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Transportation Planning Analysis Used in Small and Medium-Sized Communities

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NATIONAL RESEARCH COUNCIL

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Modlin, David G., Jr., Division of Highways, North Carolina Department of Transportation, Raleigh, NC 27611
Newnam, James T., Jr., Division of Highways, North Carolina Department of Transportation, Raleigh, NC 27611
Stephanedes, Yorgos J., Department of Civil and Mineral Engineering, University of Minnesota, Minneapolis, MN 55455

Evaluating Plan Alternatives: Energy, Safety, and Air Pollution

DAVID G. MODLIN, JR., AND JAMES T. NEWNAM, JR.

The development and "selling" of a thoroughfare plan generally tests to the limit both the technical and public relations skills of the transportation planner. A method is presented by which energy, accident, and air pollution indices may be developed for the evaluation of alternative plans. These indices can be related in a positive manner that somewhat offsets the general negative feelings aroused by talk of widenings, building new facilities, and displacing homes and businesses. The results obtained by applying the proposed analysis method were very good. The method is extremely efficient: Since all three indices are developed from the same vehicle-miles-of-travel summary, only one summary needs to be developed for each alternative to be tested.

In recent years, citizen involvement in the urban transportation planning process has been more vocal than in the past and has had a significant impact on the decision-making process (1). During the period of public meetings in which the plan is "sold" to the citizenry, tough questions are often posed to the engineer-planner, who must defend the merits of his or her work before a generally antagonistic forum. It is imperative that all available, applicable analysis tools be used in the process of evaluating plan alternatives so that a good defense of the recommended plan can be made. The analysis tools need not be complex or intricate to be useful. The purpose of this paper is to illustrate how existing techniques can be used to produce viable energy, safety, and air pollution indices by which alternative transportation system plans can be compared.

The use of the word "system" is important in that the numerical values presented in this paper involve some rather significant assumptions that would not be generally valid in the individual project-level analysis. For example, delay at individual traffic signals is assumed to be common to all alternatives; in other words, a base signal system and resulting average delays are assumed. The relations between functional classification, volume/capacity (V/C), level of service, and operating or overall travel speed are generally related to Highway Capacity Manual (2) definitions; however, the numerical indices presented are based on very average, generalized conditions. Therefore, the analyses suggested in this paper will give more reliable results when applied to the entire highway network, where deviations within analysis units will tend to offset one another.

Typically, three major areas are addressed in the analysis of alternative transportation system plans. They are existing or future capacity deficiency, damage to both public and private property, and the estimated costs of alternative improvements. The public, as well as elected officials, often have some difficulty relating these factors to the need to endorse recommended highway improvements. Maybe this is because these factors tend to foster negative thoughts: poor travel service and congestion, the taking of property, and the impact of capital expenditure on the municipal coffer. On the other hand, the use of some additional indices that generate more positive thoughts may help to dissipate some of the traditional negative feelings that often arise. For example, the amount of energy that could be saved, the number of accidents that might be prevented, and the prospect of cleaner air are all positive things that should result from the implementation of a sound, well-developed thoroughfare plan. Incorporating these concepts into our current evaluation methodology is very desirable.

METHODOLOGY

The highway network is coded in the normal manner as required in the PLANPAC/BACKPAC battery of programs (3) available from the Federal Highway Administration (FHWA). In developing the node-numbering sequence, individual facilities should be coded with consecutive node numbers to the extent possible. As will be explained later, speed adjustments will be made as a function of the V/C ratio, and consecutive numbering of facility link nodes by use of the LIBRARIAN/VS software developed by Applied Data Research, Inc., greatly facilitates these adjustments. In addition, column 65 of the standard link data format is coded to indicate one of the following functional classifications:

1. Freeways and expressways are by definition those facilities that are built to Interstate, freeway, or expressway standards.
2. Arterial facilities are those major facilities used by both local traffic and large, significant portions of the external-internal and through traffic.
3. Collector facilities are those facilities that carry major traffic volumes consisting primarily of local traffic. This is to mean everyday users of the facility.
4. Local and centroid connector facilities are those facilities that basically serve the land access function and provide access to the collector and/or arterial system.

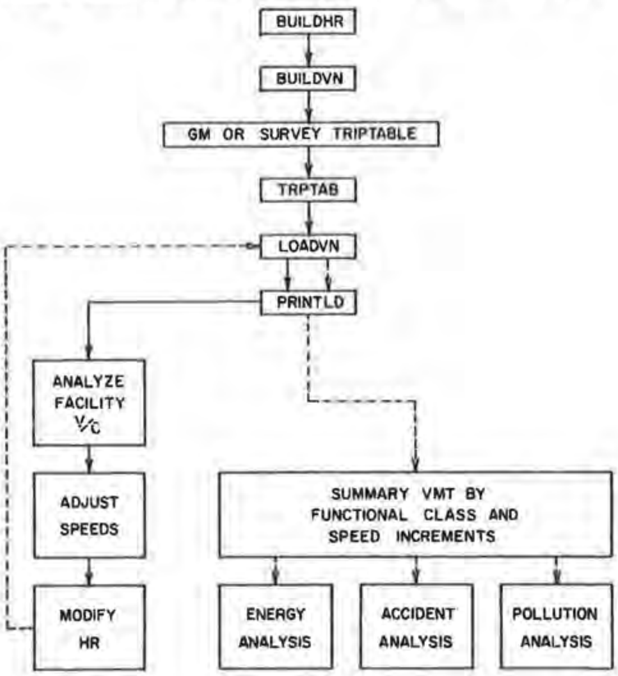
When the PLANPAC/BACKPAC planning battery is used, the following basic sequence of programs will lead to a loaded network: (a) BUILDHR, (b) BUILDVN, (c) GM (or survey trip table), (d) TRPTAB, (e) LOADVN, and (f) PRINTLD. In the base-year calibration procedure, link speeds and trip generation rates are adjusted in order to achieve good agreement between modeled and surveyed traffic volumes. The calibrated network is then ready to be loaded with the design-year trip demand. When loaded with future trips, the existing network is typically analyzed for deficiencies, and alternatives to improve traffic flow are developed and analyzed. The procedure described above is well documented and widely used and needs no further explanation.

The analysis techniques outlined in the remainder of this paper begin after the development of the calibrated network and loading of future trips on the existing or proposed alternative networks. Figure 1 shows a simplified flowchart for the suggested analysis procedure. Once future trips are loaded on the existing and proposed networks, then a good analysis of volume versus capacity is performed that may involve the application of capacity-restrained assignments. The resulting V/C ratios form the basis for adjusting the link speeds to reflect future levels of congestion and increased travel times. The speeds (travel times) initially coded, or as calibrated, in the historical record are modified to reflect the average speed indicative of the future level of service as suggested by the V/C ratio.

The objective now is to enter the new speeds into the loaded network records. Since it is not desirable to alter the calibrated trip routings at this point, the historical record containing the modified speeds for a particular network, the "original" calibrated trees and paths for a particular network, and the final "original" trip table for a particular network are used to produce the loaded network file reflecting the new speeds, which have been modified to reflect the anticipated congestion levels caused by future trip desires.

The North Carolina Department of Transportation (NCDOT) has developed computer capability for summing vehicle miles of travel (VMT) by functional classification and speed increments. The literature provides works on energy consumption rates (4-6), accident potential rates (6-11), and pollution rates (12), all based on VMT, speed, and/or functional classification. The key to correctly applying the rates, however, is the development of VMT by the proper speed increments. Following the procedure outlined in Figure 1 will produce VMT by speed groups consistent with anticipated levels of congestion.

Figure 1. Simplified flowchart for energy, accident, and pollution analyses.



ENERGY ANALYSIS

The proposed energy analysis will provide an estimate of total gallons of gasoline used daily on a systemwide basis. Functional classification and operating (or overall) speed are the key parameters. Data published in two reports (4,6) were combined with level-of-service qualifiers (12,13) to develop the information given in Table 1.

The rates given represent very average conditions and should not be used to evaluate individual projects that vary greatly in operating particulars. The published gasoline consumption rates (11) were assumed to be representative of level-of-service B operating conditions on a daily basis, and factors (3) were developed to adjust the consumption rates as a function of four average levels of congestion. For the arterial, collector, and local classifications, the level-of-service B rate was based on 5.75, 6.25, and 4.50 stops/mile, respectively.

After the V/C analysis, facilities are assigned a speed that corresponds to the indicated level of service. VMT is summarized by computer by functional classification and new speed increments. Next, a manual calculation is made by using the following equation:

$$\mathrm{TOTGAL} = \sum_{i=1}^{4} \sum_{j=1}^{n} (\mathrm{VMT}_{ij})(\mathrm{rate}_{ij}) \quad (1)$$

where

TOTGAL = estimated total gallons of gasoline used daily,
rate = rate of gasoline consumption,
i = functional classification index, and
j = speed increment index.

Two points concerning this analysis need to be made. The fuel consumption rates are representative of early 1970 vehicles. Since system alternatives are to be compared, it is the relative difference

Table 1. Energy factors for alternative plan analysis.

Functional Classification	Factor	A	B	C	D
Freeways and expressways	Avg operating speed[a] (mph)	55	50	40	30
	V/C ratio	0.50	0.62	0.75	1.00
	Level of service	B	C	D	E
	Fuel consumption (gal/mile)	0.0801	0.0817	0.0841	0.0865
	Avg miles per gallon	12.48	12.24	11.89	11.56
Arterials	Avg overall speed[b] (mph)	35	30	25	20
	V/C ratio	0.70	0.80	0.90	1.00
	Level of service	B	C	D	E
	Fuel consumption (gal/mile)	0.0931	0.1010	0.1084	0.1195
	Avg miles per gallon	10.74	9.90	9.23	8.37
Collectors	Avg overall speed[c] (mph)	30	25	20	15
	V/C ratio	0.70	0.80	0.90	1.00
	Level of service	B	C	D	E
	Fuel consumption (gal/mile)	0.0950	0.1032	0.1104	0.1216
	Avg miles per gallon	10.53	9.69	9.06	8.22
Locals and centroid connectors	Avg overall speed[d] (mph)	20	15	10	<10
	V/C ratio	0.75	0.85	0.95	1.00
	Level of service	B	C	D	E
	Fuel consumption (gal/mile)	0.0910	0.0940	0.1025	0.1165
	Avg miles per gallon	10.99	10.64	9.76	8.59

[a] Desirable operating speed = 55 mph.
[b] Desirable overall speed = 35 mph.
[c] Desirable overall speed = 30 mph.
[d] Desirable overall speed = 20 mph.

Table 2. Accident rates for alternative plan analysis.

Functional Classification	Factor	A	B	C	D
Freeways and expressways	Avg operating speed (mph)	55	50	40	30
	V/C ratio	0.50	0.62	0.75	1.00
	Level of service	B	C	D	E
	Fatalities[a]	0.68	0.84	1.39	2.65
	Nonfatal injuries[a]	27.26	33.65	55.52	106.33
Arterials	Avg overall speed (mph)	35	30	25	20
	V/C ratio	0.70	0.80	0.90	1.00
	Level of service	B	C	D	E
	Fatalities[a]	1.71	2.41	3.64	6.00
	Nonfatal injuries[a]	131.40	185.07	279.46	460.82
Collectors	Avg overall speed (mph)	30	25	20	15
	V/C ratio	0.70	0.80	0.90	1.00
	Level of service	B	C	D	E
	Fatalities[a]	1.60	2.42	3.99	7.62
	Nonfatal injuries[a]	158.42	240.03	396.05	756.09
Locals and centroid connectors	Avg overall speed (mph)	20	15	10	<10
	V/C ratio	0.75	0.85	0.95	1.00
	Level of service	B	C	D	E
	Fatalities[a]	0.42	0.80	1.98	1.98
	Nonfatal injuries[a]	59.90	115.20	285.70	285.70

[a]Per 100 million vehicle miles of travel.

between TOTGAL values that will be evaluated; these rates, even though somewhat dated, will correctly indicate the most fuel-efficient plan. Evaluated on a percentage basis, these rates versus 1980 rates should provide essentially the same numerical results. However, as new rates, in a desirable form, are published, Table 1 should be updated.

