Seattle; Portland, Oregon; Chicago; suburban northern Virginia; and San Francisco–Oakland. (The amount of data collected was dictated by the Multicorridor Project schedule, not by statistical requirements.) In each case, the hourly counts included a composite of 1 to 2 weeks' worth of weekday counts. The data set contained four downtown freeway segments, 20 central city radial segments, 24 suburban radial segments, and two suburban circumferential segments. Several

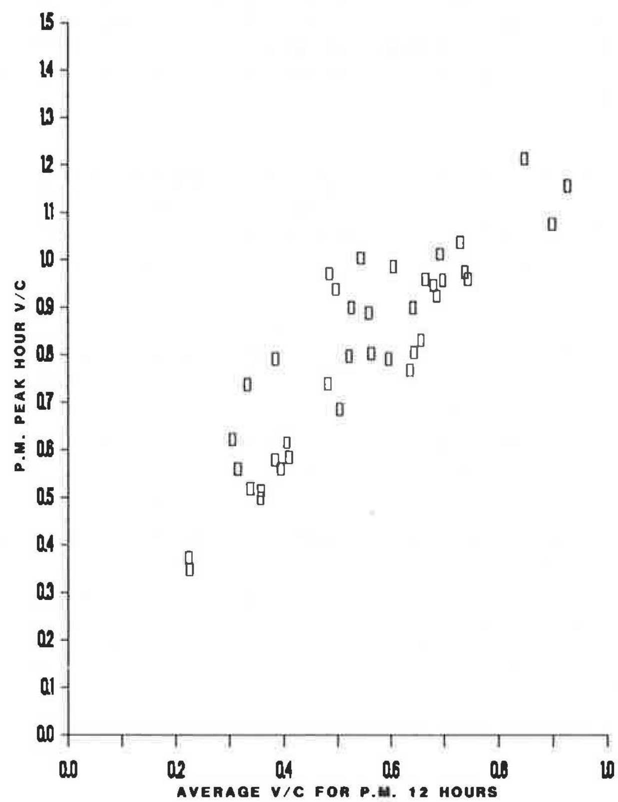

FIGURE 2 P.M. peak and 12-hr *v/c*.

of the radial segments had peak flows in both directions, whereas others were highly directional (i.e., inbound in the morning and outbound in the evening).

The analysis of the traffic counts and assignments focused on the 12-hr period from noon to midnight in order to avoid inconsistencies created by differing directional splits and peaking characteristics on different freeway segments. Because some segments experience peaking in both directions in both the morning and the afternoon whereas others only experience the typical peaking (inbound in the morning and outbound in the afternoon), two segments with similar afternoon peaks could have very different 24-hr average *v/c*'s. It would have been useful to further separate the data and analysis by freeway type—radial versus circumferential or urban versus suburban, or both—but there were insufficient data to do so. Focusing on the 12-hr p.m. period (and thereby accounting for directional split and peaking) accounts for much of the difference between different freeway types.

The relationship between p.m. peak length and the p.m. 12-hr average *v/c* for the freeway counts was assessed by comparing several characteristics of the individual counts, including the peak-hour *v/c*, the *v/c*'s for each of the 12 hr between noon and midnight, the proportional distribution of traffic volume over the 12 hr, and the number of hours during which *v/c* exceeded 0.90.

Three of these comparisons are shown in Figures 2 through 4. Figure 2 shows the peak-hour *v/c*'s for the various freeway counts plotted against their corresponding p.m. 12-hr average *v/c*. Not surprisingly, peak-hour *v/c* increases with increasing 12-hr average *v/c*. In addition, the rate of increase of peak-hour *v/c* decreases with increasing 12-hr average *v/c*, indicating that at higher 12-hr volumes the peak is more spread out. This information supports the thesis that the number of hours of congestion (i.e., length of peak period) increases with increasing 12-hr average *v/c*.

Grouping the counts by 12-hr average *v/c* yielded some interesting insights into the different traffic demand patterns on congested and free-flowing freeways. The counts were divided into five groups on the basis of 12-hr average *v/c*: 12-hr *v/c* greater than 0.8, 0.7 to 0.8, 0.6 to 0.7, 0.4 to 0.6, and less than 0.4. Proportion of 12-hr volume and *v/c* were computed for each count for each p.m. hour. These proportions and *v/c*'s were then averaged within each group. Figure 3 shows the group average hourly *v/c*'s for the five groups. Here again, the results were not surprising: the group average *v/c*'s in any given hour were higher for the groups with higher 12-hr average *v/c*.

Figure 4 shows the hourly proportions of total 12-hr volume averaged for each group of counts. The proportion of 12-hour traffic occurring in the early afternoon is somewhat higher for the groups with the lower 12-hr average *v/c*'s. This difference is much more pronounced in the afternoon peak period, when the groups with the higher 12-hr *v/c*'s have a much smaller average proportion of 12-hr traffic than do the groups with lower 12-hr *v/c*'s. This progressive flattening of the peaks with increasing 12-hr *v/c* is further evidence that the congested period on freeways increases in length with increasing 12-hr average *v/c*. The proportions reverse in the evening and night hours; the groups with low 12-hr *v/c* have the lowest hourly proportions.

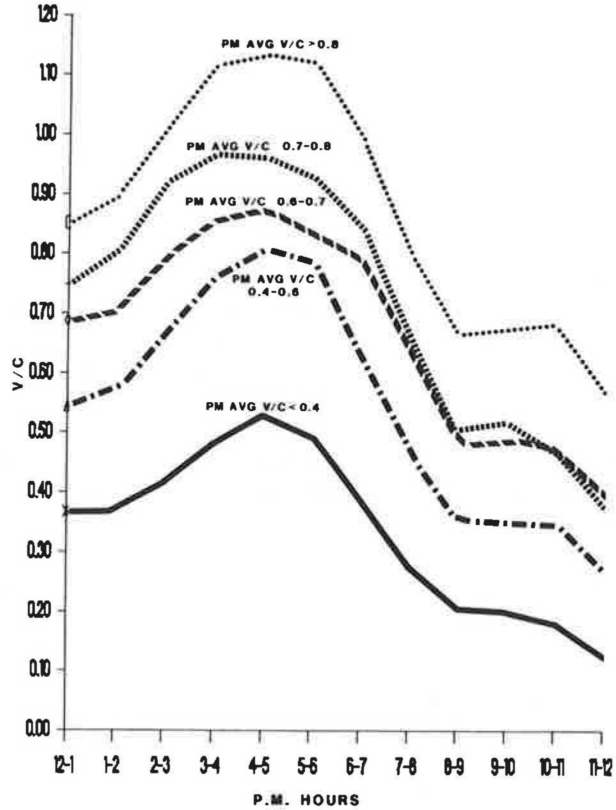

FIGURE 3 Hourly *v/c*.

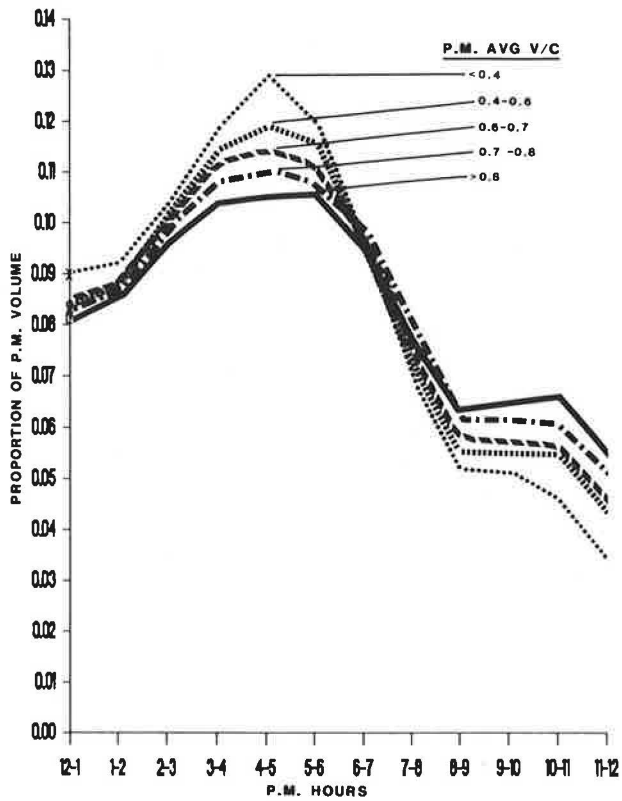

FIGURE 4 Hourly proportion of 12-hr traffic.

METHODOLOGY

Two methods of computing the number of hours of congestion were developed. The first used the hourly proportions of the 12-hr volume averaged for each group of counts. Applying the proportions to forecast freeway volumes yielded hourly volumes from which hourly v/c's were computed. The number of hourly v/c's exceeding 0.90 was then counted. The second method developed a linear regression equation to represent the relationship between 12-hr average v/c and the number of hours of v/c greater than 0.90 for the full set of individual counts.

Both of these methods require as input directional traffic volume for the 12 p.m. hr. Because the traffic assignments available were two-way daily assignments, the directional 12-hr volumes had to be estimated. The first step was to estimate the two-way traffic volume in the 12 p.m. hr. Based on the p.m. percentages computed from the available freeway counts, p.m. volume was assumed to be 60 percent of the 24-hr total. (The 25 or so freeway segments carried an average of 59.7 percent of daily traffic between noon and midnight; standard deviation of this data was only 2.6 percent.) Next, the directional p.m. volumes were determined by estimating the directional splits of two components of p.m. traffic. One component was the 20 percent of daily traffic represented by the difference between a.m. and p.m. volumes (60 percent minus 40 percent). This was assumed to be traffic making a round trip entirely during p.m. hours, and it was assumed to have a 50-50 directional split. A peak-period traffic assignment was run, and the directional splits were used to estimate the directional split of the remainder of the p.m. volume (40 percent of daily).

