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Foreword

The 1960 Highway Act made mandatory the planning of transportation facilities in urban areas. Origin and destination techniques which measure transportation patterns, now a quarter of a century old in application, are used in conducting these sophisticated urban studies. The advent of the computer has heightened the utilization of extensive sets of data in attempting to devise planning solutions to the massive urban transportation problems that exist.

The eight papers and one abstract in this RECORD are largely concerned with details of employment of the sophisticated technology used in comprehensive urban transportation planning. As such this RECORD is of chief use to those engaged in such planning and to those who must understand what is going on and how the results might be influenced by the methodology.

The first paper, by Whitaker and West, has considered in a theoretical manner many of the problems associated with trip distribution models especially the intervening opportunities model and they have formulated judgments based on their research.

Marshall presents the system for traffic simulation as done in New York State. The system is designed to select a test plan, assign traffic, summarize cost/benefit information and prepare data for an automatic plotting device.

Brown and Woehrle discuss the Tri-State Transportation Commission's approach to traffic volume estimation. They review the general concepts, inputs and methods of computation and present initial calibration results.

Balkus and Jordon set forth the Tri-State Transportation Commission's considerations in developing a regression model for projecting miles traveled by motor vehicles in a metropolitan region. Devised for the Commission's area, the model should be applicable to any urbanized area.

Zakaria and Falcocchio explain the traffic assignment process. A unique feature of their process is the fact that the capacities of highways to handle a wide range of traffic volumes can be taken into account.

Recreational travel to reservoir areas can be predicted by suitable mathematical models according to Matthias and Grecco, based on their findings from 13,000 Indiana interviews. Regression equations for two differing conditions were developed and constitute a method for prediction.

Roberts has evaluated the influence of terminal times on gravity model travel time for the small urban area studied in South Carolina. He found that the gravity model provided an adequate framework for determination of trip distribution patterns by either using the model with terminal times or without terminal times.

Characteristics of taxicab usage in Chicago were investigated by Beimborn. It was indicated that cab trips are highly oriented to the central area and involve non-work trips to a large extent. Trips are fairly short and distributed quite well over the time of day. In general taxi trips are quite different from the characteristics of urban travel taken as a whole.

The RECORD concludes with Worrall's abstract of research performed under NCHRP auspices pertaining to household travel analysis. The research was concerned with the temporal aspects and analyses of travel.

Department of Traffic and Operations

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Purdue University, Lafayette, Indiana

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Contents

The Intervening Opportunities Model: A Theoretical Consideration

ROBERT W. WHITAKER, Peat, Marwick, Livingston & Co., and
KENNETH E. WEST, Kates, Peat, Marwick & Co.

The intervening opportunities trip distribution model, though functionally more complex than its predecessors, has been subject to very little investigation except from a utilitarian point of view. While preparing a calibration computer program, an opportunity was found for a more theoretical consideration.

It was discovered that it is sometimes mathematically impossible to derive a value for L (the calibration parameter) which will cause an interchange to assume an allowable value; that is, it is sometimes not possible to calibrate the model to a base year. The reason for this is that, for a single interzonal interchange, increasing the size of L only causes a corresponding increase in the trips generated up to a certain point (the maximum). Beyond that point, any increase in L causes a decrease in the number of trips generated. This maximum is often less than the number of trips desired in a base year.

When interchanges are grouped, the same difficulty with maxima occurs. The problem is complicated by the facts that there may be more than one maximum and that one maximum may be higher than another. It is explained how the best L-factor is chosen from such a group.

•THE intervening opportunities trip distribution model is much more complex than any of its predecessors. Possibly for this reason, it has received very little purely theoretical investigation. The authors found an opportunity for such an investigation while developing a computer program to calibrate the model. Some of the findings, together with a review of the derivation of the model, are presented here in the hope that a better understanding of travel models will result.

DERIVATION

The intervening opportunities model (1) assumes that the trip interchange between an origin and a destination zone is equal to the total trips emanating from the origin multiplied by the probability that each trip origin will find an acceptable terminal at the destination. This is expressed mathematically as follows:

$$T_{ij} = O_i P(D_j) \qquad (1)$$

where

T_{ij} = the trips between origin zone i and destination zone j;

O_i = the total trip origins produced at zone i;

D_j = the total trip destinations attracted to zone j; and

$P(D_j)$ = the probability that each trip origin at i will find destination j an acceptable terminal.

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$P(D_j)$ is expressed as a function of D_j. D_j is defined as the total trip destinations attracted to zone j because the model assumes that the probability that a destination will be acceptable is determined by two zonal characteristics: the size of the destination and the order in which it is encountered as trips proceed away from the origin.

$P(D_j)$ may also be expressed as the difference between the probability that the trip origins at i will find a suitable terminal in one of the destinations, ordered by closeness to i, up to and including j, and the probability that they will find a suitable terminal in the destinations up to but excluding j; thus

$$T_{ij} = O_i \left[P(A) - P(B)\right] \tag{2}$$

where

A = the sum of all destinations for zones between, in terms of closeness, i and j and including j; and

B = the sum of all destinations for zones between i and j but excluding j.

Note that

$$A = B + D_j \tag{3}$$

It is possible to formulate the function P as follows. The probability that a trip will terminate within some volume of destination points is equal to the product of two probabilities: (a) the probability that this volume contains an acceptable destination, and (b) the probability that an acceptable destination closer to the origin of the trip has not been found. This may be expressed in differentials as follows:

$$dP = (1 - P)\, LdV \tag{4}$$

where

$$P = P(V)$$

and where

V = the volume of destination points (destination trip ends) within which the probability of a successful terminal is to be calculated; and

L = the probability density (probability per destination) of destination acceptability at the point of consideration.

Assuming L to be constant, the solution to Eq. 4 is

$$P = 1 - ke^{-LV} \tag{5}$$

where

k = the constant of integration; and

e = the constant base of the natural logarithms, 2.71828. . . .

It can be shown that k = 1 since P must be zero when V is zero. Eq. 5 thus becomes

$$\begin{cases} P(V) = 0 & V \leq 0 \\ P(V) = 1 - e^{-LV} & V > 0 \end{cases} \tag{6}$$

Eq. 6 is a cumulative probability distribution. It is nondecreasing, $P(-\infty) = 0$, and $P(\infty) = 1$. Its corresponding density function is

$$P'(V) = Le^{-LV} \tag{7}$$

The function thus derived for P(V) may be substituted into Eq. 2 letting V equal A and B; thus

$$T_{ij} = O_i\,(e^{-LB} - e^{-LA}) \tag{8}$$

Eq. 8 is the standard formulation of the intervening opportunities model. This formulation requires that destination zones be ordered according to their travel time from the origin zone. Thus, destinations are placed in sequence according to the contents of the skim trees associated with the origin.

INVESTIGATION

The intervening opportunities model as derived above is unusual among trip distribution models in that it does not guarantee the utilization of 100 percent of the origins available; that is, it has been found that all origins are seldom accounted for. The reason for this is that its cumulative probability distribution as represented by Eq. 6 approaches one only as the total destinations become very large. In practice, this means that 10 or 20 percent of the origins from a zone may easily remain unaccounted for.

Another difficulty in using the model was encountered by staff members of the Traffic Research Group of Peat, Marwick, Livingston & Co.; namely, it is not always possible to calibrate the model to a base year. This was observed while implementing a contract with the U.S. Bureau of Public Roads to program the trip distribution portion of the Urban Planning System/360 Traffic Assignment Package.

It was the group's responsibility to derive and program a calibration technique as well as to program the model itself. It was decided to develop a procedure which would locate an L-factor yielding exactly the number of trips required for any combination of zonal interchanges. This approach was somewhat novel in that all previous techniques known to the authors strive to optimize some derived trip characteristic such as vehicle-miles of travel.

It was first attempted to calculate this optimum L-factor by means of several iterative approaches. These attempts were all found to be inadequate to cope with certain interchange configurations. An investigation was initiated to discover the reason for these failures. The first and second derivatives according to L were taken of Eq. 8:

$$\frac{d}{dL} T_{ij} = O_i\,(Ae^{-LA} - Be^{-LB}) \tag{9}$$

$$\frac{d^2}{dL^2} T_{ij} = O_i\,(B^2e^{-LB} - A^2e^{-LA}) \tag{10}$$

The first derivative was then set equal to zero and was solved for a value of L, L_0, which was shown to be a maximum by demonstrating that Eq. 10 is less than zero when L_0 is substituted into it.

$$L_0 = \frac{\ln (A/B)}{A - B} \tag{11}$$

A maximum for T_{ij}, T_m, may thus be calculated by substituting Eq. 11 back into Eq. 8:

$$T_m = O_i \left[\frac{B}{A}\right]^{\frac{B}{A-B}} \left[1 - \frac{B}{A}\right] \tag{12}$$

Eq. 12 reduces to an interesting and useful form when solved for the ratio T_m/O_i and when the substitutions $A = B + D_j$ and $r = B/D_j$ are made:

$$\frac{T_m}{O_i} = \frac{r^r}{(r+1)^{r+1}} \tag{13}$$

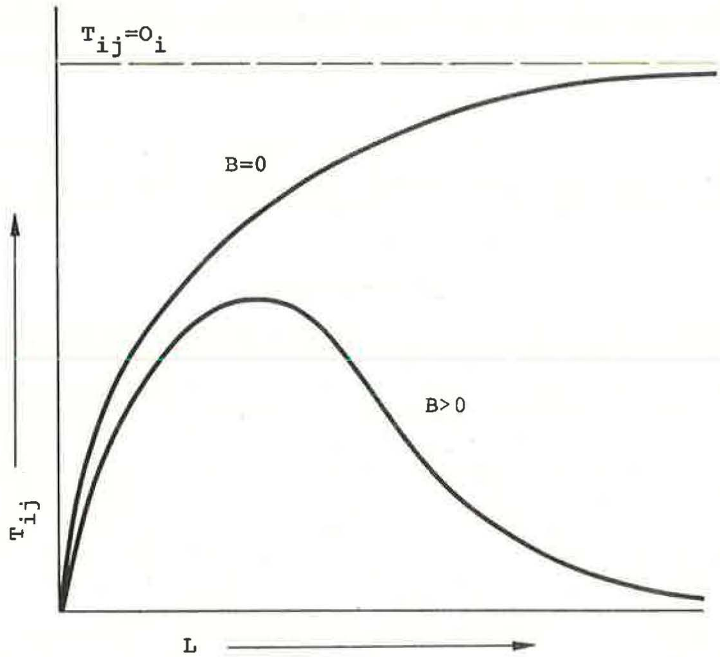

Figure 1. Graph of $T_{ij} = O_i\ (e^{-LB} - e^{-LA})$ as a function of L.

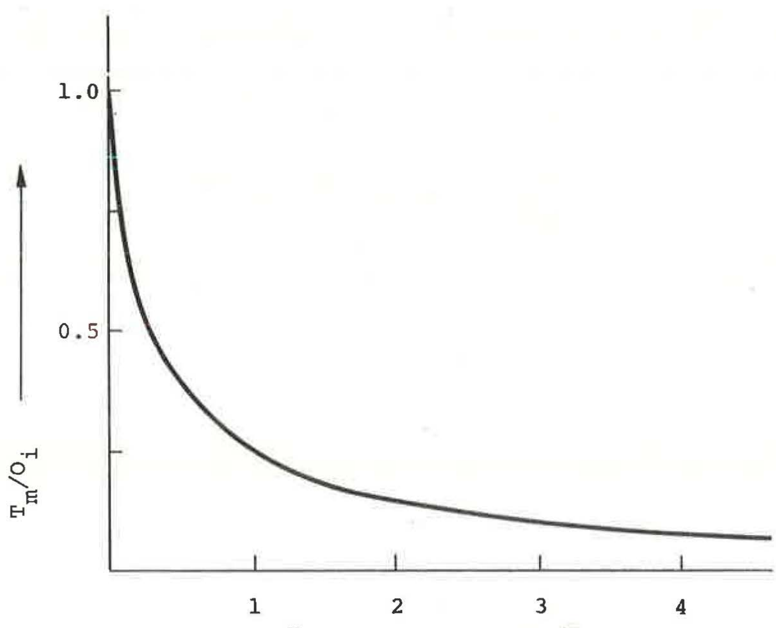

Figure 2. Graph of $\frac{T_m}{O_i} = \frac{r^r}{(r+1)^{r+1}}$.

For any interzonal interchange, so long as O_i, B and A are held constant, an increase in the value of L only causes a corresponding increase in the number of trips generated by the model up to the point L_0. Beyond that point, any increase in L causes instead a decrease in the number of trips generated. This is shown graphically in Figure 1. Note that intrazonal interchanges (B = 0) are a special case where an increase in L always causes an increase in T_{ij}. This is part of the reason that the model has been thought to overestimate intrazonal trips.

It is thus evident that there are allowable values of T_{ij} which the intervening opportunities model cannot produce. The seriousness of this limitation may be judged by studying Figure 2, a graph of Eq. 13. Where the ratio B/D_j is greater than 1, no more than 0.25 of the origins may be included in any interchange. Where the ratio is greater than 2, no more than 0.15 of the origins may be included; greater than 3, no more than 0.11; greater than 4, no more than 0.08, etc. When the ratio becomes quite large, as is usual, the maximum number of trips which the model will produce for an interchange becomes very small. As an example, when the ratio is 1000, the model will include no more than 0.0004 of the origins in any interchange.

The calibration technique actually adopted for inclusion in the System/360 Trip Distribution Package takes the limitations of the model into account. For any group of interchanges, G, the so-called best L-factor is derived. The best L-factor is defined as the first which yields the desired number of trips for interchange group G. If there is none which yields the desired number, then the L-factor yielding the most trips for group G is supplied.

It is important to understand that, although the function for one interchange may have only one maximum, the function for interchange group G may have as many maxima as there are interchanges in G. Thus, there may be a multitude of possible L-factors. The program chooses only the first in each case.

CONCLUSION

It is evident that the intervening opportunities model is mathematically limited in two ways in spite of the fact that it is a genuine probability model. In the first place, it is seldom able to account for 100 percent of the origins at any zone. In the second, it is so structured that there is a maximum number of trips which may be generated for any given interzonal interchange with an accompanying tendency to overestimate intrazonal interchanges. Thus the model is more difficult both to calibrate and to apply than has generally been found to be the case with other distribution techniques.

REFERENCE

1. Schneider, Morton. Appendix to Panel Discussion on Inter-Area Travel Formulas. HRB Bull. 253, p. 136-138, 1960.

Discussion

EARL R. RUITER, Massachusetts Institute of Technology—This discussion is written for two purposes: (a) to comment on the problem of calibrating the intervening opportunities model in the light of the limitations of the model discovered by the authors; and (b) to develop, using probability theory, an intervening opportunities model that does guarantee utilization of 100 percent of the origins available.

Calibration Aspects

The authors' empirical discovery and theoretical validation of the fact that it is not possible to duplicate any arbitrary group of trip interchanges under all conditions will be useful to future users of the opportunity model. The authors pose a real calibration difficulty, given their calibration objective, and prescribe a method of overcoming this difficulty; however, the authors' calibration objective can be improved upon. This can be done in such a fashion that the model limitation no longer poses calibration problems. If the fact that there will be errors in calculated trip interchanges is recognized, then the objective of the calibration procedure can be to minimize these errors. Thus, rather than attempting to match exactly a limited number of groups of interchanges, the error in the prediction of all interchanges can be minimized. This can be done using the concepts underlying multiple regression, by finding the L-value which will minimize the sum of the squares of the deviations between observed interchanges and calculated interchanges.

In equation form, if T_{ij}^{o} and T_{ij}^{c} are, respectively, an observed and a calculated interchange, then the error, D_{ij}, is, using the notation developed by the authors:

$$D_{ij} = T_{ij}^{o} - T_{ij}^{c} = T_{ij}^{o} - O_i\,(e^{-LB_j} - e^{-LA_j})$$

And the sum of the squares of all errors, S, is:

$$S = \sum_i \sum_j D^2_{ij} = \sum_i \sum_j [T_{ij}^{o} - O_i\,(e^{-LB_j} - e^{-LA_j})]^2$$

S is minimized when its derivative, with respect to L, is zero:

$$\frac{dS}{dL} = \sum_i \sum_j \left\{ 2\,[T_{ij}^{o} - O_i(e^{-LB_j} - e^{-LA_j})] \cdot [O_i\,(B_j\,e^{-LB_j} - A_j\,e^{-LA_j})] \right\} = 0$$

The solution of this equation for L may be a formidable task, but an iterative technique not unlike that which the authors have implemented could be used. The advantages of this calibration procedure are that all relevant data can be used and that a measure of the total error introduced by specifying an L-value is minimized.

Revised Model Derivation

An intervening opportunities model which will guarantee utilization of 100 percent of the trip origins available can be derived using the mathematical principles of random variables and probability functions. The mathematical developments in this section are based on the probability principles as presented by Wadsworth and Bryan (2).

As the authors point out, the cumulative probability distribution which underlies the intervening opportunities model (the authors' Eq. 6) is defined for all positive values of V, from zero to infinity. When the maximum value of V is V_n, Eq. 6 states that the probability of an origin finding an acceptable destination is:

$$1 - e^{-LV_n}$$

which approaches 1 as V_n approaches infinity. When it is known that all trip origins do end before V_n destinations are considered, trips should be a function of $P(V/V_n)$, the probability that a trip will end before reaching V destinations, given that it ends before reaching V_n destinations. $P(V/V_n)$ can be developed as follows:

$$P(V/V_n) = \frac{P(V, V_n)}{P(V_n)}$$

where

$P(V, V_n)$ = the probability that a trip will end before reaching V destinations <u>and</u> that it will end before reaching V_n destinations;

$P(V, V_n)$ = $P(V)$ because all V destinations are included in V_n;

$P(V_n)$ = the probability that a trip will end before reaching V_n destinations.

Using $P(V) = 1 - e^{-LV}$

$$P(V/V_n) = \frac{1 - e^{-LV}}{1 - e^{-LV_n}} \qquad (0 < V < V_n)$$

This function will distribute all trips, as can be shown by setting $V = V_n$ and observing that $P(V_n/V_n) = 1$.

The erroneous use of $P(V)$ rather than $P(V/V_n)$ in operational opportunity model programs can be corrected. For example, the Chicago Area Transportation Study has developed a corrected version, which they refer to as their "forced interchange" opportunity model (3).

In summary, the authors' discovery of limitations of the opportunity model is important, but is not an invalidation of the model. Valid calibration procedures can be devised which recognize the limitations of the model. Also, mathematically valid revisions of the model can be implemented to guarantee the utilization of 100 percent of the trip origins available at each zone.

References

2. Wadsworth and Bryan. Introduction to Probability and Random Variables. McGraw-Hill, 1960
3. Walker, S. A., III. Recent Developments in the Simulation of Transit Travel in the Chicago Area. CATS Research News, March-April 1968.

A Traffic Simulation Package Using the Opportunity Model

RALPH J. MARSHALL, Planning and Research Bureau, Planning Division, New York State Department of Transportation

This paper describes the system and components derived for the purpose of traffic simulation as done in New York State. All of the programs have been written in COBOL or ALGOL for the Burroughs B-5500. The system is designed to select a test plan from a composite network, assign traffic to the plan using the opportunity model, summarize the benefit cost data for analysis purposes and, finally, prepare data for use on an automatic plotting device.

•THE New York State Department of Transportation is responsible for all transportation planning for the State, including that for both urban and interurban areas. A major step in the planning process is the testing of alternate networks through the use of models and computer programs which simulate the networks and distribute and assign travel. The basis for this simulation and assignment process is the opportunity model developed by the Chicago Area Transportation Study. The New York planning staff, however, has adapted this model and developed others as well as concepts and computer programs which reflect further capabilities and refinements in the simulation process.

The Department acquired a Burroughs B-5500 computer in January 1966, and it was necessary to rewrite the various existing programs for simulating travel in a format suitable to this equipment. The result is a traffic simulation package, which uses the opportunity model for zonal trip interchange distribution and which consists of a system of six programs. Each program is written in COBOL or ALGOL so that the package may be easily modified to the user's particular requirements.

This paper discusses each of the six programs and those aspects of the traffic simulation process (1) necessary to the explanation of the programs.

GENERAL DESCRIPTION

A macro-diagram of the simulation package is shown in Figure 1. The six programs and the functions of each are summarized as follows:

1. Network Selection—Prepares tape of selected network links which will comprise test network.
2. Input Processing—Prepares link and zone data into tables for use in assignment.
3. Assignment—Assigns trips to network; this includes building trees, calculating interchanges, loading trees, and calculating benefit-cost.
4. VMT Comparison—Compares data from vehicle-miles of travel survey with those assigned from travel interview surveys.
5. Economic Analysis—Summarizes transportation costs, including those for vehicle operation, accidents, and travel time.
6. Plot—Prepares tapes for use by plotting equipment in plotting volume, capacity, deficiencies, and the like.

Paper sponsored by Committee on Origin and Destination and presented at the 47th Annual Meeting.

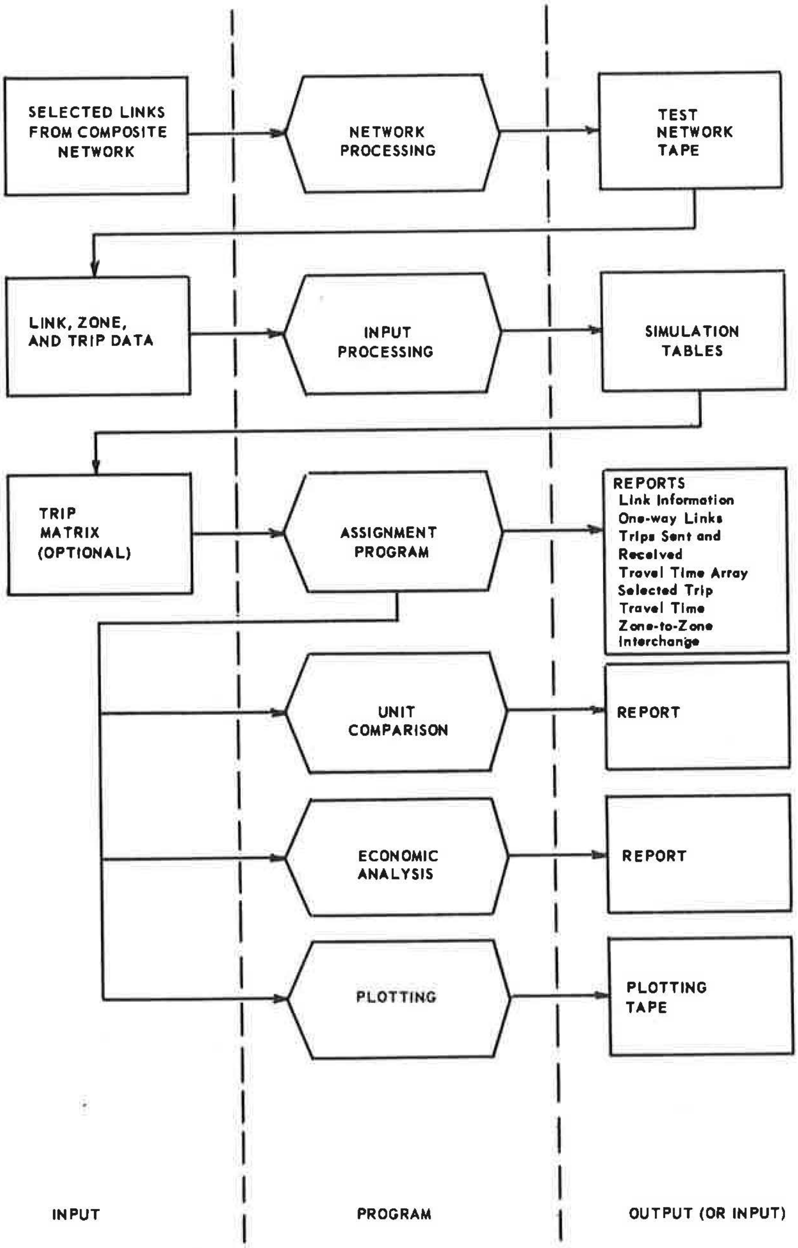

Figure 1. Macro flow diagram of traffic simulation package.

NETWORK SELECTION AND INPUT PROCESS

A technique, developed by the New York planning staff by which a composite network is delineated, has considerably reduced the amount of time and effort required in the network testing process (2). The composite network includes the existing, base-year network as well as all possible links which planners wish to test for inclusion in a recommended plan for the future. A description of each link in the composite network is coded; and when the planner has identified those links to be included in the test network, the selection program transfers the information for these links onto a tape for use in the assignment process. A further step in the preparation of the input data for the assignment process is to arrange the link descriptions and the trip data for each analysis zone into tables or disk arrays. This is accomplished by the input processing program. The data included are as follows (3):

Link	Trips and Analysis Zone
Ring and sector	Zone number
A and B node numbers	Loading sequence
Direction	Loading node
Facility type	Zone location (internal or external)
One or two direction	Number of short trips
Free-flow speed	Number of long residential trips
Design capacity	Number of long non-residential trips
Free travel time	L-value for long and short trip categories
Length	

TRAFFIC ASSIGNMENT

The traffic distribution model used in the traffic simulation process is a variation of the opportunity model developed by the Chicago Area Transportation Study. The difference in the two models is primarily that the Department's model uses a variable L for long and short trips for each traffic analysis zone (4). In practice, the L-values for zones in the same district are assigned the same value.

The traffic assignment program has been written so that a network may be tested with a maximum of 126,000 one-way links, 1,022 zones, and 4,095 nodes. The Burroughs B-5500 computer, however, is limited by a 32,000 (32K) word capacity and cannot process a network of this size. Because the program itself consumes some of this space, the practical limitation of this computer system is that any combination of two-way links and the number of nodes must not exceed 28,000 (28K) words. The other network information is not affected by the core limitation because the data are switched back and forth between core and disk as needed.

The user may select any combination of several options in the traffic assignment program[1], including:

1. Tree Building Sequence—Trees can be built in zone order or in the sequence which the user assigns to the zones.
2. Travel Time Revision or Capacity Restraint—The travel time required to traverse a link can remain unchanged or it can be changed during the processing by using the ratio of the amount of traffic assigned to the link and the amount it can theoretically handle.
3. Trip Distribution—Trip distribution may be performed by either an externally prepared trip matrix or by an internally calculated trip interchange between zones, using the opportunity model.

Most traffic simulation runs make use of the options to (a) build trees in sequence number order, (b) revise link travel time, (c) use the opportunity model to calculate trip distribution. With these options, the flow of the simulation program is as follows: The program starts to build a tree with the loading node for the first (or next) zone in the loading sequence. While this tree is being built, the zone-to-zone interchange is calculated each time a loading node (i.e., a zone's centroid) is encountered. Consequently, when the entire tree has been built for the zone under consideration, all the zone-to-zone interchanges have been calculated.

[1]An Appendix to this paper, containing a full discussion of the assignment program routines for tree building, capacity restraint, and tree loading, is available from the Highway Research Board for cost of reproduction and handling. Refer to XS-20, Highway Research Record 250.

Once the tree is finished, the program loads the trips which have been distributed on to the proper links in the tree. Because the option to revise travel time has been selected, the program checks to see whether or not it is necessary to change the travel time. The program continues to build trees for the remaining zones until all zones have been processed. It then proceeds to its output section.

Figures 2, 3, and 4 are examples of computer printouts of the traffic assignment program. Figure 2 shows the information which is given for each link in the network being tested. The information includes:

Volume (number of vehicles)	Average travel time per vehicle
Vehicle-miles of design capacity	Total vehicle hours
Vehicle-miles of assigned travel	Total travel time cost
Volume/capacity ratio	Total vehicle operating cost
Average speed	Total accident cost
Length	Total costs

In Figure 3, the one-way volumes and the final restrained travel time in 12-sec units are given for each link. Figure 4 shows the number of trips sent and received by each zone, the loading node number, and the zone sequence number.

In addition to these tabulations, there are others which can be produced for analysis purposes.

PROJECT NUMBER 28P48 MAR 26 1968 PAGE 17

A NODE NUMBER	CAPACITY CLASS	B NODE NUMBER	LINK TYPE	VMDC IN 1000S	RING	SECTOR	ZONE	DIRECTION	ASSIGNED VOLUME	AVG. SPEED	VMAV	LINK LG-	VMA- V/V- HPC	TOTAL TIME	OPERATING COST	ACCIDENT COST	TOTAL VEHICLE HOURS	TIME COST	TOTAL COST	ASSIGN. NO.	CARD NO.
1413	02	1472	2	0	1	3	014	5	1,048	00	0000	0	1	122	0	0	128	22	$22	48	71
1414	02	1415	2	0	1	3	014	5	1,713	00	0000	0	3	44	0	0	75	13	$13	48	71
1415	03	1416	1	0	1	4	014	5	9,070	00	0000	0	9	53	0	0	481	83	$83	48	71
1416	03	1417	1	0	1	4	014	5	9,070	00	0000	0	9	53	0	0	481	83	$83	48	71
1417	03	1418	1	0	1	3	014	5	9,949	00	0000	0	9	46	0	0	458	79	$79	48	71
1418	02	1419	1	0	2	3	014	5	7,865	00	0000	0	11	51	0	0	401	69	$69	48	71
1418	02	1421	2	0	2	3	014	5	2,461	00	0000	0	3	85	0	0	209	36	$36	48	71
1419	02	1420	1	0	2	3	014	5	7,865	00	0000	0	11	58	0	0	456	79	$79	48	71
1420	02	1385	1	0	2	3	014	5	7,865	00	0000	0	11	51	0	0	401	69	$69	48	71
1421	02	1422	2	0	2	3	014	5	2,461	00	0000	0	3	85	0	0	209	36	$36	48	71
1422	02	4060	2	0	2	3	014	5	2,461	00	0000	0	3	28	0	0	69	12	$12	48	71
1423	02	1424	2	0	2	3	014	5	2,758	00	0000	0	5	48	0	0	132	23	$23	48	71
1424	02	1822	2	0	2	3	014	5	2,758	00	0000	0	5	65	0	0	179	31	$31	48	71
1425	03	1394	2	0	1	1	014	5	3,148	00	0000	0	3	38	0	0	120	21	$21	48	71
1425	03	1440	2	0	1	1	014	5	1,703	00	0000	0	2	88	0	0	150	26	$26	48	71
1426	03	1396	1	0	1	2	014	5	5,065	00	0000	0	5	22	0	0	111	19	$19	48	71
1426	03	1427	1	0	1	2	014	5	2,760	00	0000	0	3	57	0	0	157	27	$27	48	71
1427	03	1428	1	0	1	2	014	5	2,760	00	0000	0	3	101	0	0	279	48	$48	48	71
1428	03	1429	1	0	1	2	014	5	8,262	00	0000	0	10	68	0	0	562	97	$97	48	71
1429	03	1430	1	0	1	2	014	5	9,058	00	0000	0	11	64	0	0	580	100	$100	48	71
1430	03	1431	1	0	1	2	014	5	10,182	00	0000	0	13	116	0	0	1,181	204	$204	48	71
1431	03	1432	1	0	1	3	014	5	9,411	00	0000	0	12	133	0	0	1,252	217	$217	48	71
1431	02	1459	2	0	1	3	014	5	1,232	00	0000	0	2	112	0	0	138	24	$24	48	71
1432	03	1433	1	0	2	3	014	5	9,905	00	0000	0	12	75	0	0	743	129	$129	48	71
1433	02	1434	2	0	2	3	014	5	1,400	00	0000	0	3	42	0	0	59	10	$10	48	71
1433	07	1461	1	0	2	3	014	5	14,360	00	0000	0	4	71	0	0	1,020	176	$176	48	71
1433	07	4060	1	0	2	3	014	5	16,353	00	0000	0	5	71	0	0	1,161	201	$201	48	71
1434	02	1435	2	0	2	3	014	5	1,400	00	0000	0	3	82	0	0	115	20	$20	48	71
1435	02	1423	2	0	2	3	014	5	2,355	00	0000	0	4	25	0	0	59	10	$10	48	71
1436	02	1437	2	0	1	1	014	5	863	00	0000	0	1	28	0	0	24	4	$4	48	71
1437	02	1438	2	0	1	1	014	5	2,830	00	0000	0	4	12	0	0	34	6	$6	48	71
1437	03	2205	2	0	1	1	014	5	2,651	00	0000	0	3	45	0	0	119	21	$21	48	71
1438	03	1439	2	0	1	1	014	5	2,462	00	0000	0	3	32	0	0	79	14	$14	48	71
1438	02	1445	2	0	1	1	014	5	5,049	00	0000	0	6	38	0	0	192	33	$33	48	71
1439	03	1440	2	0	1	1	014	5	2,307	00	0000	0	3	22	0	0	51	9	$9	48	71
1439	02	1447	2	0	1	1	014	5	155	00	0000	0	0	32	0	0	5	1	$1	48	71
1440	03	1441	2	0	1	1	014	5	1,776	00	0000	0	2	88	0	0	156	27	$27	48	71
1440	03	1446	2	0	1	1	014	5	83	00	0000	0	0	38	0	0	3	1	$1	48	71
1441	02	1442	2	0	1	1	014	5	1,776	00	0000	0	2	83	0	0	147	25	$25	48	71
1442	03	1426	1	0	1	2	014	5	4,749	00	0000	0	5	40	0	0	190	33	$33	48	71
1443	03	1442	1	0	1	2	014	5	2,746	00	0000	0	3	57	0	0	157	27	$27	48	71
1444	03	1443	1	0	2	2	014	5	2,746	00	0000	0	3	127	0	0	349	60	$60	48	71
1445	03	2206	2	0	1	1	014	5	5,049	00	0000	0	5	38	0	0	192	33	$33	48	71
1446	03	1448	2	0	1	1	014	5	83	00	0000	0	0	72	0	0	6	1	$1	48	71
1447	02	1448	2	0	1	1	014	5	155	00	0000	0	0	32	0	0	5	1	$1	48	71
1448	03	1449	2	0	2	1	014	5	166	00	0000	0	0	105	0	0	17	3	$3	48	71
1449	03	2207	1	0	2	1	014	5	166	00	0000	0	0	29	0	0	5	1	$1	48	71
1450	03	1451	1	0	2	2	014	5	12,336	00	0000	0	15	89	0	0	1,098	190	$190	48	71
1451	03	1444	1	0	2	2	014	5	7,256	00	0000	0	9	58	0	0	421	73	$73	48	71
1451	03	1452	2	0	2	2	014	5	7,254	00	0000	0	9	78	0	0	566	98	$98	48	71
1452	03	1453	2	0	2	2	014	5	7,254	00	0000	0	9	88	0	0	638	110	$110	48	71
1453	03	1455	2	0	2	2	014	5	7,326	00	0000	0	9	32	0	0	234	40	$40	48	71

Figure 2. Link information.