The second point is that a common basic level of stop delays, side friction, traffic control functions, etc., is inherent in all of the alternatives to be compared. The assumption has been made that, on a systemwide basis, deviations in traffic operations will average out and that the results of the analysis will be valid for the comparison of system alternatives.

ACCIDENT POTENTIAL ANALYSIS

The proposed accident analysis will provide an estimate of annual potential accidents as a function of functional classification and level of service being provided. Table 2 (7-12) was developed from rates published in the literature (8,10). Factors were developed, following the work of May (9) and Rykken (7,11), to modify the published accident rates to reflect four basic levels of congestion.

The rates thus developed are given in Table 2. The published rates were assumed to be representative of level-of-service C operating conditions. The data currently available address only fatal and non-fatal-injury accident rates, and the level-of-service factors developed were applied equally to both categories. In addition, the rates used to develop Table 2 are for North Carolina where available; otherwise, they are national average rates.

The functional classification, the V/C ratio(s) for a facility, and the same VMT summary developed for the energy analysis are used to estimate annual fatal and non-fatal-injury accidents by means of the following equation:

$$\text{ANNACC} = \sum_{i=1}^{4} \sum_{j=1}^{n} [(\text{VMT} \times 365)/10^8]_{ij} \times \text{RF}_{ij} + \sum_{i=1}^{4} \sum_{j=1}^{n} [(\text{VMT} \times 365)/10^8]_{ij} \times \text{RNFI}_{ij} \quad (2)$$

where

ANNACC = annual estimated fatal and non-fatal-injury accidents,
RF = fatal accident rate per 100 million VMT,
RNFI = non-fatal-injury accident rate per 100 million VMT,
i = functional classification index, and
j = level-of-service (V/C) index.

The NCDOT Traffic Engineering Division has the capability to develop accident rates, including property-damage-only rates, by functional classification and operating characteristics. Based on the results obtained during this research, it is expected that Table 2 will be updated with actual observed rates wholly applicable to North Carolina.

POLLUTION ANALYSIS

The rates used in the pollution analysis were derived directly from the Mobile 1 Mobile Source Emission Model of the Environmental Protection Agency (12). The rates represent a composite factor for a specified vehicle mix and initial running conditions. Typical emission factors are given in Table 3. The key parameters are speed and VMT.

The daily amount of pollutants emitted from mobile sources is obtained by the successive application of the following formula for each specific pollutant:

$$P_{ik} = \sum_{j=1}^{n} (\text{VMT}_j)(\text{EF}_{ijk}) \quad (3)$$

where

P = total daily mobile pollutant emitted,
EF = emission factor,
i = pollutant index,
j = speed increment index, and
k = year index.

When they become available, emission factors from the Mobile 2 program should be substituted for the Mobile 1 factors. The new factors will not change the results in evaluating alternative plans; however, the absolute values of pollutants emitted will be of use in determining the ability of the chosen alternative to meet the mobile air-quality standards.

The purpose of presenting the pollution analysis is to illustrate the significant difference in air-quality estimates when initial calibrated speeds are used in lieu of speeds adjusted to reflect the more realistic future estimated operating conditions. The procedure for deriving VMT by the "correct" speed increments recommended in this paper will pro-

Table 3. Typical emission factors for alternative plan analysis.

Speed (mph)	Pollutant (g/mile) CO 1980	CO 1981	CO 1999	HC 1980	HC 1981	HC 1999	NO_x 1980	NO_x 1981	NO_x 1999
20	63.19	57.77	21.30	6.71	5.94	2.40	3.61	3.39	1.94
25	52.30	47.96	18.14	5.76	5.08	1.95	3.80	3.57	2.09
30	44.17	40.58	15.62	5.07	4.45	1.61	4.01	3.76	2.23
35	38.21	35.15	13.72	4.57	3.99	1.36	4.19	3.94	2.35
40	34.34	31.66	12.52	4.22	3.67	1.20	4.35	4.10	2.45
45	32.40	29.94	11.97	4.02	3.49	1.11	4.53	4.27	2.55
50	31.72	29.38	11.80	3.91	3.39	1.06	4.79	4.54	2.69
55	30.73	28.45	11.34	3.80	3.29	0.99	5.25	4.99	2.94

Note: CO = carbon monoxide, HC = hydrocarbons, and NO_x = nitrogen oxides. Composite factor for vehicle mix of 80.3 percent light-duty vehicle, 5.8 percent light truck 1, 5.8 percent light truck 2, 4.5 percent heavy-duty gasoline-powered, 3.1 percent heavy-duty diesel-powered, and 0.5 percent motorcycle; 60.0° F; 21 percent cold mode catalyst, 21 percent cold mode noncatalyst, and 27 percent hot transient catalyst.

Table 4. Results of energy and accident analyses: Kinston, North Carolina.

Network	VMT	Vehicle Hours	Gallons per Day	Accidents per Year: Fatalities	Nonfatal Injury
Kinnet 05	1 428 107	39 111			
Kinnet A5	1 428 107	66 658	157 048	22.34	1872.45
Kinnet 06	1 361 843	36 658			
Kinnet A6	1 361 843	51 436	135 937	12.96	1084.05
Δ(A6-A5)	-66 264	-15 222	-21 111	-9.38	-788.40

Table 5. Results of mobile air-quality analysis: Kinston, North Carolina.

Network	VMT	Vehicle Hours	Amount of Pollutant (kg/day): CO	HC	NO_x
Kinnet 05	1 428 107	39 111	19 394.11	1 906.87	3 444.41
Kinnet A5	1 428 107	66 658	27 675.58	3 046.56	2 912.07
Kinnet 06	1 361 843	36 658	18 356.18	1 796.98	3 309.05
Kinnet A6	1 361 843	51 436	22 736.94	2 397.17	2 998.96
Δ(A6-A5)	-66 264	-15 222	-4 938.64	-649.39	+86.89

vide for more reliable estimates of air quality.

RESULTS

The procedures and analyses recommended in this paper were tested during the Kinston, North Carolina, Thoroughfare Plan update. Kinston, which has a current population of approximately 37 000, is the largest and most important urban area of Lenoir County and lies in the heart of North Carolina's Coastal Plain. In addition, the Kinston urban area supports 14 800 employees and contains 13 100 dwelling units with an average 2.82 persons/dwelling unit.

Before the numerical results of the analyses are discussed, it is appropriate to describe what was analyzed. Four networks were chosen to test the procedure:

1. KINNET 05--The existing 1979 network with the final 1979 calibrated speeds,
2. KINNET A5--The existing 1979 network with the calibrated speeds adjusted for year 2005 V/C ratios,
3. KINNET 06--The recommended thoroughfare plan network with the final 1979 calibrated speeds with capacity-restrained adjustments, and
4. KINNET A6--The recommended thoroughfare plan network with the calibrated speeds adjusted for year 2005 V/C ratios.

Each of these networks was loaded with the estimated 2005 design-year trip table. The thoroughfare plan recommends the construction of 38.25 miles of new facilities along with improvements to some existing facilities to achieve continuity in cross sections.

For the energy and accident analyses, the comparison was made between the existing and recommended thoroughfare plan networks and the speeds were adjusted for V/C ratios. Since energy and accident analyses have not heretofore been used in North Carolina studies, the comparison of unadjusted versus adjusted speeds seemed pointless in attempting to justify the merits of using adjusted speeds. It is sufficient to say that speed adjustments that correspond to estimated future operating conditions are more reasonable and give more realistic analytical results.

The numerical results from the energy and accident analyses are given in Table 4. The recommended thoroughfare plan makes significant contributions to the predicted quality of traffic flow measured in terms easily understood by any audience. In developing a "1-mile/gal gasoline saving" and a "5-mph speed improvement" on a systemwide basis, significant delays and excessive stops due to congestion are eliminated through implementation of the thoroughfare plan recommendations.

The mobile air-quality analysis used all four network options. A comparison should be made not only between A5 and A6 but also between 05 and A5 and 06 and A6. Air-quality analyses have normally been made by using the calibrated speeds for existing as well as future networks. The latter suggested comparisons will show significant differences between emission estimates using calibrated versus V/C adjusted speeds. Although the absolute value of pollutants, particularly CO and HC, increases when the adjusted speeds are used versus the calibrated speeds, it is felt that these are the most realistic values and, consequently, should be the values that are reported.

The numerical results of the mobile air-quality analysis, determined by using Mobile 1 factors, are given in Table 5. The most critical and most often cited pollutant violations in North Carolina with respect to transportation are for CO and HC.

CONCLUSIONS

The analyses described in this paper are extremely time-efficient to perform and provide alternative plan comparisons that are easily understood by any audience. In addition, the absolute numerical results obtained by the outlined procedure are superior to those obtained by the "old way of doing things". Efforts should now be directed toward updating the energy consumption rates and improving the accident rate format so that even more reliable results might be obtained.

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Mobile Source Emissions and Energy Analysis at an Isolated Intersection

DANE ISMART

A simplified technique is presented for evaluating the effect improvements will have on mobile source emissions and energy use at an isolated intersection. The procedure relates emissions of CO, HC, and NO_x and energy analysis to traffic-flow conditions at an intersection. A level of service is determined by using the critical movement analysis technique. By use of empirical data from a Federal Highway Administration report, stopped delay per vehicle is converted to the number of vehicles idling, slowing down, and stopping. Based on an NCHRP project, stopped delay per vehicle is related to level of service. The change from a base condition in idling time and vehicles stopping and slowing down as a result of an intersection improvement is used as the basis for determining the total reduction in pollutant emissions and energy use. The reductions are stated in terms of pounds and gallons as well as percentage reduction from the base condition. The procedure is designed to be a sketch planning tool for planners in small urbanized areas who have limited technical resources and data. The information necessary to use the procedure includes (a) total traffic entering the intersection, (b) turning movements, (c) number of approach lanes, (d) exclusive-use lanes, (e) approach speed, and (f) an estimate of the average upstream and downstream distance from the intersection where vehicle speeds are affected.

The procedure described in this paper will relate the emissions of air pollutants and energy to traffic-flow conditions at an isolated intersection. Traffic flow will be analyzed under the following classifications:

1. "Idling"--Vehicle hours of stopped delay,
2. "Slowdowns"--Total number of speed changes, and
3. "Stopping"--Total number of vehicles stopping.

By determining the changes in the number of vehicles idling, slowing down, and stopping, and by applying appropriate energy and emission rates, it will be possible to estimate the reduction in energy use and pollutants emitted as a result of the improvement of traffic operations at an intersection.

ENERGY USE AND EMISSION RATES

The table below (1) indicates fuel consumed and pollutant emissions for every 1000 vehicle-h of idling (January 1975 conditions for fuel consumption):

Item	Amount per 1000 Vehicle Hours
Gasoline (gal)	650
Pollutants (lb)	
Carbon monoxide (CO)	2430
Hydrocarbon (HC)	160
Nitrogen oxides (NO_x)	50

Figure 1 shows the additional fuel consumed for 1000 speed changes for various speeds [fuel consumption rates prevailing in January 1975 (2)]. This graph is used to determine the additional fuel consumed by vehicles that slow down as they approach an intersection. As a driver approaches an intersection, he will slow down his vehicle if there is a queue or if the light he approaches is in a red phase. If the queue dissipates or the signal changes before the vehicle reaches the intersection, the driver may only slow down and then return to his original speed. Figure 1 determines the additional fuel consumed based on this type of speed change.

For vehicles that stop completely, Figure 1 can also be applied. In this case, a stopped vehicle would be considered as going from the initial speed to 0 mph and then returning to the initial speed.

Figures 2-4 indicate the CO, HC, and NO_x emissions per 1000 speed changes. As was the case for fuel consumption, these figures can be applied for vehicles that slow down and stop.