For each link analyzed, then, p.m. 12-hr directional traffic volume was computed by multiplying the two-way daily volume by the following factor:

$$\text{Factor} = (0.1) + (0.4 \times \text{peak-period directional split}) \qquad (1)$$

The following linear regression equation, derived from the freeway count data shown in Figure 5, was then used to estimate the number of p.m. hours in which volume would exceed 90 percent of capacity on each link:

$$\text{No. of hours with } v/c > 0.9 = [13.92 \times (\text{12-hr average } v/c)] - 6.25 \qquad (2)$$

Using a spreadsheet, it was a fairly simple task to compute for the various freeway links the directional p.m. 12-hr volumes, the p.m. 12-hr average v/c, and finally, the hours of congestion. Lotus 1-2-3 was used for this project, but the computations are so straightforward that virtually any spreadsheet program could be used. Computations are done individually for each link, so the spreadsheet contained one row for each link. Vertically, in addition to link identification columns (e.g., road name, A Node, B Node), the spreadsheet should have five input data columns: daily traffic; capacity; peak-period traffic, A–B; peak-period traffic, B–A; and peak-period directional split (computed from the directional peak volumes). A final column is used to compute the number of hours of congestion using Equations 1 and 2.

Using the hourly proportions of 12-hr volumes and linear regression—the two methods of computing hours of congestion—yielded similar results. The linear regression method was employed for the Multicorridor Project because of its computational simplicity and the consistency of its results.

APPLICATION

The methodology just described was applied to the Multicorridor Project traffic forecasts. Estimated hours of congestion were computed for each freeway link in the Multicorridor Project study area. For all but a handful of the most congested links in the system, the computation of hours of congestion involved interpolation of the traffic count data; that is, the forecasted average v/c's for Seattle area freeways were within the range of average v/c's that actually occurred on the freeway segments for which counts were available. This made the results of the analysis more credible, because it was not necessary to extrapolate the worst freeway conditions experienced elsewhere and claim that the Seattle area should expect worse.

The results of the Multicorridor Project traffic analysis as they were presented to local elected officials and the public are shown in Figures 6 through 8, which show the number of hours of congestion on the freeway system computed from daily traffic assignments for 1980, 2000, and 2020. The first thing to note is that even though the number of hours of congestion were computed with some superficial precision, they were presented in very broad terms, as befits their actual range of accuracy.

The 1980, 2000, and 2020 congestion analysis results provided the same basic information that the more traditional

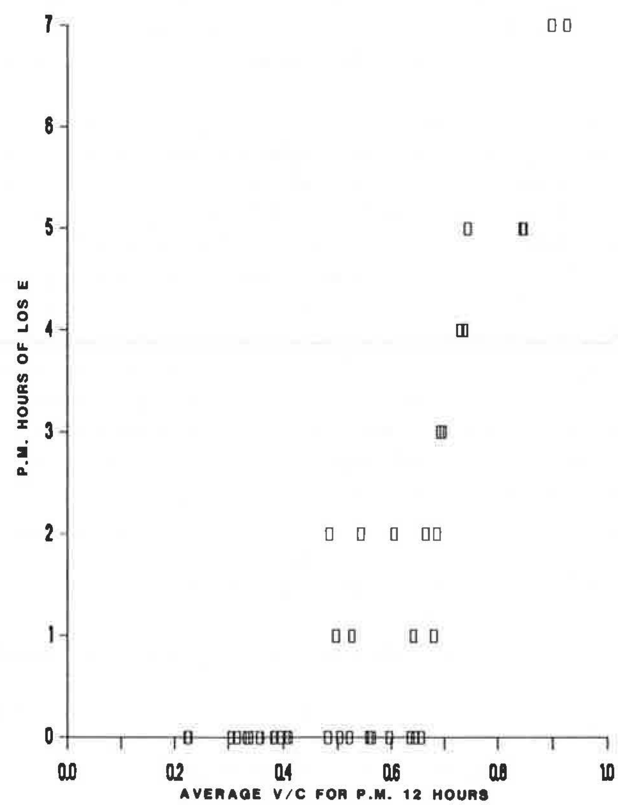

FIGURE 5 Hours of congestion.

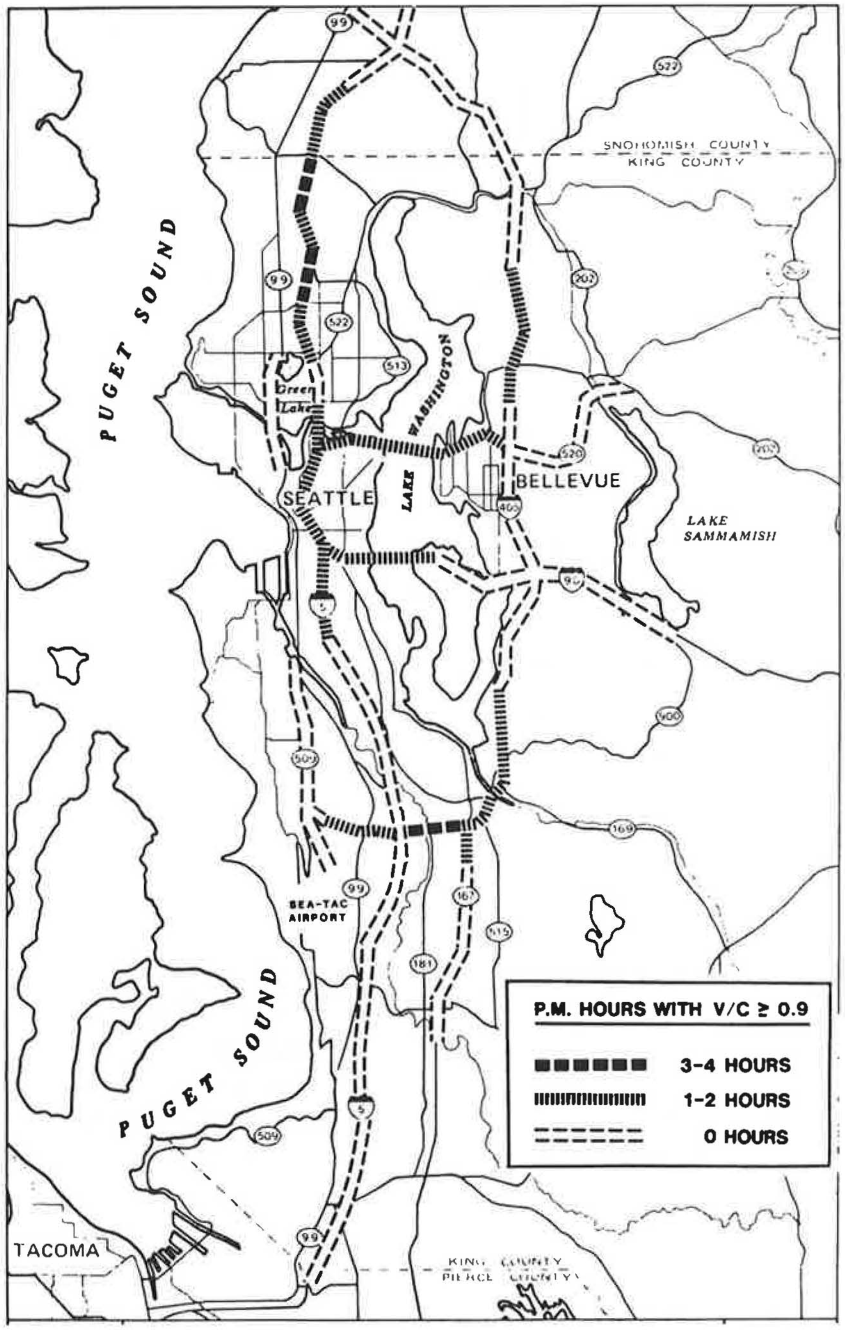

FIGURE 6 Length of p.m. peak, 1980.