A NODE	B NODE	SCALE TIME	END INDICATOR	VOLUME	A NODE	B NODE	SCALE TIME	END INDICATOR	VOLUME	PAGE 12
3401	3300	1	425	14402	3401	3404	1	425	2157	
3401	3403	1	427	3386	3403	3104	3	427	3386	
3403	3401	1	428	3473	3404	3401	1	428	1964	
3404	3400	4	431	2157	3502	3105	3	431	1614	
3502	3500	2	432	2102	3801	3800	2	432	8621	
3801	3500	9	432	11349	3801	2900	2	432	5532	
3801	3802	1	433	1375	3802	3801	1	433	1783	
3802	3700	3	434	1375	3803	2800	3	434	8826	
3803	3800	2	435	9021	4101	3400	4	435	371	
4101	4100	2	436	2526	4101	4301	2	436	2008	
4102	3500	2	436	3213	4102	3900	9	436	935	
4102	4100	3	438	3278	4104	4100	1	438	328	
4104	14102	4	440	223	4301	3400	3	440	1294	
4301	4300	2	440	2703	4301	4101	2	440	2293	
4301	12800	5	441	963	4701	4300	6	441	232	
4701	4700	2	442	1442	4701	14100	2	442	650	
5001	1401	7	442	429	5001	5000	3	442	585	
5001	4800	2	443	2991	5201	6501	3	443	11018	
5201	7000	4	443	14009	5201	5200	4	443	1727	
5201	6802	3	445	458	5801	6400	2	445	1215	
5801	16801	4	446	737	6201	6100	3	446	593	
6201	6200	2	447	602	6301	5900	1	447	1761	
6301	6800	2	447	967	6301	6302	1	447	2100	
6301	6304	1	448	1544	6302	6301	1	448	1458	
6302	6303	2	449	2100	6303	6302	2	449	1458	
6303	6800	4	450	2100	6304	6301	1	450	1021	
6304	6300	3	451	1544	6501	14303	10	451	7425	
6501	5201	3	451	14106	6501	5800	6	451	1597	
6501	6502	2	451	1195	6501	6504	3	451	1312	
6501	6600	4	452	4243	6502	6500	3	452	551	
6502	6501	2	452	691	6502	5201	3	452	467	
6502	6700	3	454	1102	6504	6501	3	454	1465	
6504	5100	2	455	1312	6701	6700	2	455	1550	
6701	6702	1	456	3861	6701	6800	2	456	1134	
6702	6701	1	459	2684	6702	7000	3	459	3861	
7202	13300	4	460	5176	7202	12502	6	460	4708	
7401	7300	3	464	674	7401	7400	3	464	914	
7601	7600	2	465	1974	7601	13502	8	465	2568	
7608	7609	2	466	11582	7608	7707	2	466	7516	
7609	7608	2	466	7516	7609	18511	4	466	9931	
7609	7600	2	467	2168	7701	7703	127	467	0	
7701	17804	3	468	4655	7701	17810	1	468	2611	
7702	7700	2	468	9026	7702	7703	1	468	1733	
7702	17805	127	469	0	7703	7700	127	469	0	
7703	7701	1	470	7266	7703	7704	2	470	3410	
7704	7707	1	472	11914	7705	7803	1	472	9304	
7706	7705	1	473	7419	7706	17903	1	473	429	
7707	7608	2	474	11582	7707	7706	1	474	7848	
7801	7704	4	475	8504	7801	7903	1	475	2308	
7802	17402	2	476	8628	7803	7802	1	476	8628	
7803	17811	1	477	676	7900	7901	1	477	1363	

Figure 3. One-way link volumes.

PROJECT NUMBER 28P48 MAR 26 1968 PAGE 1

TRIPS-SENT	TRIPS-RECEIVED	VEHICLES-VIN	EXP-VEHICLE-EQ	NODE-NUM	LOAD-NODE	ASSG-ZONE-SEQ	9=EXT RING SEC	ZONE	ASSG	CAR
2708	2455	22172			1319	1	0	1	48	774
1670	1511	10969			1351	2	0	41	48	774
2606	2436	25156			966	3	0	66	48	774
521	569	3837			1793	4	0	124	48	774
1266	1235	7566			[illegible]	5	[illegible]	[illegible]	[illegible]	[illegible]
1479	1808	7376			1873	6	0	220	48	774
322	214	1824			278	7	0	278	48	774
1966	1006	23071			436	8	0	436	48	774
10104	317	41			486	9	9	486	48	774
1536	1325	10596			298	10	0	298	48	774
23461	997	176			501	11	9	501	48	774
3748	294	32			462	12	9	462	48	774
5050	3432	65595			394	13	0	394	48	774
753	705	4286			1844	14	0	266	48	774
451	475	1662			326	15	0	326	48	774
1414	1179	12389			227	16	0	227	48	774
2186	1928	28293			1881	17	0	200	48	774
10	2	0			1640	18	0	149	48	774
645	650	3598			1598	19	0	55	48	774
95	50	203			3452	20	0	333	48	774
1143	1146	11248			1551	21	0	82	48	774
2538	2172	29331			206	22	0	206	48	774
183	63	1356			272	23	0	272	48	774
568	214	5697			417	24	0	417	48	774
1246	1354	6693			382	25	0	382	48	774
681	124	16			3902	26	9	461	48	774
2493	199	24			3912	27	9	481	48	774
35073	2596	506			499	28	9	499	48	774
6774	348	40			487	29	9	487	48	774
111	94	268			371	30	0	371	48	774
301	213	1736			213	31	0	213	48	774
1029	909	11401			1819	32	0	194	48	774
188	51	792			100	33	0	100	48	774
2550	2626	19923			1578	34	0	51	48	774
312	233	1827			3337	35	0	353	48	774
758	690	4790			1566	36	0	34	48	774
1395	685	29508			3827	37	0	408	48	774
1321	334	60			3814	38	9	440	48	774
99	1	0			497	39	9	497	48	774
341	130	2161			3301	40	0	393	48	774
20747	1344	251			3901	41	9	482	48	774
1652	1782	14070			1455	42	0	67	48	774
146	50	545			910	43	0	10	48	774
390	281	2024			1518	44	0	15	48	774
2451	2571	14716			3412	45	0	327	48	774
180	81	568			1631	46	0	137	48	774
2668	2538	27706			170	47	0	170	48	774
5243	326	39			468	48	9	468	48	774
5285	326	41			495	49	9	495	48	774
511	491	2584			299	50	0	299	48	774
792	112	19			3872	51	9	452	48	774
2338	1095	31963			3896	52	0	437	48	774

Figure 4. Trips sent and received by each zone.

INTRA 28124 INTERNAL 595492 EXT 34547

MINUTES	FREQUENCY	NON-ZERO-INT
0.	46754.	2238.
3.	133188.	13631.
6.	127905.	15497.
9.	106423.	13227.
12.	68290.	11162.
15.	45414.	10260.
18.	29452.	8693.
21.	20301.	7031.
24.	13121.	5380.
27.	8071.	3448.
30.	5343.	2218.
33.	4034.	1623.
36.	4259.	1282.
39.	3533.	935.
42.	3575.	846.
45.	3152.	721.
48.	2483.	558.
51.	1613.	326.
54.	1305.	276.
57.	889.	164.
60.	605.	110.
63.	244.	45.
66.	65.	20.
69.	7.	6.
72.	9.	8.
75.	3.	3.
78.	1.	1.
81.	0.	0.
84.	0.	0.
87.	0.	0.
90.	0.	0.
93.	0.	0.
96.	0.	0.
99.	0.	0.
102.	0.	0.
105.	0.	0.
108.	0.	0.
111.	0.	0.
114.	0.	0.
117.	0.	0.
120.	0.	0.

ELAPSED RUN TIME 6815
ELAPSED PROCESS TIME 6326
ELAPSED I/O CHANNEL TIME 379
TIME AT END OF JOB 20 7 MAR 29, 1968

Figure 5. Travel time array.

SELECTED LINKS 34640 SELECTED LINKS PAGE NO. 333 4/08/68

DEST ZONE	DEST ZN LD NODE	ORIG ZONE	ORIG ZN LD NODE	ANODE	BNODE	INT-CHGS	PATH VAL
100	10000	187	18700	10300	10301	2	71
163	16300	187	18700	10300	10301	4	111
114	11400	187	18700	10300	10301	18	79
109	10900	187	18700	10300	10301	8	82
108	10800	187	18700	10300	10301	10	89
107	10700	187	18700	10300	10301	9	85
104	10400	188	18800	10300	10301	8	97
114	11400	188	18800	10300	10301	8	105
100	10000	188	18800	10300	10301	2	97
146	14600	203	20300	10300	10301	1	249
49	4900	203	20300	10300	10301	5	215
47	4700	203	20300	10300	10301	2	211
106	10600	203	20300	10300	10301	4	207
104	10400	203	20300	10300	10301	8	201
145	14500	203	20300	10300	10301	1	243
24	2400	203	20300	10300	10301	2	241
52	5200	203	20300	10300	10301	2	235
89	8900	203	20300	10300	10301	1	251
23	2300	203	20300	10300	10301	2	235
100	10000	203	20300	10300	10301	1	201
53	5300	203	20300	10300	10301	2	231
144	14402	203	20300	10300	10301	2	288
22	2200	203	20300	10300	10301	3	233
50	5000	203	20300	10300	10301	3	225
40	4000	203	20300	10300	10301	3	241
8	800	203	20300	10300	10301	2	230
153	15300	203	20300	10300	10301	1	255
94	9400	203	20300	10300	10301	6	245
160	16000	203	20300	10300	10301	1	237
95	9500	203	20300	10300	10301	3	243
100	10000	207	20700	10300	10301	1	186
145	14500	208	20800	10300	10301	3	151
100	10000	208	20800	10300	10301	1	107
107	10700	208	20800	10300	10301	6	123
162	16200	208	20800	10300	10301	4	148
33	3300	208	20800	10300	10301	9	145
144	14402	208	20800	10300	10301	7	196
160	16000	208	20800	10300	10301	4	148
49	4900	208	20800	10300	10301	9	123
47	4700	208	20800	10300	10301	4	119
106	10600	208	20800	10300	10301	3	114
104	10400	208	20800	10300	10301	14	107
24	2400	208	20800	10300	10301	4	150
52	5200	208	20800	10300	10301	3	143
23	2300	208	20800	10300	10301	5	144
99	9900	208	20800	10300	10301	10	156
40	4000	208	20800	10300	10301	6	149
114	11400	208	20800	10300	10301	12	116
46	4600	208	20800	10300	10301	3	123
109	10900	208	20800	10300	10301	5	118
53	5300	208	20800	10300	10301	3	139
108	10800	208	20800	10300	10301	6	125
22	2200	208	20800	10300	10301	5	141
50	5000	208	20800	10300	10301	7	133
87	8700	208	20800	10300	10301	3	164
39	3900	208	20800	10300	10301	3	154

Figure 6. Selected link travel times and interchanges.

1. Travel Time Array—A tabulation is produced in which trips are grouped by travel time. Figure 5 gives the frequency of trip length in time units of 3-min increments and the number of interchange records from which these trips were accumulated.

2. Selected Links—It is possible to select links and to obtain the tabulation (Fig. 6) which lists the number of interchanges and the travel time from origin to destination zone.

3. Zone-to-Zone Trips—Figure 7 contains the zone-to-zone trip interchanges for each pair of zones. Computer-printed column headings are not shown; there are three records on each line, and each record is arrayed within six columns. The data given are: number of short trips (column 1), number of long residential trips (column 2), number of long-non-residential trips (column 3), destination zone (column 4), origin zone (column 5), and travel time in twelve second units (column 6).

The comparison is made by link type on a district basis; ring and overall totals are calculated. For each district, ring, and link time, actual and simulated vehicle-miles of travel are calculated as well as the absolute difference and percentage difference. In addition, selected statistical measures are computed for each link type and for the total. These include mean, standard deviation, standard error of estimate, and coefficient of determination. An example of the output obtained from this program is shown in Figure 8.

ECONOMIC ANALYSIS

The economic analysis program was developed to facilitate the benefit-cost analysis of the test network. An example of the output is shown in Figure 9. For each zone, district, or ring, the following items are summarized:

4	0	5	175	4	306	2	0	10	176	4	294	5	0	12	177	4	294	435
0	0	1	178	4	467	7	0	11	179	4	467	2	0	2	180	4	398	436
4	0	2	181	4	254	1	0	1	182	4	254	2	0	4	183	4	251	437
1	0	2	184	4	250	0	0	1	185	4	291	0	0	0	186	4	297	438
6	0	9	187	4	305	16	0	12	188	4	343	2	0	7	189	4	429	439
0	0	1	190	4	387	1	0	0	191	4	250	3	0	4	192	4	341	440
4	0	2	193	4	305	1	0	1	194	4	337	1	0	1	195	4	429	441
0	0	0	196	4	419	0	0	0	197	4	447	1	0	1	198	4	457	442
0	0	0	199	4	471	1	0	1	200	4	515	1	0	2	201	4	375	443
1	0	2	202	4	369	1	0	0	203	4	336	0	0	0	204	4	336	444
0	0	0	205	4	411	0	0	0	206	4	328	2	0	0	207	4	303	445
15	0	18	208	4	311	2	0	1	209	4	371	2	0	2	210	4	297	446
4	0	3	211	4	287	1	0	0	212	4	283	1	0	0	213	4	271	447
8	0	7	214	4	280	1	0	4	215	4	286	1	0	2	216	4	297	448
2	0	1	217	4	303	0	0	0	218	4	376	1	0	1	219	4	364	449
1	0	1	220	4	370	1	0	3	221	4	485	1	0	1	222	4	473	450
2	0	2	223	4	372	2	0	1	224	4	378	4	0	2	225	4	378	451
2	0	1	226	4	496	1	0	2	227	4	355	1	0	1	228	4	413	452
8	0	8	229	4	238	57	0	68	230	4	277	44	0	23	231	4	245	453
6	0	5	232	4	366	5	0	4	233	4	390	1	0	1	234	4	414	454
1	0	1	235	4	423	1	0	1	236	4	560	2	0	1	237	4	469	455
1	0	1	238	4	498	0	0	0	239	4	495	3	0	3	240	4	493	456
7	1	6	241	4	490	1	0	2	242	4	492	9	0	10	243	4	439	457
7	0	11	244	4	443	1	0	1	245	4	466	1	0	2	246	4	494	458
2	0	4	247	4	479	1	0	1	248	4	506	0	0	0	249	4	493	459
4	0	5	250	4	604	2	0	4	251	4	513	0	0	0	252	4	533	460
0	0	0	253	4	571	0	0	0	254	4	472	12	1	20	255	4	419	461
12	0	23	256	4	418	0	0	1	257	4	472	1	0	5	258	4	466	462
1	0	1	259	4	420	1	0	2	260	4	490	0	0	4	261	4	553	463
1	0	4	262	4	544	0	0	1	263	4	535	1	0	1	264	4	634	464
1	0	1	265	4	602	1	0	0	266	4	403	1	0	3	267	4	463	465
0	0	1	268	4	413	0	0	0	269	4	473	1	0	2	270	4	449	466
2	0	1	271	4	443	6	0	3	272	4	399	0	0	1	273	4	559	467
1	0	3	274	4	532	2	0	2	275	4	576	1	0	4	276	4	559	468
3	0	6	277	4	415	3	0	2	278	4	560	0	0	0	279	4	509	469
11	0	11	280	4	527	2	0	1	281	4	602	3	0	2	282	4	605	470
4	0	8	283	4	629	6	0	4	284	4	539	2	0	4	285	4	559	471
1	0	0	286	4	614	1	0	2	287	4	467	2	0	0	288	4	336	472
2	0	2	289	4	266	6	0	1	290	4	274	17	0	20	291	4	417	473
8	0	6	292	4	336	1	0	2	293	4	501	5	0	4	294	4	497	474
2	0	2	295	4	494	0	0	0	296	4	602	0	0	0	297	4	693	475
0	0	0	298	4	649	1	0	2	299	4	617	8	0	6	300	4	614	476
1	0	1	301	4	623	15	0	15	302	4	384	6	0	8	303	4	376	477
4	0	5	304	4	378	3	0	4	305	4	470	7	0	7	306	4	581	478
13	0	5	307	4	635	3	0	2	308	4	531	2	0	2	309	4	485	479
5	0	8	310	4	495	19	0	8	311	4	520	1	0	4	312	4	598	480
1	0	2	313	4	585	1	0	0	314	4	612	0	0	1	315	4	600	481
3	0	5	316	4	548	0	0	0	317	4	423	4	0	6	318	4	411	482
9	0	9	319	4	422	4	0	4	320	4	592	4	0	2	321	4	699	483
1	0	2	322	4	725	2	0	2	323	4	505	2	0	2	324	4	498	484
4	0	4	325	4	634	4	0	4	326	4	674	3	0	4	327	4	616	485
5	0	6	328	4	772	1	0	1	329	4	769	4	0	4	330	4	638	486
3	0	4	331	4	836	4	0	4	332	4	739	2	0	2	333	4	642	487
3	0	3	334	4	799	7	0	9	335	4	839	4	0	4	336	4	644	488
3	0	3	337	4	734	2	0	2	338	4	904	2	0	2	339	4	973	489
1	0	2	340	4	615	2	0	2	341	4	813	5	0	6	342	4	683	490
3	0	3	343	4	778	3	0	3	344	4	679	1	0	2	345	4	703	491
2	0	2	346	4	923	1	0	1	347	4	1041	2	0	2	348	4	943	492
2	0	2	349	4	823	1	0	2	350	4	643	10	0	12	351	4	719	493
4	0	4	352	4	818	2	0	3	353	4	919	8	0	9	354	4	818	494
3	0	4	355	4	967	2	0	2	356	4	1027	10	0	11	357	4	1001	495
5	0	6	358	4	872	3	0	8	359	4	757	7	0	9	360	4	737	496

Figure 7. Zone-to-zone trip interchanges.

Number of trips on links	Average daily speed
Link mileage	Vehicle operating costs
Link travel time (in hours)	Accident costs
Vehicle-miles of design capacity	Travel times costs
Vehicle-miles of assigned travel	Total costs
Vehicle hours of travel	

The program is designed also to summarize these data in any combination of the possible nine link types by ring, district, and zone.

PLOTTING

The plotting program prepares a tape for use by a mechanical plotter so that the results of the traffic simulation may be graphically displayed. These mechanical plots are used directly for flow analysis and also for display and reproduction. An example of a mechanical plot of traffic volumes (considerably reduced) is shown in Figure 10.

The plotting program can produce plotting tapes of the following simulation data: (a) volume of traffic assigned to a link; (b) absolute difference between link volume and capacity; (c) links on which volume exceeds capacity; and (d) links on which capacity exceeds volume.

In addition to the band representing volume and capacity data, it is possible to plot straight lines representing the network as described for the computer. This is quite useful when it is desirable to check visually a coded network for completeness.

UPSTATE NEW YORK TRANSPORTATION STUDIES 3-16-68

COMPARISON OF ASSIGNED VMT TO ACTUAL VMT -- BY DISTRICT AND LINK TYPE RUN 4

DISTRICT	LINK TYPE A				LINK TYPE B				LINK TYPE C				T O T A L			
	ACTUAL	ESTIM	DIFF	PCT	ACTUAL	ESTIM	DIFF	PCT	ACTUAL	ESTIM	DIFF	PCT	ACTUAL	ESTIM	DIFF	PCT
TOTAL	20337.	19883.	454.	2.2	11686.	11091.	595.	5.1	789n.	6603.	1287.	16.3	39913.	37577.	2336.	5.9
MEAN	616.3	602.5	92.0	17.76	354.1	336.1	81.3	25.21	239.1	200.1	69.5	17.58	1209.5	1138.7	168.6	15.48
STD DV	445.4	422.8			288.1	245.9			424.0	393.3			984.0	824.6		
STD ERR		130.40				110.56				138.47				264.46		
COF DET		0.9033				0.8527				0.8933				0.9278		

EXPLANATORY NOTES

IN ABOVE TABLE, TOTAL DIFF IS AN ALGEBRAIC SUM, TOTAL PCT IS TOTAL DIFF AS PERCENT OF ACTUAL.

MEAN DIFF AND PCT ARE AVERAGES OF ABSOLUTE VALUES. OTHER STATISTICS WERE CALCULATED USING

STANDARD FORMULAE OF DESCRIPTIVE STATISTICS. ALL VMT FIGURES ARE PRESENTED IN HUNDREDS.

CHECK OF ABOVE RESULTS, ACTUAL AND ESTIM COLUMNS --

TOTAL	20337.	19883.	11686.	11091.	7890.	6603.	39913.	37577.
MEAN	616.27	602.52	354.12	336.09	239.09	200.09	1209.48	1138.70
STD DEV	445.37	422.76	288.06	245.90	424.00	393.34	983.95	824.55

LINK TYPE IDENTIFICATION FOR THIS RUN

A = 1
B = 2
C = 3+4+5+6

2275 BENEFIT COST RECORDS 33 DISTRICTS SUMMARIZED HIGHEST DISTRICT CONSIDERED 48

Figure 8. Vehicle-miles of comparison program.

NETWORK TCDTS/41C34 TRANSPORTATION STUDY 3-29-68

TRAFFIC ASSIGNMENT ANALYSIS -- ZONE SUMMARY OF BENEFIT COST OUTPUT RUN 34

ZONE	TRIPS	LINK MILES	LINK TIME	VMC	VMT	VHT	SPD	OPTING COST	ACONT COST	TIME COST	TOTAL COST	VMT/VMC	COST PER VMT
1	278342.	7.6	0.33	1395.	587.	1934.3	30.3	1733.	588.	3348.	5669.	0.42	0.10
2	358978.	8.3	0.41	1704.	574.	2897.8	19.8	1679.	906.	5017.	7602.	0.38	0.13
3	7033.	0.6	0.02	104.	20.	81.6	24.5	53.	27.	142.	222.	0.19	0.11
4	138308.	4.5	0.22	682.	509.	2238.8	22.8	1610.	761.	3867.	6238.	0.75	0.12
5	64226.	1.8	0.12	174.	211.	1469.8	14.4	676.	618.	2543.	3837.	1.21	0.18
6	169274.	4.8	0.25	881.	292.	1103.8	26.5	816.	319.	1913.	3048.	0.33	0.10
7	76984.	3.8	0.23	301.	134.	748.9	17.9	398.	307.	1296.	2001.	0.45	0.15
8	200898.	6.9	0.40	682.	292.	1923.7	15.2	887.	671.	3329.	4887.	0.43	0.17
9	79993.	3.3	0.16	266.	110.	648.7	17.0	335.	272.	1123.	1730.	0.41	0.16
10	80652.	1.8	0.10	171.	77.	413.0	18.6	221.	154.	714.	1089.	0.45	0.14
11	157034.	7.0	0.18	1491.	679.	1400.7	48.5	2113.	240.	2424.	4777.	0.46	0.07
12	106941.	3.1	0.20	288.	164.	1504.2	10.9	585.	643.	2603.	3831.	0.57	0.23
13	77956.	3.5	0.19	385.	195.	1000.1	19.5	549.	326.	1731.	2606.	0.51	0.13
14	97376.	4.1	0.16	883.	218.	996.0	21.9	664.	381.	1725.	2770.	0.25	0.13
15	11622.	0.9	0.04	45.	43.	178.3	24.1	115.	60.	309.	484.	0.96	0.11
16	59658.	3.9	0.19	320.	156.	747.9	20.9	436.	282.	1294.	2012.	0.49	0.13
17	97962.	3.7	0.24	405.	241.	1521.4	15.8	747.	615.	2632.	3994.	0.60	0.17
18	71209.	3.0	0.20	247.	103.	682.5	15.1	326.	297.	1140.	1763.	0.42	0.18
19	50252.	2.9	0.15	286.	124.	587.2	21.1	347.	222.	1017.	1586.	0.43	0.13
20	27178.	1.5	0.09	114.	54.	317.4	17.0	164.	133.	550.	847.	0.47	0.16
21	198169.	7.1	0.40	895.	390.	1962.1	19.9	1117.	700.	3397.	5214.	0.44	0.13
22	226717.	4.9	0.33	668.	413.	2952.3	14.0	1297.	1042.	5108.	7447.	0.62	0.18
23	176728.	5.3	0.27	987.	362.	2134.8	17.0	1142.	712.	3694.	5548.	0.37	0.15
24	33844.	0.8	0.05	81.	51.	328.8	15.5	159.	137.	569.	865.	0.63	0.17
25	107384.	2.9	0.19	279.	174.	1137.4	15.3	550.	494.	1968.	3012.	0.62	0.17
26	79790.	3.1	0.17	318.	188.	1017.4	18.5	554.	417.	1760.	2731.	0.59	0.15
27	0.	0.0	0.00	0.	0.	0.0	0.0	0.	0.	0.	0.	0.00	0.00
28	25846.	1.7	0.08	162.	92.	425.7	21.6	264.	138.	736.	1138.	0.57	0.12
29	53369.	0.7	0.09	58.	81.	953.5	8.5	304.	366.	1650.	2320.	1.40	0.29
30	10615.	1.0	0.04	78.	53.	201.3	26.3	138.	61.	349.	548.	0.68	0.10
31	302793.	8.7	0.18	2621.	1257.	2165.9	58.0	4267.	303.	3748.	8318.	0.48	0.07
32	71299.	3.3	0.10	553.	210.	477.1	44.0	671.	105.	824.	1600.	0.38	0.08
33	138984.	5.7	0.26	797.	477.	1652.6	28.9	1455.	522.	2859.	4836.	0.60	0.10
34	20686.	2.8	0.14	152.	127.	597.6	21.3	354.	215.	1034.	1603.	0.84	0.13
35	44318.	3.2	0.15	185.	175.	824.0	21.2	491.	300.	1425.	2216.	0.95	0.13
36	169823.	5.7	0.28	924.	415.	1745.6	23.8	1252.	597.	3019.	4868.	0.45	0.12
37	326425.	13.7	0.48	2914.	755.	3340.4	22.6	2226.	1176.	5761.	9163.	0.26	0.12
38	50499.	2.6	0.11	247.	136.	629.2	21.6	378.	231.	1090.	1699.	0.55	0.12
39	20829.	1.9	0.07	413.	129.	908.0	14.2	425.	390.	1571.	2386.	0.31	0.18
40	16554.	1.4	0.07	122.	44.	197.3	22.3	122.	72.	342.	536.	0.36	0.12
41	7314.	0.8	0.02	62.	59.	137.5	42.9	150.	23.	238.	411.	0.95	0.07
42	36642.	1.8	0.07	219.	126.	505.0	25.0	335.	169.	873.	1377.	0.58	0.11
43	45684.	5.4	0.21	610.	257.	829.9	31.0	679.	227.	1436.	2342.	0.42	0.09
44	52282.	4.1	0.20	529.	205.	964.3	21.3	620.	358.	1667.	2645.	0.39	0.13
45	40429.	3.9	0.17	445.	130.	552.8	23.5	379.	186.	955.	1520.	0.29	0.12
46	37802.	3.9	0.18	255.	213.	934.5	22.8	577.	327.	1616.	2520.	0.84	0.12
47	38853.	1.7	0.04	559.	179.	428.5	41.8	511.	87.	742.	1340.	0.32	0.07
48	128386.	4.7	0.19	837.	335.	1157.4	28.9	943.	366.	2001.	3310.	0.40	0.10
49	330242.	7.5	0.24	2303.	498.	1531.6	32.5	1465.	392.	2649.	4506.	0.22	0.09
50	55740.	2.7	0.16	361.	150.	1042.9	14.4	496.	459.	1803.	2758.	0.42	0.18

Figure 9. Economic analysis program.

Figure 10. Traffic volumes plotted by computer.

CONCLUSION

Apart from the concepts and processes on which this traffic simulation package is based, its major significance is that it brings together in one system all the programs necessary to accomplishing traffic simulation. It is, therefore, relatively simple to use.

Moreover, because the program package has been written in COBOL or ALGOL, it can be easily adapted to the type of computer system which the user may have. Although the Burroughs B-5500, now in use by New York State, limits the size of the network which can be tested, the program has been written so that a much larger network may be tested if there is greater computer capability.

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Program, Inputs and Calibration of the Direct Traffic Estimation Method

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A new approach to traffic volume estimation is being developed by the Tri-State Transportation Commission. The Direct Traffic Estimation Method requires as inputs a coded highway network and a file of trip ends per unit area, all items of data being identified by X-Y coordinates. Output data available include link volumes, turning movements, and minimum cost trees.

The general concepts of the method are reviewed. The inputs and methods of computation are described and the initial calibration results are presented.

•THE Tri-State Transportation Commission is responsible for long-range land use and transportation planning in the 28 county New York Metropolitan Region. While the region contains 18 million residents in its 8, 000 square mile area, over 16.3 million persons live in the continuously urbanized 3, 600 square mile core (the cordon area).

In common with all urban transportation studies, Tri-State has the task of making facility traffic volume forecasts for given land use distributions and transportation networks. Typically, this work takes two distinct forms: (a) mainline volume estimates are made for various land use—transportation plan alternatives; and (b) once the basic network has been adopted, most of the subsequent work is involved with determining the effects of local changes in the network and in the land use patterns.

In this latter project-oriented work, full-scale network-wide traffic assignments are rarely justified, since the effects under study (i. e., changes in traffic volume) are usually limited to that part of the region in which the changes have taken place. Furthermore, in dealing with a region the size of Tri-State, a full-scale assignment is very costly. For this reason, and in order to permit an arbitrary degree of precision in describing the region, a new technique of traffic estimation was developed.

This new technique, Direct Traffic Estimation, permits traffic estimates to be made on any or all links in the network during a single run of the program. It thus eliminates the need to make region-wide zone to zone movements to test the consequences of localized network changes such as the addition or deletion of an interchange. Another reason for the decision to develop this approach to traffic estimation was the desire to eliminate the lumpiness in link volumes that results from placing all trips originating in a zone on the network at one point. Each square mile might be considered to be a "zone" using this approach.

Within limits, the program is indifferent to the size of the region and to the number of links in the network. Areas of special interest may therefore be described in a very fine level of detail, with a consequent improvement in sensitivity.

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METHOD

The method of Direct Traffic Estimation was developed by Morton Schneider during 1965-1966 and has since been implemented as a computer program for the IBM System/360.

The traffic volume at any point in a highway network is derived as a function of the cost of travel and the density of trip-ends in the vicinity of the point of interest. Vehicles on the link of interest are taken to distribute themselves among possible destinations according to the function

$$dV = a \int_0^\infty \frac{1}{K^2} e^{-\left(KC + \frac{a}{K}\right)} dK \, dT \tag{1}$$

in which dT is some increment of trip-ends at a distance (travel cost) of C cents from the point of interest. The constant a is related to the average trip length in the region and is measured in cents^{-1}, and K is the constant of integration. (This formula supersedes the distribution function given in reference 1 and its derivation may be found in reference 2.)

The two-way average daily traffic volume at the point of interest is obtained by integrating the function (Eq. 1) over various domains around the point of interest and then by solving

$$Q = \frac{I_n I_s{}' + I_s I_n{}'}{I_n + I_s}$$

in which the I's are the "domain integrals." The domains themselves are defined as follows. First, the region is divided into two domains by a line through the point of interest such that all points in one domain are most easily reached by going in one direction on the link of interest, while points in the second are most easily reached by going in the other. To put it another way, if in Figure 1 a vehicle is moving north on the link of interest then its destination lies somewhere in domain N. Conversely, if it is moving south it is destined for a point in domain S.

In Eq. 2, I_n and I_s are the results of integrating Eq. 1 over each main domain in turn. The quantities $I_s{}'$ and $I_n{}'$ measure the effect of links which compete with the link of interest for traffic crossing the main domain boundary. As the effect of competing links increases, I'_s and I'_n and consequently Q, becomes smaller. Conversely, if the main domain boundary is some physical barrier, such as a river, crossed only by the link of interest, then the competition effect is zero and I'_s and I'_n assume their maximum values of I_s and I_n, respectively. The traffic volume Q then becomes

$$Q = \frac{2 I_n I_s}{I_n + I_s}$$

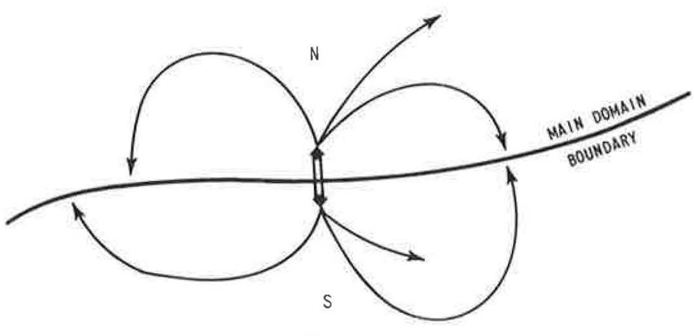

Figure 1.

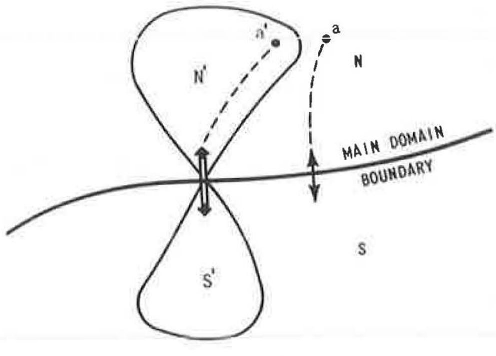

Figure 2.

The domains over which I_n' and I_s' are computed (referred to as prime domains) define the areas in the main domains which contribute traffic uniquely to the link of interest. A point is said to be in the north prime domain if and only if the link of interest lies on the least cost path between the given point and every point in the south main domain. The south prime domain is similarly defined (Fig. 2).

It follows that the prime domains exist only if the link of interest offers some travel cost advantage over competing links which cross the main domain boundary. It is seen from Eq. 2 that unless at least one prime domain exists, the traffic volume on the link of interest is zero.

A rigorous derivation of Eq. 2 is given elsewhere (1). Here, an attempt is made only to show that the traffic volume formula behaves in a reasonable way.

MAIN FEATURES OF THE PROGRAM

The computer program by which the method is implemented uses two basic files of input data. The first of these is a description of the regional highway network and is used to determine a travel costs and domain boundaries associated with each volume estimate. The second file is a description of the trip end density throughout the region, given in terms of a square mile grid, and is used in the computation of the domain integrals.

The program provides the user with two important options:

1. The number of links to be estimated is completely variable and directly determines the running time of the program. In other words, the user pays only for the output information he needs.

2. The area for which network and density data are read by the program may be a rectangle of any size (subject to core storage limitations). This allows the user to maintain input files containing large (unlimited, in fact) amounts of detailed information, but to cause the program to read only those data items pertinent to the problem at hand.

It may be noted here that the distribution function (Eq. 1) behaves in such a way that it is usually unnecessary to consider data much further away than 30 miles from the area in which estimates are being made, and that satisfactory results can often be obtained by limiting the input data to an even smaller radius.

In addition to computing link traffic volumes, the program is able to compute the turning volumes at each end of the link (i. e., the way the link volume splits among the connecting links at each end). An optional output for each specified link is the minimum-cost tree, originating at the mid-point of the link, which is used to determine the travel costs and domain boundaries associated with the link.

The program is currently being run on System/360, models 50 and 65 with 256k and 512k bytes of core storage, respectively. Table 1 summarizes the major statistics for each version of the program.

INPUT DATA FILES

The input data files to the program (DTE) are contained on a single reel of tape, which is prepared off-line from highway network and trip-end inventory files.

For each link in the highway network there is a single record in the network file which includes the X-Y coordinates of each node (link network level code), link cost, a flag to indicate if the link is one-way, and a flag to indicate if turning movements are to be computed for the link. The link coordinates are given to the nearest one hundredth of a mile and may range from 0 to 255.99 miles.

TABLE 3

CORDON AREA RANDOM SAMPLE

Ratio of $\frac{\text{DTE}}{\text{Ground count}}$ to Auto VMT

Function	Level 1	Level 2	Level 3	Total
K = 0.035	0.78	1.26	2.06	1.16
a = 0.027	1.04	1.37	2.33	1.37
a = 0.035	0.79	–	–	–
a = 0.040	0.68	1.07	1.69	0.98
a = 0.045	0.64	0.99	1.48	0.90

The network level code identifies the relative importance of the route of which a link is a part. All major activity centers are connected by Level 1 facilities. The Level 1 system includes all limited access roads. Level 2 highways include other important routes and, as a minimum, bisect the spacing between Level 1 facilities inside cordon area. Level 3 facilities include everything else that was inventoried and are generally limited to the cordon area. Approximately 30 percent of the coded mileage within the region is Level 1, 30 percent Level 2, and the remainder Level 3.