Figure 1. Additional fuel consumption for vehicle speed changes beyond consumption caused by continuing at uniform speed.

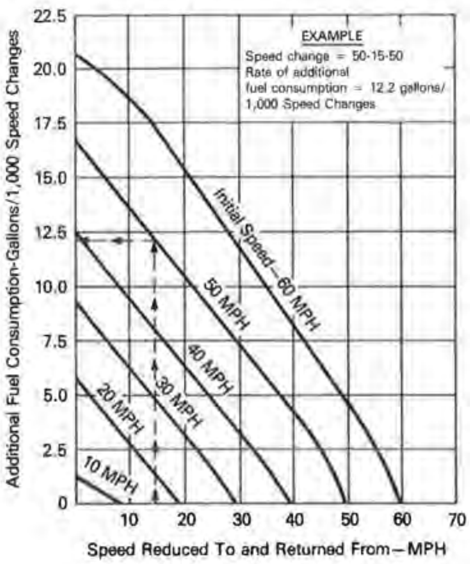


Figure 2. CO emissions for vehicle speed changes.

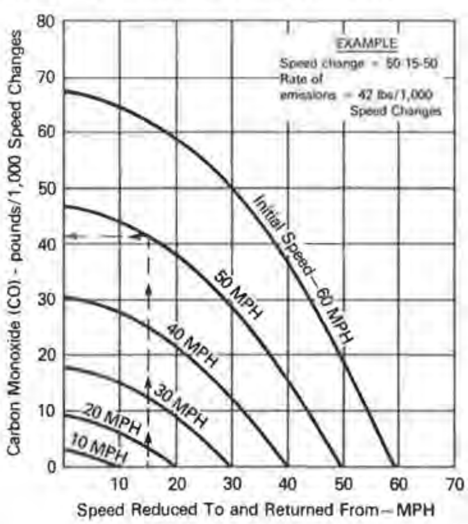


Figure 3. HC emissions for vehicle speed changes.

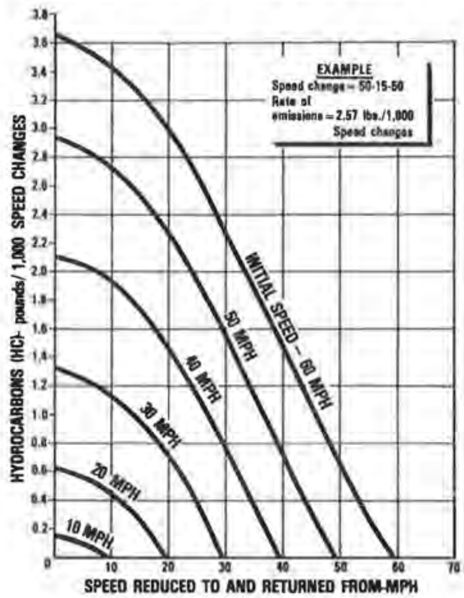
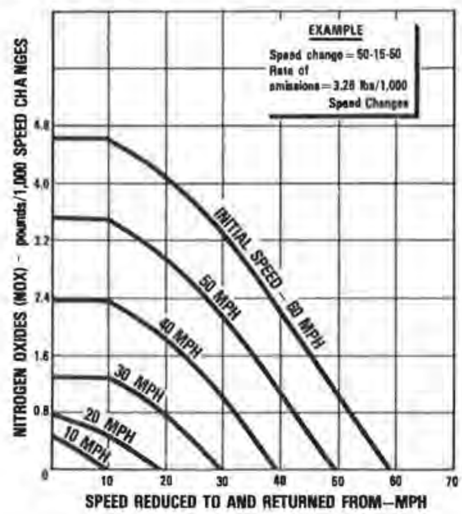


Figure 4. NO_x emissions for vehicle speed changes.



TRAFFIC-FLOW CHARACTERISTICS

In order to evaluate proposed intersection improvements, changes in traffic flow must be analyzed. The general strategy for this evaluation is to relate level of service to stopped delay per vehicle. Changes in idling, slowdowns, and stopping are based on stopped delay per vehicle.

The following relations were developed empirically from a Federal Highway Administration (FHWA) report (3).

Stopping

$$\text{Percent stopping} = 0.5497 \log_{10} (\text{ADPV}) - 0.1404 \quad (1)$$

$$\text{ADPV} = 1.3 \times \text{SDPV} \quad (2)$$

where ADPV is approach delay per vehicle (approach delay divided by the total number of vehicles passing through the intersection approach during a period of time, in vehicle seconds per vehicle) and SDPV is stopped delay per vehicle (stopped delay divided by the total number of vehicles passing through the intersection approach during a period of time, in vehicle seconds per vehicle).

Substituting SDPV for ADPV yields

$$\text{Percent stopping} = 0.5497 \log_{10} (0.3 \times \text{SDPV}) - 0.1404 \quad (3)$$

To determine the number of vehicles stopping, multiply percent stopping times the total traffic entering the intersection (TTEI) for a specific time period:

$$\text{Number of stopped vehicles} = [0.5497 \log_{10} (\text{SDPV} \times 1.3) - 0.1404] \times \text{TTEI} \quad (4)$$

To determine additional fuel consumption due to stopping, multiply the number of stopped vehicles times the additional fuel consumption rate (FCR) per speed change. The rate is obtained from Figure 1 by dividing by 1000. This will convert the rate from gallons per 1000 speed changes to gallons per speed

Table 1. Excess hours consumed per speed-change cycle beyond hours consumed by continuing at initial speed (for passenger cars).

Initial Speed (mph)	Speed Reduced To and Returned From (mph)										
	Stop	5	10	15	20	25	30	35	40	45	50
5	1.02										
10	1.51	0.62									
15	2.00	1.12	0.46								
20	2.49	1.62	0.93	0.35							
25	2.98	2.11	1.40	0.80	0.28						
30	3.46	2.60	1.87	1.24	0.70	0.23					
35	3.94	3.09	2.34	1.69	1.11	0.60	0.19				
40	4.42	3.58	2.81	2.13	1.52	0.97	0.51	0.16			
45	4.90	4.06	3.28	2.57	1.93	1.34	0.83	0.42	0.13		
50	5.37	4.54	3.75	3.01	2.34	1.71	1.15	0.68	0.35	0.11	
55	5.84	5.02	4.21	3.45	2.74	2.08	1.47	0.94	0.57	0.28	0.09

change. The equation then becomes

$$\text{Additional fuel (gal)} = [0.5497 \log (\text{SDPV} \times 1.3) - 0.1404] \times \text{TTEI} \times [\text{FCR (Figure 1)}/1000] \quad (5)$$

For pollutant emissions due to stopping, the number of stopped vehicles is multiplied by the emission rates for HC, CO, and NO_x per speed change. The rates are obtained from Figures 2, 3, and 4--i.e., by dividing by 1000 to convert the rates from pounds per 1000 speed changes to pounds per speed change. The equations for pollutant emissions (in pounds) due to stopping become

$$\text{CO} = [0.5497 \log_{10} (\text{SDPV} \times 1.3) - 0.1404] \times \text{TTEI} \times [\text{ER (Figure 2)}/1000] \quad (6)$$

$$\text{HC} = [0.5497 \log_{10} (\text{SDPV} \times 1.3) - 0.1404] \times \text{TTEI} \times [\text{ER (Figure 3)}/1000] \quad (7)$$

$$NO_x = [0.5497 \log_{10} (\text{SDPV} \times 1.3) - 0.1404] \times \text{TTEI} \times [\text{ER (Figure 4)}/1000] \quad (8)$$

Slowdowns

To determine the time lost due to vehicles slowing down but not stopping, the following equation is used:

$$\text{Slowdown delay} = \text{total approach delay} = \text{time in queue delay} \quad (9)$$

From FHWA (3),

$$\text{Time in queue delay} = (\text{stopped delay per stopped vehicle} - 11.33)/0.76 \quad (11)$$

$$\text{Total approach delay} = 1.3 \times \text{SDPV} \times \text{TTEI} \quad (10)$$

$$\text{Stopped delay per stopped vehicle} = 0.96\,(\text{SDPV}) + 11.10 \quad (12)$$

Substituting Equation 9 for SDPV in Equation 8,

$$\text{Time in queue} = \{[0.96\,(\text{SDPV}) + 11.10 - 11.33]/0.76\} \times \text{TTEI} \quad (13)$$

$$\begin{aligned}\text{Slowdown delay} &= \text{TTEI} \times 1.3 \times \text{SDPV} - \{[0.96\,(\text{SDPV}) - 0.23]/0.76\} \times \text{TTEI} = \text{TTEI} \times 1.3 \times \text{SDPV} \\ &- [1.26\,(\text{SDPV}) - 0.30] \times \text{TTEI} = 1.3 \times \text{TTEI} \times \text{SDPV} - 1.26 \times \text{TTEI} \times \text{SDPV} \\ &+ 0.30\,\text{TTEI} = \text{TTEI} \times (0.04\,\text{SDPV} + 0.30)\end{aligned} \quad (14)$$

To convert the slowdown delay in seconds to the number of vehicles slowing down in units of 1000, divide the slowdown delay by the excess hours consumed per 1000 speed-change cycles from Table 1 (4):

$$\text{Slowdowns per 1000 speed changes} = [\text{TTEI} \times (0.04\,\text{SDPV} + 0.30)]/[3600 \times \text{hours per 1000 speed changes (Table 1)}] \quad (15)$$

To determine excess energy consumption and emissions due to slowdowns at an intersection, multiply Equation 15 by the FCR and the emission rates (ERs) from Figures 1, 2, 3, and 4. The final forms of the equations for slowdowns are as follows (hours per speed change obtained from Table 1):

$$\text{Excess fuel consumption (gal)} = [\text{TTEI} \times (0.04\,\text{SDPV} + 0.30)]/(3600 \times \text{hours per 1000 speed changes}) \times \text{FCR (Figure 1)} \quad (16)$$

$$\text{CO (lb)} = [\text{TTEI} \times (0.04\,\text{SDPV} + 0.30)]/(3600 \times \text{hours per 1000 speed changes}) \times \text{ER (Figure 2)} \quad (17)$$

$$\text{HC (lb)} = [\text{TTEI} \times (0.04\,\text{SDPV} + 0.30)]/(3600 \times \text{hours per 1000 speed changes}) \times \text{ER (Figure 3)} \quad (18)$$

$$NO_x \text{ (lb)} = [\text{TTEI} \times (0.04\,\text{SDPV} + 0.30)]/(3600 \times \text{hours per 1000 speed changes}) \times \text{ER (Figure 4)} \quad (19)$$

Idling

Fuel consumption and emissions due to idling can be computed by multiplying the number of vehicles entering the intersection by the stopped delay per vehicle times the rates given in the text table at the beginning of this paper. The equations for idling would be as follows:

$$\text{Energy (gal)} = (\text{TTEI}/3600) \times \text{SDPV} \times 0.65 \text{ gal} \quad (20)$$

$$\text{CO (lb)} = (\text{TTEI}/3600) \times \text{SDPV} \times 2.43 \text{ lb} \quad (21)$$

$$\text{HC (lb)} = (\text{TTEI}/3600) \times \text{SDPV} \times 0.16 \text{ lb} \quad (22)$$

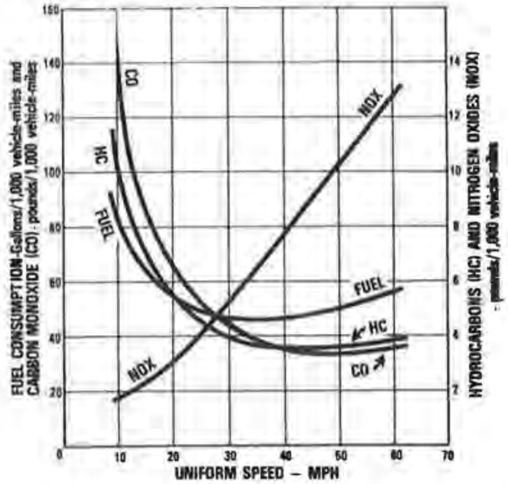
$$NO_x \text{ (lb)} = (\text{TTEI}/3600) \times \text{SDPV} \times 0.05 \text{ lb} \quad (23)$$

IMPLEMENTATION STRATEGY

The equations developed for determining energy use and vehicle emissions are based on stopped delay per vehicle. In the following table from NCHRP Project 3-28 (based on a synthesis of various data), stopped delay can be related to level of service: (V/C = volume/capacity)

Level of Service	Typical V/C Ratio	Delay Range (s/vehicle)
A	0.00-0.60	0.0-16.0
B	0.61-0.70	16.1-22.0
C	0.71-0.80	22.1-28.0
D	0.81-0.90	28.1-35.0
E	0.91-1.00	35.1-40.0
F	Varies	>40.1

Figure 5. Fuel consumption and emissions of CO, HC, and NO_x from driving 1000 miles at various uniform speeds.