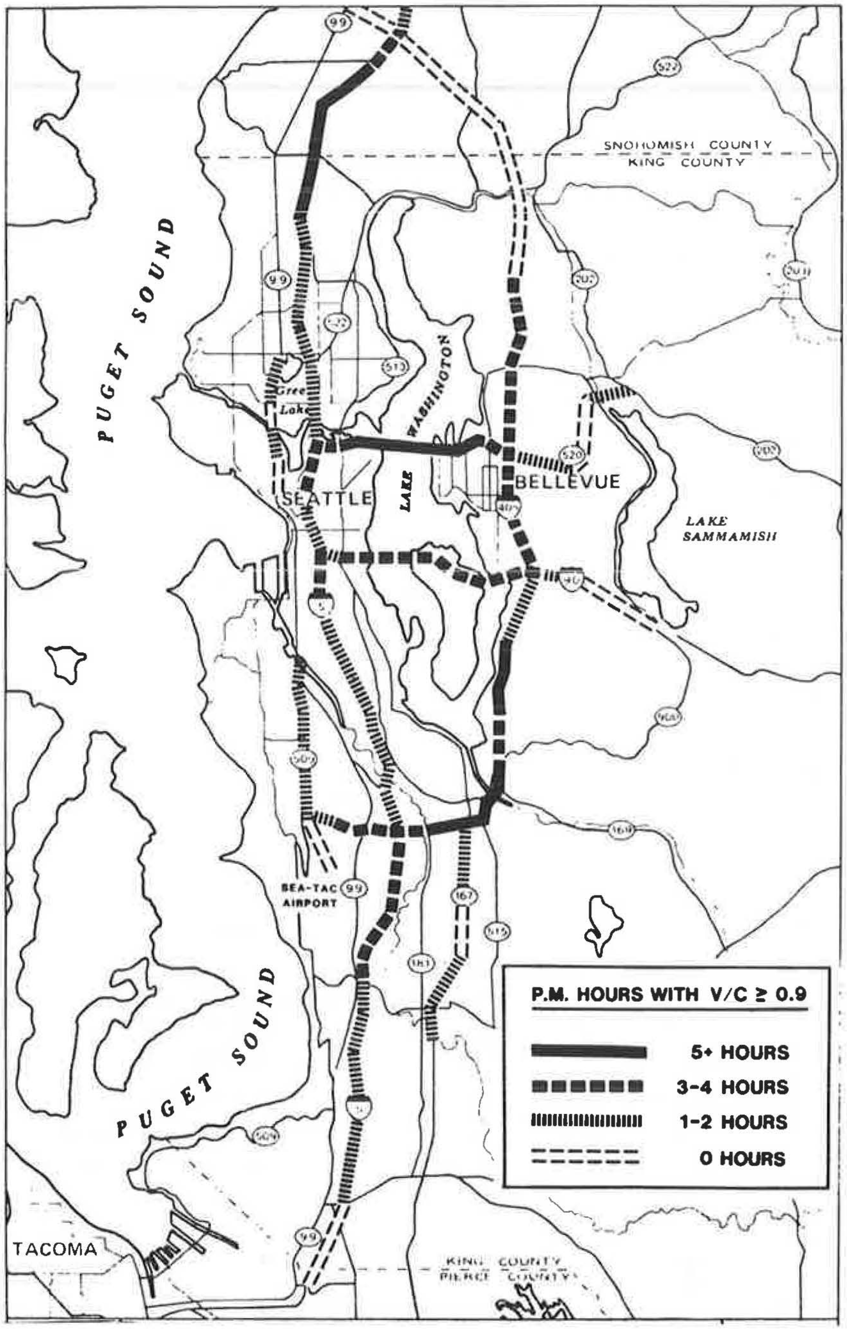

FIGURE 7 Length of p.m. peak, 2000.

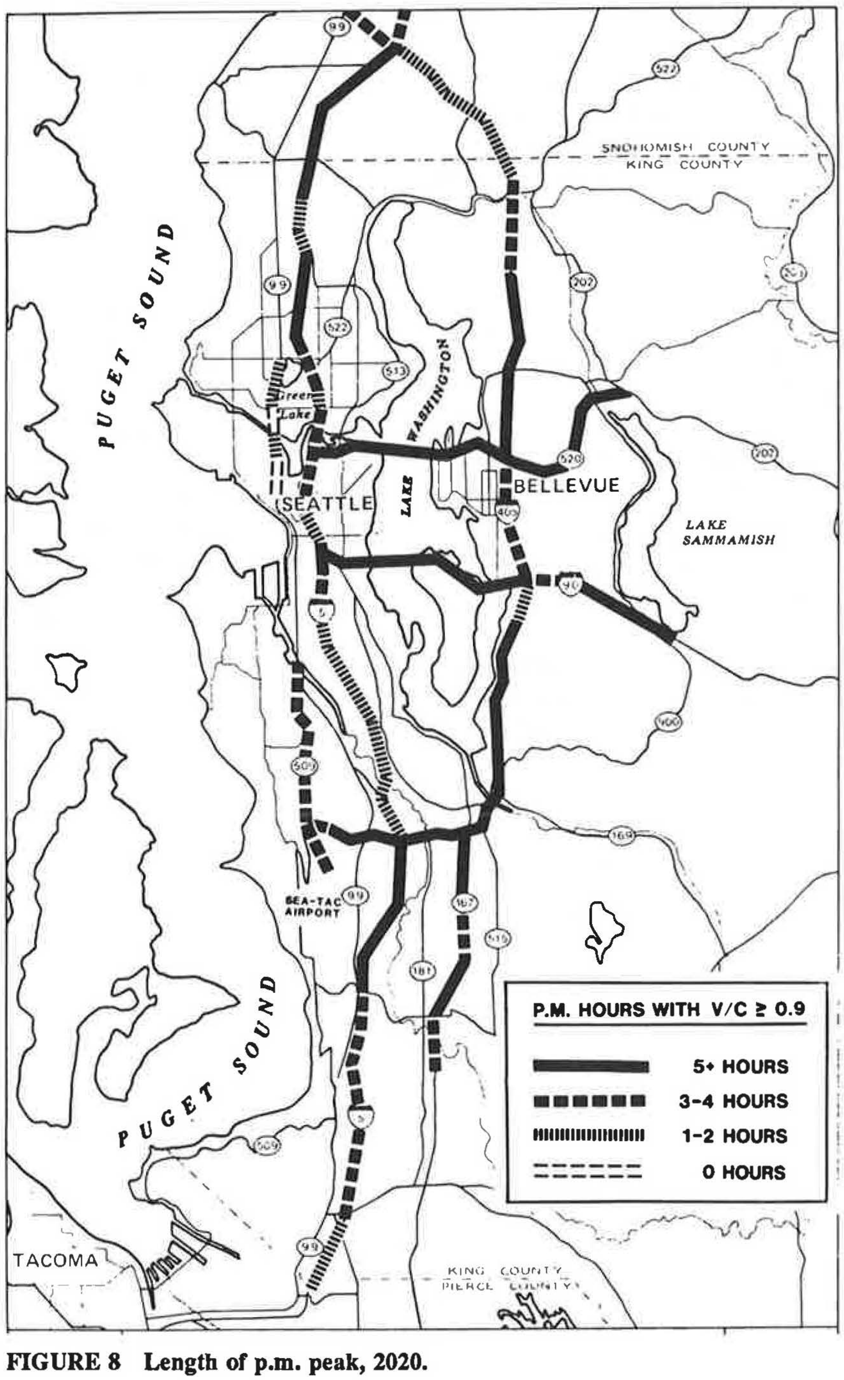

FIGURE 8 Length of p.m. peak, 2020.

analysis of peak-hour v/c ratios provides, for example, geographical distribution of congestion problems, growth of congestion over time, and identification of freeway segments with significant congestion problems. Figures 6 through 8 clearly show the increasing congestion to be expected in the future and the extent to which that increasing congestion spreads out into the suburbs. Also evident is the outward migration of the bottlenecks and constraints that control the overall capacity of the freeway system as a whole. For example, the main capacity constraint on I-5 in the North Corridor (the region's most heavily traveled corridor, extending from downtown Seattle to south Snohomish County) is currently the section just to the north of the downtown. In the future, however, I-5 congestion at the King–Snohomish County line will have more of an effect on who the corridor serves and how it operates than will the closer-in segments.

In addition, however, this congestion analysis provides some things that the more traditional analyses do not. Most important, this analysis gives transportation professionals, as well as elected officials and the public, a good sense not just of relative congestion problems (alternative a versus alternative b, or 2000 versus 2020), but also of the magnitude of those problems in absolute terms. It is easier to relate to, understand, and project what is meant by "5-hr peak period" than "peak hour v/c = 1.21." As a result, the traffic analysis for the Multicorridor Project was more informative and less distracting (from the major transit investment decision at hand) than it would have been otherwise.

By analyzing length of peak directly, the multicorridor traffic analysis also anticipated several questions that invariably arise when peak-hour v/c information is presented. When v/c's in excess of 1.0 show up, professionals and lay persons want to know how much of the excess traffic will actually materialize, how much will be diverted, and how much will divert to traveling at a less congested time. This analysis addressed these concerns before they had to be voiced.

CONCLUSIONS

The traffic analysis approach described in this paper proved to be useful in educational terms. It was also simple and straightforward computationally. The methodology for freeways could be refined by basing the relationship between average v/c and hours of congestion on more actual data and possibly using a curve-fitting technique more sophisticated than linear regression. With adequate traffic count data, the analysis could also be applied to arterials. And finally, this methodology can be used to forecast the number of hours at level-of-service F, or the so-called levels-of-service F-1, F-2, and so on, that are now gaining acceptance in traffic engineering circles.

ACKNOWLEDGMENTS

The project described in this paper was funded in part by appropriations from member jurisdictions of the Puget Sound Council of Governments, in part by support from the Municipality of Metropolitan Seattle (Metro), and in part by grants from UMTA and FHWA through the Washington State Department of Transportation.

Publication of this paper sponsored by Committee on Transportation System Management.

Economic Analysis and Potential Cost Savings Associated with Systemwide Transportation System Management Analysis of Winston-Salem Urbanized Area

EDDIE D. LEGGETT

A systemwide transportation system management (TSM) program is based on a comprehensive TSM planning analysis of an urbanized area that develops TSM alternatives to major construction projects. It consists of nine phases, four of which are programmed and applicable for use on the IBM personal computer. Five of the nine phases as they were applied to the urbanized area of Winston-Salem are discussed: (a) elimination of those capacity-deficient corridors or segments that are readily identified as requiring construction or programmed as near-future construction; (b) review and analysis of the preliminary TSM alternatives associated with those remaining corridors or segments to determine whether they are feasible TSM candidates; (c) determination of cost estimates (capital, maintenance, operational) associated with each element of the TSM alternatives, as well as the costs associated with comparable construction alternatives; (d) determination of the benefit-cost (*B/C*) ratio associated with each TSM alternative and comparative construction alternatives (programmed); and (e) determination of annual capital-cost programs for ridesharing (programmed), staggered work hours (programmed), transit mode split (programmed), traffic engineering, and comparative construction alternatives. With this planning tool, agencies are in a better position to maximize the combined effect of TSM, as well as to formulate the necessary policy directives required for implementation. A prospective TSM management program is also discussed, which presents the author's views on how current TSM programs could be improved.

Since 1985, a local team of transportation managers has been applying a transportation system management (TSM) planning methodology to the urbanized area of Winston-Salem. Their efforts have been based on earlier work and methodology developed by Leggett, which have been documented elsewhere *(1, 2)*.

The urbanized area of Winston-Salem has a population of approximately 147,200 and is located in the western Piedmont region of North Carolina. It is served by an excellent transportation system that facilitates north-south and east-west travel. Although its urban transportation system experiences traffic congestion problems during the peak periods, it is considered to serve the population's travel needs adequately. This urban system has a well-integrated network of streets, some 1,107 mi, which facilitates approximately 5,256,000 daily vehicle miles of travel. It is also supported by excellent transit and ridesharing programs, which carry approximately 11,800 and 15,863 daily person-trips, respectively.