To allow for capacity restraint (as yet not programmed), link costs are broken into two components: initial time cost and fixed cost. The initial time cost reflects the time to travel the over-the-road length of the link at a speed which is a function of the posted speed limit and the number of signals. Time is converted to cost at 2.5 cents/min. Fixed cost is taken to represent accident and operating costs at the rate of 3.5 cents/min plus all link tolls. The limits of the two link cost components are 255 cents for time and 63 cents for the fixed element. Thus, the highway network coding is very similar to the coding for all commonly used assignment programs.

The trip-end density file is based on a mapping of the region into a set of one-mile grid squares. Each entry in the file represents a square area of ¼, 1, 4, or 16 sq mi, and is identified by the X-Y coordinates of the S W corner of the square. The file entry also contains the average trip end density in trips per square mile of the square, and a square size code.

The provision for varying sizes of square, permits the user to choose the square size appropriate to the particular part of the region under consideration. In the Tri-State region, Manhattan is described by ¼-sq mi entries, the remainder of the cordon area by 1-sq mi entries; the band between the cordon and the region boundary by 4-sq mi entries, and the area beyond the region boundary (where necessary) by 16-sq mi entries.

The last file on the input tape is a table of values of the distribution function (Eq. 1) for travel cost values up to $10.24. This table is prepared by a separate FORTRAN program.

METHOD OF COMPUTATION

The computation of the traffic volume on a single link entails summing up (integrating) the contribution to each of the four domain integrals of each trip-end density square in the region. The amount of this contribution is determined by multiplying the trip ends in the square by the value of the distribution function corresponding to the cost of travel between the square and the link of interest.

The first step in this process and one which is done once only for each run of the program, is to associate each density square corner with the (up to four) closest nodes in the highway network. In this way, when the minimum cost tree is built through the highway network, from the link of interest, the minimum travel cost to each density square corner can readily be found.

During the building of the least-cost tree, the main domains are defined. A node is defined (flagged) as being in the north domain if its minimum path enters the link of interest via its north node, and similarly for south domain nodes. At the completion of tree-building, every node in the network belongs to one domain or the other. Certain links have one node in each domain, and must, therefore, straddle the main domain boundary (Fig. 3). The next step is to determine the boundaries of the north and south prime domains.

Each of the boundary-crossing links (other than the link of interest), competes with the link of interest for traffic across the main domain boundary. To determine the extent of this competition, a second tree is built simultaneously from the midpoint of

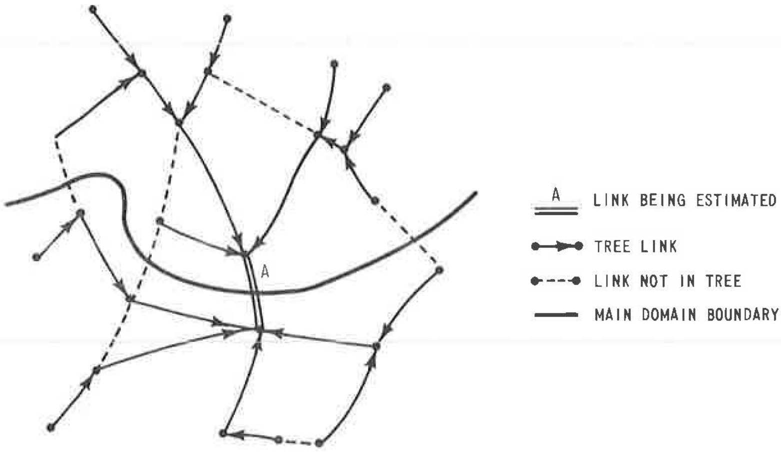

Figure 3.

each boundary link. This has the effect of changing the path values of some nodes in the original tree while leaving others the same. Those nodes whose path values are unchanged during this second tree cycle are, by definition, in the prime domains.

When the domain of each network node has thus been established, the domain integrals are computed. For each trip-end density square the nodes yielding the least travel cost to each corner of the square are found. The trip ends in the square are divided among the four corners, and the contribution of each corner and the integral to which it contributes are then determined.

CALIBRATION AND OPERATION

The initial program version used a trip decay function of the form e^{-KC} instead of the Bessel function shown in the preceding sections. Based on a random sample of 75 links, a value of 0.035 was chosen for K. Link volume estimates were run for all links falling within seven areas, each inside the cordon line and 14 by 20 miles in size. Link volume estimates were compared to ground counts and mapped for review. VMT's were summarized by network level (Table 2).

All of the VMT summaries by network level showed the same pattern. Level 1 was low, Level 2 moderately high, and Level 3 quite high. The results on Levels 2 and 3 are partly explained by the fact that DTE is estimating a component of local street travel which is not represented in the ground counts. Correcting for local streets brings the total line of Table 2 to 0.73, 0.092, 1.07, and 1.10, respectively. Except for the lowness of Level 1, these are agreeable totals.

TABLE 2

Ratio of $\frac{\text{DTE}}{\text{Ground count}}$ Auto VMT, K = 0.035

Area (14 × 20 mi)	Level 1	Level 2	Level 3	Total
New Haven, Conn.	0.85	0.98	1.34	1.01
Norwalk, Conn.	0.85	1.57	2.11	1.27
Oyster Bay, N.Y.	0.78	1.51	2.04	1.11
White Plains, N.Y.	0.71	1.75	2.14	1.08
Newark, N.J.	0.78	1.66	2.35	1.28
New Brunswick, N.J.	0.53	0.89	1.38	0.84
Long Branch, N.J.	0.25	0.84	1.21	0.74
Total	0.73	1.33	1.93	1.10

The areas summarized were also rerun multiplying the time cost component by 1.2 and the fixed cost component (including tolls) by 0.75. This had little overall impact on the results. It was hoped that this would raise Level 1 volumes.

At this stage of development, considerable experimentation was done to test the sensitivity of the system to changes in network extent or service levels. Increasing speeds or the extent of the expressway system increased total VMT, since all trip ends could be reached with less cost. Estimated volumes across screen lines on Staten Island with and without proposed highway facilities showed constant total volumes. Adding the proposed expressways had the impact of placing the bulk of VMT on the expressways and the arterials leading to the interchanges. Throughout this phase of the testing, DTE showed a very logical response to network changes.

The e^{-KC} curve with a K = 0.035 that had been used has the characteristic of virtually eliminating the contribution to the main and prime domain integrals of trip ends more than $1.00 away from the link of interest. Since facilities providing high levels of service were being underestimated and low level facilities were being overestimated, it was concluded that the shape of the decay function needed adjustment. The contribution of nearby trip ends should be reduced while the impact of those further away needed increasing. This led to the selection of a Bessel function to replace e^{-KC}.

Figure 4 shows the shape of the e^{-KC} function for K = 0.035 and Bessel curves for several values of the parameter a (see Eq. 1). It is expected that the final value of a will be in the neighborhood of 0.03 to 0.04. While the curves are shown for a cost range of up to $1.20, the program utilizes the curves up to a cost of $10.24. The trip contribution ratio is multiplied by the trip ends in a square, dT, to yield dV in Eq. 1.

A more inclusive sample of links was chosen for the calibration of the Bessel function. The number of links drawn by network level was made proportional to the cordon area mileage by network level. No links with tolls were included. A comparison of calibration results for various values of a, and K = 0.035, for this sample is given in Table 3—310 links are represented in each run, except for a = 0.035 for which only the sample's 85 Level 1 links were estimated.

The Bessel a = 0.040 had about the same results as K = 0.035 except that the high error of Level 3 was reduced considerably. Changing the function will not by itself solve the problem of increasing the accuracy of link volume estimates.

Each link volume estimate is statistically independent of the estimates on adjacent links. It depends on a reasonable description of network extent and cost, the trip end matrix, and proper definition of main and prime domains. Close examination of the outputs has shown that the relative magnitudes of the domain integrals can shift from link to link along a route. Thus, the sum of I_n and I_s will be fairly constant in a locale, but the split of this total between I_n and I_s can vary. Prime domains can also be quite sensitive to fluctuation.

The sum of I_n and I_s is also a useful measure of accessibility. Such measures have been prepared using trip ends, population, and floor space in the density matrix.

Effort is now concentrated on developing means of stabilizing the program's boundaries. Major differences in the boundaries result from very minor differences in accessing critical nodes during tree building. This must be accomplished before a final value of the a or K

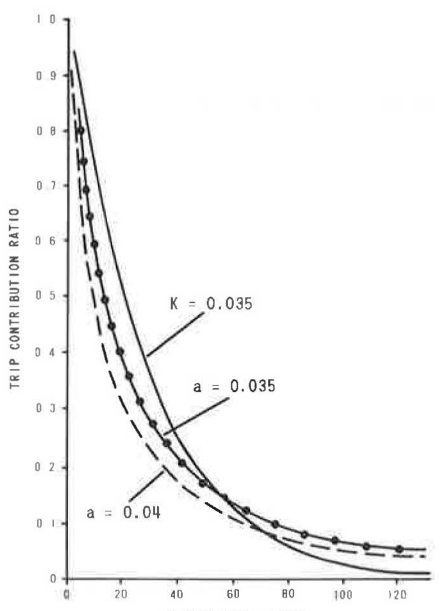

Figure 4.

TABLE 1

PROGRAM STATISTICS

Item	Model 50	Model 65
No. of 2-way links	5000	12500
No. of nodes	4095	8191
No. of density areas	5000	9135
Running time per link (sec)	5	3.3

parameter can be selected. The home interview file is also being processed to derive an empiric trip contribution ratio curve using trips that reported the use of specific East River crossings.

In making traffic volume estimates in response to highway department requests, the procedure that has been developed involves posting of I_n, I_s, I_n', and I_s' for each link. These values are smoothed and made consistent from link to link before mainline volumes are calculated.

DISCUSSION

The DTE program has been made operational and can accommodate very large networks. While the analyst can work with the outputs to make reasonable traffic volume estimates, more work needs to be done to improve the estimating capability and to reduce the amount of output manipulation. This work continues.

The reader will recognize that the principal elements of traffic assignment and distribution are present in DTE. Network description, trip ends by area, minimum path tree building and distribution (or decay) functions are a part of the working tools of any approach. What is new in this work is the combination of distribution and assignment into a single step. The use of the coordinate system provides great flexibility in the Tri-State application of the method, but is not a necessary condition for use elsewhere.

ACKNOWLEDGMENTS

No undertaking of this magnitude is possible without the support and participation of many individuals. This list must start with J. Douglas Carroll, Jr., William L. Mertz, and Edward F. Sullivan who provided many ideas and encouragement. Programming was done by G. J. H Brown, Malvin Ceren and John Bennett. Density inputs were prepared by Harold Deutschman and Stanley Feinsod. The networks were coded under the direction of A. Woehrle and Mark Roskin. Throughout all this permeates the spirit of Morton Schneider.

The preparation of this report was financed in part through Federal funds made available by the U. S. Department of Transportation, Federal Highway Administration, Bureau of Public Roads and the Federal grant from the Urban Renewal Administration of the U. S. Department of Housing and Urban Development, under the planning assistance program authorized by Section 701 of the Housing Act of 1954, as amended and in cooperation with the three states of Connecticut, New Jersey and New York.

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Regression Model for Projecting Vehicle-Miles of Travel in a Metropolitan Region

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A regression model has been developed for the projection of mileage traveled by motor vehicles in a metropolitan region. The model represents such a mileage projection for the Tri-State region, but its framework should be applicable to other regions as well. The model was built on the premise that travel parameters are influenced by the urban form which is primarily characterized by the distribution of population densities. Upon evidence of a functional relationship between population densities and trip generation rates, the vehicular travel mileage was correlated with vehicular trip-end densities. Statistically meaningful correlation between these variables verified the study's premise. Since it has been shown that population distribution patterns remain relatively stable in time, it was reasoned that the patterns of regional travel parameters should likewise remain stable. Consequently, a projective model was developed. It utilizes annual travel mileage per vehicle, which has remained stable over the past decades, and constant average miles per trip to establish the projection control totals.

•THE automobile ceased to be a novelty about three decades ago. With the perfection of mass production techniques, it has become one of the implements which characterize the civilization of the 20th century. At an average in this country, one automobile was registered for 13.1 persons in 1920, 5.4 in 1930, 3.8 in 1950 and 2.9 in 1960. The seating capacity of automobiles has long since exceeded the national population.

The ubiquity of automobiles transformed the concept of accessibility. Streets and roads had to be paved for the accommodation of the ever-increasing number of motor vehicles. Every parcel of land mapped for development must now be provided with accessibility by motor car. Automobile use has spread throughout developed areas as much as other services which are considered indispensable for maintaining the activities of an average household.

Population densities, depending upon their level, seem to embrace different sets of constraints for motor travel. A kind of kinship prevails between the forms of travel and other forms that are inherent in development density.

This study explores the relationships between vehicular trip densities, which derive from population densities, and vehicle-miles of travel. The existing relationships have been defined, and methods for projecting future travel mileage have been developed.

THE URBAN FORM AND TRAVEL

The urban form is characterized primarily by the distribution of population densities. At high development intensities not only are the structures tall, but all other facilities are arranged in a manner which conserves space. The overall spatial frame-

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work of activities is compact and its utilization intensive. As densities decrease, the compactness of activities, such as residential, shopping, entertainment, and recreation gets relaxed, and distances between activity points grow.

Because of this variation in the spatial framework of activities, the regional urban form contains services which vary with development intensities. Travel is one such service, and its forms change, especially in regions where the difference between the highest density and the lowest is great. Travel cost, convenience, safety, the utilization of travel modes, and a host of other considerations influence the forms of communication. Use of automobiles in high densities likewise is confronted by constraints which are absent in low-density areas. If the regional urban form is definable, one may reason that there should also be discernible patterns of regionwide services such as travel.

Travel mileage generated in all parts of the region depends on the number of trips made in these areas; travel mileage is a consequent of trip-making. If a generalization for the travel mileage was sought, the trip-making factor would be a logical choice for an independent variable. The trip-generating propensity stands in close relationship to population density. Since population densities depict the urban form, trip-making and travel mileage generation must show coextensiveness with such a form.

The regional urban form may be described in the most rudimentary manner by defining the structure of population density distribution. A number of studies have been made in this field. Here, however, it will suffice to quote Berry's affirmative statement (1) that "No city has yet been studied for which a statistically significant fit of the expression

$$d_x = d_o e^{-bx} \tag{1}$$

does not obtain."

where

d_x = population density d at distance x from the city center;

d_o = central density, as extrapolated in city's central business district; and

b = the density gradient.

In a more recent study of household and residential patterns for two time periods, twelve years apart, in the Greensboro, N. C. metropolitan region, Swerdloff (2) found that "gross density has consistently increased in each ring. The density gradient has flattened out slightly, indicative of a less compact population distribution in 1960 than in 1948 and characteristic of a suburbanizing region." Otherwise, curves of the same character as defined by Eq. 1 provided statistically significant descriptions of population distribution patterns.

Two aspects of these findings are pertinent to the thesis of this paper: first, that the population distribution pattern in the region is definable; and second, that this pattern remains stable in time. The distribution form does not change, but some of its parameters undergo a gradual adjustment. Consequently, should the nature of coextensiveness between regional services, such as travel, and elements of the urban form be defined, this definition could be employed for projecting the demand of services in the future.

A number of regional studies have found good correlation between population densities and trip generation. Levinson and Wynn (3) summarized the various equations which have been used for estimating trip generation from population densities, or other population characteristics and state (3, p. 50):

> The studies clearly show the reductive effect of density on total trip generation, both between various cities, and within each city. (Trip generation is generally expressed as a - bx, or a - b log x, in which x is a density function.)

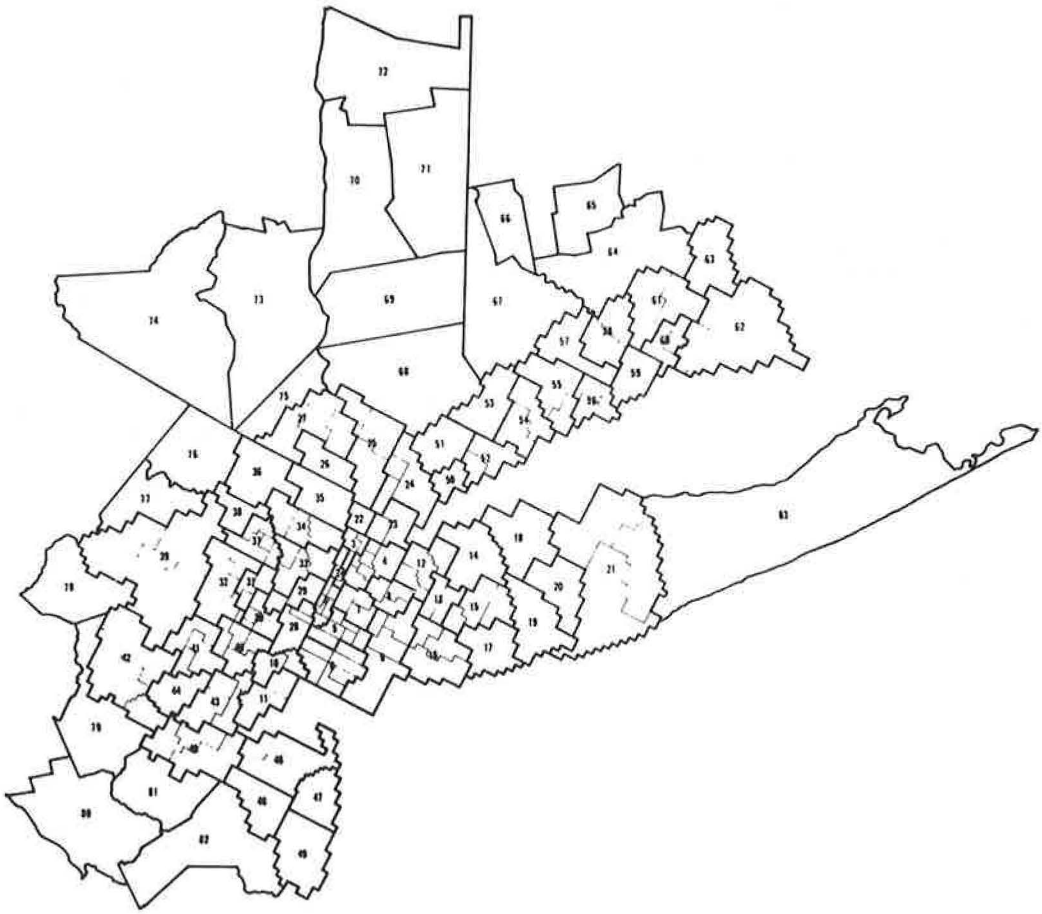

Figure 1. Analysis districts, Tri-State Metropolitan Region.

It may be stated, therefore, that trip density is a function of population density. However, it has been hypothesized that travel mileage stands in close relationship to trip density, and it may be shown that the travel mileage per square mile, m, is a function of trip density, p, which, in turn is a function of population density, δ.

$$m = f\,[p\,(\delta)]$$

This study works with vehicular trip destinations and p represents such densities. As the relationship between δ and p is outside the scope of this paper, only trip-end density will be used as the independent variable.

The hypothesis which underlies the formulation of the regression model stipulates the existence, in a regional context, of a statistically definable relationship between vehicle-miles of travel and trip-end densities. In the stipulated variation,

$$m \propto p \tag{2}$$

stating that vehicle-miles vary in a manner similar to trip densities, there is a transitive property which makes both variables equal. This property is $m/p = \bar{d}$, the average travel mileage per trip. It measures the travel intensities in areas of different levels of development and may be determined after establishing the correlation between m and p.

DISTRIBUTION OF VEHICLE-MILES AND TRIP DENSITY

In order to verify the hypothesis and to provide a statistical basis for model formulation, vehicle-miles of travel in the urbanized part (the cordon area) of the Tri-State metropolitan region (Fig. 1) were correlated to trip-end densities. The scatter diagram and regression line are shown in Figure 2.

For this purpose the cordon area, consisting of 3,509 square miles, was divided into 63 analysis districts—larger in low densities and smaller in high. (The size of analysis districts does not appreciably change the regression constants; larger aggregates, however, tend to decrease the dispersion.) In delineating the district boundaries, consideration was given to the trip distribution patterns. Under the assumption that each trip is associated with some kind of activity, the trip distribution data were looked upon as an indication of activity patterns. Each district contains one or more trip concentrations. Boundaries delineating the districts, as much as was practical from the data aggregation point of view, were located in low-density areas (Fig. 1).

Trip generation data were obtained from the Tri-State home interview survey. The highway inventory provided measurements of vehicle-miles. Both sets of data were aggregated by analysis district. In the regression analysis each district provided one observation item. District average trip density and average vehicle-miles per square mile were the two data series for which correlation was sought.

In Figure 2, the average miles of travel in each district are plotted against corresponding trip densities. These data have been fitted with a regression line, the equation of which is

$$\begin{aligned} m &= 36.09p^{0.8122} \\ R^2 &= 0.86 \end{aligned} \qquad (3)$$

(R^2 was computed from natural values; the regression equation from logarithmic transformations.)

This correlation verifies the hypothesis. Density explains 86 percent of total travel mileage variance. By this measure, there is a significant correlation between the range of trip densities in the region and vehicle-miles of travel at these densities.

According to Eq. 3, at densities of 500 trips per square mile, vehicles produce 5,600 miles of travel per square mile; at a density of 50,000 trips per square mile the corresponding mileage is 236,600; that is, for the trip density change from 500 to 50,000, or a ratio of 1:100, vehicle-miles increase only by the ratio of 1:42. This indicates that the travel mileage for this range of densities grows less than a half as much as trip densities. Evidently the average vehicle-miles per trip end varies with density.

Dividing Eq. 3 by p, the number of trip ends per square mile, an equation is obtained for the distribution of average miles per trip.

$$\bar{d} = 36.09p^{-0.1878} \qquad (4)$$

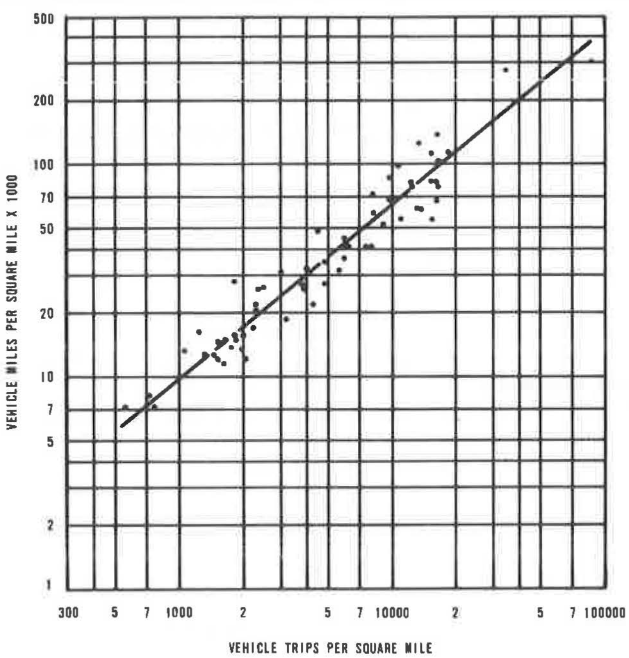

Figure 2. Regional vehicle-mile distribution.

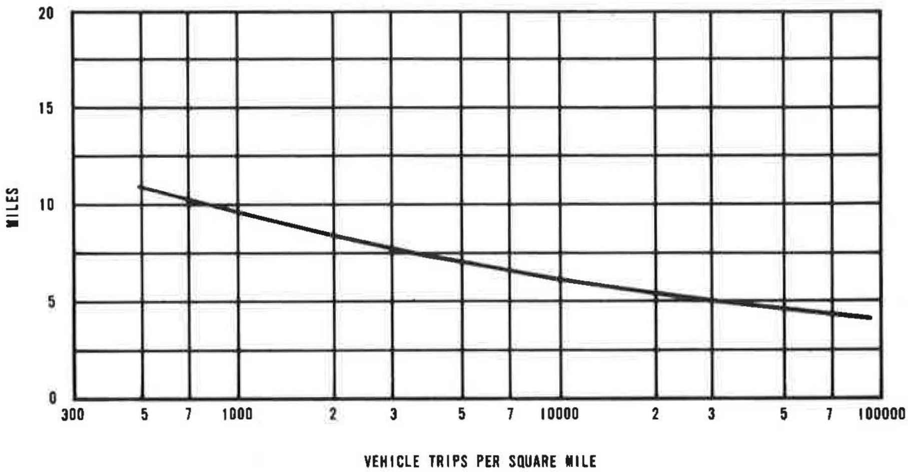

Figure 3. Vehicle-miles per average trip.

The distribution of $\bar{d}$ is shown in Figure 3. The average vehicle-miles per trip varies from about 4 miles in high densities to 11 in low.

A distinction is to be made between the average miles per trip, $\bar{d}$, and a parameter known as the average trip length, frequently denoted as $\bar{r}$. These two parameters represent different averages: $\bar{d}$ for a given area is obtained by dividing all the vehicle-miles traveled within the defined area by the number of trip destinations in this area; $\bar{r}$ is obtained by summing up the total lengths of all trips with one end in the area, regardless of whether this travel is made within the analysis district or without, and dividing this sum by the number of such trip ends in the district. Averaging these two parameters for areas as large as the entire region, $\bar{d}$ and $\bar{r}$ should tend to be equal.

Eqs. 3 and 4 defined the regional patterns of vehicle-mile variation. These equations indicate how the travel mileage intensities change in the metropolitan context with changing densities. Since it has been established that the regional urban form remains relatively stable in time, there is good reason to believe that the patterns of travel intensities will likewise remain stable. The population in different districts may grow in the future, or decline somewhat; but these changes will be reflected in trip densities and, ultimately, in travel mileage generation rates. These equations, therefore, may be utilized in constructing a tool for the prediction of travel mileage by analysis districts.

PROBLEMS ENCOUNTERED IN PROJECTIONS

In employing a regression equation for the projection of events in the future, two questions are likely to arise:

1. Will the equation which has been established on present observations remain valid in the future?
2. How may the regression equation be employed in view of the fact that most of such data observations are either above or below the regression line?

It has been shown that the pattern of travel and trip density relationships is collinear with factors characterizing the urban form. Studies which defined the urban population distribution structure by Eq. 1 and subsequent research (2, 4) demonstrated the stability of this population distribution form over time. These findings strongly suggest that the vehicle mileage distribution pattern as shown by Eq. 3 will prevail for some time to come.

Regional plans in most instances predict future population growth for most districts. Population growth and the anticipated increase in average household income lead to the projection of higher vehicle ownership rates and these to higher vehicular trip densities.

TABLE 1

VEHICLE-MILE PROJECTIONS, TRI-STATE METROPOLITAN AREA

Area	1963 VMT	1963 VMT per Trip	1963 Annual VMT per Vehicle	1985 VMT	1985 VMT per Trip	1985 Annual VMT per Vehicle
Core Counties: Manhattan, Brooklyn, Bronx, Queens, Hudson	33, 795, 644	6.48	8, 455	41, 550, 671	6.61	7, 495
Inner Counties: Essex, Passaic, Union, Nassau, Westchester, Richmond, Bergen	48, 326, 648	6.42	9, 543	77, 607, 994	6.70	10, 213
Outer Counties: Mercer, Middlesex, Suffolk, Morris, Rockland, Dutchess, Orange, Putnam, and all six Connecticut Planning areas within Tri-State Region	61, 580, 804	9.11	14, 506	128, 997, 594	8.75	12, 203
Region	143, 703, 096	7.37	10, 800	248, 156, 259	7.58	10, 465

Ratio 1963 VMT per trip to 1985 estimated from Eq. 3 = 1.05
Ratio 1963 VMT per vehicle to 1985 estimated from Eq. 3 = 1.12

Adjustment factor (mean of the two ratios) = 1.09

VMT = vehicle-miles of travel.

With future vehicular trip densities moving upward, according to Figure 3, the regional average mileage per trip would tend to decline.

The available historical data are inadequate to support or to contradict this indication. There are planners who allow for some trip length growth (5) while others believe that the average mileage per trip will remain virtually constant (6). The latter premise, however, may be substantiated by the average annual vehicle-miles of travel per vehicle.

The national average miles per vehicle has been constant for the last 20 years at about 9,670 miles (7), and it provides the most stable control figure for the estimation of vehicle-miles on a large area basis. Considering the two control figures—constant average trip miles and constant miles per vehicle—it is possible that the constancy in the latter is due to the constancy in the former. This may also be seen in Table 1, showing the estimated results for the Tri-State region.

Regional averages of miles per vehicle and miles per trip were utilized for projecting that part of the travel mileage increase over time which is attributable to causes other than the growth in trip ends. Both parameters employed for establishing control totals are regional averages. The resulting adjustment criterion, therefore, applies uniformly for the entire region. Uniform adjustment of the regional vehicle-mile totals would be attained by applying the adjustment factor either to the regional total vehicle-miles estimated from Eq. 3, or to such totals for each analysis district, or to the regional average vehicle-miles per trip.

Factors required for the development of such a criterion are denoted as follows:

M_o = present total vehicle-miles;

M_f' = future vehicle-miles estimated by Eq. 3;

M_f = final projection of total vehicle-miles;

D_o = present number of trips;

D_f = future number of trips;

N_o = present number of vehicles;

N_f = future number of vehicles.

Since some fluctuation is in evidence from year to year in the average miles per vehicle, and very likely there is also some in the average miles per trip, in developing the control criterion these two parameters may be given equal weight. Consequently, denoting the correction factor as C(t),

$$M_f = C(t) \times M_f' = \frac{M_o}{2}\left(\frac{D_f}{D_o} + \frac{N_f}{N_o}\right)$$

$$C(t) = \frac{M_o}{2M_f'}\left(\frac{D_f}{D_o} + \frac{N_f}{N_o}\right) \tag{5}$$

Eq. 5 represents the correction factor for escalating the initial estimates up to the total control figures.

Before formulating a projective tool, another obstacle resulting from irregularities in the real world remains to be resolved. Some districts generate less vehicle-miles of travel than corresponding regional averages represented by the regression line, others more. The question is whether the initial quantities will move up parallel to the regression line, as the result of increased densities, or in some other way.

Three possibilities suggest themselves for projecting individual district vehicle-mile values.

1. To converge such values on the regression line;
2. Shift vehicle-mile values to projected densities parallel to regression line; and
3. Project between these two extremes.

The regression model was constructed for the third possibility. A "corresponding density" was computed from the regression Eq. 3 for each district's actual vehicle-mile values. The projected density increases were added to the corresponding densities and future travel mileage was estimated for these new densities.

The three projection possibilities are shown in Figure 4. Segment 3′ - 3′ represents the vehicle-mile projection for the future density increase. This projection is carried over to show it in relationship with the other two projection possibilities. The projection 3 somewhat higher than projection 2—parallel to the regression line, and lower than projection 1—converging to the line (Fig. 4).

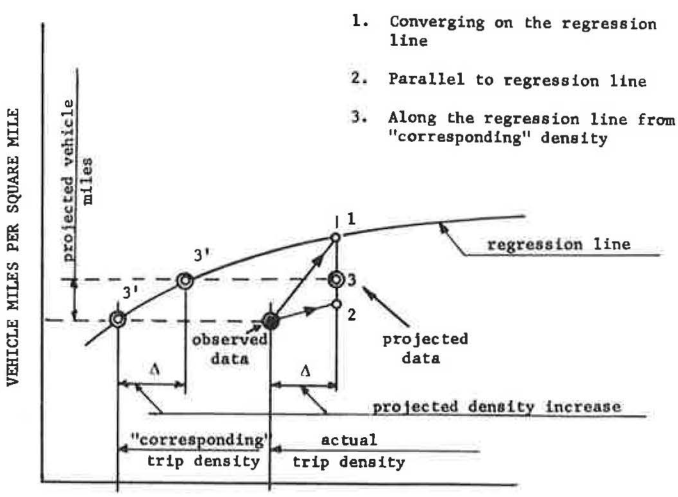

Figure 4. Three projection possibilities.

The projected trip-end density increases are primarily due to population growth. Projection 3 considers the possibility that future intensification of residential activities may tend to bring the vehicle-mile generation rates somewhat closer to the customary intensities that correspond to given densities, regardless of causes underlying the present vehicular mileage data dispersion. About the same regional total figure results whether projecting district data parallel to the regression curve or on the basis of corresponding densities. The latter technique, however, seems to produce a more realistic distribution of mileage among districts.

THE MODEL

The regression model for projecting vehicular travel mileage in a metropolitan region is built upon the regression Eq. 3. It may be rewritten in logarithmic form:

$$\log m = 1.5574 + 0.8122 \log p$$

and from this

$$\log p = 1.2312 \log m - 1.9175$$

or

$$p = \frac{m^{1.2312}}{82.70}$$

It p is computed from this equation for a given district's vehicle-mile observation m_i, the corresponding density may be denoted p_i' and the above equation restated as follows:

$$p_i' = \frac{m_i^{1.2312}}{82.70} \tag{6}$$

Denoting the amount of future increase or decrease in trip densities for the individual analysis district as Δ_i, the future trip density p_{if}', for which the vehicle-miles are to be projected, can be shown as

$$p_{if}' = p_i' + \Delta_i = \frac{m_i^{1.2312}}{82.70} + \Delta_i$$

If p_{if}' is substituted for p in Eq. 3 the projective equation for the individual analysis district becomes

$$a_i m_{if} = a_i \,(0.4364\, m_i^{1.2312} + 36.09\, \Delta_i)^{0.8122} \tag{7}$$

where

m_{if} = the projected vehicle-miles for the given district from Eq. 3, and
a_i = the size of the same area in square miles.

Thus, the projected total vehicle-miles for the entire region is equal to the sum of all districts:

$$M_f' = \sum_{i=1}^{n} a_i m_{if}$$

Incorporating the correction factor shown in Eq. 5 and substituting Eq. 7 for $a_i m_{if}$ evolves the projective model:

$$M_f = \sum_{i=1}^{n} a_i \frac{M_o}{2M_f'} \left(\frac{D_f}{D_o} + \frac{N_f}{N_o} \right) \left(0.4364 m_i^{1.2812} + 36.09\, \Delta_i \right)^{0.8122} \tag{8}$$

Constants shown in Eq. 8 are those which were derived for the Tri-State region. The framework of the model, however, should be valid for other metropolitan regions as well.

PROJECTIONS FOR THE TRI-STATE REGION

The vehicle-miles of travel were projected for the Tri-State metropolitan area from 1963 (the data collection year) to 1985. The projection was carried out at the analysis district level. A summary of the projection results is given in Table 1.

The vehicle-miles are aggregated in three rings: a core, an inner, and an outer ring. The largest travel growth is anticipated to take place in the outer ring—109 percent. The inner ring will increase its travel mileage only moderately—61 percent, and the core only 23 percent. The region's total motor vehicle travel mileage will grow 73 percent.

Table 1 also shows the relative measurements of travel: average vehicle-miles per trip, and annual miles per vehicle. These parameters were computed utilizing trip and vehicle data derived from different criteria. Consequently, Table 1 lists three independent projections of travel parameters. This juxtaposition provides a visual check of the projection results.

The initial vehicle-mile projections obtained from the regression equation have been adjusted upward by 9 percent. The constant vehicle-miles per trip criterion indicated the need for a 5 percent upward adjustment and the constant miles per vehicle, 12 percent. The average upward adjustment, therefore, amounted to 9 percent.