[Delay range is measured as stopped delay (3). Delay values relate to the mean stopped delay incurred by all vehicles entering the intersection. Note that traffic-signal coordination effects are not considered and could drastically alter the delay range for a given V/C ratio.] By relating level of service to stopped delay per vehicle, the equations in this paper could be used to analyze vehicle emissions and energy. The technique would be most applicable in determining what effect an intersection improvement, such as constructing a left-turn lane, will have on energy consumption and air pollutant emissions.

The first step in the process is to determine the level of service by using the critical movement technique (4) for both the existing intersection and the intersection with the proposed improvement. The level of service would be correlated with the stopped delay from the table above. For example, from the critical movement technique it can be determined that an existing intersection has a level of service C and a critical intersection volume of 1100. The table below (1) gives intersection capacity by level of service for the critical movement technique (* indicates a special case):

Level of Service	Capacity Range (vehicles/h) Low	High
A	0	900
B	901	1050
C	1051	1200
D	1201	1350
E	1351	1500
F	-*	1500

Level of service C has a capacity range of 1051-1200 for the critical movement technique, and the preceding table indicates that level of service C has a stopped-delay range of 16.1-22 s/vehicle. Therefore, prorating the level of service, the delay for the existing intersection would be calculated as follows:

$(1100 - 1051)/(1200 - 1051) = (X - 16.1)/(22 - 16.1)$
where X = SDPV.
$49/149 = (X - 16.1)/5.9.$
$149X = 2688.$
X = 18 s/vehicle.

The same process would be used to determine the SDPV with the proposed intersection improvement.

The second step in the process is to use the existing and improved intersection stopped-delay values in the energy and emission equations. For simplifying purposes, it is assumed that the average vehicle slowing down but not stopping will reduce its initial speed by one-half. If, for example, the initial approach speed for an intersection is 30 mph, the reduction for the average vehicle would be 15 mph. Approach speed is defined as the speed limit of the approach lanes to the intersection. By using Table 1, for this example, 1.24 excess hours would be consumed per 1000 speed-change cycles.

After the energy and vehicle emissions are computed, the third step would be to compare the results for the existing intersection with those for the improved intersection. The difference in the results between the existing and proposed intersections is the reduction in energy use and air-quality emissions (due to fewer vehicles stopping, slowing down, and idling).

APPLICATION

Significant changes in energy use and vehicle emissions as a result of an intersection improvement will only occur with a change in the level of service. For an intersection at low volumes, there may be no difference in the level of service between the existing and the improved facility. Both facilities may operate at a high level of service with low volumes. Consequently, there would be little change in the percentage of vehicles stopping, idling, and slowing down.

The evaluation for an intersection should be broken into peak and nonpeak analysis periods. The analysis periods should be based on the intersection operating at the same level of service for the entire analysis period. If there is a significant difference in the level of service in the peak or nonpeak period, it may be necessary to break down the analysis into smaller time units. The minimum time period that can be analyzed is 1 h.

Another problem in any emissions and energy analysis is the change in vehicle characteristics. In the future, the fuel consumption and vehicle emission rates will change. To compensate for this change, the rates should be modified periodically.

In evaluating an existing versus improved intersection, the percentage reduction in fuel consumption and emissions, as well as the absolute values, should be considered. To compute the percentage reduction for fuel, determine the total amount of fuel consumed at the existing intersection. At this point it has been demonstrated how to calculate the additional fuel consumed due to speed changes and idling. For the speed changes, the fuel consumed is in addition to the fuel consumed for traversing the same distance at a uniform speed. To obtain the total amount of fuel consumed, use Figure 5 (2) and add the fuel consumption indicated for a uniform speed to the consumption for speed changes and idling.

In Figure 5, the uniform speed is cross-referenced with consumption in gallons per 1000 vehicle miles. The uniform speed would be the approach speed for the intersection. By entering into Figure 5 with a uniform speed, the consumption rate can be determined. This rate is multiplied by vehicle miles at the intersection in units of 1000 to estimate fuel consumption at a uniform speed for all vehicles entering the intersection. To determine vehicle miles, an estimate must be made of the distance upstream from the intersection, where vehicles are initially affected by the intersection, and the distance downstream, where they have recovered their original speed. Normally, this distance will vary depending on the characteristics of the intersection. A reasonable estimate must be obtained from an individual who is familiar with the intersection in question.

The equation for uniform-speed fuel consumption is as follows:

$$\text{Fuel (gal)} = (\text{TTEI}/1000) \times \text{FCR (Figure 5)} \times \text{intersection distance} \quad (24)$$

After the uniform fuel consumption is added to

Figure 6. Format for total and incremental fuel consumption.

INCREMENTAL FUEL CONSUMED	BASE	ALTERNATIVE	REDUCTION (GALLONS)
Stopping			
Slowdowns			
Idling			
Σ INCREMENTAL FUEL CONSUMED			Σ REDUCTION
Uniform Speed			
TOTAL FUEL CONSUMED			

EQUATIONS

$$\text{Stopping (Gals.)} = [.5497\ \text{LOG}_{10}\ (\text{SDPV} \times 1.3) - .1404] \times \frac{\text{TTEI}}{1000} \times \text{FCR (Figure 1)} \quad (5)$$

$$\text{Slowdowns (Gals.)} = \frac{\text{TTEI} \times (.04\ \text{SDPV} + .30)}{3600 \times \text{Hours/1000 speed changes (Table 2)}} \times \text{FCR (Figure 1)} \quad (16)$$

$$\text{Idling (Gals.)} = \frac{\text{TTEI}}{3600} \times (\text{SDPV}) \times .65\ \text{Gals.} \quad (20)$$

$$\text{Uniform Speed (Gals.)} = \frac{\text{TTEI}}{1000} \times \text{FCR (Figure 5)} \times \text{Intersection Distance} \quad (24)$$

WHERE:

SDPV = Stop delay per average vehicle
TTEI = Total traffic entering intersections
FCR = Fuel consumption rate
Intersection Distance = Estimate of average total distance in miles, upsteam and downstream from the intersection (example .1 miles) where the average vehicle's free flow speed is affected.

Figure 7. Format for sum and total incremental air-quality emissions.

	Base	Alternative	Reduction
CO (Pounds)			
Stopping			
Slowdowns			
Idling			
Uniform Speed	X		
Sum			
HC (Pounds)			
Stopping			
Slowdowns			
Idling			
Uniform Speed	X		
Sum			
NO (Pounds)			
Stopping			
Slowdowns			
Idling			
Uniform Speed	X		
Sum			

Σ Reduction	
CO	
HC	
NO	

the consumption due to speed changes and idling, total fuel consumed has been determined. This value, when calculated for an existing intersection, will be used as the base for determining the percentage reduction for a proposed improvement. For example, at an intersection 1000 gal are consumed. With the construction of a left-turn lane, 100 gal less will be consumed. Therefore, the left-turn bay will reduce consumption by 100/1000, or 10 percent.

For vehicle emissions, Figures 2-4 represent total emissions for vehicles changing speeds. To simplify the analysis, it is assumed that for existing intersections most vehicles will experience a speed change at the intersection. Thus, for congested intersections, the emissions for idling, stopping, and slowing down represent total emissions. Total emissions for the existing intersection would be used just as in the analysis of energy as a base to determine the percentage reduction.

In the case of an improved intersection, total emissions would include vehicles that do not experience a speed change. The number of vehicles that do not stop or slow down can be estimated by equating it to the reduction of vehicles stopping when an existing intersection is improved.

For example, if the addition of a left-turn bay reduced the percentage of vehicles stopping from 80 to 70 percent and 4000 vehicles entered the intersection during the analysis period, it would be estimated that 400 vehicles (10 percent x 4000) will experience little interference when traversing the intersection. Then, to determine vehicle miles, the number of free-flowing vehicles would be multiplied by the distance from the intersection where vehicle movement is affected. This would be the same distance estimated for the energy analysis.

From Figure 5, pollutant emissions in units of 1000 vehicle miles for vehicles traveling at a uniform speed can be obtained. These emission rates multiplied by vehicle miles would determine the emissions for uniform-speed vehicles. The equation is as follows:

$$CO, HC, NO_x = (TTEI/1000) \times ER \text{ (Figure 5)} \times \text{intersection distance} \times \text{percent reduction of vehicles stopping} \quad (25)$$

When these emissions are combined with emissions due to slowdowns, stopping, and idling, the total emissions for an improved intersection can be calculated.

SUMMARY

The procedure described in this paper is designed as a sketch planning tool for planners. Whereas the critical movement technique is a sketch planning tool for analyzing capacity, this methodology is a tool for evaluating vehicle emissions and energy. It can be applied quickly and can provide reasonable estimates of reductions in energy use and vehicle emissions. The quick-response characteristics of the method are demonstrated by the limited amount of data necessary to do an evaluation.

To simplify the application of the technique, the equations given in this paper for pollutant emissions (Equations 9-11, 17-19, 21-23, and 24) and the formats shown in Figures 6 and 7 should be used.

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Improved Demand Estimation for Rural Work Trips

YORGOS J. STEPHANEDES

A critical review of the most widely accepted rural demand estimation models is performed. Based on data collected in two rural towns, a disaggregate specification for rural work-trip modal choice is proposed. The new model includes a set of socioeconomic and a set of policy-relevant variables and can be used for implementing a wide range of transportation policies to improve rural transit system performance. Model variables produce coefficients consistent with the notion, recently found in the literature, that rural commuters are more sensitive to fiscal variables than are urban commuters. Results from comparison tests suggest that demand prediction with the proposed specification is significantly (up to 88 percent) better than with the best of the existing models.

The evaluation of rural transportation projects that operate with federal or state support has been considered an essential part of government-subsidized transportation programs during the past decade. Transportation policies that can improve the efficiency and effectiveness of rural transit operations have recently been proposed (1), and data on performance measures for evaluating such operations are now available (1-3) and are being compiled by a number of states (4,5). In response to a need for identifying transportation policies that can also enhance rural mobility and the need to determine whether such policies will, in time, cause changes in rural economic development, a project was recently initiated (6). An immediate need for a demand estimation specification to estimate work-trip modal choice was identified.

The major objective of this study is to determine the most reliable rural demand estimation model suitable for implementing level-of-service transportation policies and sensitive to long-term mobility and economic changes that may take place in a community. This determination depends on certain basic criteria: (a) the ability of the selected model to estimate modal choice for work trips directly, (b) inclusion of level-of-service independent variables for implementing transportation policies that can improve the efficiency and effectiveness of a transit system, (c) inclusion of mobility and socioeconomic variables so that long-term changes in resident mobility and the local economy can be taken into account when modal choice is determined, (d) data availability, (e) model performance, (f) causally justifiable independent variables, and (g) the potential for model transferability to other rural areas.