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The systemwide TSM analysis consists of the following nine phases, four of which are programmed and applicable for use on the IBM personal computer:

1. Division of urbanized area into districts that radiate from the central business district (CBD) and consist of complementing corridors and their associated employment centers;
2. Collection of existing and future traffic data associated with major collectors and thoroughfares, as well as existing and future employment data;
3. Determination of existing and future capacity-deficient corridors or segments (programmed) *(1)*;
4. Development of preliminary TSM alternatives for capacity-deficient corridors designed to enable facilities to operate at a desired level of service (programmed) *(1, 3, 4)*;
5. Elimination of those capacity-deficient corridors or segments that are readily identified as requiring construction or programmed as near-future construction;
6. Reviewing and analyzing the preliminary TSM alternatives associated with those remaining corridors or segments to determine whether they are feasible TSM candidates;
7. Determination of cost estimates (capital, maintenance, operational) associated with each element of the TSM alternatives, as well as the costs associated with comparable construction alternatives *(2, 4)*;
8. Determination of the benefit-cost (*B/C*) ratio associated with each TSM alternative and comparative construction alternative (Table 1) (programmed) *(2, 4, 5)*; and
9. Determination of annual capital-cost programs (see Table 5) for (a) ridesharing (programmed), (b) staggered-work-hour (programmed), (c) transit-mode-split (programmed), (d) traffic-engineering, and (e) comparative construction alternatives.

The discussion in this paper focuses on the coordinated planning effort used during Phases 5 and 6, the methodology used to develop the cost estimates of Phase 7, the methodology

TABLE 1 BENEFIT-COST ANALYSIS: WINSTON-SALEM

Project		Do Nothing		Incremental		Best Alternative
Dist	Seg	B/C TSM-Null	B/C Cst-Null	B/C TSM-Cst	B/C Cst-TSM	
1	1	90.6	15.5	0.0	-0.8	TSM
1	2	194.2	98.1	0.0	-9.8	TSM
2	1	297.1	98.2	0.0	-3.2	TSM
3	1	75.8	17.5	0.0	-1.0	TSM
3	2	86.5	5.2	0.0	-0.3	TSM
3	3	640.9	35.8	0.0	-0.8	TSM
3	4	16.2	0.0	0.0	0.0	TSM
3	5	268.3	80.8	0.0	-3.6	TSM
4	1	52.6	10.5	0.0	-0.3	TSM
4	2	82.5	14.6	0.0	-0.5	TSM
4	3	199.1	30.5	0.0	-0.2	TSM
4	4	47.9	49.5	23.2	0.0	TSM
4	5	129.2	35.0	0.0	-3.2	TSM
4	6	100.3	57.2	0.0	-4.4	TSM
4	7	74.0	23.9	0.0	-0.9	TSM
4	8	318.9	29.4	0.0	-0.2	TSM
4	9	117.4	14.2	0.0	-0.2	TSM
5	1	40.6	0.0	0.0	0.0	TSM
5	2	91.8	100.8	19.8	0.0	TSM
5	3	435.3	1.6	0.0	-0.1	TSM
5	4	264.0	28.2	0.0	-0.7	TSM
5	5	129.2	57.4	0.0	-2.5	TSM
5	6	565.7	73.7	0.0	-0.8	TSM
5	7	161.2	47.6	0.0	-1.7	TSM
5	8	207.2	0.0	0.0	0.0	TSM
6	1	210.4	93.0	0.0	-3.0	TSM
6	2	98.3	16.4	0.0	-0.7	TSM
6	3	20.9	18.5	0.0	-7.9	TSM
6	5	902.5	0.0	0.0	0.0	TSM
6	6	58.1	8.3	0.0	-0.5	TSM
6	7	153.1	22.9	0.0	-0.6	TSM
6	8	198.0	20.7	0.0	-0.6	TSM
6	9	157.7	1.8	0.0	-0.1	TSM
7	1	57.9	1.9	0.0	-0.1	TSM
7	2	105.3	8.4	0.0	-0.2	TSM
7	3	155.6	0.9	0.0	0.0	TSM
7	4	50.8	4.2	0.0	-0.1	TSM
7	5	19.6	10.4	0.0	-0.2	TSM
7	6	183.3	66.7	0.0	-0.3	TSM
7	7	88.9	6.9	0.0	-0.3	TSM
8	1	194.9	110.9	0.0	-1.4	TSM
8	2	477.1	64.6	0.0	-0.3	TSM
8	3	110.3	31.8	0.0	-0.7	TSM

Note: Under Do Nothing, when B/C (Cst-Null)=0.0 indicates construction was not considered as an alternative and only B/C (TSM-Null) was evaluated.

: Under Incremental, a 0.0 indicates this alternative was not evaluated because its comparable alternative under Do Nothing produced a greater B/C ratio which makes it the defender.

used to determine the *B/C* ratios of Phase 8, the methodology used to determine the annual capital-cost estimates of Phase 9, and the prospective TSM management process.

The comprehensive TSM analysis has been designed to determine TSM alternatives that can be used to defer or replace construction alternatives. Once the urbanized area has been divided into districts, the analysis proceeds with the determination of existing (1983 = base year) and forecast (2000 = design year) transportation demands for the area's major collectors and thoroughfares, as well as the existing effects of TSM on such corridors (i.e., ridesharing; staggered work hours, if known; and transit mode split).

For each of the major collectors and thoroughfares, the local thoroughfare plan was used to determine

- Vehicle capacity at level-of-service E during the base year:

$$EVC = \sum_{S=1}^{n} [(\text{segment's distance}_s/\text{total distance}) \times (\text{segment's capacity}_s)] \text{ or most restricting capacity}$$

- Existing average daily traffic (*ADT*) volume during the base year:

$$EADT = \sum_{S=1}^{n} [(\text{segment's distance}_s/\text{total distance}) \times (\text{segment's } ADT_s)] \text{ or segment's highest ADT}$$

- Existing peak-hour directional flow factor (*PHDF*).
- Existing peak-hour traffic volume factor during the base year (*PHF*).
- Forecast ADT for the design-year volume (ADT_{di}).

The local ridesharing and transit programs provided the following TSM elements for each of the designated corridors:

- Peak-hour vehicle occupancy rate during the base year (VOR_{bi}).
- Peak-hour percent transit mode split during the base year (TMS_{bi}).
- Peak-hour percent staggered work hours during the base year (SWH_{bi}) (this was not evaluated for Winston-Salem).

Of the 180+ corridors or segments evaluated in Winston-Salem, 100 proved to have an existing or future capacity problem. However, only 43 of these were determined to be feasible TSM candidates, whereas the remaining 57 problem areas were judged definitely to require a construction solution.

COORDINATED TSM PLANNING PROCESS

During this phase of the study, the management-level transportation professionals responsible for each of the TSM elements (i.e., the transportation planner, ridesharing coordinator, transit director, and traffic engineering manager) collectively reviewed and evaluated the potential for the preliminary TSM alternatives associated with each of the 100 capacity-deficient corridors or segments. Each alternative consisted of at least one and in most cases a combination of the following TSM options:

- Vehicle occupancy rate (*VOR*)
- Staggered work hours (*SWH*)
- Transit mode split (*TMS*)
- Traffic engineering (*TE*)

These alternatives were developed from a computer model on the basis of operational characteristics of a particular corridor or segment [i.e., base-year capacity (EVC_{bi}), estimated ADT volumes during base year and design year ($EADT_{bi}$ and $EADT_{di}$), vehicle occupancy rate during base year (VOR_{bi}), and transit mode split during base year (TMS_{bi})].

For a given corridor, the designated magnitude of each TSM alternative (i.e., which one is designated to accommodate the percent of peak-hour person or vehicle trip demand in excess of the facilities' peak-hour vehicle capacity at level-of-service D) is directly associated with the existing effects of TSM. For example, if the corridor or segment has no transit, transit is not considered as an option, and if it has lower-than-average participation from ridesharing and staggered work hours, these options are weighted accordingly. Therefore, corridors of this nature place greater emphasis on traffic engineering, which makes them easy to eliminate when the traffic-engineering option requires extensive construction.

This element of the planning process is considered a valuable planning tool. The professionals are provided with a potential alternative based on a corridor's operational characteristics without having to perform the cumbersome calculations associated with TSM evaluations. For each alternative, the number of additional person-trips required to be accommodated was given for the non-traffic-engineering options, and the number of vehicles was given for traffic-engineering options. With these values the professionals had the necessary components to evaluate the potential for their programs to effectively manage a particular traffic congestion problem.

Criteria used by each of the non-traffic-engineering professionals to determine whether their individual program could accommodate the additional number of person-trips associated with each option were based on the concentration of employment centers, past employee-employer participation, nature of employment base, available land resources for park-and-ride facilities, and available transit service within the particular area of interest.

For each of the traffic-engineering options, traffic engineers evaluated the ability of the existing corridor or segment to accommodate additional vehicles on the basis of improving intersection geometrics, re-marking existing pavement, improving signal optimization, minor widening of shoulders, and making reversible lanes to enhance the existing roadway capacity.

This coordinated planning process consisted of approximately five meetings. Initially the group eliminated the following types of projects: those projects programmed and budgeted as immediate Transportation Improvement Program (TIP) construction needs that were scheduled to be built in the near future, those that could not be mitigated by TSM because of severe capacity problems or their nonsupporting TSM criteria, and those being complemented by ongoing or near-future construction projects.

On completion of the elimination process, the professionals examined the details of each TSM option and the associated operational characteristics of each corridor or segment to determine the feasibility of the TSM alternative. As with the foregoing case, a number of these projects were also dropped and designated as construction projects.