The average vehicle-miles per trip end, while varying between the three rings, remains practically equal for 1963 and 1985 in each ring. The average mileage per vehicle, however, varies. The core indicates a drop in such mileage from 8,455 to 7,495. A similar drop is also in evidence for the outer ring, from 14,506 to 12,203. The projected increase in the automobile ownership rates might account for this change. In the core area, the future automobile ownership might have been projected to increase as a result of higher median household income. Similar projections for the outer ring may have anticipated the expansion of ownership of second and third cars. The total regional vehicle-mile figures maintains reasonably equal average miles per trip and miles per vehicle for 1963 and 1985.

CONCLUSIONS

A statistically significant correlation between vehicular trip-end densities and vehicle-miles of travel on an analysis district basis provides a rationale for the formulation of a regression model by which the future vehicle-miles in such districts may be projected. The regression equation which resulted from this correlation defines the pattern by which the vehicular travel mileage changes with changing trip densities. Stability in annual miles per vehicle and a constant regional average trip mileage provide control totals for assessing the change of travel miles over time resulting in an adjustment factor for the Tri-State region of the order of 9 percent. The application of the regression model in projecting the vehicle-miles traveled in the Tri-State region by 1985 demonstrated the workability of this tool.

Estimates of total vehicle-miles and the defined variation pattern of the vehicular travel mileage provide the basis for further steps in travel analysis.

ACKNOWLEDGMENT

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The Traffic Assignment Process of the Delaware Valley Regional Planning Commission

THABET ZAKARIA, Sr., and JOHN FALCOCCHIO, Sr., Delaware Valley Regional Planning Commission

This paper reports on the traffic assignment process of the Delaware Valley Regional Planning Commission. It describes the concepts of traffic assignment used and it briefly explains the computer process. The DVRPC assignment is an "all or nothing" assignment which utilizes travel cost as the tree trace variable.

Four sensitivity tests were made to evaluate the inputs to the traffic assignment package and the results of each test are reported. Tests 1 and 2 were unrestrained assignments. These tests were conducted to determine if turnpike tolls and accident costs should be included as components of total travel costs. Tests 3 and 4 are restrained assignments which were made to find out the advantage of calculating the 2KD value (peak-hour fraction) for each input to the capacity restraint program on the basis of generalized route and area types.

The district assignment calibration indicates that the district simulated volumes are sufficiently accurate to serve as a guide in establishing the overall design requirements for freeways and high-type arterials.

A zone assignment, using tests 1 and 3, was made for the portion of Mercer County within the cordon area for the purpose of determining the degree of improvement in output that it has over district assignment.

The "real error" concept is introduced as a method which may be used in reporting the accuracy of the traffic assignment outputs. This method seems to be superior to the usual RMS measure used in statistics, since it takes into consideration the capacity of highways to accommodate a wide range of traffic volumes.

•TRAFFIC assignment may be defined as the allocation of trip interchanges on the routes or transportation facilities whether they are highways or public transportation facilities. Because it is concerned with the many variables involved in the travel behavior of people, traffic assignment is the most complex step in the traffic simulation model.

Since transportation facilities have certain physical capacities such as vehicles or passenger per hour or per day, a comparison between the assigned volumes (demand) and the capacity of the existing, or of the existing plus the proposed transportation facilities (supply), would clearly indicate the deficiencies or the surpluses of individual transportation facilities as well as the whole system. The knowledge of traffic supply and demand is one major criterion used to evaluate objectively the adequacy of proposed highway and mass transit facilities. This information is an end result of the traffic assignment program.

The purpose of this paper is to discuss and report the concepts and calibration outputs of the highway traffic assignment program used by the Delaware Valley Regional Planning Commission. The assignment outputs or results, are reported at the district level for the whole DVRPC cordon area and at zonal level for the portion of Mercer County lying within this area. The DVRPC cordon area includes five counties in Pennsylvania and four counties in New Jersey covering an area of about 1,200 sq mi, with

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a 1960 population of 4 million people and over 2,200 mi of highway test network consisting of less than 11 percent (230 mi) of turnpikes, freeways and high-type arterials.

TRAFFIC ASSIGNMENT CONCEPT

The traffic assignment concept used by the Delaware Valley Regional Planning Commission is based on the assumption that people are rational and behave in their travel in a way that minimizes their total travel cost. Total travel cost is defined as the sum of travel cost elements, such as operating and time costs, incurred by persons making trips between any pair of zones or districts. This concept has been adopted by DVRPC because it was found that the concept of least total time tends to overassign traffic to high-speed facilities such as expressways and high-type arterials. For example, consider two districts, A and B, which are linked only by two paths: a freeway and a high-type arterial. The following data are obtained from the trace variables that were used in the calibration of 1960 highway network.

Item		Freeway	Arterial	Difference
Distance between A and B	(mi)	4.2	3.8	0.4
Speed on route	(mph)	54.0	44.0	10.0
Time of travel	(min)	4.6	5.2	0.6
Time and operating cost (travel cost)	(cents)	26.5	24.1	2.4

These data show that the arterial link between A and B is 0. 6 min slower but 2. 4 cents cheaper than the freeway link. On the basis of the minimum travel time criterion and "all-or-nothing" assignment, all trips between A and B will take the freeway link. But all trips will take the arterial link on the basis of the minimum travel cost criterion.

A brief identification of the travel cost components, which were summed up to trace the least total cost paths follows.

1. Time Cost—The time value is the major cost component in traffic assignment. Although it is recognized that the time value is dependent on many variables—such as the time of travel, income, type of transportation facilities, purpose of trips, and age composition of travelers—DVRPC adopted \$1. 50 per vehicle hour and \$1. 00 per person hour. These values seem to be reasonable as average time values for all trips in the DVRPC cordon area since they produced assigned values close to the 1960 ground counts on the Delaware River toll bridges and on a few expressways and turnpikes. Since the distance and speed are known for each link in the highway network, the computer determines consecutively the time for each link, time cost, and its aggregated time via all possible routes, and from the latter computes the time cost for each route according to the unit time cost established.

2. Operating Cost—The operating cost to auto users includes the cost of fuel, oil, tires and maintenance. The DVRPC has adopted the auto operating cost estimated by Hoch for the Chicago Area Transportation Study (4). This operating cost per vehicle mile can be established as a factor of speed. Like time cost, operating cost can be obtained by the computer since the speed and distance for each link are known.

3. Turnpike and Bridge Tolls—Turnpike and bridge charges are two other elements of travel cost. They are coded manually on the links where auto drivers must pay them. Bridge tolls, if any, are added to the operating cost and included in all the sensitivity tests. But turnpike tolls are included in a separate test to find out whether or not these tolls are to be considered as a predictive variable in the assignment model. Also, turnpike tolls are coded into the link data card for providing cordonwide summaries of tolls.

4. Accident Cost—Another element of travel cost is accident cost which is considered in tracing the least total travel cost paths in order to test the effect of accident cost on freeway assigned volumes. Like time and operating costs, accident cost can be computed for each link since the distance and speed are coded.

Table 1 shows travel cost component used for traffic assignment at various selected speeds.

TABLE 1
1960 TRAVEL COSTS IN THE DVRPC REGION

Average Speed (mph)	Cost (cents per veh-mi)			
	Operating	Accident	Time	Total Travel
10	4.62	4.30	15.00	23.92
15	3.84	2.70	10.00	16.54
20	3.39	1.80	7.50	12.69
30	2.91	0.80	5.00	8.71
40	2.74	0.40	3.75	6.89
50	3.28	0.30	3.00	6.58
60	3.90	0.30	2.50	6.70

Parking charges are excluded in all sensitivity tests because they are included in tracing the minimum travel cost paths for the trip distribution model. The rationale for excluding parking charges from traffic assignment program is that they are paid by auto users in the district of destination regardless of the routes they choose in their travel.

Two other elements of travel cost not considered in the DVRPC assignment program are the inconvenience and discomfort costs. These two elements are the least known in travel behavior. Although they may be important criteria in the selection of the route of travel between the districts of origin and destination, inconvenience and discomfort costs are nevertheless very difficult to quantify in dollar terms per vehicle-mile, unlike the other travel cost elements previously mentioned.

In summary, the DVRPC assignment program is all-or-nothing assignment and utilizes the concept of least travel cost. The travel cost elements used in tracing the least travel cost are time cost, operating cost, accident cost, and turnpike and bridge tolls.

COMPUTER PROCESS OF DVRPC HIGHWAY ASSIGNMENT SYSTEM

Generally, the DVRPC highway assignment system is similar to that prescribed by the Bureau of Public Roads in its Traffic Assignment Manual (1). It consists of a series of programs which allocates trip interchanges between districts or zones in a region to a relevant transportation network. This assignment process is accomplished according to the following general steps:

1. Preparation of the input data.
2. Preparation of the network.
3. Loading of traffic on the network.
4. Output of unrestrained assigned volumes.
5. Restrained assignment.

In step 1, two functions are performed: (a) the number of trips from each subarea (district or zone) to all subareas are converted into a binary magnetic tape suitable for input to step 3; and (b) the preparation of all data pertaining to network and area characteristics is accomplished by the link data preparation and build area description programs. The program of link data preparation generates the link data tape necessary for steps 2, 4 and 5. The DVRPC link data preparation program considers area (i.e., CBD, urban, suburban, rural and open rural) and route types which are the determinants of speeds, capacities and the travel cost components. The build area description program generates the area description tape necessary for step 2. This tape includes area information such as load node numbers, area code, cost-time conversion factors and turn penalty.

In step 2, two programs are run: first, the build network description program which edits and summarizes the information of the link data tape in order to be suitable for use in the computation of minimum paths and subsequently in the actual assignment of traffic volume to the network; and second, the build trees program which actually employs Moore's algorithmic procedure to compute, on the basis of least travel cost, minimum cost paths or trees through the network from each subarea centroid to all other nodes in the highway network.

The actual assignment of the simulated traffic interchanges on the links of the highway system is accomplished in step 3 by running the load network program. The DVRPC load network program conforms substantially to the all-or-nothing network loading described by the Bureau of Public Roads (1).

In step 4, the results obtained in steps 1 and 3 are summarized and represented in different forms by the output loaded network program. This program requires as input

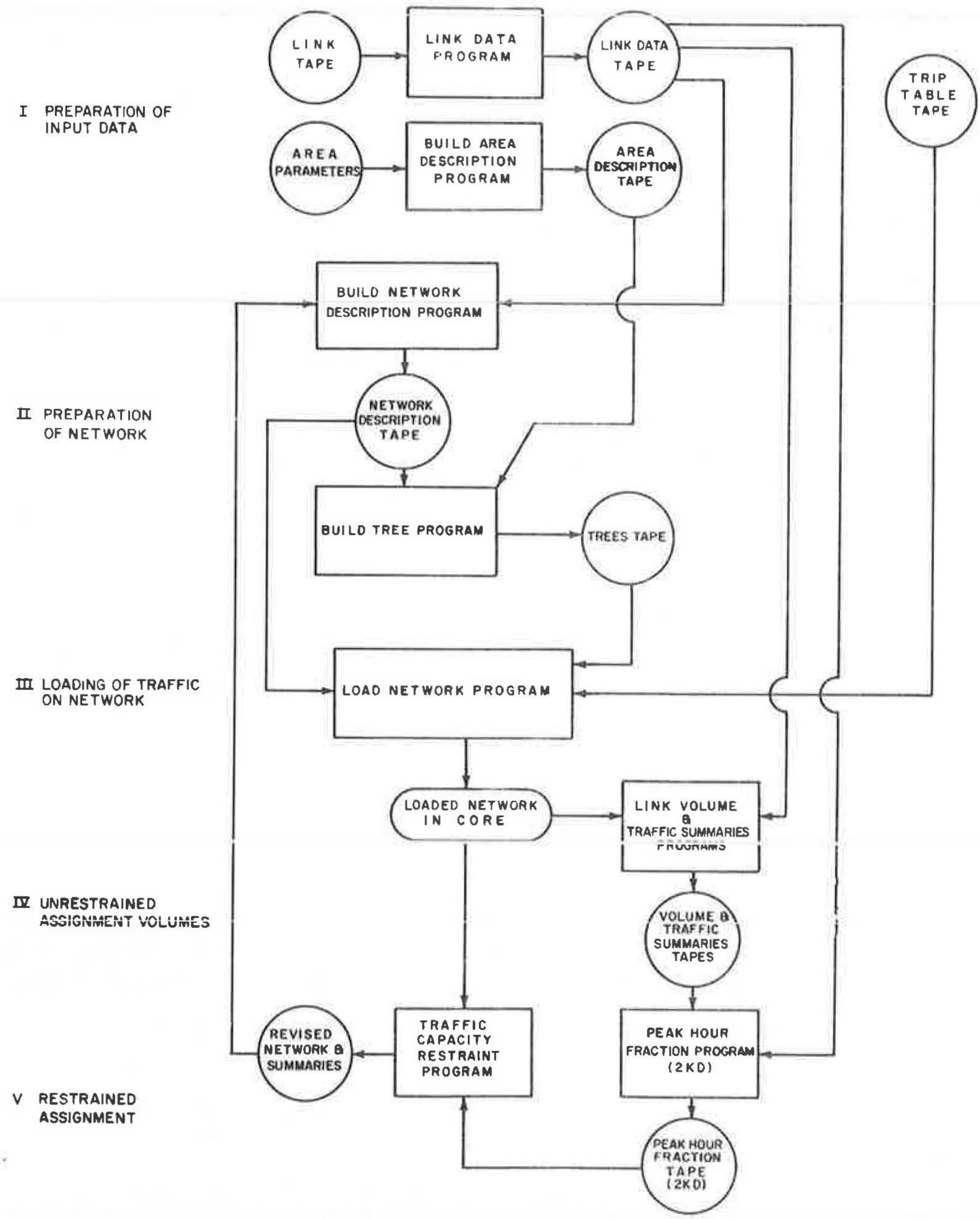

Figure 1. The highway assignment system of DVRPC.

tape the link data tape and the link and turn volumes. The program produces an output tape which is subsequently converted to sets of cards employed in printing and plotting (E. A. I. data plotter) individual link and turn volumes as well as traffic summaries such as vehicle-miles of travel, vehicle hour, vehicle time cost, vehicle operating cost, accident cost, toll charges, miles of route, vehicle speeds, total cost, and vehicle density. The restrained assignment is accomplished in step 5 which adjusts the traffic volume assigned in step 3 on the basis of the capacity restraint concept. According to this concept, new values of speed, time and route travel costs are computed. The computation of average daily restrained speed requires, as input, the following: daily duration of delay, peak-hour fraction, peak-hour speed, and off-peak speed.

The former Penn-Jersey staff developed a program for computing the peak-hour fraction from a linear regression function of home to work trips. The peak-hour fraction program uses, as input, the total assignment volumes obtained from the output loaded network program of step 4, a tape record of assigned home-to-work trips and the link data tape. By using the peak-hour fraction, the link data tape and load network tape, the capacity restraint program produces a tape containing revised network loadings, new time, operating and accident cost values as well as corresponding sets of prints and data plotter cards for detailed analysis.

TABLE 2

INPUT VARIABLES TO HIGHWAY ASSIGNMENT TESTS

Input Variables	Unrestrained Runs		Restrained Runs	
	Test 1	Test 2	Test 3	Test 4
X-var. (operating cost)[a]	LU	LU	LU	LU
Turnpike tolls	NA	A	NA	NA
Y-var. (time cost)	LU	LU	LU	LU
Z-var. (accident cost)	NA	A	NA	NA
Off-peak speed	NA	NA	LU	LU
Average daily speed	LU	LU	C	C
Peak-hour speed	NA	NA	C	C
Min. avg. daily speed	NA	NA	LU	LU
Delay rate	NA	NA	C	C
Max. delay rate	NA	NA	LU	LU
Daily duration of delay	NA	NA	C	C
Hourly capacity	NA	NA	NA	LU
2 KD value	NA	NA	LU	C
24-hr capacity	NA	NA	LU	C

Notes: LU = input obtained from look-up table, except manual coding.
NA = input not used or not applicable in test.
A = input used or applicable in test.
C = computation as specified.

[a]The other component of (X-var.) is the bridge tolls which are included in all tests.

Figure 1 is a work flow chart showing the main steps of DVRPC's highway assignment system.

TRAFFIC ASSIGNMENT SENSITIVITY TESTS

To evaluate the DVRPC traffic assignment concept, several sensitivity tests were designed and run to simulate 1960 traffic volumes. The calibration of the assignment model was made on the basis of different input specifications. Four sensitivity tests were designed and tested at the district level for the whole DVRPC cordon area, and only two sensitivity tests (tests 1 and 3) were run at the zonal level for that portion of Mercer County which is included in the cordon area.

Table 2 gives the major input variables used in each test to trace the minimum cost paths required for traffic loading. Tests 1 and 2 are unrestrained assignment runs, but 3 and 4 are restrained. Test 2 differs from test 1 in that the turnpike tolls and accident costs in test 2 are included in the computation of minimum travel cost paths in order to test the effect of these costs upon traffic assigned to high-type arterials, freeways and turnpikes. The main difference between test 3 and test 4 is that the peak-hour fraction (2KD) in test 4 is computed for each link in the system as a function of the ratio of home-to-work trips to total trips rather than those 2KD values which are input in test 3 on the basis of generalized field observations. Another difference between test 3 and test 4 is that the daily capacity is computed for each highway link in test 4 but the daily capacity is input in test 3 according to certain values specified by route and area types.

Table 3 gives the speeds which are input into the tests. Off-peak speeds have been estimated by route and area types on the basis of speed and delay runs (7). Peak-hour speeds were estimated theoretically by adding the delay rate during peak hour to off-peak speed rate; i. e.:

Peak-hour speed rate = Off-peak speed rate + delay rate in peak hours

The average daily speeds (Table 3) were used in the unrestrained assignment runs (tests 1 and 2). But for tests 3 and 4, average daily speed was computed for each link on the basis of the following relationship:

Average daily restrained speed rate = (daily duration of delay) (peak-hour fraction) (peak-hour speed rate) + [1 - (daily duration of delay) (peak-hour fraction) (off-peak speed rate)]

TABLE 3

OFF-PEAK, PEAK, AVERAGE DAILY AND MINIMUM AVERAGE DAILY SPEEDS (MPH) BY AREA AND ROUTE TYPE, 1960

Route Type	CBD				Urban				Suburban				Rural				Open Rural			
	OPS	PHS	ADS	MADS	OPS	PHS	ADS	MADS	OPS	PHS	ADS	MADS	OPS	PHS	ADS	MADS	OPS	PHS	ADS	MADS
Turnpike (0)	—	—	—	30	—	—	—	32	—	—	—	35	60	60	60	38	60	60	60	42
Freeway (1, 2)	40	25	37	30	45	30	43	32	50	36	48	35	55	45	54	38	60	60	60	42
Multi-high (8, 5) (controlled)	24	17	23	18	29	18	27	21	39	25	37	28	44	39	44	31	50	50	50	35
Multi-high (9) (uncontrolled)	22	17	21	16	27	17	25	20	37	22	34	26	42	37	42	30	50	50	50	35
Multi-low (3)	17	11	16	13	22	13	20	16	32	20	30	23	37	26	26	26	45	35	44	32
Other (4)	12	8	11	10	17	14	17	13	27	23	27	20	32	24	31	23	42	42	42	30

Notes: a. Average daily speeds (ADS) are used in tests 1 and 2 except for links manually coded.
b. Off-peak speeds (OPS) are used in tests 3 and 4 except for links manually coded.
c. The minimum values for ADS are used when the computed average daily speeds in any iteration of the restrained assignment program (tests 3 and 4), are lower than MADS.

The restrained average daily speeds computed for tests 3 and 4 were subject to the minimum average daily speeds constraint in Table 3. These minimum average daily speeds are computed by assuming that one-third of daily travel occurs at 5.0 mph and the other two-thirds occur at off-peak speeds. This assumption is made in order to determine the lowest possible daily speed on each of the heavily congested links in any area in the region.

On the basis of previous speed and delay studies, three equations were used to estimate the delay rates used in tests 3 and 4. As is evident, these delay functions are dependent on the route type of the link.

Turnpikes and freeways:

$$\text{Delay rate} = 2.5 \left(\frac{\text{daily assigned volume}}{\text{daily estimated capacity}}\right) 5$$

Multilane high-type facilities:

$$\text{Delay rate} = 3.0 \left(\frac{\text{daily assigned volume}}{\text{daily estimated capacity}}\right) 5$$

All other routes:

$$\text{Delay rate} = 3.5 \left(\frac{\text{daily assigned volume}}{\text{daily estimated capacity}}\right) 5$$

The delay rate function is subject to a constraint of a maximum delay rate, expressed as minutes per mile. Figure 2 shows the relationship between the delay rates and speeds as functions of the daily volume to daily basic capacity ratios for urban highway facilities. The minimum peak speeds on all highways are set up to be 5.0 mph when the volume-capacity ratio becomes equal to or larger than 1.2. The higher the off-peak speeds, the higher are the maximum delay rates.

In order to compute the average daily restrained speeds, the daily duration of delay was computed as a function of volume to capacity ratio. One formula was used to estimate the duration of delay during peak hours.

$$\text{Daily duration of delay} = \left[2.5 \left(\frac{\text{daily assigned volume}}{\text{daily estimated capacity}}\right) - 1\right] - 2$$

It is further assumed that the maximum duration of delay is 10 hours.

Highway basic rather than practical capacities were used in the capacity restrained program. For test 3 the daily basic capacities (Table 4) were used to compute the ratios of daily assigned volumes to daily basic capacities. (The V/C_b values are used in the computation of: delay rates and daily duration of delay; both are subsequently used to

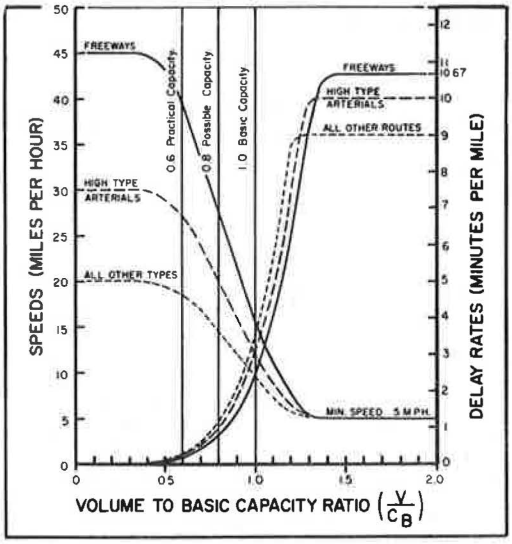

Figure 2. Delay rates and speeds related to volume-capacity ratios for urban highways.

estimate the average daily restrained speeds.) For test 4, however, the daily basic capacities were computed for each highway link as follows:

1. Estimate the peak-hour fraction (2KD) where K is defined as the ratio of two-way peak hour volume to two-way daily volume and D is the ratio of heavy directional peak hour volume to two-way peak hour volume. The value of 2KD is estimated by a linear regression function (5):

$$2KD = 0.0256 + 0.39\,(WR)$$

where

$$WR = \left[\frac{\text{home-to-work trips (assigned)}}{\text{total trips (assigned)}}\right]$$

2. Estimate the daily basic capacity by dividing the hourly capacity into the peak hour fraction (2KD); i.e.,

$$\text{Daily basic capacity} = \frac{\text{hourly capacity}}{2KD}$$

To obtain realistic values for the estimated daily capacities, upper and lower values for 2KD were established on the basis of 1960 available data of cordon and screen line traffic count stations.

Thus, if the 2KD values computed by the model were lower than 0.08 or higher than 0.20, the daily basic capacities of Table 4 were used rather than using the aforementioned formula to compute these capacities. For test 3, however, the following 2KD values were input into the link data program rather than estimating them by the regression equation.

Values	CBD's and Urban Areas	Suburban Areas	Rural and Open Rural Areas
2KD Values (arterials)	0.10	0.12	0.14
2KD Values (freeways)	0.11	0.13	0.15

TABLE 4

HIGHWAY BASIC CAPACITIES BY ROUTE AND AREA TYPES
(Vehicles per Lane)

Route Type	CBD		Urban		Suburban		Rural and Rural Open	
	Hourly	Daily	Hourly	Daily	Hourly	Daily	Hourly	Daily
Freeways (0, 1, 2)	2, 500	22, 700	2, 500	22, 700	2, 500	19, 200	2, 500	16, 700
High-type (cont.)(8)	700	7, 000	900	9, 000	1, 320	11, 000	1, 680	12, 000
High-type (uncont.)(9)	600	6, 000	800	8, 000	1, 200	10, 000	1, 600	11, 400
Low-type (3)	450	4, 500	600	6, 000	900	7, 500	1, 200	8, 500
Other routes (4)	370	3, 700	500	5, 000	750	6, 200	1, 000	7, 100

Notes: a. Basic daily capacities are used in the restrained assignment program (test 3).
b. Basic hourly capacities are used in the restrained assignment program (test 4).

RESULTS OF ASSIGNMENT RUNS

District Assignment Tests

The results of the four sensitivity tests are reported here and evaluated first in terms of statistical measures and then on a link-by-link comparison. The tests were conducted for the whole of the DVRPC cordon area (see Fig. 3).

Figure 3. Traffic assignment study area of the DVRPC 9 county region.

To measure the accuracy of each of the four sensitivity tests, statistical measures were run for eight types of link groups: all master count stations, all river crossings, all turnpikes, all freeways, all high-type arterials, all low-type arterials, all links in Pennsylvania, and all links in New Jersey. The statistical measures were the following:

1. The mean of ground counts ($\bar{X}$) for each link group.
2. The mean of the simulation errors ($\bar{E}_1$) and the mean of these simulation errors expressed as a percent of ground counts ($\bar{E}_2$),

$$\bar{E}_1 = \frac{\sum_{i=1}^{n} (Vai - Vgi)}{n}$$

$$\bar{E}_2 = \frac{\sum_{i=1}^{n} \frac{(Vai - Vgi)}{Vgi} \times 100}{n}$$

These measures indicate the average error of overassignment or underassignment in each link group.

3. The root mean square error in vehicles (RMS), and in percent of the mean of ground counts (% RMS).

$$RMS = \sqrt{\frac{\sum_{i=1}^{n} (Vai - Vgi)^2}{n}}$$

$$\% RMS = \frac{RMS}{\bar{X}} \times 100$$

As is known in statistics, about 67 percent of the links of each group should lie within one root mean square. Of course, the lower the RMS is, the better are the simulation results.

4. The coefficient of correlation (r) between the assigned volumes and ground counts. This measure indicates the quality of fit of the simulated result to the actual counts.

$$r = \sqrt{1 - \frac{(RMS)^2}{(Sx)^2}}$$

where

n = number of links in each group considered,
Vai = the assigned volume to link i,
Vgi = the ground count to link i, and
Sx = the standard deviation of the ground counts in each link group considered.

The results of these statistical measures for the eight link groups mentioned above are given in Table 5 for each sensitivity test. Test 1 produced better results than test 2, indicating that the inclusion of turnpike tolls and accident costs in tracing the minimum paths did not improve the outputs of the unrestrained assignment program. A characteristic common to both tests is a general overassignment to all link types except for turnpikes. The best results were obtained in the assignment to river crossings and turnpikes but the assignment to freeways and arterials resulted in high simulation errors which were considerably reduced in test 3 and 4.

TABLE 5

COMPARISON OF SIMULATION RESULTS BY ROUTE AND AREA TYPES FOR TESTS 1, 2, 3, AND 4

Link Group	Test No.	Mean of Ground Count, $\overline{X}$ (veh)	Mean of Simulation Error: $\overline{E}_1$ (veh)	Mean of Simulation Error: $\overline{E}_2$ (%)	Root Mean Square Error: RMS (veh)	Root Mean Square Error: RMS (%)	Coefficient of Correlation, r (%)
1. Master control stations	1	18, 227	3, 362	17	15, 154	83	—
	2	18, 227	3, 257	14	17, 077	93	—
	3	18, 227	-1, 264	-4	10, 898	60	71
	4	18, 227	-215	-2	9, 700	53	78
2. River crossings	1	27, 020	886	4	6, 107	23	96
	2	27, 020	1, 418	2	8, 354	31	93
	3	27, 020	2, 649	12	10, 409	38	89
	4	27, 020	2, 974	10	10, 431	38	89
3. Turnpikes	1	20, 765	-1, 470	-7	4, 875	24	49
	2	20, 765	-6, 114	-27	8, 468	41	—
	3	20, 765	795	3	3, 228	16	84
	4	20, 765	180	1	2, 804	14	88
4. Freeways	1	29, 374	8, 556	64	22, 909	78	—
	2	29, 374	12, 932	79	26, 938	92	—
	3	29, 374	7, 768	41	19, 062	65	32
	4	29, 374	8, 471	44	19, 422	66	27
5. High-type arterials	1	36, 019	22, 278	74	34, 586	96	—
	2	36, 019	30, 133	100	43, 554	120	—
	3	36, 019	8, 498	31	17, 501	48	49
	4	36, 019	9, 526	35	18, 770	52	35
6. Low-type arterials	1	17, 081	3, 003	35	16, 891	99	—
	2	17, 081	3, 218	35	16, 940	100	—
	3	17, 081	2, 213	39	12, 443	73	—
	4	17, 081	2, 542	39	12, 776	75	—
7. Links in Pa.	1	13, 907	2, 086	37	16, 290	117	—
	2	13, 907	3, 076	37	17, 231	122	—
	3	13, 907	2, 185	40	11, 766	84	—
	4	13, 907	2, 306	39	11, 996	87	—
8. Links in N.J.	1	12, 470	1, 972	21	11, 684	94	—
	2	12, 470	2, 475	27	13, 066	113	—
	3	12, 470	1, 057	15	8, 463	68	—
	4	12, 470	1, 261	16	8, 827	71	—

The calibration results of tests 3 and 4 were obtained through an iterative process.

First Iteration—The vehicle trips were loaded on the minimum cost paths which were traced on the basis of the restrained speeds that were computed by the capacity restraint program. These restrained speeds were calculated on the basis of V/C ratio in which the assigned volume (V) was taken from the output of test 1.

Second Iteration—The speeds were obtained from averaging the speeds of the first iteration and the minimum average daily speeds. The purpose for this iteration was to get maximum restrained speeds at a reasonable rate to reduce the overassignment on freeways and high-type facilities.

Third Iteration—The minimum cost paths for this iteration were traced on the basis of restrained speeds that were computed by the capacity restraint program. The volume used in the calculation of V/C ratio was taken from the second iteration.

It was found that the three iterations were satisfactory to dampen the fluctuations in tracing alternative minimum cost paths. Large fluctuations, however, were purposely controlled by inputting reasonable parameters in the equations of the restrained assignment program.

The average of the assignment outputs of the first and third iterations produced the best calibration results. The statistical measures in Table 5 indicate that test 3 resulted in better assignment than test 4. However, the assignment results of test 4 are generally comparable to those of test 3. The estimation of the peak-hour fraction (2KD) from the assignment of home-to-work trips on the highway network (which requires several computer runs) did not prove to be significantly superior to inputting values for 2KD.

The restrained tests gave the best simulation volumes of all link groups considered except for the river crossing links. The cause of this occurrence was studied and it was found that the restrained speeds produced multiple river crossings in the trace of the minimum travel cost paths. For example, the tracing of some selected trees showed

TABLE 6

COMPARISON OF ASSIGNED VOLUMES AND GROUND COUNTS AT HIGH-TYPE ARTERIALS AND LOW-TYPE ARTERIALS

Link Identification	1960 Ground Counts	Unrestrained			Restrained			
		Test 1 Diff.	Test 2 Diff.	ADS	Test 3		Test 4	
					Diff.	ADRS	Diff.	ADRS
Sections of high-type arterials:								
US 130	33, 000	+ 17	+ 27	37.0	+10	38.3	+15	39.0
US 130	46, 000	+ 57	+ 83	37.0	+21	32.2	+34	37.5
Roosevelt Blvd.	70, 000	+ 81	+ 91	37.0	- 5	32.8	-11	33.0
West Chester Pike	19, 200	+ 22	+ 22	42.0	+36	32.0	+29	42.0
Sections of low-type arterials:								
Broad Street	36, 000	- 1	+ 75	20.0	-26	16.0	-13	16.0
Chestnut-Walnut	32, 800	- 23	- 32	16.0	-12	13.0	-33	14.0
Market Street	11, 800	- 44	- 14	16.0	-38	17.0	+52	17.0
5th-6th Streets	24, 200	- 1	+ 36	20.0	+ 9	21.5	+ 5	21.5
City Line Avenue	34, 600	- 25	- 24	30.0	+14	23.0	-22	25.6
City Line Avenue	26, 000	- 20	- 21	20.0	-11	18.7	-31	19.0
15th-16th Streets	20, 000	- 26	- 13	20.0	+37	19.0	+67	19.0
Belmont Avenue	16, 600	+102	+103	30.0	- 8	27.5	+ 8	27.5
Lancaster Pike	18, 600	+152	+138	30.0	+26	25.3	+26	26.3

Notes: Applicable to Tables 6, 7, and 8.

a. Diff. = difference between assigned volume and ground count expressed as percent of ground count.
b. ADS = average daily speed.
c. ADRS = average daily restrained speed.

TABLE 7

COMPARISON OF ASSIGNED VOLUMES AND GROUND COUNTS AT TURNPIKES AND EXPRESSWAYS

Link Identification	1960 Ground Count	Unrestrained			Restrained			
		Test 1 Diff.	Test 2 Diff.	ADS	Test 3		Test 4	
					Diff.	ADRS	Diff.	ADRS
Pennsylvania Turnpike:								
Valley Forge-Norristown	20, 524	-38	-48	60.0	-39	60.0	-38	60.0
N.E. Ext'n-Ft. Washington	21, 421	- 5	-36	60.0	+ 8	60.0	+ 6	60.0
Ft. Washington-Willow Grove	21, 245	+12	-16	60.0	+16	60.0	+14	60.0
Willow Grove-Philadelphia	20, 513	+ 3	-26	60.0	+ 1	60.0	+ 2	60.0
New Jersey Turnpike:								
Bordentown-Tpke. Jctn.	32, 613	+16	-32	60.0	+17	60.0	+14	60.0
Mt. Holly-Camden	24, 902	0	-40	60.0	+11	60.0	+ 2	60.0
Camden-Woodbury	20, 919	- 8	-28	60.0	- 1	60.0	- 8	60.0
Woodbury-Swedesboro	17, 773	+10	- 5	60.0	+20	60.0	+ 9	60.0
North-South Freeway:								
NJ 47-NJ 168	21, 000	+65	+69	54.0	+70	53.9	+69	55.0
NJ 168-NJ 534	17, 200	+23	+24	54.0	+21	55.0	+21	55.0
Schuylkill Expressway:								
Valley Forge-US 202	16, 798	+16	+14	54.0	+10	52.5	+28	55.0
Maudyunk-City Line	47, 005	- 6	+ 9	48.0	- 1	48.0	+ 1	49.8
Girard-Spring Garden	88, 666	+18	+39	43.0	+12	38.8	+ 9	40.3
University-34th St.	35, 145	+ 8	+50	43.0	+25	45.0	+27	45.0
7th St.-Front St.	50, 000	+ 2	+ 7	48.0	+ 9	49.3	+11	50.0

TABLE 8

COMPARISON OF ASSIGNED VOLUMES AND GROUND COUNTS AT MASTER COUNT STATIONS AND RIVER CROSSINGS

Link Identification	1960 Ground Count	Unrestrained			Restrained			
		Test 1 Diff.	Test 2 Diff.	ADS	Test 3		Test 4	
					Diff.	ADRS	Diff.	ADRS
Master stations:								
US 202	15, 600	- 6	- 3	31.0	- 78	23.0	- 35	28.0
US 309	19, 800	+ 25	+ 59	48.0	- 11	50.0	+ 33	50.0
NJ 541	3, 000	- 18	- 27	31.0	- 17	32.0	- 19	32.0
US 30	7, 600	+ 80	+ 77	36.0	+ 62	37.0	+ 86	37.0
River crossings:								
Chester Ferry	2, 639	- 39	- 45	8.0	+ 23	8.0	- 15	8.0
Walt Whitman	50, 086	+ 1	+ 6	45.0	+ 27	34.5	+ 29	37.0
Ben Franklin	69, 104	+ 2	+ 5	45.0	0	33.0	0	38.4
Tacony-Palmyra	46, 602	+ 4	+ 9	35.0	- 5	28.0	+ 46	30.0
Burlington-Bristol	18, 738	+ 19	+ 34	30.0	+ 59	30.0	- 6	28.0
Turnpike Bridge	12, 764	- 7	- 55	60.0	- 17	60.0	- 13	60.0
Trenton Freeway	15, 833	+178	+206	50.0	+125	44.3	+124	49.6
Bridge Street	18, 922	- 65	- 76	25.0	- 70	25.0	- 72	25.0
Calhoun Street	18, 271	+ 6	+ 9	25.0	- 17	23.0	- 19	23.5
Yardley	7, 030	+ 50	+ 37	20.0	+ 29	14.0	+ 35	17.8

TABLE 9

SYSTEM CHARACTERISTICS OF THE MERCER COUNTY TRAFFIC ASSIGNMENTS

Items	Assignment	
	District	Zone
Miles of route	107	161
Highway links	182	574
Approach links	32	211
No. of internal load nodes	13	105
Average trip ends per internal load node	41, 400	7, 500
Average population per internal load node	15, 640	1, 940

that trips originating in the Trenton area with a destination in the vicinity of the Philadelphia International Airport would cross the Delaware River at the Trenton Freeway Bridge into Pennsylvania; thus proceeding in Pennsylvania along US 13 and again crossing the Delaware River at the Burlington Bristol Bridge into New Jersey and continuing the journey in New Jersey along US 130 to the Walt Whitman Bridge; and using this bridge to cross once more the Delaware to reach the destination.