A critical review of the most significant existing demand estimation models is performed first. This review includes a summary of performance characteristics that emphasizes effectiveness and the drawbacks of each model from the limited tests found in the literature. Subsequently, a new demand estimation specification for rural work-trip modal choice is proposed and compared with the best of the existing models. The comparison tests are based on six data sets collected in two rural towns over a three-month period.

The major findings can be summarized in two parts. First, results from tests of the performance of the existing rural demand models (7,8) are mostly in agreement with previous studies (7-11). More specifically, the existing models are found to be easy to comprehend but hard to apply to a specific trip purpose, as in work-trip estimation. Furthermore, because of the lack of a strong causal justification and the dearth of appropriate level-of-service variables, their use for policy analysis is not warranted. Finally, they result in significant estimation errors; because of this and the above characteristics, their potential for transferability is questionable at best. These observations reinforce the need for the development of more rigorous and more accurate rural demand estimation specifications.

This is accomplished by the proposed specification, which, in agreement with recent research findings, results in an increased importance of travel cost and household income in rural areas. The tests show that the proposed specification performs better than the existing ones.

BACKGROUND

Review of Rural Estimation Models

The existing approaches to modeling the steady-state demand sector of the rural transportation system fall into three general categories: (a) attitudinal studies, simple survey tabulations, and rough trip-rate estimates (12-15); (b) mathematical techniques based on aggregate analysis (7-9); and (c) disaggregate mathematical techniques (1).

Lack of rigorous analysis does not justify the use of approaches belonging to the first category for reliable policy analysis. Methods in the second category have relied on simple regression techniques (7,9) or used cross-classification techniques in combination with probabilistic assumptions (8). Due to their structure and assumptions, these methods often result in models that are descriptive rather than causal, models with large forecasting errors, questionable transferability properties, and little applicability to policy analysis. When such models are used, the sensitivity of prediction to errors in parameter estimates can be high, and the lack of emphasis on level-of-service variables makes prediction insensitive to proposed changes in transportation policy. On the contrary, disaggregate models are capable of capturing the causal relations between transportation level of service, household socioeconomic characteristics, and travel behavior and therefore provide a more meaningful analysis of various transportation policy options (16).

Aggregate Models

The first comprehensive work in this area (7) aimed "to produce forecasting methods at area-wide and route levels that are specific enough to enable local planners to use [the] method as the basis for initial operations of small-scale transit systems...and simple enough to be applied by local planning staff personnel." Five econometric models were presented, two applicable to fixed-route systems and three to demand-responsive systems. For each kind of system, there are models at the county (macro) level, and at the route (micro) level. By using regression analysis, route ridership is forecast as a (log) linear function of aggregate route characteristics such as total population along routes and route length and destination population.

The choice of independent variables is often arbitrary, and specifications are correlative rather than causal; e.g., excessive attention is paid to achieving a high R^2, but little attention is given to identifying variables that cause a specific ridership to be created. Ridership estimates are not sensitive to changes in transportation policy, household socioeconomic characteristics, or competing alternative levels of service. Furthermore, parameter estimates and statistical measures may be biased due to simultaneity and zone-size variance, respectively; the sensitivity of prediction to errors in parameter estimates can be high. It is concluded that the models are simple and easy to implement but are based on questionable assumptions, have limited applicability, do not contribute to a better understanding of the transit structure, and do not achieve their stated objectives; i.e., they cannot be safely implemented for forecasting purposes, and they cannot form a reliable basis for initial operations in rural areas (1).

The second mathematical approach to modeling rural transit ridership was a response to the deficiencies of the previous approach. Its objective was "to develop techniques of demand estimation which are...simple to understand, easy to apply, and low cost in nature,...offer the possibility of transferability, [and are] capable of identifying the needs generated by specific target populations along routes, such as the elderly, carless, or households with low income" (8).

The Poisson model was introduced as a technique superior to ones previously (7) used. It is a simple and appealing model but is subject to criticisms similar to those directed at previous research. Independent variables used for cross classification are rather arbitrary. Ridership estimates are insensitive to changes in transportation policy and to the level of service of competing alternatives. Although regression methods are criticized, they are used to improve on the Poisson model when it proves to be a poor performer (and the specification chosen is correlative rather than causal). Finally, the model is based on questionable assumptions (e.g., that the decision to ride the bus is a random event or that such events for rural households are independent of each other) and does not contribute to a better understanding of the transit structure, a fact acknowledged by its authors (8). Although some of the objectives, such as low cost, ease of application, and need identification, are satisfied, three are not met: The model is confusing, it is not accurate, and it does not have potential for transferability (1).

A more recent modeling attempt (9) used simple regression and was developed for demand-responsive service. It could be criticized along earlier (7) lines. The major existing models developed for fixed daily rural service that were relevant to this study and could be tested are summarized as follows: The macromodel (7) is expressed as

$$\text{LOG (RTPASS/MO)} = -0.353 + 0.407 \text{ LOG BMILES} + 0.533 \text{ LOG FREQ} + 0.611 \text{ LOG RESTRPOP} - 0.123 \text{ LOG COMPBMS} \quad (1)$$

where

RTPASS/MO = round-trip passengers per month,
BMILES = total vehicle miles per month,
FREQ = average monthly round-trip frequency,
RESTRPOP = people who may use the system (00s), and
COMPBMS = monthly vehicle miles of competing systems in the area.

The micromodel (7) is expressed as

$$\text{LOG(OWPASS/DAY)} = 6.344 + 0.697 \text{ LOG FREQ} - 2.547 \text{ LOG D} + \text{LOG POP}_o + \text{LOG POP}_d \quad (2)$$

where

OWPASS/DAY = one-way passengers per day on a specific route;
FREQ = round trips per day on that route;
D = round-trip distance from farthest origin point served to main destination (miles);
POP_0 = population of area traversed minus population of largest city, which is defined as the destination population (00 000s); and
POP_d = population of the largest city traversed (00 000s).

The Poisson mode (8) is expressed as

$$T = 0.003\ 05\ R^{1.396}\ U^{0.935} \quad (3)$$

where

T = trip ends per operating day,
R = route mileage, and
U = number of dwelling units within 0.25 mile of a route.

Disaggregate Models

The inadequacies of aggregate modeling techniques for rural transportation demand estimation led to an early attempt to formulate rigorous disaggregate specifications (1). A limited analysis was conducted of the effects on ridership of certain transportation level-of-service attributes and of socioeconomic characteristics of individuals. The limited scope of the study could only result in an indication that rural residents are more sensitive to travel cost than urban residents. This was a significant conclusion because previous researchers (7-9), using aggregate analysis, had decided that this particular characteristic did not play a significant role in rural ridership estimation. Furthermore, in the course of the study it became evident that disaggregate demand estimation for rural transportation was feasible. It was determined that more work was needed to measure the effect of a number of level-of-service variables on demand modal choice before such demand models could be used for policy analysis.

PROPOSED MODEL

Because of the disadvantages of the existing models, it was decided that a new model should be developed that should fulfill the criteria set forth at the beginning of this paper. In addition, the new model should make efficient use of data and should be at least as accurate as previous prediction approaches. Given its known characteristics and advantages over aggregate methods, it was decided that a disaggregate formulation should be adopted.

For predicting the choice of transportation mode to work from among three modes--transit, drive alone, and shared ride--a multinomial logit model structure was chosen. The statistical properties of the logit model and its successful application in analyzing discrete modal choice are well documented (17-19) and are not restated here. The particular form of the model used was as follows:

$$P(m:M_t) = \exp(X'_{it}\theta) / \sum_{j\epsilon M_t} \exp(X'_{jt}\theta) \quad (4)$$

where

$P(m:M_t)$ = probability of worker t selecting mode m from choice set $M_t \equiv$ {transit, drive alone, rideshare},
X_{mt} = vector of independent variables for alternative m and worker t, and
θ = vector of coefficients estimated by using the maximum likelihood method (17).

The vector of independent variables (X_{mt}) can be expressed in the general form

$$X_{mt} = X_m(L_m, S_t) \quad (5)$$

where L_m is a vector of level-of-service characteristics of mode m and S_t is a vector of socioeconomic characteristics of worker t.

VARIABLES AND DATA

Three level-of-service variables and six socioeconomic variables were included in the logit formulation. These variables and their expected coefficients are summarized in Table 1. The level-of-service variables are defined as in urban work-trip modal-choice models (16). Of the socioeconomic variables, the variable automobiles per household worker is introduced as a replacement for automobiles per licensed driver and workers per household; it is hypothesized that the former is of direct and overriding concern in rural areas, where individual workers have been found to be increasingly dependent on the automobile (10). A dummy variable is introduced to associate home ownership with driving alone, which is a significant expense in rural areas and would most likely be expected of homeowners. Finally, length of residence is introduced to account for long delays involved in the decision to ride a transit vehicle or share a ride in rural areas, a sociological characteristic also pointed out in the literature (1,10). Automobile availability per licensed driver for shared ride is not assigned an expected sign in Table 1 as a result of two observations: (a) It has been shown that in urban areas the effect of this variable on shared ride is less than it is on drive alone, and (b) across-the-board increased automobile availability in rural areas, when combined with the previous observation, may result in an unpredictable effect on shared ride.

Approximately 500 households from the rural towns of Cloquet and Le Sueur, Minnesota, were contacted, and household characteristics were recorded for those who were potential riders of the commuter rural transit service. Sample demographic and socioeconomic characteristics are summarized in the following table:

Characteristic	Cloquet	Le Sueur
Estimated population	12 000	4200
Estimated population growth (%)	5	12
Percentage of total population		
Age (years)		
18-64	51.6	50.4
$\geq$ 65	13	12.2
Below poverty level (1970 Census)	6.5	4.2
Household income	19 190	20 120
People per household	2.9	3.8
Workers per household	1.4	2.4
Licensed drivers per household	1.9	2.5
Automobiles per household	1.6	2.1
Own residence (%)	98	78
Length of residence (years)	22	8.7

These data were supplemented by information on level-of-service characteristics of the transportation system. To minimize the effect of a variety of trip choices on the choice of mode to work, only simple home-based trips were considered, i.e., trips from home to work to home. The final sample of 77 observations was divided into two subsamples: 40 Cloquet observations and 37 Le Sueur observations. A disaggregate model was then developed for each subsample to allow evaluation of model transferability. Finally, a model was developed for the complete sample so that higher statistical significance could be obtained.

ESTIMATED COEFFICIENTS

Three basic disaggregate models to estimate rural work-trip modal choice were derived from the Minnesota data--one from the Cloquet sample (model 1), one from the Le Sueur sample (model 3), and one from the combined Minnesota sample (model 5). These models are presented in Table 2. The previously stated hypotheses about the positive influence of home ownership on driving alone and of length of residence on using transit and carpooling are reflected by the parameters associated with variables DROWN and RESL, respectively. The two parameters have the expected sign and, in the combined sample model, are significant at the 8 and 7 percent levels, respectively. The two variables were not included in the Cloquet model, since almost all Cloquet respondents owned their home and length of residence was uniform across individuals. A third hypothesis being entertained--that automobile availability per worker has a positive influence on driving alone and carpooling--is reflected in model 5 by the parameter associated with variable AAPW. That parameter is also of the expected sign and is significant at the 5 percent level.

For all estimated coefficients, significance improved drastically when the sample size increased, as seen in Table 2, with the exception of the in-vehicle travel time coefficient (IVTT). All other coefficients in the combined Minnesota model are significant over the 8 percent level. Very short commuting trips in Le Sueur probably account for the perceived lack of importance of IVTT in that town.

For the convenience of prospective model users, two alternative models were derived for each town and these are also presented in Table 2. Models 2 and 4 differ from models 1 and 3, respectively, in that the former two do not use the variable AAPW but, rather, its components. In addition, automobile availability per licensed driver (AALD) was not found to be significant for work-trip modal choice in Cloquet and was not included in any demand model for that town.