Throughout the evaluation of each preliminary TSM option, the various options were altered, if necessary, to produce a TSM package more applicable to a particular corridor or segment. For example, if an individual indicated that his program could not adequately accommodate the required persons or vehicles, an informal dialogue would begin. One person might remind the transit manager of a transit route that could be extended to the particular project area. The transit manager might respond by acknowledging the fact that the particular route extension was possible and that the idea had great potential. Therefore, the demand of one or all of the other TSM options would be reduced to make the TSM alternative feasible. This type of dialogue would continue throughout the meeting and produce an invaluable exchange of ideas.

This multimodal planning process enabled the professionals to maximize the combined effects of their programs, thereby enabling them to identify a greater number of projects that might be significantly affected by TSM.

BENEFIT-COST ANALYSIS AND PROJECT COST DETERMINATION

In order to assess the economic benefits associated with the 43 TSM projects, each TSM alternative and a comparative construction alternative were subjected to an economic analysis by using the traditional benefit-cost ratio (*B/C*) method *(5)*. This method determines the ratio of benefits to costs after each project or alternative has been discounted with respect to its expected life at a minimal attractive rate of return [i.e., all projects or alternatives were discounted to an equivalent uniform annual cost (EUAC) on the basis of their expected life, which was judged to be the difference between the design year and the year the problem occurs plus 1, and a minimal attractive rate of return of 10 percent]. In other words, both TSM and construction alternatives were designed to accommodate the capacity problem from the year of inception through the design year 2000. Projects with *B/C* ratios greater than 1.0 are considered economically attractive and those with *B/C* ratios less than 1.0 are considered not to be economically efficient.

When projects with different benefits and costs are compared, it is necessary to conduct an incremental benefit-cost analysis. First the *B/C* ratio for each project alternative is compared with the null alternative. If the *B/C* ratios of both alternatives are greater than 1.0, incremental benefit-cost analysis is used to determine which alternative is more economically advantageous. The process is a repeat of the direct *B/C* ratio approach, with the exception of replacing the null alternative with the alternative (TSM or construction) that has the highest B/C ratio. This alternative then becomes the "defender" and the benefits and costs of the other alternative are compared with those of the defender. If the B/C ratio in this comparison is less than 1.0, the costs associated with the defender produce a greater rate of return; however, if the B/C ratio is greater than 1.0, the costs associated with the other alternative produce a greater rate of return. To illustrate the procedure of the incremental benefit-cost analysis, an actual project has been evaluated as follows (Table 2).

The basis alternative (null) is taken as the defender:

$$B/C = \frac{R_{null} - R_{TSM}}{(C_{TSM} + M_{TSM}) - M_{null}}$$
$$= \frac{(12{,}590{,}765 - 9{,}680{,}800)}{(15{,}253 + 6{,}453) - 2{,}700}$$
$$= \frac{2{,}909{,}965}{19{,}006}$$
$$= 153.1$$

$$B/C = \frac{R_{null}\ R_{const}}{(C_{const} + R_{const}) - M_{null}}$$
$$= \frac{(12{,}590{,}765 - 9{,}741{,}766)}{(121{,}930 + 5{,}401) - 2{,}700}$$
$$= \frac{2{,}848{,}999}{124{,}631}$$
$$= 22.9$$

Because both the TSM and construction alternatives produce B/C ratios greater than 1, which implies that they are economically advantageous at the same minimum attractive rate of return, it is necessary to conduct an incremental benefit-cost analysis to determine which investment should be pursued. The TSM alternative has a greater B/C ratio than the construction alternative, so it is regarded as the defender against the construction alternative in the incremental benefit-cost analysis, as follows:

$$B/C = \frac{\text{incremental benefits}}{\text{incremental costs}}$$
$$\frac{R_{null} - R_{const}}{(C_{const} + M_{const}) - (C_{TSM} + M_{TSM})}$$
$$\frac{9{,}680{,}800 - 9{,}741{,}776}{(121{,}930 + 5{,}401) - (15{,}253 + 6{,}453)}$$
$$= \frac{-60{,}976}{105{,}625}$$
$$= -0.6$$

As can be seen, the construction alternative produces a negative B/C ratio when compared with the TSM alternative, suggesting that the TSM alternative is the more attractive investment. In both cases, the TSM costs are less than the construction costs. In other words, the total of implementation and maintenance costs as well as the user costs incurred by the TSM alternative is less, implying that the TSM alternative provides greater benefits for less cost.

The results of this analysis (Table 1) identify each project by a particular district and segment number. For each of the TSM alternatives, with the exception of the two projects designated 4, 4 and 5, 2, the investment costs and the road user costs were less than those costs associated with the comparative construction alternative. In most cases, as reflected by the B/C ratios, these costs were considerably less because their improvements do not extend beyond the peak hours, as do the construction improvements.

In the case of Projects 4, 4 and 5, 2, the TSM investments over the life of the projects were greater than the construction investments. But estimated TSM user costs were less as a result of the reduced number of vehicles to be operating on the facility by the design year, which explains why the construction alternatives proved to be less cost-efficient in the incremental analysis. In other words, the TSM investments provide a greater rate of return per dollar investment to the extent that their benefits accruing to the public outweigh the benefits of the less expensive construction alternatives. Projects of this nature would be considered the least desirable TSM candidates and would most likely be better served by construction.

Table 3 presents two examples of the proposed TSM and comparative construction alternatives. Table 3 is designed to provide a general overall description of the specific actions and cost requirements of, as well as economic benefits associated with, each TSM project and its comparative construction alternative. With this generalization, decision makers can quickly view the potential merits associated with each project recommendation.

To simplify the analysis, the project costs for the construction capital program and the project costs for the transit and traffic engineering portion of the TSM capital program are programmed for the year in which the capacity problem occurs. It is realized that this is not always the case because of the time required for completion of preliminary engineering and construction, which take at least a year or more for construction projects. For the ridesharing and staggered-work-hour options of the TSM capital program, project costs are prorated and spread out over the life of the project.

The two elements of the economic analysis used to determine the B/C ratio for the construction and TSM alternatives, as set by AASHTO, are road user costs and highway costs. The calculation of these costs has been based on urban arterial highway sections, which are assumed to have homogeneous operational characteristics.

TABLE 2 PROJECTS, DISTRICT 6, SEGMENT 7

Alternative	Investment ($)	Capital Costs (C) ($)	Operating and Maintenance Costs (M) ($)	User Costs (R) ($)	B/C Ratio
Null			2,700	12,590,765	
TSM	125,099	15,253	6,453	9,680,800	153.1
Construction	1,000,000	121,930	5,401	9,741,776	22.9

NOTE: Annual items: i = 10 percent; $n = (YR_{di} - YR_{pi}) + 1$.

TABLE 3 PROJECT ALTERNATIVES AND ECONOMIC VALUES USED TO DETERMINE RECOMMENDED ACTION: WINSTON-SALEM

Corridor					Description of Improvements[a]		Costs ($)				B/C Ratio		Recom-mended Action
							TSM		Construction				
District	Segment	Miles	Lanes	Problem	TSM	Construc-tion	Capital	M&O	Capital	M&O	TSM	Construc-tion	
. . .													
6	7: Country Club Road between Silas Creek and Lindburgh	0.8	2	1983	RST = 57 (CP); SWHT = 30 (B&A); TMST = 79; TET = F	+2 lanes	RST = 17,820; SWHT = 10,680; TMST = 81,599; TET = 15,000; TOT = 125,099	TMST = 30,773; TET = 22,147; TOT = 52,921	1 million	44,292	153.1	22.9	TSM
6	8: Country Club Road between Old Vineyard and Peacehaven	1.3	2	1984	RST = 55(CP); SWHT = 28(B) and 29(A); TMST = 72; TET = F	+2 lanes	RST = 16,200; SWHT = 9,600; TMST = 74,369; TET = 15,000; TOT = 115,169	TMST = 28,047; TET = 33,990; TOT = 62,036	1.3 million	67,980	198.0	20.7	TSM
. . .													

[a]Description of improvements is as follows:

RST (Ridesharing Task) = number of carpools required (CP); CP = number of person-trips to be accommodated/[persons per carpool minus existing vehicle occupancy rate (VOR) of specific corridor] (i.e., Project 6,7 above was based on 98 person-trips to be accommodated, 3 persons per carpool, and 1.28 existing VOR).

SWHT (Staggered-Work-Hour Task) = number of additional person-trips to be diverted before (B) and after (A) the peak periods (i.e., Project 6,7 above was based on 60 additional person-trips to be diverted by design year 2000).

TMST (Transit Mode-Split Task) = number of additional person-trips to be accommodated by design year 2000.

TET (Traffic Engineering Task) = repave and restripe to add an additional lane (F) (i.e., Project 6,7 was based on 161 vehicles to be accommodated by design year 2000).

Construction = roadway widening to add an additional lane in each direction (i.e., Project 6,7 above was based on 332 vehicles to be accommodated).

Road user costs (*RUC*) are basically those incurred by the traveling public that result from the operational characteristicsof both highway and vehicle. The three factors that were used to calculate those costs are vehicle operating costs (*VOC*) travel-time costs (*TTC*), and accident costs (*AC*) (i.e., $RUC = VOC + TTC + AC$).

The computational procedures used to determine the various road user costs are detailed elsewhere *(5, 6)*. However, the procedure used to estimate these costs for the TSM alternative has been altered *(2)*.

TSM alternatives are designed to manage the peak-hour traffic volumes, and the cost reductions and costs incurred by these alternatives reflect the resulting improvements to peak-hour traffic congestion. To account for the improvements occurring during the peak hours, it is necessary to evaluate the peak-hour traffic volumes. The design-year estimated ADT ($EADT_{di}$) and the desired estimated ADTs required to maintain level-of-service D ($EADT_{di}$), or vehicle capacity at level-of-service D, are converted to peak-hour volumes.