In general, the district highway assignment resulted in an overassignment, which is a characteristic of a coarse network. The best assignments were made on the turnpikes and the poorest assignments were made on low-type arterials; freeways and high-type arterials were generally good.

The vehicle summaries produced by the four tests were very close to actual field data. The difference between the vehicle-miles of test 3 and those obtained from the route and intersection data collected from the field was less than 2 percent (22. 36 million vs 22. 73 million).

The link-by-link comparison of assigned volumes to ground counts for randomly selected sections of turnpikes, freeways, high-type arterials, low-type arterials, master count stations, and all river crossings, is given in Tables 6 through 8. Tests 3 and 4 produce generally lower simulation errors than the unrestrained runs of tests 1 and 2. The best results are found for turnpikes, freeways, and high-type arterials. Neither test 3 nor 4 was better than the unrestrained tests for the bridge crossing links. The reason for this discrepancy was previously given.

Overall, the link-by-link comparison indicates that the calibration of the district assignment produces simulation volumes of sufficient accuracy for establishing general design requirements of freeways and high-type highway facilities throughout the DVRPC cordon area.

Mercer County Assignment

For Mercer County, a zone highway assignment was run and compared to the outputs of the district assignments. The comparison and evaluation of the outputs of the two assignment systems were made to assess the amount of improvement made in the calibration of traffic volumes due to the characteristics of the zone system.

The zone and district systems are given in Table 9. The average number of trip ends per zone loaded on the zone network is considerably less than that loaded on the district network (7500 vs 41, 400). The zone network contains 54 additional miles of routes; this additional mileage makes the zone network much denser than the district network (182 links vs 574), and thus it provides a greater combination of route choices for trip interchanges in the area. Figures 4 and 5 show the zone size and network density of Trenton's CBD for the district and zone assignments. These maps illustrate typical differences between the zone and the district networks. The zone centroids (from which trips are loaded) are more uniformly distributed throughout the area than those of the district. In turn, each load node of the zone network is connected to the adjacent highways by more approach links than in the district system, resulting in a more uniform loading of trips. Since the cost trace variables for both the district and zone networks are identical, the differences between the simulated volumes of the zone assignment and those of the district assignment are due to the change in network density and in the size of traffic zones.

Certain capacity characteristics for links located in the suburban and rural areas were revised to describe the zone network in a more precise manner than was necessary for the district assignment; however, these changes are not considered to have a significant influence on the comparison between the zone and district systems.

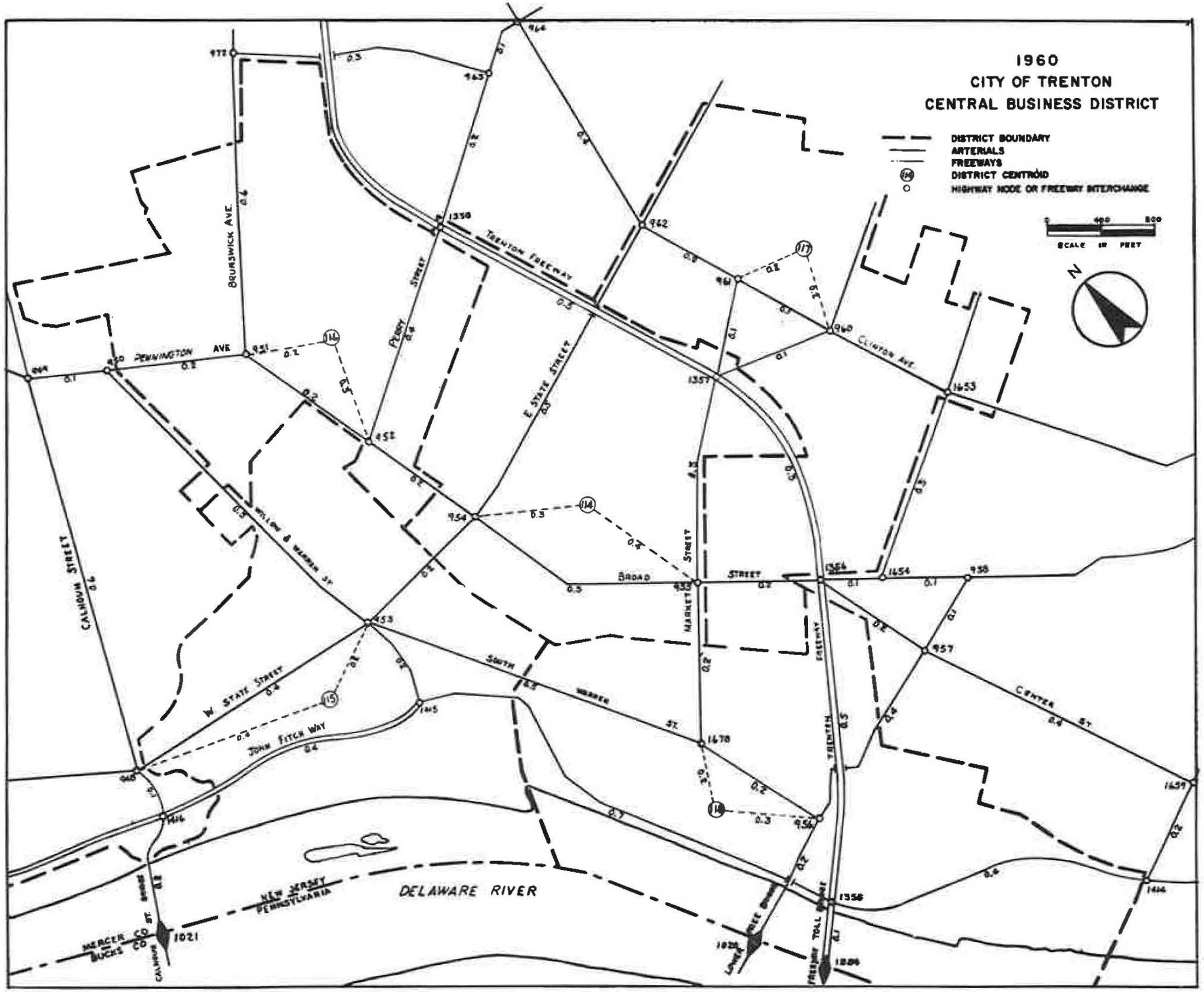
1960
CITY OF TRENTON
CENTRAL BUSINESS DISTRICT
DISTRICT BOUNDARY
ARTERIALS
FREEWAYS
DISTRICT CENTROID
HIGHWAY NODE OR FREEWAY INTERCHANGE
SCALE IN FEET
N
TRENTON FREEWAY
BRUNSWICK AVE.
PENNINGTON AVE
PERRY STREET
E STATE STREET
CLINTON AVE.
BROAD STREET
MARKET STREET
WILLOW & WARREN ST
CALHOUN STREET
W STATE STREET
JOHN FITCH WAY
SOUTH WARREN ST
CENTER ST
TRENTON FREEWAY
DELAWARE RIVER
NEW JERSEY
PENNSYLVANIA
MERCER CO
BUCKS CO
CALHOUN ST. BRIDGE
LOWER FREE BRIDGE
FREEWAY TOLL BRIDGE

Figure 4.

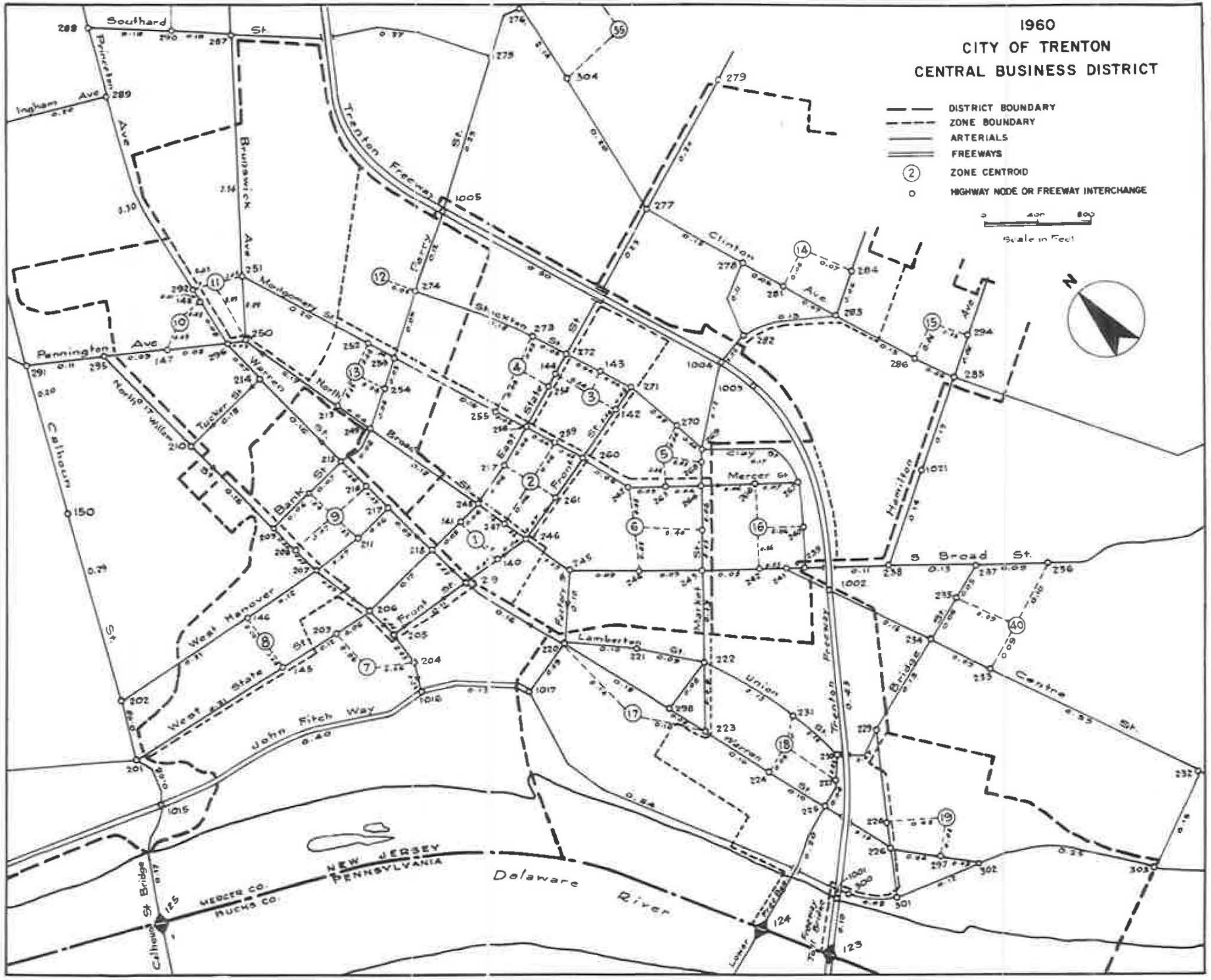
1960
CITY OF TRENTON
CENTRAL BUSINESS DISTRICT
DISTRICT BOUNDARY
ZONE BOUNDARY
ARTERIALS
FREEWAYS
ZONE CENTROID
HIGHWAY NODE OR FREEWAY INTERCHANGE
Scale in Feet
Southard St.
Princeton Ave
Ingham Ave
Brunswick Ave
Trenton Freeway
Perry St.
Clinton Ave
Montgomery St.
Pennington Ave
Warren St.
North Willow St.
Tucker St.
Calhoun St.
North Broad St.
Shipton St.
East State St.
Front St.
Bank St.
West Hanover St.
West State St.
John Fitch Way
Lamberton St.
Market St.
Mercer St.
Clay St.
Hamilton Ave
S Broad St.
Bridge St.
Centre St.
Union St.
Delaware River
NEW JERSEY
PENNSYLVANIA
MERCER CO.
BUCKS CO.
Calhoun St Bridge
Lower Ferry Rd.
Toll Bridge

Figure 5.

TABLE 10

ROOT MEAN SQUARE ERRORS FOR DISTRICT AND ZONE HIGHWAY ASSIGNMENTS—MERCER COUNTY

Volume Group	Unrestrained Assignments				Restrained Assignments			
	Zone		District		Zone		District	
	RMS	%RMS	RMS	%RMS	RMS	%RMS	RMS	%RMS
0 1, 999	1, 253	111	560[a]	56[a]	1, 827	165	560[a]	56[a]
2, 000- 4, 999	5, 210	159	5, 364	162	4, 315	129	5, 171	151
5, 000- 9, 999	5, 962	83	9, 814	137	4, 573	64	5, 583	77
10, 000-14, 999	6, 524	55	14, 154	120	5, 517	46	10, 273	87
15, 000-19, 999	10, 522	66	18, 338	112	8, 997	56	11, 550	71
20, 000-24, 999	10, 428	48	16, 787	81	5, 052	24	17, 276	81
25, 000-35, 000	9, 308	31	9, 801	31	9, 488	31	7, 337	23
Freeways	5, 970	29	17, 899	87	6, 776	33	15, 977	77
High-type arterials	12, 265	67	13, 474[b]	51[b]	10, 187	53	13, 555[b]	63[b]
Low-type arterials	6, 746	76	13, 687	116	5, 468	61	9, 415	79
All links	7, 073	73	14, 000	111	5, 776	60	10, 059	79

[a]Only one link in this volume group. [b]Only three links in this volume group.

TABLE 12

REAL ERROR OF TRAFFIC ASSIGNMENT FOR MERCER COUNTY

Design Volume Range	One-Half Design Tolerance (±)[a]	Real Error (%)	
		Zone Calibration	District Calibration
0- 4, 999	99	41	52
5, 000- 9, 999	35	29	42
10, 000-14, 999	21	25	66
15, 000-19, 999	16	40	55
20, 000-24, 999	12	12	69
25, 000-34, 999	17	14	6

[a]As a percent of group mean volume; these volumes were computed on the basis of the mean of ground counts to reflect the 1960 actual traffic distribution by volume ranges.

TABLE 11

LANE REQUIREMENTS BASED ON PRACTICAL CAPACITY AND ECONOMIC VOLUME RANGES FOR PROPOSED HIGHWAY FACILITIES[a]

Range of Average Daily Design Volume	CBD			Urban			Suburban			Rural and Open Rural.		
	L	H	F[b]	L	H	F	L	H	F	L	H	F
0- 4, 999	2	–	–	2	–	–	2	–	–	2	–	–
5, 000- 9, 999	4	–	–	4	–	–	4	–	–	4	–	–
10, 000- 14, 999	–	4	–	4	–	–	4	–	–	4	–	–
15, 000- 19, 999	–	6	–	–	4	–	–	4	–	4	–	–
20, 000- 24, 999	–	6	–	–	6	–	–	4	–	–	4	–
25, 000- 34, 999	–	–	4	–	–	4	–	6	–	–	6	–
35, 000- 44, 999	–	–	4	–	–	4	–	–	4	–	–	4
45, 000- 64, 999	–	–	4	–	–	4	–	–	6	–	–	6
65, 000- 94, 999	–	–	6	–	–	6	–	–	8	–	–	–
95, 000-135, 000	–	–	8	–	–	8	–	–	–	–	–	–

[a]Based on references (2, 3).
[b]L = low-type arterial; H = high-type areterial; and F = freeway.

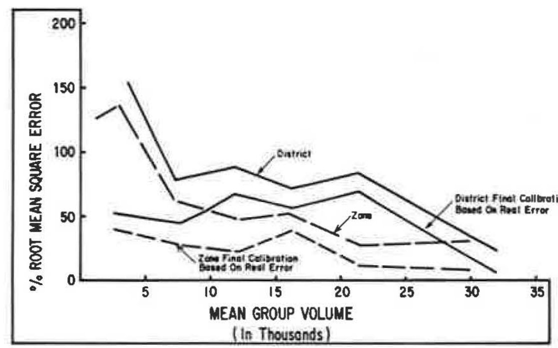

Figure 6. RMS error vs mean group volume.

The zone assignment was calibrated in four computer runs: one unrestrained and three restrained assignment runs. The average of the first and third restrained runs was found to give the best results. The same variables used in the district calibration were applied to calibrate the zone assignment. As in the case of the cordonwide district highway assignment, the evaluation of the Mercer County zone assignment and the comparison of its results with those of the district assignment were made through statistical analysis and direct comparison of ground counts to simulated volumes for selected sections of the highway system.

The statistical evaluation of the Mercer County assignments was made by comparing the assigned volumes with the ground count within selected traffic volume ranges; these volume ranges were established to reflect significant differences in lane capacity requirements. In addition, statistical measures were calculated for freeways, high-type arterials, low-type arterials, and all route types combined. The root mean square error (RMS) was used as the criterion for measuring the accuracy of the assignment. This error both in vehicles and as a percent of the mean of ground counts is given in Table 10 for the zone and district assignments. The zone assignment produces significantly better results than the district assignment since there is a reduction in RMS error for almost all link groups. The lowest RMS values were obtained for the high volume groups of 20, 000 vehicles or more. Freeways received the best assignments with an RMS of 33 percent and low-type arterials the poorest with an RMS of 61 percent. The low-type arterials usually carried an average daily volume of well below 10, 000 vehicles per day.

As it is used in statistics, the RMS value here represents the error of the traffic assignment. In measuring the accuracy of traffic assignment, however, the statistical significance of the RMS should be appraised in conjunction with the fact that while highways are usually designed to accommodate a fixed design volume, they in effect have a capacity to serve certain ranges of volume demand. For example, a six-lane freeway in the CBD can accommodate a traffic volume varying from 65, 000 to 95, 000 vehicles per day, representing a range of 30, 000 vehicles. This range is equivalent to ±RMS (15, 000) which is equal to 19 percent of the mean. It is reasoned that when a freeway is assigned a volume that falls anywhere between 65, 000 and 95, 000, the "real error" of the traffic assignment may be considered insignificant if the ground count also falls anywhere between this range.[1] The allowable design volume ranges are given by route and area type in Table 11 and they are defined here as design tolerance volumes. When these tolerances are expressed in terms of a percentage deviation of the mean of each volume group, the result is defined here as the real error of the traffic assignment. These real errors are listed in Table 12 for both the zone and district systems.[2]

In Figure 7, the overall improvement of the zone assignment results over those of district assignment is brought into focus both in terms of RMS and real error. The

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[1] The real error concept does not purport to, either replace the RMS as the measure of assignment accuracy, or justify the error of traffic assignment. Rather, it is introduced to relate the RMS to practical highway design. Thus, if a six-lane freeway is designed to accommodate the mean volume of 80,000 vehicles per day when the RMS is 19 percent, then the peak-hour volume per lane will range between 1,375 and 1,740 vehicles. This range of peak-hour volume per lane is the equivalent of about 65,000 to 95,000 vehicles per day and it is acceptable for design purposes. It is seen, that although we would like to design this freeway to accommodate 1,500 vehicles per lane in the peak hour, we would not require a change in the number of lanes if the peak-hour volume per lane is between the range of 1,375 and 1,740 vehicles.

[2] The real error of traffic assignment as defined in this paper represents not only that error which is attributable to the traffic assignment process, but it also includes errors resulting from other sources such as the O and D survey, the accuracy of the ground counts, and the trip table used. The outputs of simulated volumes for future years will depend not only on the accuracy of the traffic assignment process but also on the accuracy of each component model of the traffic simulation process, such as the car ownership, trip generation, modal split, and trip distribution models.

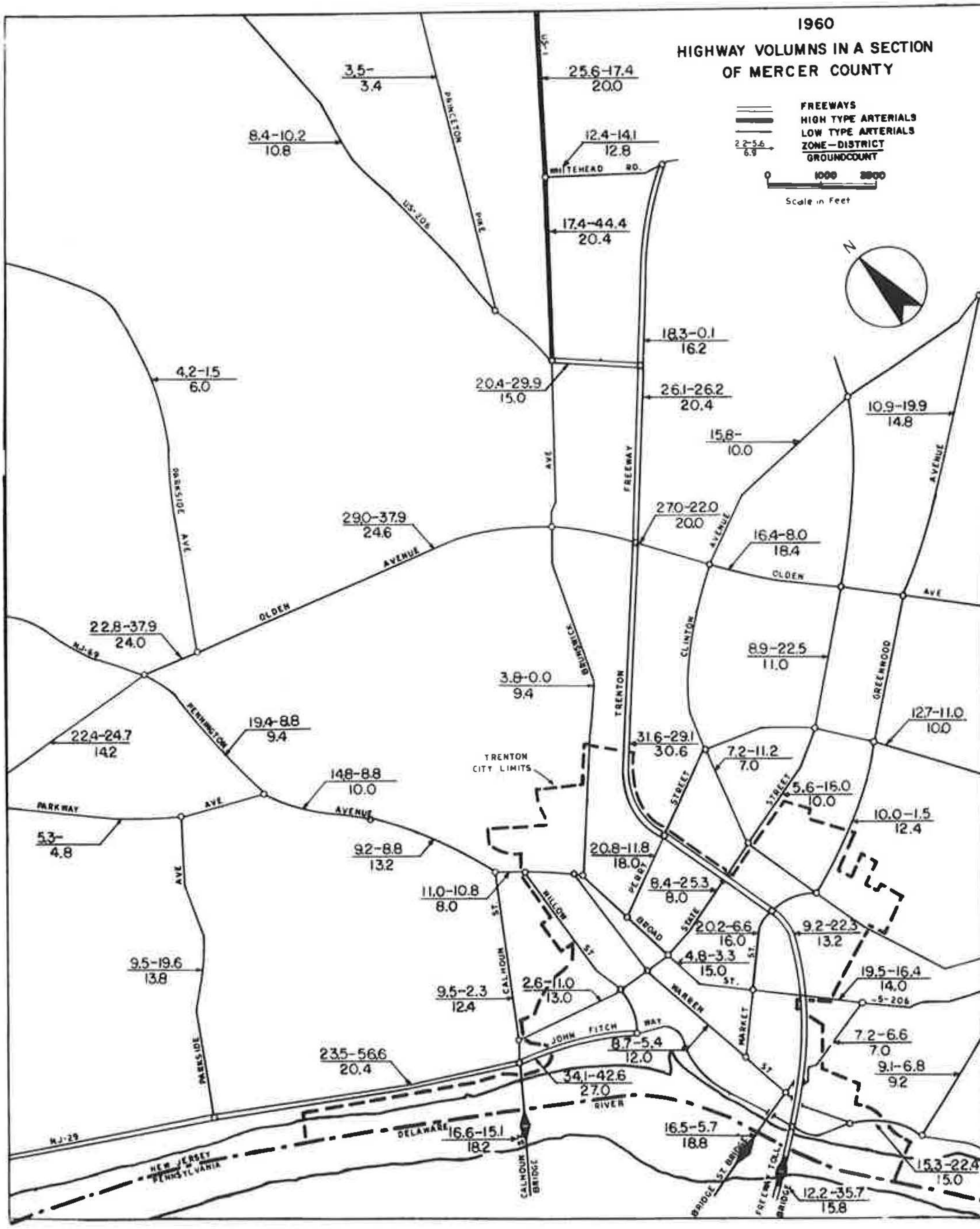
1960
HIGHWAY VOLUMNS IN A SECTION
OF MERCER COUNTY
FREEWAYS
HIGH TYPE ARTERIALS
LOW TYPE ARTERIALS
ZONE—DISTRICT
GROUNDCOUNT
Scale in Feet
N
PRINCETON PIKE
US-206
U.S.-1
WHITEHEAD RD.
FREEWAY
TRENTON
BRUNSWICK AVE
OLDEN AVENUE
PARKSIDE AVE
NJ-69
PENNINGTON AVENUE
PARKWAY AVE
CLINTON AVENUE
GREENWOOD AVENUE
PERRY STREET
STATE STREET
BROAD ST.
WARREN ST.
MARKET ST.
WILLOW ST.
CALHOUN ST.
JOHN FITCH WAY
DELAWARE RIVER
NJ-29
NEW JERSEY
PENNSYLVANIA
CALHOUN ST. BRIDGE
BRIDGE ST. BRIDGE
FREEWAY TOLL BRIDGE
TRENTON CITY LIMITS
US-206
3.5- / 3.4
25.6-17.4 / 20.0
8.4-10.2 / 10.8
12.4-14.1 / 12.8
17.4-44.4 / 20.4
18.3-0.1 / 16.2
4.2-1.5 / 6.0
20.4-29.9 / 15.0
26.1-26.2 / 20.4
10.9-19.9 / 14.8
15.8- / 10.0
29.0-37.9 / 24.6
27.0-22.0 / 20.0
16.4-8.0 / 18.4
22.8-37.9 / 24.0
8.9-22.5 / 11.0
3.8-0.0 / 9.4
12.7-11.0 / 10.0
22.4-24.7 / 14.2
19.4-8.8 / 9.4
31.6-29.1 / 30.6
7.2-11.2 / 7.0
14.8-8.8 / 10.0
5.6-16.0 / 10.0
10.0-1.5 / 12.4
5.3- / 4.8
9.2-8.8 / 13.2
20.8-11.8 / 18.0
11.0-10.8 / 8.0
8.4-25.3 / 8.0
20.2-6.6 / 16.0
9.2-22.3 / 13.2
4.8-3.3 / 15.0
9.5-19.6 / 13.8
2.6-11.0 / 13.0
19.5-16.4 / 14.0
9.5-2.3 / 12.4
7.2-6.6 / 7.0
23.5-56.6 / 20.4
8.7-5.4 / 12.0
34.1-42.6 / 27.0
9.1-6.8 / 9.2
16.6-15.1 / 18.2
16.5-5.7 / 18.8
15.3-22.4 / 15.0
12.2-35.7 / 15.8

Figure 7.

greatest accuracy was obtained for high-volume roads and the least accuracy for low-volume roads. This conforms with the findings of other transportation studies.

Actual field survey data of route miles and traffic counts produced 1.409 million vehicle-miles; and the zone and district assignments produced 1.409 and 1.523 million vehicle-miles, respectively. The excellent agreement produced by the zone assignment is due to the system characteristics of the zone network such as additional highway mileage, smaller zone size, fewer number of trips loading from a zone load node and shorter highway links. The combination of these system characteristics, therefore, has resulted in better paths for traffic assignment.

A detailed analysis of simulated volumes is shown in Figure 7 which illustrates some freeways, high-type arterials, and low-type arterials in the fringe area of Trenton. The zone assigned volumes, the district assigned volumes and the observed ground counts are shown for each highway link. These volumes provide a link-by-link comparison of zone and district assignments. A good simulation was obtained on the Trenton Freeway by both the district and zone assignments; notable were the improvements made on the John Fitch Way, Olden Avenue, US 1, and sections of routes in the CBD such as Warren and Perry Streets. On these highway sections, the overall differences between the assigned volumes and the observed link volumes for the zone and district assignments were as follows:

Route	Zone	District
John Fitch Way	+14%	+49%
Olden Avenue	+16%	+30%
US 1	+ 2%	+42%
Warren Street	-27%	-55%
Perry Street	+11%	-34%

These results indicate that the zone simulated volumes are of significantly greater accuracy than those of the district assignment.

CONCLUSIONS

The traffic assignment process used by the Delaware Valley Regional Planning Commission is based on the concept that trip interchanges are assigned to those routes which offer the least resistance to travel. The resistance parameter used is travel cost. This represents a departure from the practice used by other transportation studies which consider travel time as the travel resistance parameter. Travel cost was used in order to minimize the amount of overassignment to freeways and expressways which normally become considerably overloaded when travel time is used.

Only the cost components which were quantifiable were included in the computation of the total travel resistance. These were the vehicle operating costs, vehicle time costs, bridge tolls, accident costs and turnpike tolls. These five cost components were then combined into two sets of total travel costs; one set including all five components and another combining only the vehicle operating costs, vehicle time costs, and bridge tolls. For each set of these travel costs an unrestrained assignment was run (tests 1 and 2) and it was found that test 1, which included only the vehicle operating costs, vehicle time costs and bridge tolls, produced better unrestrained assignment outputs. Consequently, only these three components of travel cost were the ones used in the calibration of the restrained assignment tests 3 and 4.

The results of tests 3 and 4 proved that the extra efforts associated in the calculation of the 2KD value (peak-hour fraction) did not produce more accurate assignment results than did test 3 where the 2KD's were input into the capacity restraint program. The analysis and evaluation of the assignment results of test 3 showed that the use of travel cost as the parameter of travel resistance produced simulated volumes of acceptable accuracy and that these volumes can be used as a guide in the design of major highway facilities such as freeways and high-type arterials. The RMS errors for turnpikes and freeways were 16 percent and 65 percent respectively, and 48 percent for high-type arterials.

A satisfactory calibration was obtained through the use of mechanical means built into the assignment program without the need of manual adjustments of the input variables used in the assignment package. This is considered a very desirable property since it permits a realistic evaluation of the accuracy of the calibration process and thus provides reliable estimates of future simulated volumes.

As the direct result of smaller zone size and denser highway network, the zone assignment for Mercer County produced much better simulation of traffic volumes than the district highway assignment. The real error of the zone assignment for all the link volumes groups considered varied from 12 percent to 41 percent, demonstrating that the zone calibration for Mercer County produced assignment outputs that could effectively be used in highway design.

This paper shows that the use of the real error concept in the assessment of traffic assignment outputs is valuable since it is directly related to highway lane design volumes.

We believe that when using the zone outputs for highway design the errors of traffic assignment can be reduced if the simulated link volumes are combined to reflect the general traffic demand on the particular highway.

ACKNOWLEDGMENT

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Simplified Procedure for Estimating Recreational Travel to Multi-Purpose Reservoirs

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W. L. GRECCO, Purdue University

This research reports the results of a study concerned with the development of a model that can be used to predict recreational trips to new reservoirs in Indiana. The model utilizes only road distance, county population, and the influence of other similar facilities as the parameters affecting attendance. A technique was developed illustrating how the model can be used to predict future attendance and traffic volumes.

Three parks were used in the study. Data were collected by conducting interviews of 25 percent of arriving trips at the park entrances. Over 13,000 interviews were conducted over a 2-yr period. Yearly distributions of trips by trip purpose and frequency were investigated.

The prediction model was developed by using nonlinear regression analysis to determine the parameters of distance, population and the influence of other parks. Two equations were developed, one for the condition where there is no other park closer to a county than the park under consideration and the other where there is another park closer. Together, the two equations constitute the prediction model.

•THE control and use of water resources is of major importance to the economic life of the United States. Flood control, irrigation, and hydroelectric power were originally the three purposes considered in the cost analysis for justification of the construction of dams and their reservoirs. However, not until recent years have the recreational benefits been generally included in the economic analysis or even recognized as an economic factor.

Recreation is now a big business in this country. A substantial portion of the Gross National Product is devoted to recreational pursuits in all areas of the nation.

Traffic patterns have changed because of the proportionate increase in personal expenditures for recreational purposes. Many rural highway sections serving recreational facilities now carry their peak travel loads on weekends.

The development of the future highway network must take into account the traffic generating abilities of a recreational park on a reservoir. A recreational facility is of little value without access. On some routes, peak volumes result from trips made for recreational purposes. On many routes, weekend traffic volumes exceed the weekday volumes and the increase is due mainly to recreational travel (1).

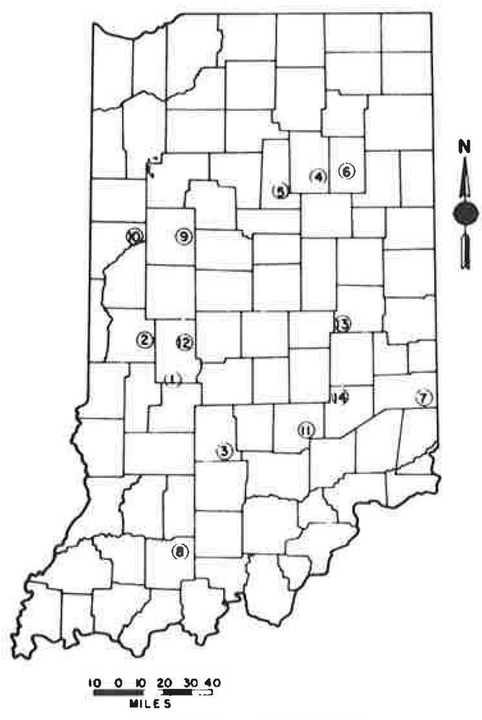
Future highway planning must take into consideration the traffic generating capabilities of this type of recreational facility. There is no point in having a well-developed park that is difficult to reach: the public will not go to such a park in numbers great enough to utilize the investment made in developing the park.

Paper sponsored by Committee on Origin and Destination.

Water is a recreational magnet and will attract large numbers of people for recreational purposes. The multi-purpose dams and their reservoirs are natural recreational attractions and traffic generators. The recreational potential of a reservoir cannot be fully utilized unless transportation planning coincides with reservoir development plans so that an adequate transportation system is available as the recreational demand grows. The agencies responsible for planning must have some means of determining demand prior to construction so that the best use can be made of the available resources of land and money. At the present time, little factual information is available that can be used by planners to estimate the recreational demand. Many reservoir sites are located in areas with poor existing transportation facilities. Usually existing roads were designed for rural traffic of low volumes, and as such, these roads cannot begin to accommodate the traffic generated by a reservoir and its recreational facilities.