The combined Minnesota rural work-trip modal-choice model is again presented in Table 3 along with two existing urban models. An inspection of the model coefficients confirms the observation found in the literature (1) that rural residents are more sensitive to travel cost than urban residents. Furthermore, remaining household income (RHINC) is seen as having an influence on rural modal choice greater than in urban areas by an order of magnitude, which also indicates the increased importance of financial considerations for transportation decisions in rural areas. Finally, it should be noted that the increased importance placed by urban commuters on OVTT in relation to IVTT is also observed in rural commuting and is of the same order of magnitude.

MODEL TESTING AND EVALUATION

Method

In testing the demand estimation models, six data sets were used. The following table summarizes these data sets and gives the monthly transit ridership for each data set:

Location	Data Set	Transit Route	Round-Trip Passengers per Month
Cloquet	1 }	Cloquet-Potlatch	292
	2 }	Cloquet-Diamond Match	
Le Sueur	3 }	Le Sueur-Green Giant	157
	4 }	Le Sueur-Hospital	
	5	Le Sueur-Telex	268
	6	Henderson-Telex	268

Because of its small size, data set 2 could not be used alone but only in combination with data set 1. Similarly, data set 4 had to be used in combination with data set 3. Six estimation models were tested: Macromodel (7), Micromodel (7), Poisson model (9), disaggregate Cloquet model 1, disaggre-

Table 1. Rural work-trip modal-choice model: definition of variables.

Variable Code	Definition	Expected Sign of Coefficient
D_a	1 for drive alone, 0 otherwise	
D_s	1 for shared ride, 0 otherwise	
OPTC/HINC	Round-trip out-of-pocket travel cost (¢) ÷ household annual income (1968$)	Negative
IVTT	Round-trip in-vehicle travel time (min)	Negative
OVTT/DIST	Round-trip out-of-vehicle travel time (min) ÷ one-way distance (miles)	Negative
$AALD_a$	Number of automobiles per licensed driver for drive alone, 0 otherwise	Positive
$AALD_s$	Number of automobiles per licensed driver for shared ride, 0 otherwise	Unknown
WPH_s	Number of workers in the household for shared ride, 0 otherwise	Positive
$AAPW_{a,s}$	Number of automobiles per household worker for automobile and shared ride, 0 otherwise	Positive
$RHINC_{a,s}$	Household annual income − 800 (number of persons in the household) for drive alone and shared ride (1968$), 0 otherwise	Positive
$DROWN_a$	1 for own residence and drive alone, 0 otherwise	Positive
$RESL_{t,s}$	Length of residence (years) for transit and shared ride, 0 otherwise	Positive

Note: a = drive alone, s = shared ride (carpool), and t = transit.

Table 2. Work-trip modal-choice model for rural Minnesota.

Variable	Cloquet Model 1[a]	Cloquet Model 2	LeSueur Model 3[a]	LeSueur Model 4	Combined Minnesota Model 5[a]
D_a					
Coefficient	-2.390	-1.854	-6.525	-5.912	-6.356
t-statistic	-0.715 3	-0.793 1	-2.340	-2.036	-2.933
D_s					
Coefficient	-3.192	-3.497	-8.694	-5.639	-6.832
t-statistic	-0.899 8	-1.209	-3.150	-1.311	-3.378
OPTC/HINC					
Coefficient	-77.620	-69.541	-114.72	-155.88	-136.99
t-statistic	-0.513 1	-0.492 4	-0.732 8	-0.974 2	-1.437
IVTT					
Coefficient	-0.059 46	-0.059 00	-0.013 85	-0.015 48	-0.029 31
t-statistic	-1.199	-1.198	-0.274 5	-0.311 3	-0.983 3
OVTT/DIST					
Coefficient	-0.589 0	-0.582 4	-0.494 9	-0.461 6	-3.583
t-statistic	-2.266	-2.284	-2.324	-2.454	-3.583
$AALD_a$					
Coefficient	-	-	-	1.166	-
t-statistic				0.571 7	
$AALD_s$					
Coefficient	-	-	-	-3.208	-
t-statistic				-0.692 8	
WPH_s					
Coefficient	-	0.597 4	-	0.247 7	-
t-statistic		0.643 0		0.409 9	
$AAPW_{a,s}$					
Coefficient	0.290 1	-	1.673	-	1.286
t-statistic	0.215 2	-	1.536	-	1.667
$RHINC_{a,s}$					
Coefficient	0.000 183	0.000 155 8	0.000 508	0.000 542	0.000 427 5
t-statistic	0.573 8	0.513 2	1.679	1.778	2.057
$DROWN_a$					
Coefficient	-	-	1.214	0.852 9	1.471 7
t-statistic			0.887 6	0.637 5	1.406
$RESL_{t,s}$					
Coefficient	-	-	0.037 46	0.040 94	0.012 37
t-statistic			0.530 9	0.576 0	1.500
Sum of chosen probabilities	21.83	21.89	26.64	27.74	49.27
$L^*(\hat{\theta})$	-29.11	-28.98	-16.14	-16.80	-47.14
$L^*(0)$	-43.94	-43.94	-40.65	-40.65	-84.59
$\rho^2 = 1 - [L^*(\hat{\theta})/L^*(0)]$	0.34	0.34	0.60	0.59	0.44

Note: $L^*(\hat{\theta})$ = log likelihood at convergence and $L^*(0)$ = log likelihood at zero.

[a]Selected for testing and evaluation.

gate Le Sueur model 3, and disaggregate combined Minnesota model 5.

Four error measurements were computed for each data set and model. These measurements included (a) absolute error (AE) and (b) percentage of absolute error (PAE), defined as a percentage of actual ridership. For data sets that were themselves combinations of other data sets, the sum absolute error (SAE) was computed to measure the total absolute error of the component data sets. Percentage of sum absolute error (PSAE) was also calculated for SAE as a percentage of actual ridership. These error measurements are defined as follows:

$$AE = |\text{actual ridership} - \text{estimated ridership}|,$$

$$PAE = |\text{actual ridership} - \text{estimated ridership}| / \text{actual ridership},$$

$$SAE = \sum_{i=1}^{N} |\text{actual ridership} - \text{estimated ridership}|_i, \text{ and}$$

$$PSAE = \sum_{i=1}^{N} |\text{actual ridership} - \text{estimated ridership}|_i / \sum_{i=1}^{N} \text{actual ridership}_i$$

where N is the total number of component data sets within a data set.

In testing the three aggregate models (Macro, Micro, and Poisson) certain application problems were encountered. For example, in both Le Sueur and Cloquet, the transit systems only serve work trips at specific destinations. The market for these systems is therefore smaller than the general population. The aggregate models tested do not seem to be suited for handling these cases since the values of independent variables such as RESTRPOP, POP_0, and POP_d in the Macromodel and Micromodel become very small and may lead to inaccurate results.

Other variables in the aggregate models also appear to be unclear in some applications. The variable BMILES in the Macromodel makes no distinction between deadhead miles and miles driven with passengers aboard. In certain cases, such as the Le Sueur system, which has one route between Le Sueur and Henderson 6 miles away, the deadhead miles are a significant portion of the total bus miles. In Cloquet, all service is within the city and deadhead miles are also further reduced as twice a day the bus drops off workers of one shift and leaves with workers from the previous shift without having to deadhead to the plant. These two situations are quite different, and it is unlikely that this model accurately handles both cases. Similar problems exist in applying the variable R, used by the Poisson model to account for system route mileage. Finally, it should be noted that, when applying the Macromodel and Micromodel, no corrections were made for fare, since in both cities the transit fare is the "base fare".

Results

The absolute error (AE) measurement for the six models tested is presented in Table 4 in two ways.

Table 3. Transferability of work-trip modal-choice model: rural versus urban.

Variable	Rural Minnesota	Urban[a] New Bedford	Urban[a] Los Angeles
D_a			
Coefficient	-6.356	-2.198	-2.746
t-statistic	-2.933	-2.648	-4.85
D_s			
Coefficient	-6.832	-1.535	-1.830
t-statistic	-3.378	-1.535	-3.95
OPTC/HINC			
Coefficient	-136.99	-87.33	-24.37
t-statistic	-1.437	-1.576	-2.07
IVTT			
Coefficient	-0.029 31	-0.019 9	-0.014 65
t-statistic	-0.983 3	-0.484 9	-2.25
OVTT/DIST			
Coefficient	-0.480 8	-0.101 3	-0.186 0
t-statistic	-3.583	-2.903	-4.02
$AALD_a$			
Coefficient	-	2.541	3.741
t-statistic		3.674	7.19
$AALD_s$			
Coefficient	-	0.449 9	0.609 3
t-statistic		0.847 8	1.58
WPH_s			
Coefficient	-	0.187 4	0.081 0
t-statistic		1.249	0.46
$AAPW_{a,s}$			
Coefficient	1.286	-	-
t-statistic	1.667		
$RHINC_{a,s}$			
Coefficient	0.000 427 5	0.000 072	0.000 083
t-statistic	2.057	1.279	2.31
$DROWN_a$			
Coefficient	1.471 7	-	-
t-statistic	1.406		
$RESL_{t,s}$			
Coefficient	0.012 37	-	-
t-statistic	1.500		
BW_a[a]			
Coefficient	-	1.026	0.810 1
t-statistic		3.769	3.28
$DTECA_s$[a]			
Coefficient	-	0.000 60	0.000 27
t-statistic		0.766 5	2.23
Sum of chosen probabilities	49.27	N.A.	N.A.
Log likelihood at convergence	-47.14	-256.5	-391.2
Log likelihood at zero	-84.59	-436.4	-930.0
ρ^2	0.44	0.41	0.58

[a]Models and variables introduced in report by Atherton and Ben-Akiva (16).

First, the error value is given so that conclusions on model performance can easily be drawn; evidently, lower errors indicate better model performance. Second, each model is compared with the Micromodel, and the deviation of its error with respect to that of the Micromodel is presented. A negative deviation means that the model in question has a greater error than the Micromodel and is therefore less desirable. A positive deviation implies that the model has a smaller error than the Micromodel and is therefore more desirable. Table 4 also includes a relative error measurement (PAE), which indicates the relative size of the absolute error with respect to the actual ridership value.

From the test results and the relative performance comparisons of Table 4, the following conclusions can be drawn:

1. At all times and for any individual data set, the proposed disaggregate specification performs substantially (up to 88 percent) better than the Micromodel. To be sure, this conclusion is drawn from testing the disaggregate models on a town different from that used in model development.

2. In testing model performance on combined data sets, the sum absolute error (SAE) again reveals the superiority of the disaggregate models. This conclusion can be drawn from the following table in

Table 4. Estimation errors of six demand models.

Location	Data Set	Estimation Model	AE	PAE	Improvement Over Micromodel (%)
Cloquet	1 and 2[a]	Macromodel[b]	230	79	-58
		Micromodel[b]	146	50	-
		Poisson[c]	260	89	-78
		Cloquet[d]	0	0	100
		Le Sueur[e]	47	16	68
		Combined[f]	18	6	88
Le Sueur	3 and 4[a]	Micro	275	176	-
		Cloquet	32	21	88
		Le Sueur	56	36	80
		Combined	50	32	82
	5	Micro	90	34	-
		Cloquet	81	30	10
		Le Sueur	17	6	81
		Combined	27	10	70
	6	Micro	262	98	-
		Cloquet	122	46	53
		Le Sueur	40	15	85
		Combined	50	19	81

[a]Treated as one data set.
[b]Developed by Burkhardt and Lago (7).
[c]Developed by Newman and Byrne (8).
[d]Developed with disaggregate data from Cloquet (model 1).
[e]Developed with disaggregate data from Le Sueur (model 3).
[f]Developed with disaggregate data from the combined Cloquet-Le Sueur sample (model 5).

which the Cloquet model, when applied to combined Le Sueur data sets, performs substantially better than the Micromodel (data sets 3 and 4 are treated as one data set):

Data Set	Estimation Model	SAE	PSAE	Improvement Over Micro-model (%)
3 and 4, 5	Micro	365	86	--
	Cloquet	113	27	69
	Le Sueur	73	17	80
	Combined	77	18	79
3 and 4, 5, 6	Macro	652	94	-4
	Micro	627	91	--
	Cloquet	235	34	63
	Le Sueur	113	16	82
	Combined	127	18	80

3. At all times and for any data set, the proposed disaggregate specification developed by using the combined Minnesota data performs substantially better than the Micromodel.