The difference between these peak-hour volumes ($PHEADT_{di} - PHEADT'_{di}$) is the number of vehicles required to be reduced, diverted, or accommodated by the TSM alternative to allow the existing facility to operate at level-of-service D. Therefore, the only difference in the process used to compute the road user costs for TSM alternatives is the procedure used to determine the reduction in the design-year ADTs that result from the TSM improvement.

The process used to estimate the capital-cost and maintenance-and-operating-cost elements of the highway costs for the construction alternative differs somewhat from that for the TSM alternative. The highway costs for the construction alternative were provided by the Thoroughfare Planning Unit of the North Carolina Department of Transportation (NCDOT), Winston-Salem's Traffic Engineering Section, and the NCDOT Maintenance and Equipment Branch. The capital costs consisted primarily of drainage, pavement, right-of-way acquisition, and signalization costs, and maintenance and operating costs included equipment, materials, labor, and administration. Project-specific detailed engineering estimates prepared for each alternative construction project were used for capital cost estimates. Maintenance and operating costs ($769.00/lane-mile) were average and were derived by dividing the total annual urban highway maintenance and operating costs of the state's urban system by the total lane miles for the state's urban system.

When the costs are developed for a TSM alternative, which acts to defer the need for construction, it is necessary to determine or estimate the costs associated with implementing and continuing the TSM alternative during the peak hour. These costs are somewhat subjective and very sensitive to an area's political, social, and economic environment.

The TSM alternatives consisted of various combinations of the TSM actions identified earlier: increasing the vehicle occupancy rate, the effects of staggered work hours, and the transit mode split and improving operational characteristics through traffic engineering. For each of these options, the estimated capital costs and maintenance and operating costs associated with their initial implementation and continuation were determined from ongoing TSM activities both within the state and in Winston-Salem.

The capital costs for the traffic engineering options were determined by the Winston-Salem traffic engineering staff. Any roadway improvements (e.g., intersections, paving, widening) were based on the same criteria as was the construction alternative, and any signalization or signing was based on purchase price and installment costs.

Before the capital costs associated with the non-traffic-engineering TSM options are estimated, it is necessary to determine the number of additional peak-hour person-trips that each of these options is to accommodate or divert by the design year. As discussed earlier, these person-trips are determined during Phase 4 of the analysis, when the preliminary TSM alternatives are developed. These alternatives are designed to accommodate a portion of the estimated design-year peak-hour person-trip demand as required to enable the facility to operate at level-of-service D.

For the vehicle-occupancy-rate and staggered-work-hour TSM improvements, a cost of $20 a person to divert an individual from a single-occupancy vehicle (SOV) to a high-occupancy vehicle (HOV) was used to estimate the capital costs. This figure was derived by evaluating the annual average costs and participation of the state's various ridesharing programs (i.e., cost per person = state's total annual ridesharing costs divided by estimated total ridesharing participation). Although this cost is subjective and sensitive to an area's environment, it was believed to be representative of Winston-Salem's ridesharing program. However, this cost was believed to be higher than the actual costs of diverting individual travel from the peak hour, but it was used in order to preclude the probability of underestimating the cost of the staggered-work-hour option. Also, the estimate is believed to be ambitious in light of existing transportation policy, which is based on support for construction alternatives.

In order to estimate the capital costs associated with improvements in the vehicle occupancy rate ($VORC_i$) and in staggered work hours ($SWHC_i$), it is necessary to determine the cumulative effect associated with the number of person-trips to be accommodated by these improvements. The cost of $20 a person is considered to be a continuing cost and each trip accommodated by these options requires this expenditure to be maintained from the date that an individual begins participating through the design year. Therefore, the annual costs associated with the person-trips of these options, which are prorated from the inception of the capacity problem through the design year, have to be incurred each year during this period.

The methodology used to estimate the capital costs associated with the TSM transit mode-split option ($TSMC_{TSMi}$) was based on the purchase price of a bus ($135,000) with an expected life of 15 years; the capacity of each route, determined by the number of buses operating, frequency of operation, number of seats per bus (40), and number of standees allowed per bus during the peak hour (15); the existing ridership for each route during the peak hour; and the number of additional person-trips that each route was designated to accommodate during the planning period by the TSM analysis.

For the planning period, the TSM transit mode split option was designated to accommodate 1,307 additional person-trips during the peak hour, requiring an additional 10 buses to be added to the area's existing peak-hour fleet of 40 buses.

In order to associate a capital cost with each of the TSM transit mode split options, a transit mode split person-trip ratio was determined for each option. This ratio was derived by dividing the number of additional person-trips to be accommodated by each transit mode split option ($PTTSM_{TSMdi}$) by the total number of additional person-trips to be accommodated by transit for the entire urban area ($TPTTMS_{TSMj}$). The result was multiplied by the total transit capital costs ($1.35 million).

The transit and traffic engineering options were the only ones considered to have costs that could be adequately termed maintenance and operating costs as compared with the construction alternative. However, the cost of $20 a person associated with the ridesharing and staggered-work-hour program was judged adequate to cover administrative and operating costs.

The maintenance and operating costs associated with the traffic engineering option (TE_{TSMi}) were determined by the Winston-Salem traffic engineering staff. These costs were negligible in most cases, with only those projects requiring roadway widening and major intersection improvements being considered to have increased maintenance and operating costs. Also, the maintenance and operating costs of the null alternative (i.e., $769/lane-mile) are included as part of these costs.

The methodology used to estimate the transit maintenance and operating costs was based on the number of additional buses to be added during the planning period (10), the number of years that the additional buses would be operational during the planning period (44), and the estimated annual peak-hour maintenance and operating cost associated with each bus ($11,571). In order to associate a maintenance and operating cost with each of the TSM transit mode split options, the transit mode split person-trip ratio ($PTTMS_{TSMdi}/TPTTMS_{TSMj}$), as used to estimate the capital costs, was multiplied by the total operation subsidy ($509,124).

CAPITAL-COST PROGRAMS

The development of the capital-cost program for the systemwide TSM analysis was not based on Winston-Salem's available transportation revenues. Instead, the program was based on the funding needed to finance the TSM needs package in its entirety, as well as an alternative construction needs package. This approach was taken in order to properly document the potential cost saving associated with the implementation of the systemwide TSM program, as well as to document the required expenditures needed to finance an ongoing long-range TSM program responsive to the area's multimodal transportation needs.

The annual financial requirements (i.e., capital-cost portion, as discussed earlier) were determined by associating a cost with each element of the TSM alternatives and a cost with each of the comparative construction alternatives. These costs were then totaled to develop the TSM and construction capital-cost programs shown in Table 5 (discussed later).

As stated earlier, the capital costs for the ridesharing and staggered-work-hour options include both administrative and operational costs, whereas the capital costs for the other TSM options and construction alternatives do not. However, because of the variable nature of these costs, no effort was taken to separate them, and no appreciable error in capital-cost estimates is believed to be associated with these additional costs.

Table 4 gives the annual capital costs and the total capital costs associated with each of the TSM improvements for the project discussed earlier (District 6, Segment 7: Country Club Road between Silas Creek and Lindburgh), as well as the annual capital costs and total capital costs associated with the entire TSM improvement.

Based on the design level-of-service D v/c ratio of 0.85, this project's v/c ratio of 0.88 (ADT/EVC = 14,000/16,000) suggests that this facility has an existing capacity problem. Moreover, by the design year 2000 it is estimated that it will have an ADT of 19,180 ($EADT_{di}$), which produces a v/c ratio of 1.20.

In order for capacity problems that occur most often during the peak hour to be evaluated by TSM, the existing vehicle capacity or ADT based on level-of-service D ($EADT_{Di}$ = 16,000), the base-year ADT ($EADT_{bi}$ = 14,000), and the design-year ADT ($EADT_{di}$ = 19,180) were converted to peak-hour volumes as follows:

$$PHEADT_{Di} = 816$$

$$PHEADT_{bi} = 840$$

$$PHEADT_{di} = 1,151$$

These volumes indicate that the facility's existing peak-hour capacity at level-of-service D ($PHEADT_{Di}$ = 816) at present cannot adequately accommodate 24 vehicles (24 = 840 − 816), and will not be able to adequately accommodate 335 vehicles (335 = 1,151 − 816) by the design year.

On the basis of this capacity deficiency, it was estimated that an additional lane in each direction costing $1 million would be required if this deficiency were to be accommodated by a construction alternative.

Alternatively, it was estimated the capacity deficiency could be mitigated by TSM for a capital cost of $125,099, which consisted of the following peak-hour improvements:

- Increase the existing VOR of 1.28 to 1.38 by the design year through a ridesharing program (VOR_{TSMdi} = 1.38), which would enable the facility to accommodate an additional 98 person-trips at a cost of $17,820;
- Divert 4 percent of the person-trip demand by design year through a staggered-work-hour program (SWH_{TSMi} = 0.04), which would enable the facility to divert 60 person-trips from the peak hour at a cost of $10,680;
- Increase the transit mode split of 1 percent to 6 percent by design year (TMS_{TSMdi} = 0.06), which would enable the facility to accommodate 79 additional person-trips at a cost of $81,599; and
- Increase existing capacity by 19.82 percent, which was scheduled during the base year (TE_{TSMdi} = 0.1982) and would enable the facility to accommodate an additional 161 vehicles at a cost of $15,000.

The non-traffic-engineering improvements were designed to reduce or divert a portion of the peak-hour vehicle demand, whereas the traffic-engineering improvement was designed to increase the peak-hour level-of-service D roadway capacity.