PURPOSE AND SCOPE

The flood control projects that have been and are being developed in Indiana produce many reservoirs which are suitable for recreational purposes (Fig. 1). The Indiana Department of Natural Resources is responsible for the development and operation of recreational facilities at such reservoirs. Very little information is available for plan-



MAJOR PROJECTS

COMPLETED	UNDER CONSTRUCTION	AUTHORIZED	PLANNED BUT UNAUTHORIZED
1. CAGLES MILLS	5. MISSISSINEWA	8. PATOKA	12. BIG WALNUT
2. RACCOON	6. HUNTINGTON	9. LAFAYETTE	13. BIG BLUE
3. MONROE	7. BROOKVILLE	10. BIG PINE	14. DOWNEYVILLE
4. SALAMONIE		11. CLIFTY CREEK	

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Figure 1. Reservoirs in Indiana.

ning recreational facilities. No one can accurately say how many sites are needed to satisfy the demand for recreation in a given area. No one knows what effect a reservoir park has on attendance at another park in the same general area.

The Department early in 1965 asked the Joint Highway Research Project at Purdue University to conduct a research program that would develop information that could be used for planning recreational developments at future reservoir sites. Three reservoirs were suggested, two had been in operation for several years and the third was in the process of being opened for public use although few facilities were available. The two developed parks are Lieber State Park on Cagles Mill Reservoir and Raccoon State Recreation Area on Mansfield Reservoir. The third park is located on Monroe Reservoir.

Funding for the project was provided by the U. S. Bureau of Public Roads of the U. S. Department of Transportation and the Indiana State Highway Commission through the Joint Highway Research Project. The purpose of the research was to develop a method that could be used to predict recreational attendance at planned reservoirs based on the characteristics of the recreational facilities, population, the distance from population centers to the planned reservoir, and the influence of other reservoirs in the vicinity. The determination of the growth patterns of attendance at new facilities as compared to established reservoirs is a secondary objective.

The park facilities at the reservoirs are similar in type. Boat launching ramps at various locations around the reservoirs are provided; 5 at Raccoon, 2 at Cagles Mill, and 9 at Monroe. Each boat ramp is provided with paved roads, parking area, and, usually, picnic grounds. These ramps may or may not be located in the main park.

Raccoon and Cagles Mill each have one beach several hundred feet long. Swimming is permitted only at the beaches with adequate lifeguard personnel and control equipment as well as diving boards and bath houses. Monroe is to have two beaches operated by the State and one operated by the U. S. Forest Service. These will be similar to the ones at Raccoon and Cagles Mill.

The camp grounds, beaches, concession stands, boat rentals, picnic areas, and bath houses are located within the main park at Raccoon and Cagles Mill. There are hiking trails available. In general, each park is well kept by personnel who know and take pride in their work. The recreational facilities available at each park are similar and it is difficult to visualize what additional types of facilities would be useful at this type of park.

The need for outdoor recreational areas can only increase. The Midwest as a region has 29 percent of the population of the 48 contiguous states, but only 12 percent of the recreational acreage (2). The use of flood control reservoirs for recreational purposes can provide a substantial portion of the needed public recreational areas and every effort should be made to utilize such areas in the most efficient manner for the benefit of the public.

Proper utilization of these facilities will require an adequate highway system. The purpose of this research was to provide a simplified method for estimating future traffic volumes for new facilities of this type.

DATA COLLECTION

In order to acquire sufficient data for the study, collection was made over a period extending from June 1965 until October 1966 at all three parks. Figure 1 shows the location of the three reservoirs. Data collection was begun early in the planning stage of the project in order to take full advantage of the summer season of 1965.

The primary source of data was a 25 percent interview of vehicular trips arriving at the parks. The 25 percent sample was considered adequate for analytical purposes and did not create a disruption in traffic flow. Each interview took approximately 20 seconds.

The number on Indiana passenger car licenses includes the number of the county of residence of the listed owner. This was recorded and used as the county of origin for the trip. The driver was asked the purpose of the visit; the number of adults and children were determined. Children were considered to be persons under 12 years of age since no charge is made for admittance of persons under 12. Note was made of any equipment carried such as a boat, house trailer, or camping trailer. Time of day, date, park and place of entry (main gate or isolated boat ramp) were recorded.

The interviews were conducted at the gatehouses at Raccoon and Cagles Mill. No gatehouses were in operation at Monroe during this period. The advantage of conducting interviews at the gatehouses was that the vehicles were already stopped in order to pay fees, and no further disruption of traffic was necessary. Also, it was possible to determine which vehicles had already paid and thus duplication of interviews was prevented. Vehicles on park business were not charged admission and were not included in the sample. It was therefore possible to exclude all those vehicles which were not entering the park for the first time. Unfortunately, this was not possible at the boat ramps at Raccoon or Monroe; however, the volumes at the boat ramps were low enough so that it was possible to ask if the trip had entered the park previously, without causing an undue delay to traffic. At Cagles Mill there is an attended gatehouse at the only isolated boat ramp.

The majority of the interviews were conducted over the weekend periods, from Friday afternoon to Sunday afternoon, during the summer months. Weekends were selected randomly. During 1965, the parks were visited every two weeks beginning early in June and continuing through August. Raccoon was visited one weekend and Cagles Mill and Monroe the next weekend throughout the summer. Periodic visits were made during the fall and winter and also during the spring of 1966, in order to determine the yearly distribution of trips. During the 1966 summer season, visits were made to each park every third weekend. Weekday visits were made in June and August only.

The general procedure for weekends was to begin at 2 p. m. on Friday and interview until 9 or 10 p. m. On Saturdays, interviewing would begin at 9 a. m. and continue until 8 p. m. On Sundays, interviewing would begin at 9 a. m. and continue until 5 p. m. The hours were selected on the basis of observations made at Raccoon Park. After about 9 p. m. on Fridays, few arrivals were noted, and few arrived before 9 a. m. on any day of the week. The parks were open 24 hours a day during the summer, but interviews were conducted only during the stated hours. The park records on attendance showed that on weekends the arrivals during the interview period usually accounted for about 90 percent of the total visitors on Saturdays and Sundays and about 75 percent on Fridays. Weekday interviews were conducted in essentially the same manner as were the weekend interviews.

PREPARATION OF DATA

The data were summarized by means of the IBM 7094 computer utilizing FORTRAN IV. The large number of data items precluded any attempt at hand calculation. Over the 2-yr period, 13,340 samples were collected.

Since the visitors were asked to state the purpose of their visit, many multiple purposes were stated. Most trips to a reservoir are probably made for more than one purpose; however, in this study only the stated purposes were recorded since these were considered to be the purposes which inspired the trip. No effort was made to determine if, in fact, the stated purposes were actually accomplished. The fact of interest was what attracted the visitor to the park, not what he actually did once he had arrived at the park. The trip purposes considered were boating, camping, fishing, picnicking, swimming, hiking, looking, and others.

Some trip purposes were not compatible with multiple listing. Looking and other categories were not listed with multiple purposes. For instance, a trip purpose given as "boating and looking around" was classified only as "boating." A camping trip which also lists picnicking as a purpose does not logically make sense, as an overnight camping trip without meals is hardly feasible. Some of these trips (camping and picnicking) may have appeared on the summation sheets but they were summed with the camping trips for analytical purposes. A trip for which boating and swimming were given as the reasons for making the trip could not justifiably be counted as boating rather than swimming or vice versa as no reason existed for making an arbitrary judgment as to what category in which to place the trip. A multi-purpose trip of swimming and boating could not be listed as both a swimming and a boating trip. The solution to this problem was to list each trip as a separate entry.

TABLE 1

SAMPLING DAYS AND EXPANSION FACTORS

Park	1965 Sampling Days	1965 Expansion Factor	1966 Sampling Days	1966 Expansion Factor
Raccoon	107	0.287	128	0.342
Cagles Mill	79	0.210	75	0.200
Monroe	75	0.200	80	0.212

A summation program was used to determine the trips to each park for each year from each county in Indiana and Illinois as well as from other states. The trips from each county could be determined also by type, that is boating trips from each county to each park could be determined. The number of people, or adults and children, could be determined in the same manner as were the number of trips.

When county trip totals were determined, it became apparent that over 80 percent of all trips came from within 125 miles of a park. For the purpose of this analysis, no counties beyond 125 miles of the closest park were considered. The observed trips per county beyond this range were so few as to be insignificant.

In order to standardize the trip rate from any particular county, a unit of measure was selected as trips per 1000 population. There is a large variation among county populations. Marion County contains 785,000 people, while Union County contains 6000. Obviously, the total number of trips from the two counties will vary even if the distances to a park are the same. The use of a trip rate will tend to normalize the disparity of population differences.

The trips from each county were converted into trips per 1000 population. Observed trips were divided by a factor, called the expression factor which is the percentage of all trips to a park that were sampled in a year. The total trips were obtained from the

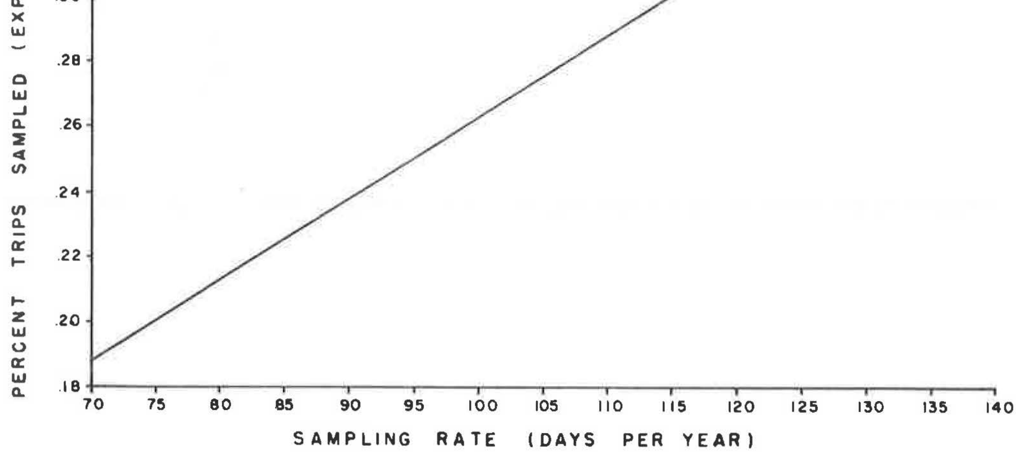

Figure 2. Expansion factors.

Department of Natural Resources' weekly tally sheets which were available for 1965 and 1966. A sampling day is a day of weekday sampling. If a weekend day, Friday, Saturday, or Sunday, was used, a multiplier of five was used as weekend days produce about five times as many trips as a weekday day. Therefore, one weekend produces 3 x 5, or 15 sampling days. The total sampling days for each park and each year are given in Table 1.

The observed trips from a county were multiplied by four to reflect the 25 percent sample. The resulting value was then divided by the expansion factor from Figure 2, determined from the number of sampling days in order to obtain the total annual trips from that county to a park. By dividing the total trips, or boating, swimming, camping, or picnicking trips by the county population in thousands, the trip rate for any desired trip purposes can be obtained.

The county population data projections for 1965 and 1966 were linear projections of the 1950 and 1960 census data (3). The trip rates for each county for both years, to all three parks were computed for total, boating, camping, picnicking, and swimming trips.

The distance figures were developed from the center of each county to the center of each park. Road miles were measured using the primary highway system.

ANALYSIS

Development of Prediction Model

A normal plot of the trip rates versus distance of the various counties from a reservoir produced a curved line. A plot of the same data on a semilogarithmic graph produced a straight line, indicating that an exponential type of function should describe the trip rates in terms of distance. This result was expected since the relationship between trip frequency and distance has been shown to be exponential (4). The relationship is based on the premise that a trip desires to be as short as possible; a person making a trip for any purpose will usually go no further than is necessary to satisfy the purpose for which the trip is being made.

For determing the trip rate, the function used was:

$$Y = A e^{-Bx} \tag{1}$$

where

Y = trips per 1000 population from a county to a reservoir,
A = Y intercept of nonlinear regression curve,
e = base of natural logarithms,
B = rate of change of nonlinear regression curve, and
x = distance in tens of miles from a county to a reservoir.

Two regression curves were developed; one is to be used for counties that are closest to the specified park and the other for trips to a park from a county that is not the closest. Trips were observed from a particular county to more than one park. If the assumption that a trip desires to be as short as possible is correct, then the characteristics or the parameters of Eq. 1 should be different for each case. Case one is the condition where there is no park closer to a county than the park under consideration. Case two is the condition where one or more parks are closer to the county than the park under consideration.

In accordance with these conditions, the counties were separated into two groups for each park. All counties that were closer to the park considered than to any other park were placed into one group. The other group contained all counties that were closer to a park other than the one under consideration.

Monroe was not considered for the purposes of estimating the parameters because the park was not fully operational during this period. No beaches were open for swimming, only extremely limited camping and picnicking facilities were available, and fishing was not permitted. Only boating could be considered to be in normal operation at Monroe. The road network serving the area was inadequate. The roads were narrow, winding, and mostly unpaved.

In 1966, Raccoon Park did not permit swimming because the reservoir pool level was too low. For this reason, the 1966 data for Raccoon were not included in this portion of the analysis.

Eq. 1 is nonlinear in that the function is not linear for the parameters; it is not of the linear form

$$Y = B_0 + B_1Z_1 + \ldots + B_nZ_n + \epsilon \tag{2}$$

The estimation of the parameters by the method of least squares can be made by using a logarithmic transformation into the form

$$\ln Y = \ln A - Bx \tag{3}$$

This approach assumes the errors in the original function are multiplied and are therefore additive in the transformed model. The logarithmic transformation may not give good estimates of the parameters because the transformed model is not the same function as the original (5).

A nonlinear regression analysis was used to obtain the parameters A and B. The method selected was a minor variant of SHARE 3094 (6). This program finds the estimates of the parameters A and B in the function $Y = Ae^{-Bx} + \epsilon$ by minimizing

$$\Sigma\, \epsilon^2 = \Sigma\, (Y - \hat{Y})^2 \tag{4}$$

where ϵ is the residual error and $\hat{Y}$ is the estimate of Y. In order to solve $\Sigma\, \epsilon^2 = \Sigma\, (Y - \hat{Y})^2$, the partial differential equations of $Y = Ae^{-Bx}$ with respect to the parameters must be used since

$$\frac{\partial(\Sigma\, \epsilon^2)}{\partial \hat{A}} = -2\, \Sigma\, (Y - \hat{Y})\, \frac{\partial \hat{Y}}{\partial A} = 0 \tag{5}$$

and

$$\frac{\partial(\Sigma\, \epsilon^2)}{\partial \hat{B}} = -2\, \Sigma\, (Y - \hat{Y})\, \frac{\partial \hat{Y}}{\partial B} = 0 \tag{6}$$

are the normal equations which must be solved simultaneously for $\hat{A}$ and $\hat{B}$, the estimates of A and B.

The required partial differential equations were

$$\frac{\partial \hat{Y}}{\partial A} = e^{-\hat{B}x} \tag{7}$$

and

$$\frac{\partial \hat{Y}}{\partial B} = x\hat{A}e^{-Bx} \tag{8}$$

These two equations were substituted into Eqs. 5 and 6. The equations may be solved by finding the values of A and B which will minimize Eq. 4. This is the method used by the SHARE 3094 program. It is an iterative technique which requires an initial estimate of the true parameters.

The initial parameters were developed by estimating parameters for a linear transformation of the data for the total trips to Raccoon for 1965 and to Cagles Mill for 1965 and 1966. The results were such that a value of 250 was selected as the initial estimate for A and a value of 0.466 was selected for B. A similar procedure was used for intervening parks with the resulting estimates of 120 for A and 0.466 for B. The values used proved satisfactory as initial estimates.

Using the data for total trips, a linear transformation of the form $\ln Y = \ln A - Bx$ was made for the purpose of testing if the various regression lines produced could be considered parallel (7). A standard F test using analysis of variance techniques which compared the variance between the individual slopes and the variance about the individual lines using the mean squares s_1^2 and s_2^2, respectively, showed the following variance ratio:

$$\frac{s_2^2}{s_1^2}(2, 82) = \frac{0.776}{0.368} = 2.108 < 2.35$$

for an α level of 0.10. The hypothesis is that the lines are parallel and since s_2^2/s_1^2, distributed as $F(2, 82)$ where 2 and 82 are the degrees of freedom for s_2^2 and s_1^2, is less than 2.35 it cannot be said that the lines are not parallel. The α level was chosen to be relatively high in order to reduce the chance of making a type II error, or to accept the hypothesis when in fact it is false. It was thought to be more advantageous to accept a higher probability of making a type I error or to reject the hypothesis as false when it actually is true (7). If the lines produced by the transformed equation can be accepted as being parallel under the above conditions, it should be safe to conclude that the actual nonlinear regression lines are also parallel; that is the slopes of the lines can be considered equal. This approach was used because there is no satisfactory way to perform an analysis of variance for the nonlinear case.

The number of iterations required to estimate the parameters was usually less than 10 and in no case was a computer force off used because the number of programmed iterations had been exceeded. The initial parameters are then used to calculate an estimate of A and B. The new estimate is then used to get a better estimate. This process continues until a satisfactory answer is reached.

An iterative technique for estimating parameters of a nonlinear system may or may not work satisfactorily depending on the form of the iterative technique used. The SHARE program used for this research employs the method known as Marquardt's compromise (8). This method is a compromise between the linearization (or Taylor series) and the steepest descent methods (6). Its chief advantage is that it seems to be applicable to a greater range of problems than the two other methods. Taylor series may not converge as it is a linear form. Marquardt's compromise method almost always converges and does not slow down as does the steepest descent method which often converges very slowly and often requires changes in scale. For nonlinear problems, no particular method of iteration can be considered best because for a particular problem, modification of any method may result in quicker convergence. A satisfactory answer is one which satisfies the criteria imposed.

Several criteria for stopping are available. When the slope of $\Sigma (Y - \hat{Y})^2$ is near zero; that is, when the partial derivatives approach zero, the criteria are satisfied. In the SHARE program the value of the slope is considered to be near zero when the actual value is less than 0.0001. There are two additional ways in which the SHARE program may be satisfied, when the changes in A and B become too small for an iteration or when any predetermined number of iterations have been made. In this case the standard convergence criteria supplied with the program were used. The Epsilon Test was used to determine convergence (6). This test is passed whenever

$$\frac{|\delta_j|}{\tau + |b_j|} < \epsilon, \text{ for all } j \tag{9}$$

where

b_j = value of the jth parameter,
τ = constant used in convergence test (10^{-3}),
ϵ = convergence criteria (5×10^{-5}), and
δ_j = increment to b_j.

The program produces 10 sets of parameters, A and B, for use in Eq. 1. The equations developed are to be used to predict trip rates for the total trips, boating, swimming, picnicking, and camping trips for both cases. The 1965 data for Raccoon and Cagles Mill were used as well as the 1966 data for Cagles Mill. The values were averaged for each category and the average values of A and B were used to produce straight line plots on a semilogarithmic graph for estimating purposes (Figs. 3 and 4).

Using estimating lines for trips to the closest park and intervening park, one is able to produce over 95 percent of the total trips to Raccoon for 1965 and to Cagles Mill for 1965 and 1966. This estimate (95 percent) is considered to be entirely adequate for future planning purposes.

Standard statistical tests such as simultaneous significance tests for multiple contrasts in the analysis of variance were run on the trips to the parks for the various trip purposes (9). The trips were converted to percentages in order to account for the difference in total observed trips which resulted from the differences in the number of observation days as well as the differences in the annual attendance at each park. Of interest in these tests was whether or not there were any significant differences for the various trips due either to parks or to years.

These tests showed that there existed no significant difference between Raccoon and Cagles Mill for the purpose of boating, picnicking, and swimming. From this, it is inferred that the attraction of each park is the same and the only difference in trips ar-

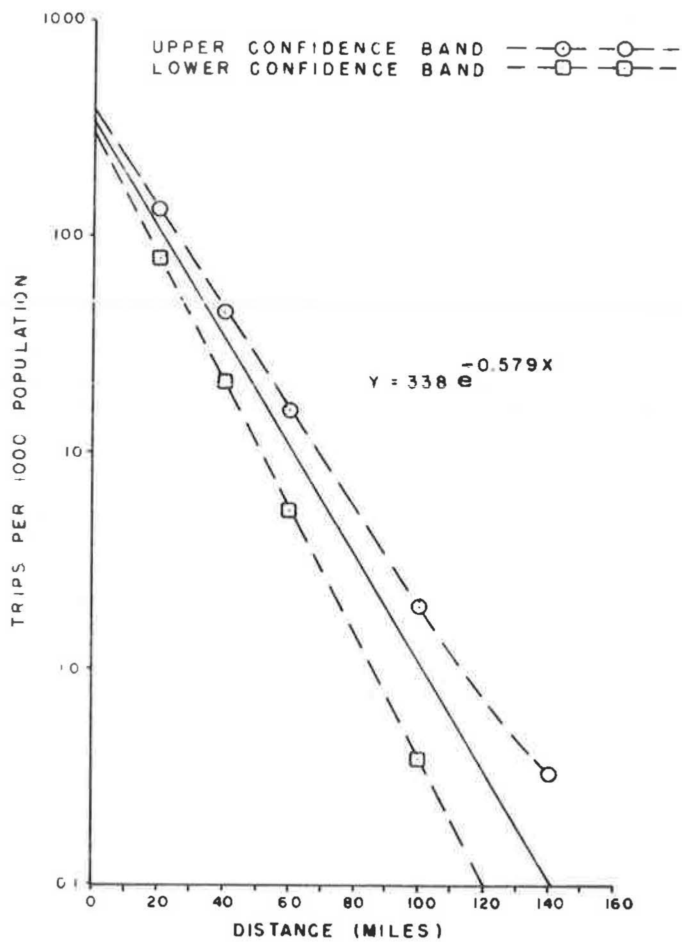

Figure 3. Total trips to closest park.

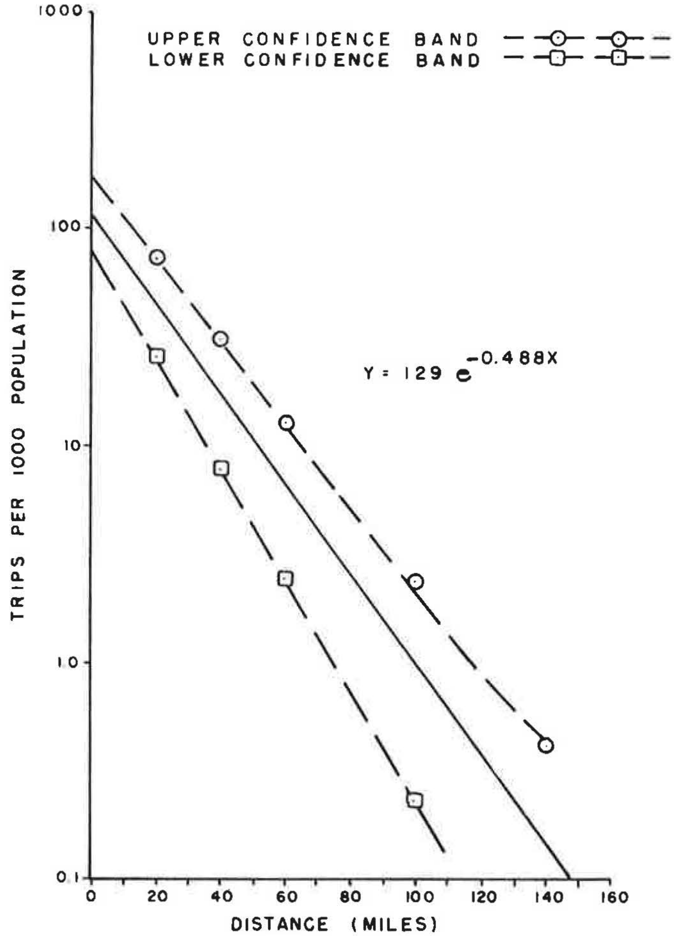

Figure 4. Total trips with intervening park.

riving at each park can be attributed to the population distribution around eack park. More of the population is closer to Raccoon than to Cagles Mill. The fact that the estimating line for total trips to the closest park has a higher X intercept (338.4 vs 129.3) than the line for total trips to a park with an intervening park, validates the assumption that a recreational trip desires to be as short as possible.

For the two primary curves (Figs. 3 and 4), confidence intervals were computed by determining the lines for the average values of the upper and the lower confidence bands as determined by the standard convergence criteria. The lines appear to diverge as X

TABLE 2

SINGLE PURPOSE TRIPS IN PERCENT OF TOTAL ANNUAL TRIPS

Activity	Raccoon 1965	Cagles Mill 1965	Cagles Mill 1966
Boating	11.1	9.6	18.2
Camping	3.2	4.8	5.1
Picnicking	6.5	5.7	6.5
Swimming	12.3	12.4	15.2

TABLE 3

MULTI-PURPOSE TRIPS IN PERCENT OF TOTAL ANNUAL TRIPS

Activity	Raccoon 1965	Cagles Mill 1965	Cagles Mill 1966
Boating	36.9	37.6	47.1
Camping	18.4	25.7	13.4
Picnicking	32.3	36.4	21.6
Swimming	38.1	55.6	31.9

TABLE 4

TRIP PURPOSE IN PERCENT OF TOTAL ANNUAL TRIPS, AVERAGED FOR ALL PARKS

Activity	Percent
Boating	40.0
Camping	19.9
Picnicking	30.0
Swimming	42.0

TABLE 5

AVERAGE TRIP PURPOSE IN PERCENT FOR JUNE, JULY, AND AUGUST

Activity	Raccoon 1965	Cagles Mill 1965	Cagles Mill 1966
Boating	36.0	36.0	33.0
Camping	18.6	21.0	16.0
Picnicking	32.0	33.3	25.5
Swimming	42.0	55.1	45.5

increases, but actually the confidence interval decreases as X increases. The largest value of the confidence band occurs when X is zero. This is apparently because fewer observations are available for the smaller values of X than are available for the larger values of X. In other words, fewer counties are closer to the parks than are further from the parks, using 50 miles as a division point.

Trips by Purpose

In order to determine what percentage of the total trips each trip purpose produces, two tables were developed. Table 2 gives the percentage of total trips contributed by each single purpose; no multi-purpose trips are included. Table 3 is considered to be more useful in explaining the trip purposes because it gives the multi-purpose trips as well as the single purpose trips for each purpose.

One conclusion is that swimming is the most preferred activity; followed in order by boating, picnicking, and camping (Tables 4 and 5).

Arrival distributions were plotted to determine the arrival patterns both for total trips and for each trip purpose. Considerable variations exist among the days of the weekend (Figs. 5, 6, and 7). The only major difference between parks was noted in the magnitude of trips per hour. This effect however, has already been explained as being due to the population distribution around the parks.

The values for the arrival distributions were obtained by averaging the summer weekend observations for June, July, and August. Less than 30 percent of the total annual trips to a park are made before June (Fig. 8). By the end of August, more than 90 percent of the total annual trips have been made. The arrival rate for the months other than June, July, and August will be much less than those plotted. These figures may be used as the average arrival rates and daily distributions for the parks involved.

The maximum number trips observed in one day was a Sunday when 1348 trips were sampled at Raccoon State Recreation Area.

Figure 9 shows the trips rates by purpose. The values were determined in the same manner as were the values for total trips. The plots show the relative attractiveness of each activity.

Of interest is the relationship of the curves to each other in terms of distance. For distances of less than 30 miles, swimming is the largest trip producer; beyond 45 miles,

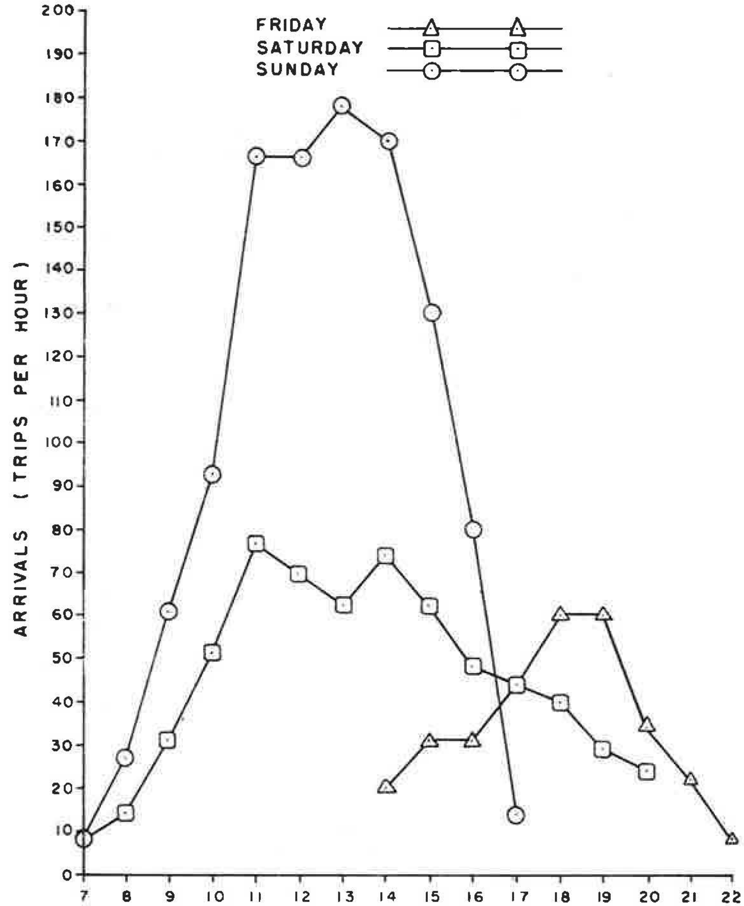

Figure 5. Total trip arrivals, Raccoon, 1965.

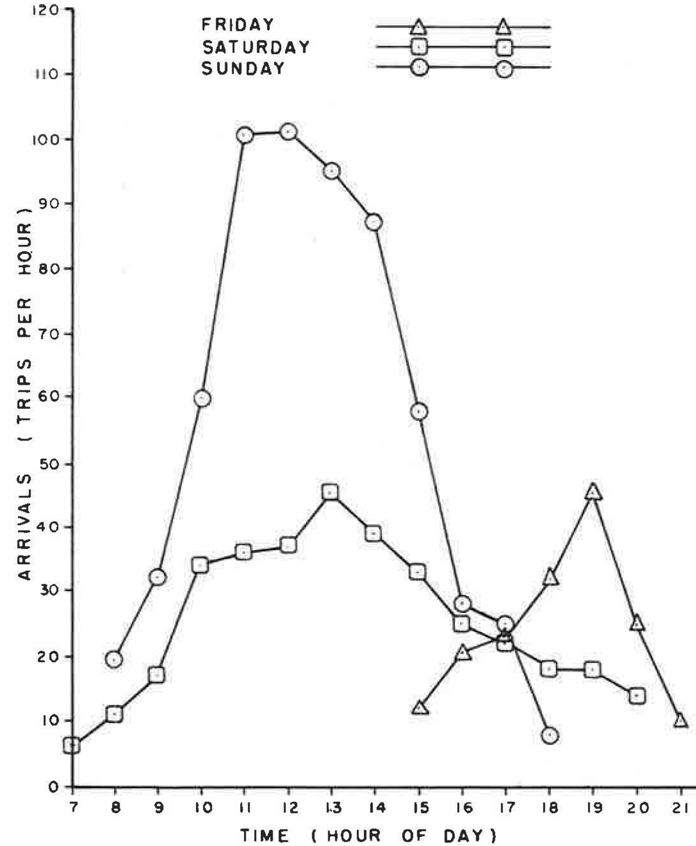

Figure 6. Total trip arrivals, Cagles Mill, 1965.

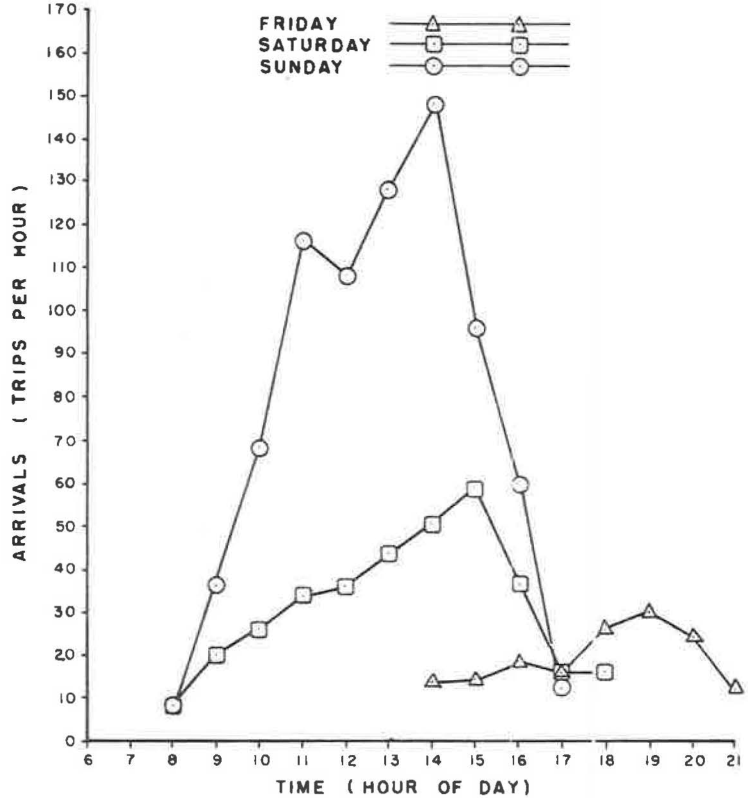

Figure 7. Total trip arrivals, Cagles Mill, 1966.

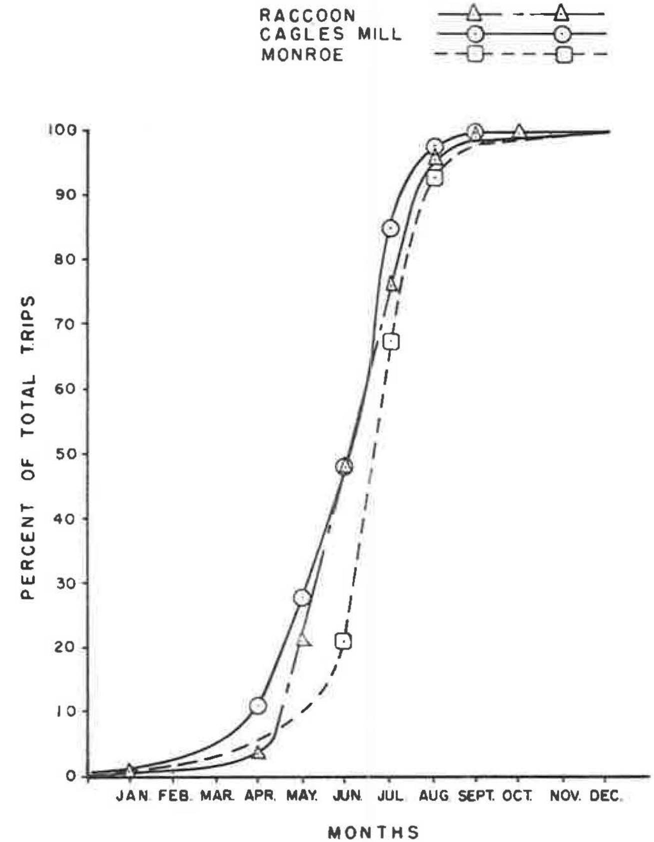

Figure 8. Cumulative trips.