4. The Macromodel and Poisson model perform substantially (up to 78 percent) worse than the Micromodel. Although not indicated in Table 4 and the table above, the error always represents underestimation. This observation supports previous remarks on the performance of the Poisson model (8,9) but not on that of the Macromodel (9,11).

CONCLUSIONS

A disaggregate demand specification was developed to estimate rural work-trip modal choice. The inclusion of a set of policy-relevant variables allows the use of the model for implementing a wide range of transportation policies to improve transit system performance. The inclusion of mobility and socioeconomic variables allows one to take into account long-term changes in resident mobility and the local economy when determining modal choice. Although parameters did not change appreciably across the models developed for different towns, their statistical significance in general increased as the sample size increased.

When the rural specification is compared with existing disaggregate urban specifications, it is seen that variables associated with financial considerations are more important to rural commuters than they are to urban commuters. However, the increased importance placed on OVTT over IVTT is found to apply to rural and urban commuters in a similar fashion.

The test results suggest that, in the prediction of rural work-trip modal choice, the disaggregate specification developed here performs better (up to 88 percent better) than the best existing aggregate models for all locations and at all times. Of the existing models, the Micromodel appears to perform better than the Macromodel or the Poisson model, which consistently underestimate the demand.

Future work will include further testing of the disaggregate specification developed here. In particular, larger data samples will make it possible to identify a system of market segmentation so that the model can be tested on aggregate data with small aggregation bias. Research is planned toward developing improved specifications to increase the model sensitivity to a larger variety of policy options. For example, the model could be extended to handle additional modes of work travel or to include additional policy and socioeconomic variables. Research is also needed in developing similar specifications for additional trip purposes so that a more complete set of travel patterns for rural residents can be estimated.

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Synthesized Through-Trip Table for Small Urban Areas

DAVID G. MODLIN, JR.

Research performed to develop an improved and simple-to-use set of models that would facilitate the synthesis of a through-trip table for urban areas of less than 50 000 population is described. The effects of functional classification, average daily traffic, percentage of trucks, route continuity, and urban area population were determined to be significantly correlated with through-trip patterns. A least-squares analysis led to the development of a set of simple multiple regression expressions that estimate (a) the percentage of through-trip ends at each station and (b) the distribution of these trip ends among stations. The relations developed are simple to apply. The introduction of the new parameters, especially route continuity, appears to have improved the accuracy of the resulting trip table as compared with previous applications of the technique.

The Planning and Research Branch of the North Carolina Department of Transportation (NCDOT) is responsible for implementing the 3-C (continuing cooperative, and comprehensive) planning process as mandated by the Federal-Aid Highway Act of 1962. In addition, the Branch provides planning services on a contractual basis to any of the smaller urban areas that wish to develop a thoroughfare plan. To date, 165 municipalities have taken advantage of this service and 133 of the thoroughfare plans developed have been mutually adopted by the individual areas and NCDOT.

The factors leading to the implementation of various elements of a given thoroughfare plan are many and varied in nature. However, the basic element in the determination of the need for and the structure of a given thoroughfare plan is traffic volume. An analysis of existing volumes provides the basis for the determination of deficient transportation corridors. Growth in traffic volumes and the corresponding need for new and/or improved facilities may be anticipated by using some future land use plan and an understanding of the causal relations of trip generation and attraction. Typically, in major planning studies these relations are established from data available from the external, internal, truck, and taxi origin-destination (O-D) surveys. Mathematical expressions are developed that simulate the traffic patterns determined by the O-D surveys. Alternative transportation systems are then evaluated by using the developed simulation models.

The cost today of conducting O-D surveys in order to develop unique simulation models for individual small urban areas is prohibitive. Recent cost factors reported by NCDOT are the following: portable traffic counter per installation per week, $5.55; hourly machine count per installation per week, $14.00; classification count per 8-h count, $92.00; external station interview per interview, $1.00; and internal home interview per interview, $25.00. Additional costs are incurred in the processing of the raw O-D data into final report form.

The escalating costs described above mandate a synthetic procedure for determining travel patterns in small urban areas. Abundant literature exists that both describes and documents acceptable procedures for synthesizing internal-internal trips by modeling techniques. Given that the number of external-internal trips produced at each station could be determined by modeling techniques, then a proved procedure exists for their distribution among internal traffic zones.

The models discussed in this paper build on and validate previous attempts to synthesize through-trip (external-external) patterns by using multiple regression analysis and selected variables that are routinely available. Since the average daily traffic (ADT) at a cordon station is the sum of the external-external and external-internal traffic, if one can be estimated, the other is known. Thus, by successfully applying the procedures described in this paper, the total travel patterns for small urban areas can be synthesized and the benefits of long-range planning can be achieved at a minimum cost.

REVIEW OF LITERATURE

The Planning and Research Branch of NCDOT has been particularly successful in developing and applying techniques for synthesizing internal-internal travel patterns (1). This ability to synthesize internal-internal travel patterns helps to make long-range planning available to small urban areas (those with a population of less than 50 000). However, since the early 1970s, the cost of external O-D surveys has become a significant, and restrictive, factor in the provision of long-range planning services. To be viable, long-range planning must consider the data derived from the external O-D survey.

In 1969, a study (2) required by Section 17 of the 1968 Federal-Aid Highway Act provided a set of parameters that suggested a solution to the problem of estimating through trip patterns. By using this information, data routinely available from the annual count program, and published external O-D data from small urban areas, I was successful in developing a procedure for estimating through-trip patterns (3,4). The technique used multiple regression analysis to develop two models. The first model estimated the percentage of through-trip ends at each cordon station and the second, a composite model made up of six individual equations, estimated the distribution of the through trips among cordon stations. The result of the application of the models is a triangular through-trip table.

In 1978, Pigman (5) published a report following my work that also included the results of a comparative cross-classification analysis. Pigman concluded that the regression analysis technique provided fewer data problems and provided sufficient accuracy to make its use appropriate for planning purposes.

Pigman (5) also confirmed the importance of urban area population, ADT at the external station, and the percentage of trucks as estimators of through trips. The significance of the impact of functional classification was not proved in the estimation of through-trip productions; however, the distribution models were based on the functional classification of the origin station. The models developed by Pigman are considerably simpler and consequently less tedious to use than others (3) that have been reported. In achieving simplicity, however, there appears to be some minor loss of "statistical accuracy"

Pigman's work and continued interest by NCDOT to improve on its ability to offer transportation planning services have renewed interest in the simulation of through-trip movements. That interest and the desire to test the importance of new parameters and several modifications of old ones led to the effort reported in the remainder of this paper.

MODEL DEVELOPMENT

External O-D surveys of 14 cities and towns scattered throughout North Carolina that have populations ranging from 6600 to 50 500 were the source of data for the analyses. Based on the work of others (3,5) and the experience of NCDOT staff, a basic set of independent variables had already been identified. The experiences of the staff in applying models (3) previously developed suggested that route continuity should be important in the distribution phase and that modified forms of other parameters might prove to be more useful.

External-External Generation Model: Percentage of Through Trips

The external-external trip model estimates the percentage of through-trip ends at each external cordon station. Data on urban area population, urban area employment, ADT, percentage of trucks excluding panels and pickups, and percentage of panels and pickups were tabulated for 14 urban areas (see Table 1). Multiple linear regression analysis was used to derive a prediction equation.

The total number of observations used in the analysis was 241. Models were developed under two scenarios: (a) Functional classification was significant, and (b) functional classification could be

Table 1. O-D reports used in developing through-trip estimation model.

Urban Area	Year Conducted	Urban Area Population	Urban Area Employment	No. of External Stations	ADT	Trucks (%)
Lincolnton	1975	18 500	9 400	17	220-5 980	1.8-10.1
Dunn-Erwin	1973	17 300	7 250	21	280-15 210	2.0-19.6
Tarboro-Princeville	1973	13 500	10 500	9	1 190-5 800	4.4-11.2
Mount Airy	1974	22 900	13 750	19	340-9 010	4.2-21.9
Statesville	1971	37 000	16 050	24	90-11 200	2.5-19.9
Hickory	1973	50 500	38 650	30	340-28 190	2.0-18.0
Sanford	1977	21 900	14 200	23	270-11 170	0.3-25.7
Farmville	1976	6 600	3 550	7	1 550-5 320	5.4-16.3
Boone	1976	16 000	6 200	8	880-12 220	3.6-9.3
Shelby	1975	31 500	13 600	23	150-12 370	1.3-12.8
Canton	1972	10 000	4 150	12	90-18 000	2.1-18.1
Morganton	1970	16 500	16 500	18	210-12 400	1.6-16.2
New Bern	1975	25 350	8 650	10	180-11 400	0.6-10.5
Monroe	1974	15 900	8 300	20	300-16 110	1.3-19.5

Table 2. O-D reports used in developing through-trip distribution models.

Functional Classification	O-D Reports	ADT	Through-Trip Ends (%)	No. of Observations
Interstate	Dunn-Erwin, Hickory, Canton, and Morganton	10 000-28 190	48.1-97.5	135
Principal arterial	Tarboro-Princeville, Mount Airy, Hickory, Farmville, Boone, Shelby, and New Bern	3 660-16 230	26.6-71.8	179
Minor arterial	Tarboro-Princeville, Mount Airy, Farmville, Shelby, and Morganton	2 220-8 700	20.2-50.9	85
Major collector	Tarboro-Princeville, Mount Airy, Hickory, Farmville, Boone, Shelby, Morganton, and New Bern	1 550-8 150	6.6-25.4	166
Minor collector	Mount Airy, Boone, Shelby, and Morganton	1 020-2 400	6.2-18.1	86
Local	Mount Airy, Hickory, Boone, Shelby, Morganton, and New Bern	450-2 400	4.5-18.9	118

ignored. Under the first assumption, the data were grouped by functional classification and four equations were developed; under the second, a single equation was developed. The second case proved to be the better one. The equation that was simplest, represented all functional classes, and gave the best predicting ability (R = 0.86) was

$$\underset{(6.13)}{Y = 9.29 - 0.000\,31\ UP} + \underset{(12.72)}{0.0026\ ADT} + \underset{(8.07)}{1.48\ TRK} \tag{1}$$

where

Y = percentage of through-trip ends of the ADT at the external station,
UP = urban area population,
ADT = average daily traffic at the external station,
TRK = percentage of trucks excluding panels and pickups at the external station, and
( ) = t-value of the coefficient.

This equation is the exact form of the one developed by Pigman (5); even the coefficients are remarkably similar. This finding should validate the applicability of the technique and lend credence to the hypothesis (4) that the models might be transferable.

External-External Distribution Model: Percentage Distributed Among Stations

The second model developed is really a composite model in which equations for each of five functional classifications estimate the distribution of trip ends among stations. The initial groupings by functional classification and O-D reports used in this analysis are given in Table 2. The independent variables that proved to be significant were ADT at the destination station, percentage of trucks excluding panels and pickups at the destination, percentage of through trips at the destination, and route continuity as a dummy variable.

The recommended equations for the distribution phase are given in Table 3. The addition of route continuity in the data set was beneficial, and the development of an ADT attraction factor, following Pigman (5), also proved to be significant. The distribution equations are much simpler and statistically better than those previously reported (3) and are as simple as, and give better R^2 values than, those reported by Pigman (5).

STATISTICAL RESULTS OF MODEL DEVELOPMENT

Statistics provides a basis for judging the worth of prediction equations such as those presented in this paper. The format of each equation can be defended logically, and this is a first test. The ease with which the values of the independent variables can be determined at a base year and the confidence with which they can be projected to a design year are also considerations in model development. The variables in the final equations are routinely available and can be projected with reasonable confidence.