A composite of these annual improvements and costs, as shown in Table 4, is the method used to develop the TSM capital-cost program. For the ridesharing and staggered-work-hour improvements, the scheduling of annual costs is based on

TABLE 4 TSM ANNUAL CAPITAL COSTS AND ASSOCIATED PERSON OR VEHICLE TRIPS TO BE ACCOMMODATED: WINSTON-SALEM

Year	Ridesharing		Staggered Work Hours		Transit		Traffic Engineering		Total Costs
	NAPT/Y	Costs	NAPT/Y	Costs	NAPT/Y	Costs	NAVT/Y	Costs	
1983	5	$ 100	3	$ 60	4	$ 0	15	$15,000	$ 15,600
1984	5	200	3	120	4	0		0	320
1985	5	300	3	180	4	0		0	480
1986	5	400	3	240	4	0		0	640
1987	5	500	3	300	4	0		0	800
1988	5	600	3	360	4	8,159		0	9,119
1989	5	700	3	420	4	0		0	1,120
1990	5	800	3	480	4	0		0	1,280
1991	5	900	3	540	4	0		0	1,440
1992	5	1,000	3	600	4	0		0	1,600
1993	6	1,120	3	660	4	0		0	1,780
1994	6	1,240	3	720	5	16,320		0	18,280
1995	6	1,360	4	800	5	0		0	2,160
1996	6	1,480	4	880	5	8,160		0	10,500
1997	6	1,600	4	960	5	0		0	2,560
1998	6	1,720	4	1,040	5	8,160		0	10,920
1999	6	1,840	4	1,120	5	32,640		0	35,600
2000	6	1,960	4	1,200	5	8,160	161	0	11,320
Totals	98	$17,820	60	$10,680	79	$81,599	161	$15,000	$125,099

NAPT/Y = Number of Additional Person-Trips per year required to be accommodated during the peak hour.

NAVT/Y = Number of Additional Vehicle-Trips per year required to be accommodated during the peak hour.

an annually prorated portion of the costs associated with their design-year improvements, beginning with the year that the facility is expected to exceed capacity through the design year. Although the transit improvement is prorated in the same manner as the foregoing two options, the scheduling of capital cost is treated somewhat differently as a result of the manner in which expenditures are required to purchase the additional transit stock. For example, the cost to purchase the 10 buses required to meet design-year transit demand during the planning period (one during 1988, two during 1994, one during 1996, one during 1998, four during 1999, and one during 2000) is shown in Table 4 for these years, which is when additional transit capacity is needed. During those years, the costs of each transit improvement include a weighted portion of the capital expenditure required to purchase additional transit stock.

Unlike the foregoing TSM options, the traffic engineering costs are scheduled for the year in which the capacity problem has been estimated to occur, as are the construction costs.

Figure 1 presents a comparison of the capital costs required to fund the 15-year capital improvement programs for both the construction and TSM scenarios detailed in Table 5. These scenarios are based on 1983 base data. As shown in Figure 1, because the needs associated with the 1983 and 1984 costs have not yet been met, these costs have been carried forward to 1985.

Not surprisingly, the construction scenario would require a capital layout many times larger than the TSM scenario, with 1985 requiring the largest initial capital expenditure for both scenarios. The capital layout of $2.90 million for TSM during 1985 comprises 48.01 percent of total TSM program costs ($6.04 million), whereas the capital layout of $16.60 million for construction during 1985 comprises 50.61 percent of total construction program costs ($32.08 million). These initial costs associated with both scenarios represent a substantial portion of the program needs that have not yet been met, indicating that Winston-Salem's transportation funding requirements have not kept pace with its transportation demands.

The potential capital-cost savings associated with the implementation of the systemwide TSM program are overwhelming when compared with those for the construction program. A comparison of the TSM and construction cumulative capital costs associated with the 15-year capital improvement program (1985 to 2000) costs as shown in Figure 1 reveals the following:

1. TSM cumulative capital costs for 1985 ($2.90 million) comprise 19.52 percent of comparable construction capital costs ($16.60 million); that is, the construction program is 4.72 times more costly than the TSM program, thus indicating a potential cumulative capital-cost savings of $13.70 million associated with the TSM program.

TABLE 5 COMPARISON OF ANNUAL CAPITAL COSTS BETWEEN TSM AND CONSTRUCTION ALTERNATIVES: WINSTON-SALEM

Year	Transportation System Management (TSM)					Construct
	Rideshare	Stg. Wr. Hr	Transit	Traf. Eng.	Total	
1983	$ 1680	$ 400	$ 0	$ 2754800	$ 2756880	$ 14493200
1984	3940	960	0	117000	121900	1700000
1985	6440	1560	0	10600	18600	405000
1986	9100	2200	0	77400	88700	1165000
1987	12040	2920	0	309600	324560	1942100
1988	15760	3760	135000	75000	229520	1330000
1989	19600	4660	0	205000	229260	1042200
1990	24720	5840	0	215300	245860	4293000
1991	29940	7020	0	0	36960	0
1992	35420	8260	0	21800	65480	1400000
1993	41720	9600	0	0	51320	1539000
1994	48140	11020	270000	20300	349460	2100000
1995	54800	12520	0	13200	80520	80000
1996	61640	14140	135000	0	210780	534600
1997	68520	15820	0	0	84340	0
1998	75640	17740	135000	31000	259380	55000
1999	82800	19740	540000	0	642540	0
2000	90080	21780	135000	0	246860	0
Totals:	$ 681980	$ 159940	$ 1350000	$ 3851000	$ 6042920	$ 32079100

2. TSM cumulative capital costs for 1990 ($4.02 million) comprise 15.24 percent of the comparable construction capital costs ($26.37 million); that is, the construction program is 5.60 times more costly than the TSM program, thus indicating a potential cumulative capital-cost savings of $22.35 million associated with the TSM program.

3. TSM cumulative capital costs for 1995 ($4.60 million) comprise 14.60 percent of the comparable construction capital costs ($31.50 million); that is, the construction program is 5.85 times more costly than the TSM program, thus indicating a potential cumulative capital-cost savings of $26.90 million associated with the TSM program.

4. TSM cumulative capital costs for 2000 ($6.04 million) comprise 18.87 percent of the comparable construction capital costs ($32.08 million); that is, the construction program is 4.31 times more costly than the TSM program, thus indicating a potential cumulative capital-cost savings of $26.04 million associated with the TSM program.

As shown by both Table 5 and Figure 1, the successful implementation of the systemwide TSM program (recognizing the need for innovative TSM policy changes and separate TSM funding sources) would have the ability to generate substantial cost savings and at the same time provide an acceptable level of service comparable to the construction program. The potential cumulative cost savings ($26.04 million) associated with the 15-year TSM capital program suggest that every dollar invested in TSM would have the ability to defer or replace $5.31 in construction costs or generate $4.31 in savings. The potential to increase the purchasing power of scarce tax dollars would be great. However, in the real world, it is obvious that neither of these programs would be implemented in its entirety.

The primary advantage of such an analysis is that it identifies the potential cost savings that could be realized from TSM. Also, it establishes a comprehensive approach designed to estimate TSM program needs based on future transportation demand, comparable to the highway construction program. This process also enables TSM professionals to document the economic advantages of modal trade-offs in their efforts to justify specific budget requirements.

For example, consider the recurring political implications associated with the highway construction program when it needs additional tax dollars. Without supporting documentation based on a comprehensive long-range planning approach designed to forecast transportation needs, the justification for public tax dollars would become even more burdensome. Not only does this urban highway planning approach help to justify revenue needs, it also aids in the documentation of needed policy changes required to generate such revenues. Similarly, the TSM capital-cost program provides an analogous approach in documenting revenue needs and the necessary lead time required to implement the difficult policy changes needed to generate public acceptance of innovative TSM measures. In other words, if TSM is to be more successful in improving urban transportation systems, its planning process has to be at least as comprehensive as, and an integral part of, the urban highway planning and construction program.

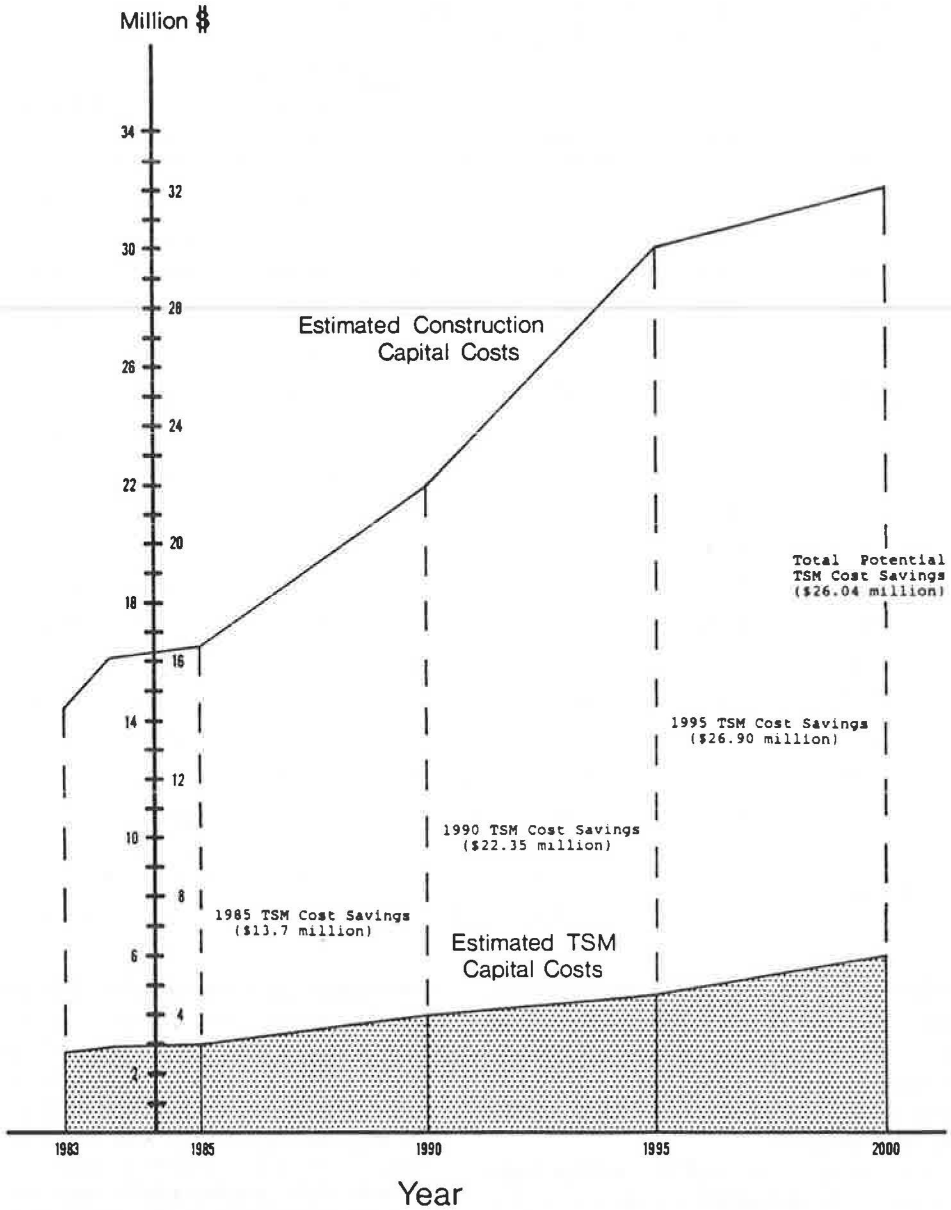

FIGURE 1 Comparison of TSM and construction cumulative capital-cost program.