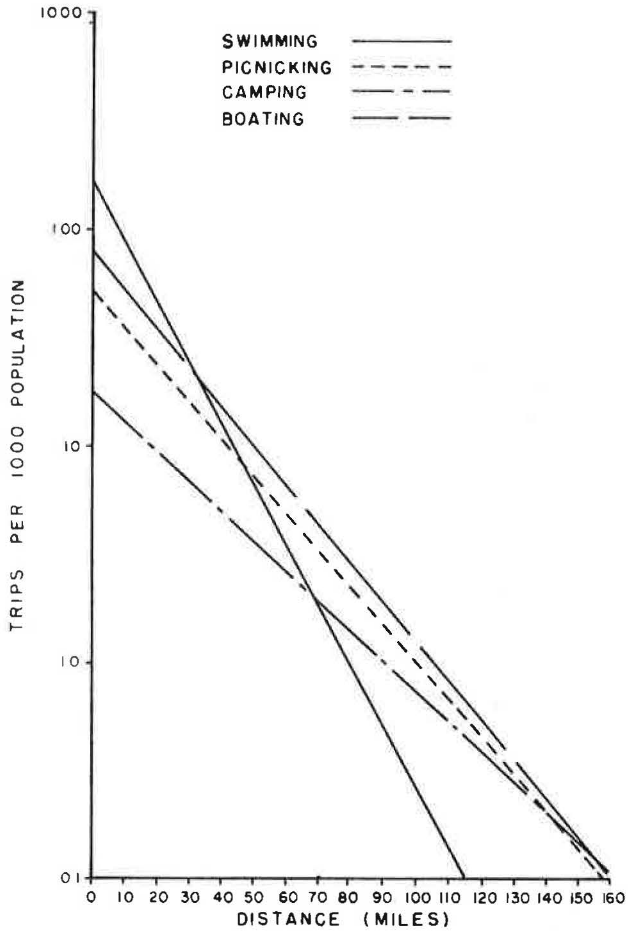

Figure 9. Trip rates by purpose for closest park, 1965.

TABLE 6

AVERAGE NUMBER OF PERSONS PER TRIP

Park	1965	1966
Raccoon	3.72	3.40
Cagles Mill	3.84	3.63
Monroe	3.80	3.63

picnicking is the most attractive; and camping is the most attractive beyond 70 miles. The curves for boating, picnicking, and camping tend to converge with an increase in distance which indicates that as the distance increases the trip purpose has less effect on the trip rate. It may also indicate that more multi-purpose trips are made for longer distances than for shorter distances.

The curve for swimming reflects the fact that swimming, as a separate activity, can be satisfied closer to home than can most other activities. Also, the high rate for short distances reflects the desire of many people for a short duration trip for a swim. This can also be inferred from the arrival distributions which show the swimming arrivals to be later in the day.

Persons Per Trip

The average number of persons per trip is given in Table 6. The values were obtained by dividing the total number of people sampled who visited a park in a year by the

TABLE 7

PERCENT OF TOTAL ANNUAL TRIPS OCCURRING ON MAXIMUM VOLUME WEEKEND

Year	Park	Total Annual Trips	Maximum Weekend Volume	Percent of Total
1965	Raccoon	57, 146	2, 778	4.8
1965	Cagles Mill	30, 695	2, 431	7.9
1966	Cagles Mill	41, 322	3, 329	8.1

total number of trips sampled in that same year. There is no significant difference among the values, the average value being 3.69 persons per trip.

Maximum Volume

Some knowledge of the maximum expected weekend volume could be of interest to planners. To determine the maximum weekend attendance, the maximum number of trips for a weekend for Raccoon and Cagles Mill were found for 1965 and 1966. The Department of Natural Resources weekly tally sheets were used to obtain the exact number of trips for Cagles Mill in 1966. Sufficient data for 1965 were not available since only abbreviated tally sheets were made available. For this reason, the observed study values for the highest volume weekend were used. The results are given in Table 7. The average value is 6.9 percent.

The maximum number of trips occur on Sunday. For any weekend when weather conditions are similar, Sunday will have the maximum volume of arriving trips. For the maximum weekends in Table 7, the daily breakdown is listed in Table 8.

For planning purposes, the maximum volume that will be expected can be computed easily from the estimated total arrival trip volume. Since the Sunday trips of the maximum volume weekend account for, on the average, 53.5 percent of the total trips, 0.535 multiplied by 6.9 percent of the total annual trips will give an estimate of the maximum number of trips that can be expected in one day. Therefore, approximately 3.7 percent of the total annual trips can be expected as the maximum daily volume.

CONCLUSIONS

The objective of this research was to develop a method of predicting attendance for recreational purposes at new reservoirs. The method developed appears to be an effective tool for predicting future recreational attendance at reservoir parks. The model $Y = Ae^{-Bx}$ is able to produce accurate trip estimates to a reservoir if the trip rates are placed into two categories, trips to the closest park and trips with an intervening park.

The model for the case of total annual trips to the closest park is $Y = 338.4e^{-0.5791X}$. When there is an intervening park, the model becomes $Y = 129.3e^{-0.4875X}$. Distance measured in tens of miles and population measured in thousands are the two variables necessary to use the equations which will produce annual trip rates for a county.

The method is able to predict future attendance with reasonable accuracy based on distance, population, and the influence of similar parks. Previously developed models required many socioeconomic and park characteristics variables which are difficult to measure and evaluate and extremely difficult to project (10). The model developed is probably as accurate and is much simpler to use. The model is adequate for advanced planning purposes and can be used to predict reservoir attendance and traffic volume estimates.

TABLE 8

DAILY TRIPS ON MAXIMUM VOLUME WEEKEND

Year	Park	Sunday	Saturday	Friday	Percent of Weekend Trips Occurring as Sunday Trips
1965	Raccoon	1, 419	865	494	51.0
1965	Cagles Mill	1, 276	748	407	52.5
1966	Cagles Mill	1, 938	766	625	57.2

Statistical evidence indicates that there is no significant difference between parks of similar type with regard to their ability to attract visitors. There is one attraction rate for trips to a reservoir that is closer to the point of trip origin than another reservoir and another attraction rate for trips when there is another reservoir closer to the point of trip origin than the reservoir being considered. The difference in attraction rates substantiates the assumption that a trip desires to be as short as possible. Generally, people will not go past a park to get to another that has similar facilities.

The county trip rate to a park is also directly related to the distance between the county and the park; the longer the distance, the smaller the trip rate per 1000 population for that county.

The attendance at any proposed site is dependent on the population, its distance from that park, and the location of other similar parks with regard to the location of the population. These parameters are easily understood and readily available to any planning agency. Distance can be determined from an official state highway map. Population data and projection techniques are common tools for the planner (11).

Growth of the trip rates is a possibility, but was not included in this report because a 2-yr time period is not sufficient for an accurate examination of the possible changes in trip rates. The continuing phase of this project should investigate possible growth of the trip rate.

The trip rate appears to be a handy tool. The high correlation index values (R^2) for total trips (0.84 to 0.97) indicates that the trip rate technique was effective in eliminating county population variations. The correlation index values also tend to validate the assumption that socioeconomic factors and park facility quantities can be considered uniform in that all parks tend to draw trips from what is essentially a uniform cross section of all types of social and economic population groups.

Additional work is contemplated which will test the stability over time of the proposed trip rates and will evaluate travel time as a factor in lieu of travel distance.

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An Evaluation of the Influence of Terminal Times on Gravity Model Travel Time Factors

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This paper presents the findings of an investigation to determine the influence of including or excluding terminal times in the development of travel time factors (friction factors) for a gravity model traffic analysis for a small urban area. This investigation was made by developing travel time factors for the urban area of Rock Hill, S. C. using two procedures. The first procedure developed the factors using only over-the-road driving time as a measure of spatial separation. The second used total travel time (over-the-road driving time plus terminal time) as a measure of spatial separation. Internal auto driver trips for purpose of home-based work, home-based non-work, and non-home-based and truck trips were included in the analysis.

Separate assignments to the street network were made of the trips reported in the home interview and the trips developed by the two gravity models.

Statistical tests comparing the assigned link volumes and CBD zonal interchanges indicated that both gravity models reproduced adequately the trip distribution patterns of the home interview data. The travel time factors developed using both driving time and terminal times were found not to differ significantly from those using only driving time, except in the case of non-home-based trips.

It was concluded that while there were basic differences requiring further investigation the gravity model provided an adequate framework for determining trip distribution patterns using either the model with terminal times or the model without terminal times.

•TO provide a framework for sound decision-making in developing transportation networks, reliable forecasts of future travel must be developed. With these forecasts, proposed alternate transportation systems may be tested and analyzed for the services which they will provide, comparing service benefits of each system with estimated costs.

Two of the key phases in the forecast of future travel patterns are trip distribution and traffic assignment. These phases provide the quantitative data on travel needed to properly plan transportation facilities. The traffic assignment techniques provide an estimate of the probable traffic on each segment of a transportation network. The need for accurate, reliable traffic assignments has accelerated the development of various procedures capable of synthesizing zone-to-zone movements for alternate configurations of land use and transportation facilities. These procedures provide for distributing the trips emanating from each zone in the study area to other zones. Several such procedures, generally referred to as traffic models, have been developed by various organizations throughout the country. The model which has been most widely applied is the so-called gravity model.

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In May 1964, the University of South Carolina entered into a contract with the South Carolina State Highway Department to perform certain technical phases of the Rock Hill Area Transportation Study. One phase of that study required the calibration of a gravity model for the Rock Hill area suitable for use in the development of future travel patterns.

The development of this model would not have been unusual except for the fact that the agreement required the development of travel time factors (friction factors) using both over-the-road driving time alone and total travel time (over-the-road driving time plus terminal time), as a measure of spatial separation, and to evaluate the difference, if any, in these factors.

To date most of the O-D studies which have used the gravity model to develop future travel patterns have subscribed to the theory that terminal times are necessary to obtain reliable travel time factors; while this has been widely accepted, studies have not been carried out to verify the necessity of using terminal times, especially in small urban areas.

Rock Hill is a small urban area located in York County in the north-central portion of South Carolina. The location of Rock Hill in relation to some other urban areas is shown in Figure 1. The study area (Fig. 2) has a population of approximately 40,000, of which 29,500 are within the city limits. The economic base of the area is primarily the textile industry.

GRAVITY MODEL THEORY

The gravity model adapts the Newtonian gravitational concept to the distribution of urban travel patterns. It employs the concept that the interchange of trips between zones in an urban area is dependent upon the relative attraction between the zones and the spatial separation between them as measured by an appropriate function of distance (1). This function of spatial separation adjusts the relative attraction of each zone for the ability, desire, or necessity of the trip maker to overcome the spatial separation between the zones.

In early uses of the gravity model, the mathematical form of the model was used and the exponent b was determined empirically. Early studies have shown that the exponent of travel time varies from 0.5 up to 3.0 depending upon the importance of the trip purpose. In addition to the variation of the exponent by trip purpose, Voorhees has

Figure 1. Location map: Rock Hill, South Carolina.

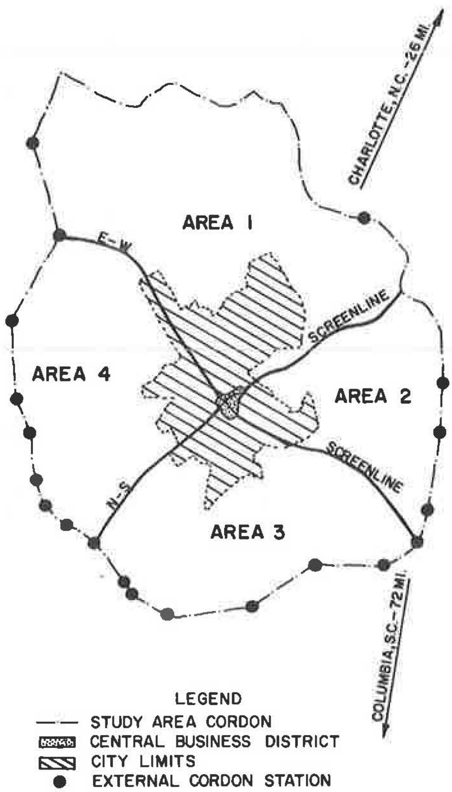

Figure 2. Rock Hill Study Area.

shown that the exponent may not remain constant but will increase as the spatial separation increases (2). This is particularly true where terminal times are not added to driving times in determining spatial separation. However, a constant exponent expresses the areawide effect of spatial separation on trip interchange as a linear logarithmic function of travel time.

To overcome the restriction of linearity for the travel time function and to simplify the computational requirements of the model, later studies have made use of the following form of the model:

$$T_{i-j} = P_i \frac{A_j F_{(t_{i-j})} K_{i-j}}{\sum_{x=1}^{n} A_x F_{(t_{i-x})} K_{(i-x)}}$$

where

T_{i-j} = trip produced in zone i and attracted to zone j;

P_i = trips produced by zone i;

A_j = trips attracted by zone j;

$F_{(t_{i-j})}$ = empirically derived travel time factor which expresses the average areawide effect of spatial separation on trip interchange between zones which are t_{i-j} apart; and

K_{i-j} = a specific zone-to-zone adjustment factor to allow for the incorporation of the effect on travel patterns of defined social or economic linkages not otherwise accounted for in the gravity model formulation.

The use of a set of travel time factors to express the effect of spatial separation on zonal trip interchange, rather than the traditional inverse exponential function of time, simplifies the computational requirements of the model (3). The above form of the model allows for a nonlinear travel time function which allows consideration for the effect of spatial separation generally increasing as the travel time increases.

CALIBRATING THE MODELS

In using the gravity model for trip distribution, several decisions must be made as to the type of model to be developed. In small urban areas, these decisions are somewhat more simplified than for larger urban areas. It was decided to calibrate a 24-hr model using total daily vehicular trips with both origin and destination within the study area and off-peak driving time as a measure of spatial separation.

The trips reported in the O-D study and used in this study were stratified into the following purpose categories: home-based auto driver work, home-based auto driver non-work, non-home-based auto driver, and truck.

Two procedures were followed in developing the gravity model trip distribution curves. The first used over-the-road driving time alone as a measure of spatial separation; the second used total travel time (over-the-road driving time plus a terminal time for each end of the trip). Interzonal driving times were obtained through the standard tree building computer program while an estimate of terminal time was made for each zone. This estimate was based on the type and intensity of land development within each zone and were made on the basis of judgment and a knowledge of the particular zones. Two CBD core zones were assigned terminal times of four and three minutes. One highly developed zone adjacent to the CBD was assigned a terminal time of 3 minutes. Other highly developed zones both in and outside the CBD were assigned terminal times of 2 minutes, and residential and relatively undeveloped zones were assigned terminal times of 1 minute. The minimum terminal time which could be assigned was 1 minute. Of the 99 zones in the study area, 12 were assigned terminal times greater than 1 minute. Eight of the ten zones in the CBD were assigned times of more than 1 minute. Intrazonal driving times were estimated on the basis of the average driving time from the zone centroid to all points on the edge of the zone.

Determining Travel Time Factors

The optimum set of travel time factors was developed for each trip purpose category by a process of trial and adjustment. This process has been well documented (5, 6, 7) and will not be explained in detail here. Briefly, the travel time factors F_{i-j} were developed in an iterative procedure which was continued until the synthetic trips calculated for each trip length interval closely matched the O-D trips reported for the same interval. Any convenient set of travel time factors may be used to start the iteration procedure; however, in this study an initial set of travel time factors was developed for each trip purpose category using a straight-line curve fitted to the O-D trip length frequency distribution. These factors, together with zonal productions and attractions and travel time matrices, were used to obtain an initial gravity model estimate of zone interchanges. After comparing the resulting synthetic interchanges with the observed interchanges, the initial sets of travel time factors were revised to produce more accurate results. These revisions were made on the basis of comparing the overall trip length frequency distribution curve of the gravity model with that of the actual O-D interchanges. The process was repeated until acceptable criteria were met (5). The completion of the calibration process produces a set of travel time factor curves for each trip purpose which together with projected productions and attractions is used to develop a future trip matrix. In addition to the travel time factor curves, there is a synthetic trip matrix for the total trips available for comparison with the reported O-D trip matrix. The O-D and synthetic trip matrices can be statistically compared directly or assigned to a street network for comparison.

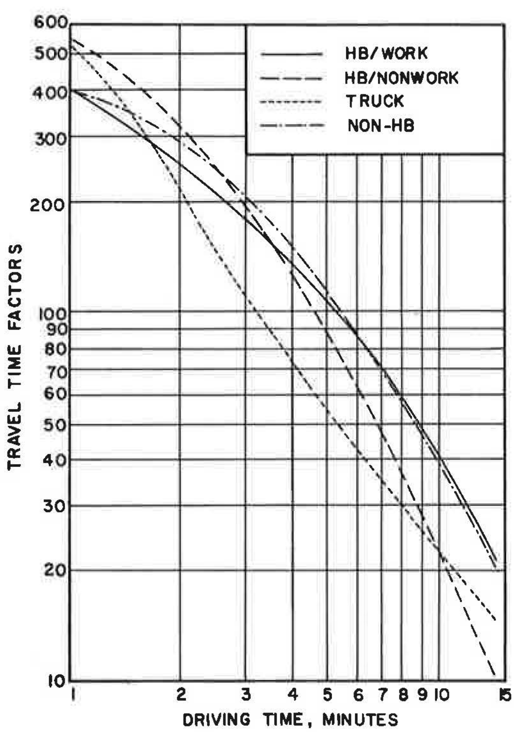

Figure 3. Travel time factors without terminal times.

Developing the travel time factor curves using two procedures resulted in two sets of travel time factors available for comparison. The travel time factors developed from the first procedure using driving time alone are shown in Figure 3. All of the curves shown, with the exception of that for trucks, have a concave shape and could not be approximated with a constant exponent. The concave shape of these curves is undoubtedly due to the absence of zonal terminal times.

Following the development of the travel time factors using driving time only, the model was calibrated using total travel time. The initial set of travel time factors used for this model was obtained by adding

2 minutes to each time interval of the factors developed for the model with no terminal times. Since the minimum terminal time added in any zone was 1 minute, it was necessary to add two minutes to the previous travel time factors in order to make a valid comparison of the travel time factors for a gravity model both with and without terminal time. This resulted in the minimum trip time possible being 3 minutes since the minimum intrazonal time was 1 minute. Using the travel time factors developed by this procedure as a first trial, distributions were made for the gravity model and the resulting trip length frequencies for each purpose were compared with the O-D distributions. On this step, two of the four categories of trip purposes were found to be within the limits of criteria as previously established without additional adjustment. The two remaining internal purposes required one further calibration to fulfill the established criteria.

The travel time factors developed from the second procedure using total travel time are shown in Figure 4. Two of these four curves (home-based work and home-based non-work) have the same travel time factors as shown in Figure 3 with the exception of the addition of the 2 minutes to each time interval. The two remaining curves (non-home-based and trucks) are slightly different (Fig. 5). In the original research carried out for the Rock Hill study, an evaluation of the various travel time factor curves and the trip length frequency distribution curves (not shown in this paper) shows that the non-home-based travel time factors are significantly different when terminal times are used, while the difference in the curves for truck trips can probably be attributed to the model being slightly better fitted in the calibration procedure (8). These findings indicate that there is a significant difference in the travel time factors for non-home-based

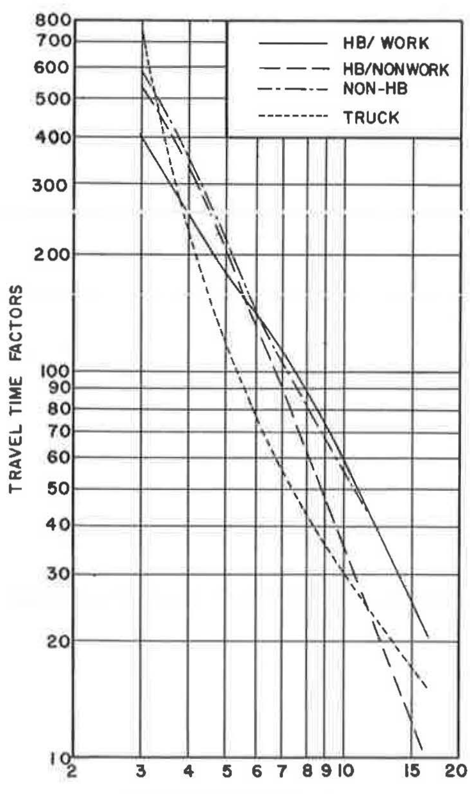

Figure 4. Travel time factors with terminal times.

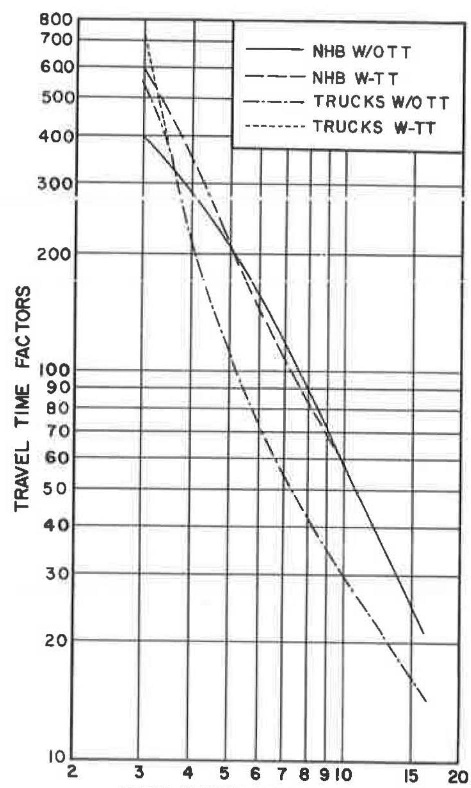

Figure 5. Comparison of travel time factors with and without terminal times.

trips when terminal times are used and when they are not used. This statement should not be interpreted as a conclusion that assignments made to a street network using a distribution made with these sets of travel time factors would be different to the point of influencing a major transportation decision.

These curves, which are a function of total travel time, unlike those which are a function of driving time alone, could, with the exception of trucks, be approximated very closely with a constant exponent.

TESTING THE MODELS

The trip distribution which is produced by the calibrated gravity model is a "synthetic" distribution and is therefore an approximation of actual conditions. It must be realized that variations between the actual and the synthetic conditions developed using mathematical models are inherent in any approximation process. To determine if the models used to forecast future travel patterns are adequate, various statistical tests are made to analyze how well these models reproduce the existing O-D travel patterns. The statistical tests are generally applied only when the gravity model has been calibrated to a trip distribution pattern obtained from an O-D survey.

Comparison of Street Network Assignments

After each of the gravity models had been calibrated, the internal gravity model and O-D trip interchanges were assigned, by the all-or-nothing method, to the existing street

TABLE 1

COMPARISON OF O-D AND GRAVITY MODEL ASSIGNMENTS WITHOUT TERMINAL TIMES

Link Volume	No. of Links[a]	Average Link Volume O-D	Average Link Volume G.M.	Mean Difference	Percent Mean Difference	Standard Deviation	Test Statistic t	Table Value, $t_{\alpha/2;\ n-1}$[b]
0-999	748	347	381	34	9.8	60	15.49[c]	1.96
1000-1999	241	1359	1399	40	2.9	133	4.67[c]	1.96
2000-2999	92	2429	2417	12	0.5	168	0.69	1.99
3000 and over	43	4424	4352	72	1.6	177	2.67[c]	2.02
All links	1124	891	918	27	3.3	110	8.17[c]	1.96

[a]Total links with zero gravity model volume and zero O-D volume, 140; total links with zero gravity model volume and non-zero O-D volume, 6.
[b]5 percent significance level.
[c]Significantly different.

TABLE 2

COMPARISON OF O-D AND GRAVITY MODEL ASSIGNMENTS WITH TERMINAL TIMES

Link Volume	No. of Links[a]	Average Link Volume O-D	Average Link Volume G.M.	Mean Difference	Percent Mean Difference	Standard Deviation	Test Statistic t	Table Value, $t_{\alpha/2;\ n-1}$[b]
0-999	755	349	379	30	8.6	59	13.97[c]	1.96
1000-1999	237	1361	1398	37	2.7	130	4.38[c]	1.96
2000-2999	91	2438	2436	2	0.1	157	0.12	1.99
3000 and over	44	4380	4294	86	2.0	287	1.98	2.02
All links	1127	888	912	24	2.7	109	7.39[c]	1.96

[a]Total links with zero gravity model volume and zero O-D volume, 141; total links with zero gravity model volume and non-zero O-D volume 2.
[b]5 percent significance level.
[c]Significantly different.

network, and a simple statistical analysis of the differences between the actual and synthetic volumes were made for each of the models. The results of these tests for the comparison of the O-D and the gravity model volumes both with and without terminal times are given in Tables 1 and 2, respectively. The tables show the average O-D and gravity model link volume, the difference and percent difference in the average volume, the standard deviation of the difference, and Student's t statistic for each volume grouping. The t statistic results from Student's t test for the equality of means where the observations are paired (9). The observations in this case are the individual network links and the pairing effect results from the network being loaded with the O-D trip distribution and then being tested against the same network loaded with each of the gravity model distributions.

Tables 1 and 2 indicate that the difference in the O-D and gravity model volumes is never large enough to influence a transportation decision. The mean differences between the O-D and gravity model volumes are, in all cases, not greater than 10 percent and the majority of the link volumes show considerably less than 10 percent difference. Three of the volume groupings and the total grouping for the model with terminal times show mean differences and percent mean differences, which are lower than those found for the model without terminal times. The standard deviation of the differences follows the same pattern, although this difference is extremely small.

While it appears that there is no practical difference in the O-D and gravity model values shown, the t statistic indicates that there is a significant difference statistically in several of the volume groupings and in the total grouping for both of the models. The three volume groups which show no significant difference are those with a relatively low number of links. This can be explained in part by the fact that the t statistic value

TABLE 3

COMPARISON OF O-D AND GRAVITY MODEL ASSIGNMENTS WITHOUT TERMINAL TIMES[a]

Link Volume	No. of Links	Average Link Volume		Mean Difference	Percent Mean Difference	Standard Deviation	Test Statistic t	Table Value, $t_{\alpha/2;\ n-1}$[b]
		O-D	G.M.					
0-999	11	552	636	84	15.2	133	2.10	2.23
1000-1999	21	1537	1584	47	3.1	157	1.37	2.09
2000-2999	9	2437	2451	14	0.6	141	0.30	2.31
3000 and over	16	5991	5876	115	1.9	225	2.04	2.13
All links	57	2739	2742	3	0.1	188	0.12	2.00

[a]Includes only those links of the street network located within the CBD.
[b]5 percent significance level.

TABLE 4

COMPARISON OF O-D AND GRAVITY MODEL ASSIGNMENTS WITH TERMINAL TIMES[a]

Link Volume	No. of Links	Average Link Volume		Mean Difference	Percent Mean Difference	Standard Deviation	Test Statistic t	Table Value, $t_{\alpha/2;\ n-1}$[b]
		O-D	G.M.					
0-999	11	552	626	74	13.4	142	1.73	2.23
1000-1999	21	1537	1551	14	0.9	162	0.40	2.09
2000-2999	9	2437	2449	12	0.5	104	0.35	2.31
3000 and over	16	5991	5782	209	3.5	244	4.26[c]	2.13
All links	57	2739	2702	37	1.4	209	1.34	2.00

[a]Includes only those links of the street network located within the CBD.
[b]5 percent significance level.
[c]Significantly different.

is calculated as the product of the mean difference and the square root of the number of observations divided by the standard deviation of the differences. The lower the number of observations, the lower the t statistic. This seems to indicate that where statistical comparisons of traffic assignments such as these are made, the extremely high number of observations (links) tend to show a significant difference in cases where a practical difference may not occur.

To evaluate this phenomenon further, the same comparisons were made for those links in the CBD only. This comparison, which has relatively high average link volumes and a low number of observations for each grouping, is given in Tables 3 and 4 for each of the models. Again, the mean differences for both models have no practical difference. However, only one of the volume groupings for the two models shows a significant difference statistically. This can probably be attributed in part to the lower number of observations and to the higher standard deviations of the mean differences, which result in lower t statistics. Again three of the volume groupings for the model with terminal times show mean differences, percent mean differences, and standard deviations which are lower than those found for the model without terminal times. However, in this case, the total grouping shows a greater difference in the model with terminal times. Although the differences appear to be smaller between the O-D and gravity model with terminal times than between the O-D and gravity model without terminal times this difference is believed to be insignificant.

Comparison of CBD Zonal Interchanges

While the results of the loaded street network comparison were revealing, it was felt that a large number of the trips made over these links were between zones which would be influenced very little by the presence or absence of terminal times. Therefore, in an attempt to isolate the influence of terminal times on the zonal interchanges, those zones in the CBD which had terminal times greater than 1 minute were analyzed for zonal interchanges with each other using the paired observations technique. It was felt that the influence of terminal times would be the greatest on intra-CBD trips because the terminal time as a proportion of the total trip time would be larger for these trips than for any others. The results of these analyses are given in Tables 5, 6, and 7. Tables 5 and 6 compare the O-D zonal interchanges with the gravity model interchanges for the model with and without terminal times; Table 7 shows a comparison of the zonal interchanges produced by each of the models. The same values are shown in these tables as were shown for the street network assignments.

TABLE 5

COMPARISON OF O-D AND GRAVITY MODEL ZONAL INTERCHANGES WITHOUT TERMINAL TIMES[a]

Origin Zone	Average Zonal Interchange[b]		Mean Difference	Standard Deviation	Test Statistic t	Table Value, $t_{\alpha/2;\ n-1}$[c]
	O-D	G.M.				
1	101	128	27	49	1.55	2.37
2	18	21	3	4	2.12	2.37
3	11	11	0	7	—	—
4	9	11	2	6	0.94	2.37
5	28	32	4	6	1.89	2.37
6	30	32	2	9	0.63	2.37
8[d]	7	8	2	7	0.81	2.37
9	24	29	5	7	2.03	2.37
All zones	28	34	6	19	0.89	2.37

[a]Includes only those zones within the CBD assigned a terminal time greater than 1 minute.
[b]Average number of trips originating in the origin zone and terminating in all other zones within the CBD.
[c]5 percent significance level; 8 observations per zone.
[d]Zone 7 was within the CBD but was not assigned a terminal time greater than 1 minute.

TABLE 6

COMPARISON OF O-D AND GRAVITY MODEL ZONAL INTERCHANGES WITH TERMINAL TIMES[a]

Origin Zone	Average Zonal Interchange[b]		Mean Difference	Standard Deviation	Test Statistic t	Table Value, $t_{\alpha/2;\ n-1}$[c]
	O-D	G.M.				
1	101	110	9	21	1.24	2.37
2	18	18	0	4	—	—
3	11	9	2	6	0.94	2.37
4	9	9	0	5	—	—
5	28	26	2	5	1.13	2.37
6	30	28	2	7	0.81	2.37
8[d]	7	7	0	6	—	—
9	24	23	1	6	0.47	2.37
All zones	28	29	1	9	0.31	2.37

[a]Includes only those zones within the CBD assigned a terminal time greater than 1 minute.
[b]Average number of trips originating in the origin zone and terminating in all other zones within the CBD.
[c]5 percent significance level; 8 observations per zone.
[d]Zone 7 was within the CBD but was not assigned a terminal time greater than 1 minute.

TABLE 7

COMPARISON OF GRAVITY MODEL ZONAL INTERCHANGES[a]

Origin Zone	Average Zonal Interchange[b]		Mean Difference	Standard Deviation	Test Statistic t	Table Value, $t_{\alpha/2;\ n-1}$[c]
	G.M. (w/oTT)	G.M. (w/TT)				
1	128	110	18	28	1.28	2.37
2	21	18	3	4	2.12	2.37
3	11	9	2	2	2.83[d]	2.37
4	11	9	2	3	1.89	2.37
5	32	26	6	6	2.83[d]	2.37
6	32	28	4	7	1.61	2.37
8[e]	8	7	1	3	0.94	2.37
9	29	23	6	7	2.43[d]	2.37
All zones	34	29	5	11	1.29	2.37

[a]Includes only those zones within the CBD assigned a terminal time greater than 1 minute.
[b]Average number of trips originating in the origin zone and terminating in all other zones within the CBD.
[c]5 percent significance level; 8 observations per zone.
[d]Significantly different.
[e]Zone 7 was within the CBD but was not assigned a terminal time greater than 1 minute.

There is no statistical difference for any of the origin zones for either of the two models; however, it is believed that the model with terminal times is more closely approximating the O-D zonal interchanges. This is indicated by the lower mean differences and standard deviations for this model. The t statistic is lower as a result.

Table 7 shows that there is a significant difference in the synthetic zonal interchanges for the two models for zones 3, 5, and 9 while the other origin zones show no significant difference. This lends further evidence to the conclusion that the model using terminal times is reproducing the CBD zonal interchanges more accurately than the model without terminal times.

An evaluation of the findings of the street network comparison and the CBD zonal interchange comparison seems to be indicating contradictory findings. That is, the street network comparison shows no practical differences in the two models, while a comparison of the CBD zonal interchanges shows a difference. This difference can be resolved by considering the fact, mentioned previously, that the CBD street network links would be carrying a large number of trips which would not be influenced by the presence or absence of terminal times. These trips are those which are passing through the CBD to some other zone in the area of those trips which begin outside and terminate inside the CBD.

Comparison of Accessibility Indices

The denominator of the gravity model formula, called the accessibility index, is used as a measure of a zone's accessibility to the attractions of a particular trip purpose. For instance, that zone which shows the largest accessibility index value for home-based work trips is the zone with the greatest accessibility to employment throughout the study area. The accessibility index is calculated using travel time factors and zone attractions. Since the travel time factors are meaningful only in their relationship to each other, it follows that zone accessibility indices are important only in their relationship to each other. Table 8 gives the accessibility indices of each trip purpose for the gravity model both with and without zone terminal times. Both the maximum and minimum accessibility are given, with the corresponding zone number and the ratio of the maximum accessibility to minimum accessibility. In all purpose categories, the gravity model with zonal terminal times has an accessibility ratio which is lower than that for the model without terminal times. In all internal trip purpose categories except home-based work, the zone which had the maximum accessibility for the model without terminal times did not have the maximum accessibility for the model with terminal times. The significance of these results becomes important when attempting to use the gravity model as a measure of accessibility for alternate street systems. If terminal times are not used in the gravity model then any zone which has a terminal time reduction in the future will not show an increase in relative attraction in proportion to what it should show. This would undoubtedly affect the CBD more than other parts of an urban area.

Screenline Comparisons

Comparisons were made of the O-D volumes and the gravity model volumes crossing both of the screenlines which had been established in the Rock Hill area. The first of these was the Southern Railroad tracks running in a general east-west direction and designated as the north-south screenline (direction of vehicular movement) and the Southern Railroad tracks running in a general north-south direction and designated as the east-west screenline. Each of the screenlines runs through the central area of Rock Hill and intersects to divide the study areas into four separate areas (Fig. 2).

The results of the O-D and gravity model screenline crossings are given for the model with and without terminal times in Table 9. The north-south screenline shows

TABLE 8

COMPARISON OF ZONAL ACCESSIBILITY INDICES[a]

Trip	Without Terminal Times			With Terminal Times		
	Accessibility Index		Ratio Max/Min	Accessibility Index		Ratio Max/Min
	Maximum	Minimum		Maximum	Minimum	
Home-based work	20514	4546	4.51	14470	4091	3.54
Zone No.	(59)	(68)		(59)	(52)	
Home-based non-work	67487	10779	6.26	52041	9965	5.22
Zone No.	(1)	(32)		(12)	(32)	
Non-home-based	53575	13928	3.85	47079	12409	3.79
Zone No.	(6)	(32)		(12)	(68)	
Trucks	11688	2490	4.69	9372	2400	3.91
Zone No.	(1)	(32)		(12)	(32)	

[a]Denominator of the gravity model formula.