Table 4 summarizes selected statistical results for the models. The measures presented seemed to portray models that are indeed appropriate for planning purposes: They are simple, statistically sound, and reasonably accurate.

A further test is one of model performance on a station-by-station and movement-by-movement basis. A decision was made to compare the performance of the models presented in this paper with that of models previously developed (3) by using the original test cites, Ahoskie and Wilson. Based on root-mean-square error (RMSE), the model recommended in this paper for estimating percentage of through-trip ends yielded significantly poorer results for both Ahoskie (1965) and Wilson (1964) O-D data than did the previously reported models (3). Percentage of trucks seemed to be the cause of the poorer results. The old models (3) were developed by using an average 1965 data base, whereas the models reported in this paper are based on average 1975 data. Could it be that over the 10-year period an assumption that the truck-car through-trip relation had remained

Table 3. External-external trip distribution models.

Functional Classification of Origin Station	Distribution Equation	R^2
Interstate	Y = -2.70 + 0.21 PTTDES + 67.86 RTECON (5.48) (23.28)	0.96
Principal arterial	Y = -7.40 + 0.55 PTTDES + 24.68 RTECON (6.22) (9.09) + 45.62 ADT/CD (2.66)	0.87
Minor arterial	Y = -0.63 + 86.68 ADT/CD + 30.04 RTECON (8.18) (11.38)	0.86
Major collector	Y = -1.08 + 0.000 79 DESADT + 0.47 PTKDES (4.21) (2.43) + 31.78 ADT/CD (3.45)	0.69
Minor collector and local	Y = -0.40 + 109.42 ADT/CD (15.37)	0.73

Note: Y = percentage distribution of through-trip ends from an origin station to a destination station, PTTDES = percentage of estimated through-trip ends at destination station, RTECON = route continuity (1 = yes, 0 = no), ADT/CD = ADT at destination station divided by the sum of ADT at all stations, DESADT = ADT at destination station, PTKDES = percentage trucks excluding panels and pickups at the destination station, and ( ) = t-value for the coefficient.

Table 4. Statistical results for models.

Model	Total Observations	Mean of Dependent Variable	R^2	Standard Error	Coefficient of Variation
Percentage through-trip ends	241	20.58	0.74	9.66	47
Distribution of through-trip ends					
Interstate	134	5.22	0.92	5.36	103
Principal arterial	179	8.95	0.76	9.35	104
Minor arterial	85	8.24	0.74	6.63	80
Major collector	166	7.22	0.48	7.74	107
Minor collector and local	204	6.46	0.53	6.83	106

Table 5. Analysis of stability of through-trip-end estimation model.

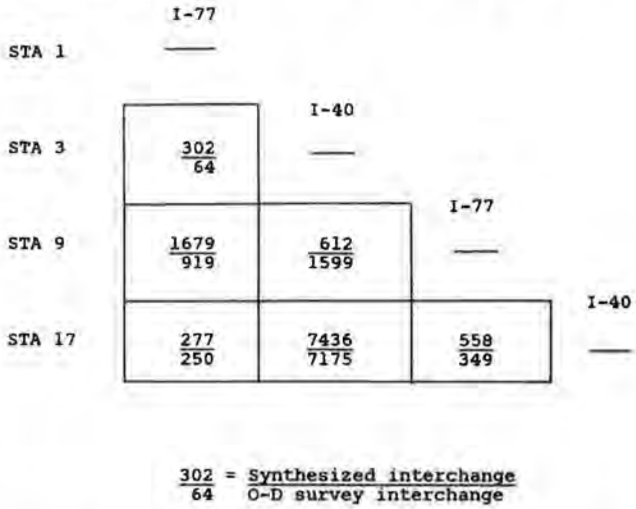
Item	1958	1973
Cordon ADT	22 798	29 790
Number of trucks excluding panels and pickups	2 705	2 407
Percentage of trucks	11.87	8.08
Through-trip ends	6 676	6 586
Percentage through-trip ends of cordon ADT	29.28	22.11
RMSE for trip-end-estimation model performance (%)	10.37	9.40

constant was indeed invalid?

As a beginning point in attempting to answer this question, the average percentage of trucks of the station volumes for the 1965 and 1975 periods was analyzed. On a 1965 basis, both Ahoskie and Wilson had more than 11 percent trucks in the traffic stream crossing the cordon, whereas the average percentage of trucks crossing the cordon in the 1975 data base was 7.1 percent. It appeared that the composition of the traffic stream had indeed changed over time and that a performance comparison that used the old test data to compare the new models with those previously reported would be invalid.

Fortunately, external O-D surveys had been conducted in Elizabeth City during two time periods, 1958 and 1973. The availability of these data provided the opportunity to test in a limited manner the hypothesis that the relation of percentage of trucks in the traffic stream to the production of through trips was changing over time. A summary of the pertinent analysis data is given in Table 5. The

Figure 1. Distribution results with major intersecting routes: Statesville, North Carolina.



models recommended in this paper performed better when compared with the 1973 O-D data than when compared with the 1958 data. This result tends to confirm the hypothesis that through-trip production models have been dynamic during the past two decades and that periodic reevaluation of the models is required in order to maintain their maximum efficiency.

Now that there was an explanation for the adverse comparison of the old versus new models utilizing the 1965 vintage O-D data, a test of the new through-trip estimation model was performed on 1975 O-D data for Laurinburg, North Carolina, a city with an urban area population of 22 500 and 20 external cordon stations. The model performed extremely well, yielding an RMSE of 7.6 percent. One station, Secondary Road 1601, had an extraordinarily high proportion of trucks, 18.3 percent, compared with the station volume of 240 vehicles/day. Removing this station from the analysis yielded an RMSE of 6.3 percent.

The next test was that of the distribution models. Again, the new distribution models were tested against those previously reported (3) by using the 1965 Ahoskie O-D data. The old distribution models had previously given an RMSE of 87.8 trips; in comparison with the same data, the new models gave an RMSE of 88.1 trips. Given that the basis for the distribution was the trip ends estimated by the generation model, it can be concluded that the new distribution models are superior to the old ones. This statement is derived from the fact that the through-trip ends estimated by the new generation model based on the 1965 Ahoskie test data had an RMSE of 13.6 percent compared with that of 11.5 percent for the old models (4). Therefore, the starting point for the new distribution models had 18 percent more error than the old models. The new models not only are much simpler but also produce a better distribution.

A problem discovered in applying the old through-trip distribution models was the poor performance of the models in handling the case of the intersection of two major facilities--e.g., I-40 and I-77 in Statesville, North Carolina. This situation was tested with the new models, and the results are shown in Figure 1. For route continuity, the paired stations are 1 and 9 and 3 and 17. The results are much better than those for the old distribution models.

Table 6. Input data for example model application.

Station No.	Functional Classification	ADT	ADT/Cordon	Trucks (%)	Route Continuity
1	Major collector	1550	0.075	5.7	No
2	Major collector	2200	0.107	7.3	No
3	Major collector	1560	0.076	5.4	No
4	Principal arterial	4380	0.214	16.3	With 6
5	Minor arterial	3130	0.153	9.7	With 7
6	Principal arterial	5320	0.260	16.2	With 4
7	Minor arterial	2360	0.115	14.4	With 5

Note: Urban population = 6600.

Table 7. Selected distribution results.

Origin Station	Destination Station	Calculated Percentage	Adjusted Percentage	Factor
1	2	7.49	10.63	1.4196
	3	5.11	7.25	
	4	16.84	23.91	
	5	10.81	15.35	
	6	18.99	26.96	
	7	11.20	15.90	
Total or avg		70.44	100.00	
4	1	6.86	6.19	0.9019
	2	10.55	9.51	
	3	6.68	6.02	
	5	15.94	14.38	
	6	53.92	48.63	
	7	16.93	15.27	
Total or avg		110.88	100.00	
5	1	5.87	5.89	1.0032
	2	8.64	8.67	
	3	5.96	5.98	
	4	17.92	17.98	
	6	21.91	21.98	
	7	39.38	39.50	
Total or avg		99.68	100.00	

EXAMPLE APPLICATION OF MODELS

A short example is offered to demonstrate the ease with which the recommended models can be applied and to illustrate some of the mathematical detail in developing the estimated through-trip table. The input data required are given in Table 6.

Estimates of the percentage of through-trip ends at each external station are developed by using the major-collector equation given in Table 3. Then each percentage is multiplied by the corresponding station ADT to produce an estimate of the number of through-trip ends passing the station. These trip ends will be used in the distribution phase. The results for the example are given below (total trips = 3575):

Station No.	Through-Trip Ends Estimated Percentage	Calculated No.
1	19.71	306
2	23.76	523
3	19.29	301
4	42.75	1872
5	29.74	931
6	45.05	2397
7	34.69	819
Total		7149

The next step is to match each station, according to its functional classification, with the proper equation as given in Table 3. The correct equation for each distribution is chosen according to the station from which the trip ends are to be distributed, the origin station. For example, from station 1, major collector, to all other stations, one would choose Equation 4 from Table 3.

In applying any of the distribution equations, the resulting sum of the estimated percentages from one station to all others does not generally add up to 100 percent. The percentages should simply be factored so that their resulting sum is 100 percent. The results of the distribution phase for this example are given in Table 7 for stations 1, 4, and 5.

In applying the proper distribution equation at each station, the estimated two-way trip interchange between a particular origin station and all other destination stations is generated. Two-way trips are distributed because the dependent variable initially estimated was trip ends. The distribution procedure, when completed, results in two estimates of two-way trip interchange for every pair of stations, each having acted as an origin station once. The value used for this triangular trip matrix is taken as the average of the two values.

After the estimated trip interchanges are averaged and the trip matrix is summed, the total number of trip ends at individual stations will vary from the values predicted by the initial equation. This results because of the averaging procedure. A FRATAR factor is determined, and the trip table is balanced and adjusted to the initial predicted number of through-trip ends at each station. Figures 2 and 3 present the results of the above procedures for the example problem.

Figure 2. Synthesized through-trip table.

STA. NO.	1	2	3	4	5	6	7
1	0						
2	33 37 40	0					
3	22 22 22	38 35 22	0				
4	73 94 116	129 154 178	72 92 113	0			
5	47 51 55	83 82 81	46 51 56	269 218 167	0		
6	82 118 153	146 190 235	81 115 149	910 1018 1127	204 280 355	0	
7	49 48 47	86 78 69	48 48 47	286 215 143	368 353 339	378 276 174	0
TOTAL	370	576	363	1791	1035	1997	1018
DESIRED TOTAL	306	523	301	1872	931	2397	819
FRATAR FACTOR	0.827	0.908	0.829	1.045	0.900	1.200	0.805

Figure 3. FRATAR adjusted through-trip table.

STA. NO.	1	2	3	4	5	6	7
1	0						
2	24	0					
3	13	25	0				
4	70	124	69	0			
5	37	67	37	186	0		
6	130	229	125	1268	349	0	
7	30	56	32	156	255	293	0
TOTAL	304	525	301	1873	931	2394	822

SUMMARY AND CONCLUSIONS

The purpose of the research discussed in this paper was to try to improve the methodology for synthesizing a through-trip table for small urban areas. New parameters were introduced in an attempt to alleviate problems discovered in using previously developed models. The new variables that proved to be significant, both in leading to simpler models and avoiding old problems, were route continuity as a dummy variable and station ADT developed as an attraction factor.

Both models continue to reflect the importance of trucks in the estimation and distribution of through trips. The importance of this factor has varied since the mid-1960s. During the mid-1970s, the increased availability of the automobile and an expanded standard of living diminished the correlation between trucks and through trips. The important point is that, as relations among the independent parameters change, the models, to remain valid, must be updated.

Overall, the models presented are adequate for long-range planning purposes. They are extremely easy to apply and produce results that are reasonable and sufficiently accurate for planning purposes.

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