PROSPECTIVE TSM MANAGEMENT PROGRAM

Although the demands on the urban transportation system continue to grow, elected officials and professionals continue to be pressured to plan for an even larger urban highway infrastructure with only limited concern for any financial constraints. The compounding effect of inflation, coupled with project overplanning that exceeds available funding many times over, accelerates urban demand for funds at an unmanageable rate.

Perhaps an analogy can be drawn to the business community, where one key to any successful business is the development and implementation of an effective management plan. Such a plan is designed to maximize profits by ensuring that investments are made in an effort to produce the greatest rate of return. When the TSM input costs (TSM capital-costs program = $6,042,920) are compared with the construction input costs (construction capital-costs program = $32,079,100), both of which could produce the same output (operation of the facility at level-of-service D), it is evident that TSM maximizes the benefits of its available revenue potential. Of course, few businesses have a board of directors as large and diverse in opinion as the general motoring public.

Another attribute of the business approach is the provision of a marketable product in response to consumer demand. In U.S. government context, that product, of course, typically would be additional urban highway access and expansion, with TSM relegated to a much less frequently demanded commodity.

Although the concept of TSM is well understood, it continues to be treated as a short-term demand-responsive measure. Unlike the planning and development of the urban highway system, TSM has not grown or developed as an integral part of the transportation system. As a result, many professionals, guided largely by public pressure, remain reluctant to consider TSM as a workable alternative to construction.

It is judged that this systemwide TSM planning methodology can help provide officials and professionals with a necessary tool to better consider TSM options, resulting effects, and necessary steps to bring about the results.

The effective management of urban transportation systems requires that TSM planning encompass both short- and long-term horizons, as does the development of the urban highway system. The TSM requirements (i.e., designated actions to accommodate a targeted portion of the transportation demand) ideally should be conceived well in advance of their need (i.e., before a system begins to operate at an undesired level of service) in order to be given the opportunity to grow and develop as an integral part of the urban transportation system.

If TSM were to be planned in this manner, it is the author's perception that policy initiatives designed to support TSM could be considered and developed in advance of a corridor's congestion problem and before commuting habits were well established. Before the year of the estimated congestion problem, a portion of the number of person-trips estimated to be accommodated or diverted from the peak hour to maintain level-of-service D could be targeted by the various TSM programs. This gradual process of implementation over time would likely produce the necessary modal changes.

As is the case with any new product perceived to be of lesser quality and designed to capture a portion of an existing product's market that has been proven and accepted by the consumer, precise timing and adequate exposure are required to gradually introduce the attributes of the new product to an otherwise reluctant consumer if the desired market share is to be obtained. In the same context, TSM should be introduced at an appropriate time before a corridor develops a congestion problem in order to provide the necessary lead time to capture a designated portion of the traffic demand.

Of course, current transportation policy, or lack thereof, generally supports the use of the SOV for the daily work trip until the highway system becomes congested. Then the public may be requested to consider alternative transportation modes, which consist of both voluntary and mandatory measures. These measures are sometimes designed to persuade the public to instantly shift to a higher-occupancy vehicle, which restricts the freedom of movement they enjoy by using their own vehicle. Consequently, the introduction of TSM measures at this time (i.e., when a corridor exceeds its desired vehicle capacity) is often viewed as undesirable and the public's resentment of sporadic changes often precludes the ability of TSM to extend the economic life of a corridor.

In the majority of urban areas, major congestion problems currently exist and the introduction of TSM measures on such corridors is not provided the luxury of being gradual. Therefore, the effective implementation of TSM requires a strong commitment from both an urban area's political leaders and those responsible for the management of sizable employment centers (say, those with 50+ employees) in order to mitigate the congestion to an acceptable level of service. With the comprehensive systemwide TSM planning approach, which estimates the number of person-trips required to be accommodated or diverted during the peak hour from the inception of the capacity problem through the design year, as done in the Winston-Salem study, it is possible to determine the percent of congestion generated by each employment center. The number of person-trips to be accommodated can be equally distributed among the area's employment centers on the basis of the percent of each center's employment of the total area's employment.

Collectively, this concept acts to place an equal burden and responsibility on each employment center, which is a much more systematic approach than merely contacting only those few with, say, 200+ employees. In turn, the employers have well-established annual goals (i.e., based on the number of persons), which can be used to measure their effectiveness, instead of vague, nondescriptive goals that are not directly associated with transportation demand. Moreover, if a few major employers are not singled out as those contributing to the congestion, a unified approach will most likely generate a greater level of participation among them. In addition, this amount of congestion associated with each employment center could be converted into a "congestion tax" to develop corporate tax incentives designed to bring about greater participation.

It is essential to inform employers of the urban congestion problem and to make it apparent that their business-related activities are the source of the problem during the peak hours. The business community bases their decisions on facts and not good will, so it is important that they see the supporting documentation showing where they fit into the overall picture. To appeal to them, a more factual approach detailing the congestion problem and benefits of TSM is needed, that is,

1. The magnitude of the existing and future urban congestion problems,
2. The source of the existing and future urban congestion problems as they relate to each employment center,
3. The economic impact these problems may potentially have on business activities (i.e., deterioration of the economic base caused by businesses relocating or new business locating in fringe areas, which could result in loss of employees or shoppers), and
4. The economic advantages of extending the life of existing highway facilities through effective transportation management techniques (i.e., tax-dollar savings).

This approach, coupled with the traditional approach, which relies heavily on TSM marketing techniques designed to inform the employer and employee of personal benefits of TSM, should produce far greater results than either approach alone.

Also, the transportation professionals responsible for these various programs need to better coordinate their planning function in an effort to develop a qualitative and comprehensive plan that parallels the urban highway planning process. Appropriately, this plan should provide the factual data necessary to justify program goals and budget requirements. Moreover, the alternatives of this plan should complement the construction alternatives.

To best achieve this objective, the scheduling and timing of program needs should be designed to extend the economic life of the existing system before it is enlarged. This does not imply that new roadways needed to induce economic growth should not be built. Instead, the scheduling and timing of TSM alternatives should be based on their potential and directed toward managing the existing system in an effort to minimize capital expenditures. As with the case with most urban areas, the existing congestion problems preclude this process and, as stated earlier, a strong commitment from the political and business community is required.

This commitment is essential to obtain the required policy changes and without quantitative documentation supporting TSM benefits, it is highly unlikely that such support will be pledged. Such an initiative has to be undertaken by an area's transportation professionals and adopted as part of the urban planning process. Policy initiatives detailing the implementation of this planning process have to be supported by top elected and appointed officials (mayors, council members, planning board members, etc.). In turn, these officials have to effectively communicate to the public and business leaders the source of the congestion problems and the advantages of managing the urban transportation system.

As a result of the potential economic benefits to be derived from the deferment or replacement of construction alternatives, TSM planning demands a well-conceived comprehensive approach coupled with an effective implementation mechanism designed to exhaust its potential.

SUMMARY AND CONCLUSIONS

The traditional reality that TSM has a minimal effect at best suggests that it generally is a less desired product or is not properly structured to penetrate the market. Regardless of which may be the case, financial implications of TSM make it necessary to revise the existing structure in an effort to better identify its market share as well as the strategies necessary for it to effectively penetrate the market.

The systemwide TSM program described here is offered as a tool and a step toward this end. It can provide the data necessary to determine an employment center's contribution to the capacity problem and thus its corresponding responsibility to help mitigate this problem. It can detail the effect that higher vehicle occupancy rates, increased transit frequencies, and traffic engineering improvements would have on an urban corridor. In short, it can provide the technical base from which public implementation policies could be developed in order to better incorporate TSM into solutions to the urban transportation dilemma.

The urbanized area of Winston-Salem is considered to have one of the state's more progressive and successful TSM programs. Over the years, its ridesharing and transit efforts have received national attention as a result of their effective implementation and management. With the area's strong TSM commitment and successful TSM track record, it is believed that the systemwide TSM planning approach will further enhance the area's ability to continue effective transportation management techniques.

The systemwide TSM analysis has been well received by the engineering, planning, and transit staff of the Winston-Salem urbanized area and is planned to be updated in the fall of 1988. To date, a portion of the traffic engineering capital costs (from 1985 to 1992) has been submitted as part of the area's Capital Improvements Program. Portions of the remaining TSM capital costs are planned to be included as part of an urban needs package that will be submitted to the public as justification for a bond referendum. Also, the project designated District 5, Segment 6, of this analysis together with other TSM improvements have been submitted as a TSM demonstration project to the Board of Transportation for funding.

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Publication of this paper sponsored by Committee on Transportation System Management.