TABLE 9

COMPARISON OF SCREENLINE CROSSINGS O-D VS GRAVITY MODEL ASSIGNMENTS TO STREET NETWORK

Screenline	Without Terminal Times			With Terminal Times		
	O-D Crossings	G. M. Crossings	Ratio G. M. /O-D	O-D Crossings	G. M. Crossings	Ratio G. M. /O-D
East-West	32716	34040	1.040	32716	34060	1.041
North-South	29484	30446	1.033	29484	30544	1.036

slightly better results than the east-west screenline for both models; however, the difference is small and cannot be considered significant. The results of the models with and without terminal times are practically the same when comparing the sets of data from each model; however, there is apparently no significant difference in the two sets of comparisons.

CONCLUSIONS

In that phase of the original research study dealing with the Rock Hill area gravity model there were a considerable number of conclusions which were reached on the basis of evaluation of the study findings. Most of these conclusions, as would be expected, simply substantiated the findings of earlier studies. However, some of these conclusions were relatively new and will need further investigation in later studies. The conclusions which are presented here are those which were reached after a careful evaluation of the data contained in this paper only.

It would appear that the findings indicate that the use of terminal times in the gravity model for small urban areas is not critical. However, there is sufficient evidence to warrant further study of the use of terminal times in the gravity model for large urban areas where the numerical variation of the terminal times between zones may be high. It is therefore concluded that the gravity model formula provides an adequate framework, within the normal limitations of accuracy expected, for determining trip distribution patterns for Rock Hill using either the model with terminal times or the model without terminal times.

There is a significant difference in the travel time factors for non-home-based auto driver trips when terminal times are used than when not used. This conclusion does not necessarily imply that the trip distribution resulting from the use of different time factors would necessarily show a significant difference.

When over-the-road driving time alone is used as a measure of spatial separation, the relationship between travel time factors and time cannot be expressed by a constant exponent. However, the reverse of this is true when using total travel time; that is, the relationship between travel time factors and time can be closely approximated by a constant exponent. It follows that when travel time factors are developed for a gravity model using driving time alone these factors cannot safely be changed to include terminal times by simply adding an interval of time for this terminal time. It should be noted that the key words are "safely changed." The travel time factor curves for two of the four trip purposes were found to adequately reproduce the O-D trip length frequency distribution curves for the model using terminal times simply by adding an interval of 2 minutes to the curves using no terminal time.

The inclusion or exclusion of zone terminal times significantly affects the relationship of the zone accessibility indices.

The comparison of the screenline crossings for the gravity models with and without terminal times showed no significant difference.

In summary, it appears that an adequate measure of spatial separation for small urban areas, considering all of the various factors involved in applying the gravity model, is over-the-road driving time alone.

ACKNOWLEDGMENTS

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The opinions, findings, and conclusions expressed in this paper are those of the author and not necessarily those of the U. S. Bureau of Public Roads.

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Characteristics of Taxicab Usage

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This paper presents an investigation of the characteristics of taxicab usage in the city of Chicago. The taxicab appears to operate as a hybrid mode, combining many of the advantages of both automobile and transit. Restrictions on the numbers of cabs that may operate may have a substantial effect on the level of taxicab service.

The data showed that characteristics of taxicab travel appear to be quite different from the characteristics of urban travel as a whole. The taxicab is highly oriented to the central area and used to a large extent for non-work trips. The trips are generally short, and are fairly uniformly distributed over the time of day. It was also found that taxicabs are used mainly by housewives and persons in high income groups. Considerable difference was found between the trips destined to the central area and to the non-central area. Non-central area trips seemed to be made by women and non-drivers for "to home" trips while central area trips were largely of the business or recreational type and were made by persons of white collar occupation groups.

•THE transport systems in an urban area could be thought of as a dichotomy. On one hand, there are user owned systems, i.e., automobiles, which are fixed neither to a route nor to a schedule. On the other hand, there are publicly owned or franchised systems which have specified routes, fares, and schedules. The taxicab, however, fits neither of these categories in that it combines the characteristics of both systems. The taxicab could be thought of as a public automobile, with neither a fixed route or schedule, available at a specified rate of fare for use by anyone who manages to find one. The taxicab, like the automobile, provides access to all points and requires no in-route delays for transfers. However, unlike the automobile, parking is not required, a fare is charged, and it is available to all.

In this manner, the taxicab is similar to many recent proposals for personal transport or adaptive routing systems. Upon a close examination, these systems bear a striking similarity to the taxicab. It may be interesting to speculate the feasibility of an expanded and low cost taxicab system to handle many of the circulation problems facing our cities.

Despite its unique characteristics, relatively little attention has been paid to the taxicab in recent transportation studies. The procedure usually used has been to include taxicab trip data with data on automobile trips. Very little research has been done about the nature of taxicab trips as such, nor have many attempts been made to assess the role of the taxicab in urban transportation. This paper describes a study of the characteristics of taxicab travel and compares them to the characteristics of all travel using data from the Chicago area. The nature of the present operation of taxicabs and their regulatory framework will also be examined to better understand the current status of the taxicab and its future potentials.

The study was conducted by examining data collected by the Chicago Area Transportation Study (CATS) in 1956. The internal trip data from the home interviews (Number

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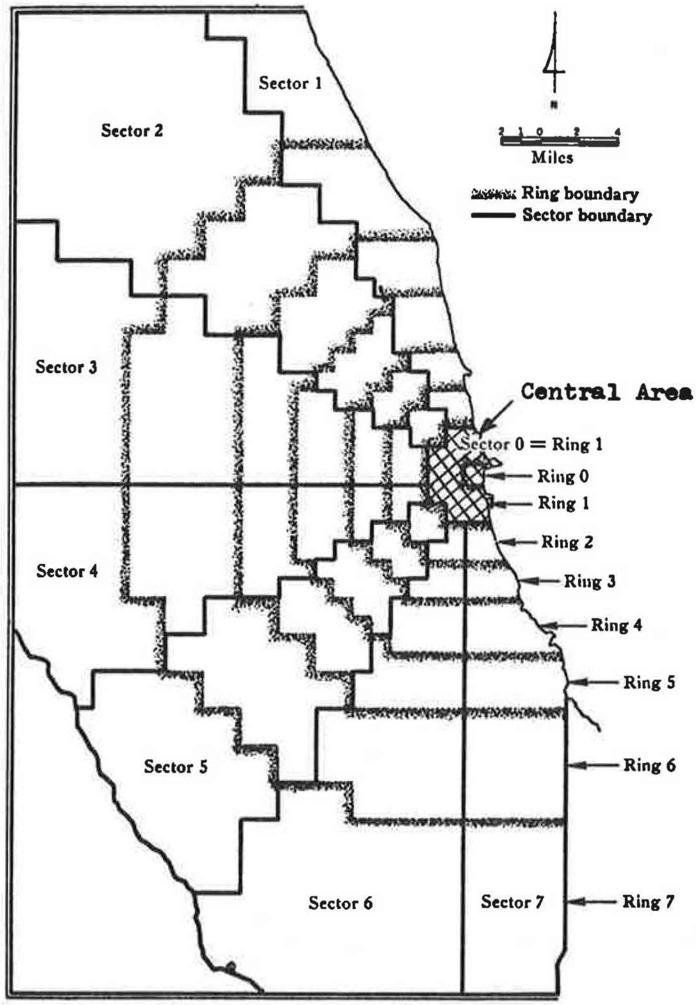

Figure 1. Chicago: analysis rings and sectors. (Source: Meyer, Kain and Wohl, "The Urban Transportation Problem," Harvard University Press. Copyright 1965 by the RAND Corporation.)

2 card) were used and those trips where one of the modes of travel was taxicab were separated from the file of all internal trips and analyzed. These trips were then compared to trips by all modes as reported by CATS. The home interview cards provided extensive data on the nature of the trip and the person making it. All data were expanded to population tables using expansion factors given on the cards. A total of 2,041 cards were analyzed which expanded to 100,506 trips. The data were split into those trips destined to the central area of Chicago (the loop and the area immediately surrounding it, Fig. 1) and those destined outside the central area. These two groups of data were then compared. In addition, the characteristics of taxicab trips were compared to the characteristics of trips by all modes.

REGULATION OF TAXICABS

A taxicab and its driver are subject to many controls and regulations imposed by labor unions, cab companies, and local governments. The extent and effect of these controls are very hard to quantify; however, they should be taken into consideration when studying the role of the taxicab in urban transportation. Such real-world conditions

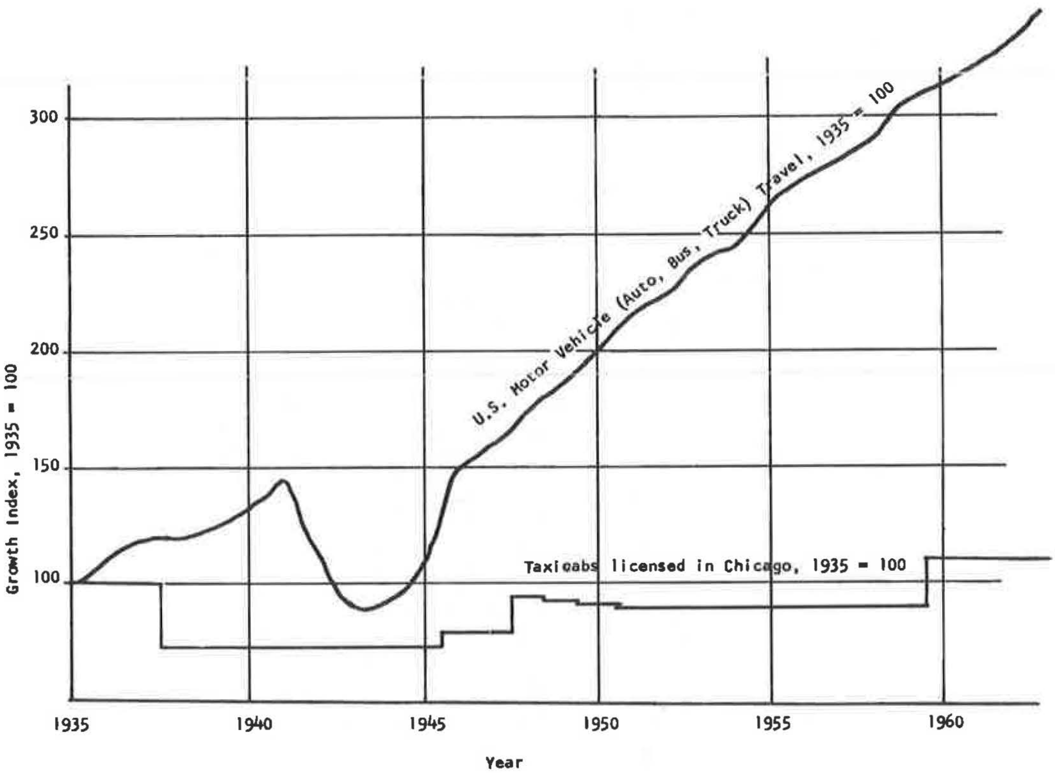

Figure 2. Comparison of the growth of motor vehicle travel with the number of taxicab licenses in Chicago, 1935-1963.

may have a significant effect on conclusions made from a study of this sort. In Chicago, the cab companies and their drivers are regulated by the Public Vehicle Commission. The Chicago City Council has control over such matters as fares, the number of licenses issued and the establishment of regulations.

One important regulation which must be considered when examining the characteristics of taxicab usage is the limitations which are generally imposed upon the number of licenses that may be issued. This restriction allows only those cabs with a permit or medallion to operate. Generally, no new permits are issued and only those persons who held the license the year before (or to whomever they transfer it) will be permitted to operate a taxicab. The actual cost of such a transfer may be well over $12,000 in some cities. This restriction on the number of permits places an upper limit on the number of cabs, which in turn affects the number of trips which can be made by taxicab. In many cases, the number of permits issued has not grown as rapidly as the demand. In Chicago, for example, the number of permits has been held constant at 4600, or about one for every 770 people, since 1960. Figure 2 shows how the number of permits has varied since 1935, and also the growth of automobile travel over the same period on a percentage basis. The supply of taxicabs has risen at a much slower rate than the amount of automobile travel. This is generally true for most major cities where licenses are restricted.

The basis for such restrictions on the number of taxicabs is open to some question. A sharp contrast can be seen by looking at the taxicab service in Washington, D. C., one of the few cities which does not impose restrictions upon entry into taxicab operation. There is a much higher number of cabs in Washington (one for every 77 persons vs one for every 772 persons in Chicago, 1967) lower fares and heavier use. An earlier study of taxicab service made at Northwestern University in 1958 (1, p. 80) indicated that Washington had a level of taxicab service 5 to 13 times higher than Chicago and other large cities. Indices of demand such as population, number of hotel rooms, retail sales, and number of air passengers were used to compare the taxicab service in the cities studied.

Perhaps the primary reason that taxicab service is not at a higher level at present is that this regulation may stymie initiative and growth of usage. Without a change in this policy, the taxicab will continue to play a minor role in urban transportation. Feasible means of removing the restriction on the number of licenses issued may be difficult to find, since those operating cabs would object to the loss of the asset they have with the scarce franchise.

SPATIAL DISTRIBUTION OF TRIPS

The CATS data examined in this study indicated that a total of 100,506 trips were made daily in the Chicago area by persons in taxicabs at the time the data were collected (1956). Of these trips, 86,521 were made with taxicab as the primary mode of travel. In the remainder of the trips, taxicabs were used at either end of a linked trip.

Table 1 shows how this total number of trips was distributed between the central and non-central areas. The taxicab trips were highly concentrated in the CBD and the adjoining area (the central area) and generally were quite short. Two-fifths of all taxi trips were destined to the central area, whereas only 11.4 percent of all travel was destined to this area. In the central area taxicab person trips accounted for 3.62 percent of all person trips; outside the central area person trips by taxicab accounted for only 0.65 percent of all the trip destinations.

The spatial distribution of taxicab trips can be examined further by looking at the taxicab trip destinations by ring and sector. The taxicab trips were grouped by ring and sector (Tables 2 and 3). The rings and sectors are defined in Figure 1. Since the rings and sectors vary considerably in size and population, the number of trips per unit of area or population is given. The number of trips per square mile in each ring decreases out from the center and the number of trips per 1000 population remains nearly constant outside the center, indicating that each ring beyond the central area is fairly homogeneous in the production of trips.

When the trips were grouped by sector, some variation was found. The number of trips per square mile and per 1000 population is largest in the first sector (north, along Lake Michigan), and then drops to its lowest in the south and southwest sectors. These rates increase again near the lake. Chicago is a city that developed principally by sectors. Each sector is roughly homogeneous in character, but quite different from other sectors. The rings are the opposite, they are heterogeneous in character, but similar to each other. Sector 1, where taxi trips were high, is the so-called Gold Coast and North Shore area, where the average incomes are high. Sector 5, where the number of taxicab trips was low is generally an industrial area with low-income housing. Sector 6, another low sector, is an area of low-income, non-white housing. The data give a rough indication of the relationship between income level and taxicab usage. Those sectors where the incomes were higher had a relatively high amount of taxicab usage; where the incomes were low, the amount of taxicab usage was low.

Purpose of Trips

Table 4 indicates the distribution trips by taxicab and other modes by trip purpose. It shows how taxi trip purposes compare to trip purposes by other modes and to trip purposes by all modes combined. A comparison is also made between central and non-central destined trips.

It was found that taxicab trips generally have a different distribution of trip purposes than trips by all modes. Further, taxicab trips destined to the central area have a different distribution of trip purposes than those destined outside the central area. For all taxicab trips, there is a smaller proportion of work trips and larger proportion of social-recreation and personal business trips than for trips by all modes of travel. For those taxicab trips destined to the central area, the percentage of home trips is nearly half that of trips destined to the non-central area while the portion of work trips and

TABLE 1

ORIGIN-DESTINATION—TABLE AND EXPANDED TAXICAB PERSON TRIP TABLE

Origins	Destinations		Row Totals
	Central Area	Non-Central Area	
Central area	27, 099	16, 073	43, 172
Non-central area	15, 067	42, 267	57, 334
Column totals	42, 166	58, 340	100, 506

TABLE 2

TAXICAB PERSON TRIP DESTINATIONS BY RING

Ring	Trips	Land Area (sq mi)	Trips per Square Mile	Population 1000's	Trips per 1000
1	42, 166	13. 6	3100	323	130. 5
2	10, 000	26. 1	383	745	13. 4
3	13, 318	41. 2	323	962	13. 8
4	14, 744	85. 0	173	1, 286	11. 5
5	7, 413	129. 2	57	755	9. 8
6	7, 916	293. 7	27	655	12. 1
7	4, 949	647. 7	8	444	11. 1

TABLE 3

TAXICAB PERSON TRIP DESTINATIONS BY SECTOR

Area	Trips	Land Area (sq mi)	Trips per Square Mile	Population 1000's	Trips per 1000
Sector:					
1	17, 342	97. 5	178	771	24. 4
2	6, 648	241. 6	28	757	8. 8
3	9, 276	203. 7	46	807	11. 5
4	6, 944	158. 0	44	540	12. 9
5	1, 940	173. 8	11	345	5. 6
6	5, 097	222. 0	23	748	6. 8
7	11, 110	129. 6	86	939	11. 8
District:					
01	18, 813	1. 2	15, 677	5	3, 762. 6
11	23, 353	12. 4	1, 883	318	73. 4

TABLE 4

PERCENT OF TRIPS BY TRIP PURPOSE AND MODE

Trip Purpose	Mode								
	Auto	Auto Pass.	Suburban Railroad	Subway or El.	Bus	Taxi to Central	Taxi to Non-Central	All Taxi	All Trips
Ride	0. 0	3. 9	0. 0	0. 0	0. 0	0. 0	0. 0	0. 0	0. 0
Home	41. 4	44. 3	46. 6	48. 0	46. 5	27. 9	52. 9	42. 4	43. 5
Work	23. 4	8. 8	40. 9	34. 6	23. 5	23. 2	6. 4	13. 4	20. 5
Shop	6. 2	5. 5	2. 8	4. 0	4. 6	4. 2	2. 3	3. 1	5. 5
School	0. 4	2. 0	0. 8	2. 1	6. 3	0. 4	1. 1	0. 8	1. 9
Social-recreation	11. 0	20. 9	4. 0	4. 0	8. 4	13. 6	18. 5	16. 5	12. 7
Eat meal	2. 2	3. 3	0. 0	0. 6	0. 7	7. 1	1. 6	3. 9	2. 1
Personal business	10. 6	11. 0	4. 9	6. 7	10. 0	23. 6	17. 2	19. 9	10. 3
Serve passenger	4. 8	0. 3	0. 0	0. 0	0. 0	0. 0	0. 0	0. 0	2. 4
Total	100. 0	100. 0	100. 0	100. 0	100. 0	100. 0	100. 0	100. 0	100. 0

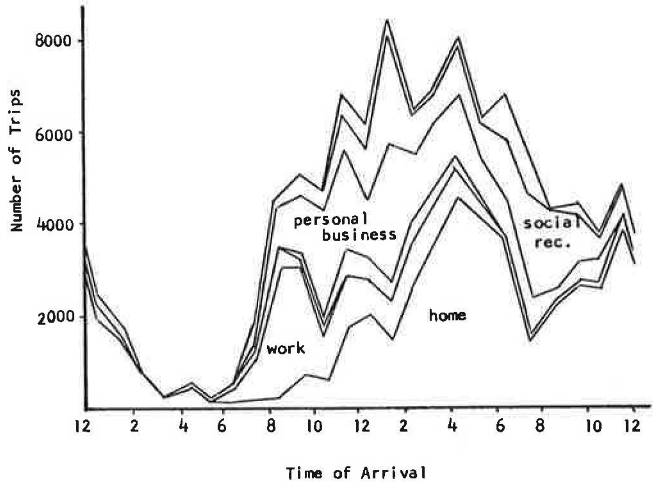

Figure 3. Hourly distribution of taxicab person trips by trip purpose.

business trips is higher. More than half of the non-central area destined trips (52.9 percent) were to home trips.

Trip Purpose Vs Time of Day

The variation of taxicab trips by purpose over the day is shown in Figures 3 through 6. Figures 3 and 4 compare taxicab trips to trips by all modes (2, p. 35). Taxicab travel does not have the same peaking characteristics as all travel does. Instead of two distinct peaks (as in Fig. 4), taxicab travel is fairly uniform over the day with small peaks near the middle of the day. Taxicabs are not used for home-to-work and work-

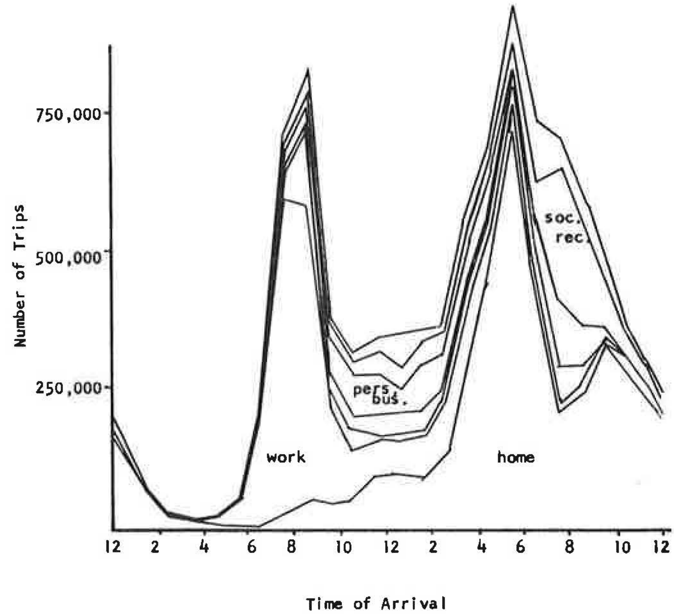

Figure 4. Hourly distribution of all trips by trip purpose.

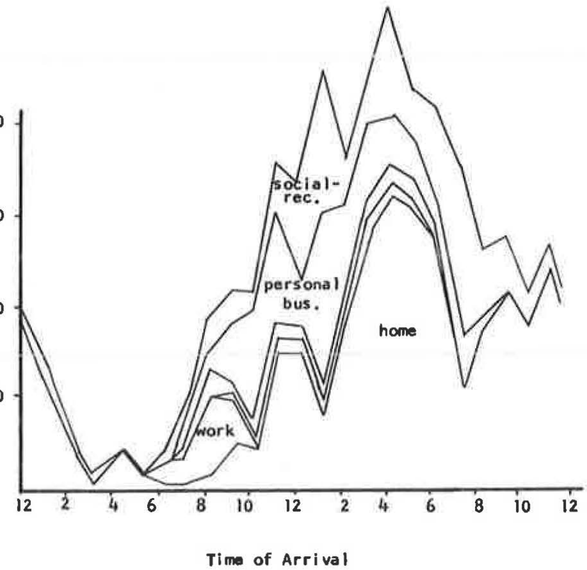

Figure 5. Hourly distribution of taxicab person trips destined to the non-central area.

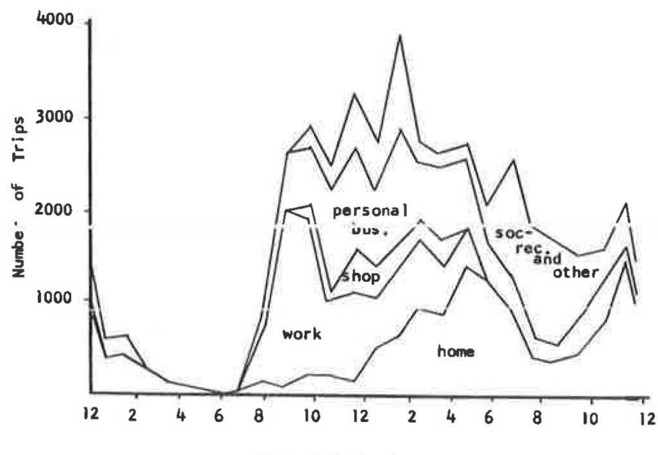

Figure 6. Hourly distribution of taxicab person trips destined to the central area.

to-home trips in as large proportions as in trips by all modes. A peaking of to-home and to-work trips occurs, but the use of taxicabs for personal business and social-recreation purposes during other times of the day more than fills up the period between the morning and evening rush periods. Figures 5 and 6 show that the pattern of taxicab trips destined to the central area differs from the pattern of the non-central area destined trips. The non-central area trips peak at 5:00 p. m. with another smaller peak at 1:00 p. m., the large number of home-destined trips causing the evening peak. Other substantial proportions of the trips are for personal business and social-recreational purposes. Only a very small portion (6.4 percent) of the trips destined outside the central area are for the purpose of going to work.

The taxicab trips destined to the central area had no sharp peaks, but a fairly high level of usage throughout the day, with the maximum amount of travel occurring at

TABLE 5

AVERAGE DISTANCE, TIME, AND TRIPS PER DWELLING UNIT OF TAXICAB PERSON TRIPS

Trip Purpose	Total No. of Trips	Average Distance	Average Time	Average Trips per DU
Ride	0	0.000 mi.	0.000 hr.	0.000
Home	35,547	3.169	0.366	6.282
Work	10,954	2.164	0.300	8.975
Shop	2,560	1.802	0.230	7.804
School	685	1.514	0.291	10.228
Social-recreation	14,829	3.174	0.324	9.211
Eat meal	3,697	1.996	0.239	8.780
Personal business	18,249	2.792	0.296	6.952
Serve passenger	0	0.000	0.000	0.000
Change mode	0	0.000	0.000	0.000
Total	86,421	2.863	0.326	7.584

Average speed = 8.8 airline mph.
Average length of trip (miles) by other modes: all trips, 4.3; suburban RR, 13.3; subway-elevated, 7.2; auto driver, 3.5; auto passengers, 3.9; bus, 3.6; and taxicab, 2.9.

TABLE 6

AVERAGE DISTANCE, TIME, AND TRIPS PER DWELLING UNIT OF TAXICAB PERSON TRIPS DESTINED TO THE CENTRAL AREA

Trip Purpose	Total No. of Trips	Average Distance	Average Time	Average Trips per DU
Ride	0	0.000 mi	0.000 hr.	0.000
Home	11,164	2.461	0.335	5.355
Work	7,780	1.785	0.274	9.493
Shop	1,367	1.998	0.274	6.993
School	63	1.576	0.300	4.952
Social-rècreation	5,249	2.203	0.259	8.999
Eat meal	2,862	1.423	0.214	9.618
Personal business	18,769	2.635	0.298	6.759
Serve passenger	0	0.000	0.000	0.000
Change mode	0	0.000	0.000	0.000
Total	37,254	2.226	0.291	7.450

Average speed = 7.6 airline mph.

TABLE 7

AVERAGE DISTANCE, TIME, AND TRIPS PER DWELLING UNIT OF TAXICAB PERSON TRIPS DESTINED TO THE NON-CENTRAL AREA

Trip Purpose	Total No. of Trips	Average Distance	Average Time	Average Trips per DU
Ride	0	0.000 mi.	0.000 hr.	0.000
Home	24,383	3.494	0.381	6.707
Work	3,174	3.093	0.364	7.707
Shop	1,193	1.578	0.179	8.734
School	622	1.507	0.290	10.762
Social-recreation	9,580	3.706	0.360	9.328
Eat meal	835	3.960	0.327	5.904
Personal business	9,480	2.937	0.295	7.131
Serve passenger	0	0.000	0.000	0.000
Change mode	0	0.000	0.000	0.000
Total	49,267	3.339	0.352	7.449

Average speed = 9.5 airline mph.

1:00 p. m. A large portion of trips to the central area are made for work, personal business, and social-recreational purposes. A smaller portion of the trips are made to home than to the non-central area.

That taxicab trips do not have definite peaks during the day as travel in general does may be due to a number of different reasons. First, it may be that taxicabs are used for different purposes at different times of the day due to some inherent characteristic of taxi travel. Or it might be because taxicabs are being used to capacity at nearly all times and the actual pattern of use is quite different than the desired pattern of use. Because of the small number of cabs, it may be that they are being used near capacity throughout the day. People who would like to use a taxicab at peak hours may be prevented from doing so because of the small number of cabs available.

Trip Length

The majority of the analysis is presented using those trips which listed the taxicab as one of the modes in a linked trip. Using those trips where the primary mode was taxicab did not change the conclusions substantially except when trip length was examined. The data here are presented using primary mode taxicab trips only. It was found that the trips with a primary mode other than taxi tended to be much longer and raised the average trip lengths substantially, hence they are not included in these tables. The results obtained are given in Tables 5, 6, and 7.

The average taxicab trip was shorter than the average trip by other modes. Taxicab trips had an average length of 2.9 mi while all trips averaged 4.3 mi (2, p. 120). For trips by taxicab, travel time averaged about 20 min, or an average speed of 8.8 airline miles per hour. Work trips by taxicab were an average of 2.2 mi, while work trips by all modes were an average of 5.3 mi. Social-recreation trips were 3.1 mi by taxicab, 4.3 mi by all modes, and home trips were 3.2 mi by taxicab vs 4.4 mi by all modes.

The trips destined to the central area were somewhat shorter in distance than those destined outside the central area (2.2 vs 3.3 mi). The difference in time was somewhat less (17.5 vs 21 min for the non-central area) resulting in a lower average speed in the central area (7.6 airline mph) than outside the central area (9.5 airline mph). The differences in trip length by trip purpose were more varied. For example, the difference in the length of home trips was fairly large (3.5 vs 2.5 mi) while the difference in the length of personal business trips was not as large (2.9 vs 2.6 mi). In these comparisons, the central area destined trip lengths are the lower numbers and the non-central area trips are the higher numbers.

A taxicab is used a large amount of time during the day. Based on a total of 3700 taxicabs, an average trip length of 0.326 hr, an estimated occupancy rate of 1.1 passengers per cab and a total of 100,506 trips per day at the time of the study, the average taxicab was in use for revenue purposes 7.9 hr per day. Time spent cruising, returning to cab stands, etc., apparently occupies the rest of the day. If the total number of trips was underestimated because of under sampling of transients, this figure may be somewhat low.

Trip Production

Data were also available for the number of trips made per dwelling unit of the people who make taxicab trips (Tables 5, 6, and 7). The average taxicab user came from a household that made 7.46 trips per day, while the average household in the area produced 6.12 trips per day (2, p. 115). This implies that the people who make taxicab trips are persons who, in general, make more trips per day than the population of trip makers. The average number of trips per dwelling unit in the central and non-central areas was also tabulated and the aggregate rates were nearly the same for both areas. There was some difference when stratified by trip purpose (Tables 6 and 7).

Land Use at Trip Ends

It was possible to examine the land use at taxi trip origins and destinations. The predominant land use destination was residential land use; this accounted for 55.1 percent

TABLE 8

LAND USE AT ORIGIN AND DESTINATION OF TAXICAB PERSON TRIPS

Land Use	Taxicab Trip Origins Trips	Origins Percent	Taxicab Trip Destinations Trips	Destinations Percent	Percent of all Trip Dest.
Residential	50,581	50.4	55,361	55.1	>54.9
Agriculture, forestry, fisheries	61	0.1	65	0.1	
Durable manufactures	1,066	1.1	1,054	1.0	} 7.6
Nondurable manufactures	1,440	1.4	1,494	1.5	
Transportation, comm., util.	6,920	6.9	3,874	3.9	2.7
Retail	12,347	12.3	9,977	10.0	} 24.0
Services	17,188	17.1	16,511	16.4	
Wholesale trade and commerce	997	1.0	1,434	1.4	
Public buildings	7,564	7.5	8,559	8.5	7.7
Public open space	2,248	2.2	2,083	2.1	3.1
	100,506	100.0	100,506	100.0	100.0
Sub-classifications:					
Medical and health services	4,513	4.5	4,897	4.9	
Eating and drinking places	6,065	6.0	4,640	4.6	
Hospitals, mental inst., etc.	2,773	2.8	3,152	3.1	
Indoor amusements and rec.	3,981	4.0	2,947	2.9	
General merchandise group	3,268	3.3	2,762	2.8	
Office buildings, N. E. C.	2,268	2.6	2,782	2.8	
Financial, ins., and real est.	2,363	2.4	2,180	2.2	
Personal services	876	0.8	1,256	1.2	

of all the trip destinations by taxicab. The next largest major land use at trip destination was service land which includes such subclassifications as medical and health services, indoor amusements and recreation, and office buildings. The pattern of land use at the destinations of all trips (2, p. 113) was quite similar to that of the taxicab trips. The distribution of land use at the origin of the taxicab trips was similar to the distribution of land use at the taxicab trip destinations. A summary of this land use data is given in Table 8.

The degree to which taxicabs are used to serve the terminals of intercity transportation modes was also examined. The land use coding provided information on the number of person trips by taxicab to and from the following land uses:

Land Use	Origins	Destinations
Railroads	1346	1537
Air transportation	2173	1099
Water transportation	0	0
Highway transportation	94	0
Total	3613	2636

The number of taxicab trips to or from these land uses was fairly small when compared to the total number of trips by taxicab. The share of taxicab trips to communication, transportation, and public utility land use was somewhat greater than that portion of all trips (3.9 vs 2.7 percent). It should be remembered that these trips are from data collected in 1956, when the amount of air travel was relatively low and that these land use categories include nonterminal areas.

Other Modes Used in Connection with Taxicab Trips

The mode of travel was given in the data as the first mode, primary mode, and the last mode of travel of a linked trip. Where there was only one mode used, as in the

TABLE 9

MODES OF TRAVEL USED IN CONJUNCTION WITH TAXICAB PERSON TRIPS

Mode	Destination of Trip					
	Central Area		Non-Central Area		Entire Area	
	Trips	Percent	Trips	Percent	Trips	Percent
Taxicab as the primary mode	37, 254	88. 3	49, 267	84. 4	86, 521	86. 0
Taxicab as the first mode in a linked trip	466	1. 1	4, 034	6. 9	4, 500	4. 5
Taxicab as the last mode in a linked trip	3, 400	8. 1	4, 373	7. 5	7, 773	7. 8
Taxicab as the first and last mode in a linked trip	1, 046	2. 5	666	1. 2	1, 712	1. 7
Primary mode used where taxicab was given as the first mode:						
Auto driver	0	0. 0	150	0. 3	150	0. 1
Suburban RR	1, 053	2. 5	3, 086	5. 3	4, 139	4. 1
Subway or el	369	0. 9	560	1. 0	929	0. 9
Bus	90	0. 2	904	1. 5	994	1. 0
Primary mode used where taxicab was given as the last mode:						
Auto driver	213	0. 5	0	0. 0	213	0. 2
Auto passenger	62	0. 1	128	0. 3	190	0. 2
Suburban RR	2, 945	7. 0	1, 837	3. 1	4, 782	4. 8
Subway or el	429	1. 0	1, 580	2. 7	2, 009	2. 0
Bus	317	0. 6	122	0. 2	368	0. 4

TABLE 10

TAXICAB PERSON TRIPS BY AGE, SEX, RACE, AND DRIVER

Characteristic	All Destinations		Central Area Destinations		Non-Central Area Destinations	
	Trips	Percent	Trips	Percent	Trips	Percent
Sex:						
Male	41, 404	41. 2	22, 197	52. 5	19, 207	32. 9
Female	59, 102	58. 8	19, 969	47. 5	38, 133	67. 1
Total	100, 506	100. 0	42, 166	100. 0	58, 340	100. 0
Race:						
White	90, 623	90. 2	40, 405	95. 7	50, 218	86. 1
Non-White	9, 883	9. 8	1, 761	4. 3	8, 122	13. 9
Total	100, 506	100. 0	42, 166	100. 0	58, 340	100. 0
Drivers:						
Male	28, 184	28. 0	17, 856	42. 4	10, 328	17. 7
Female	17, 024	16. 9	7, 871	18. 8	9, 153	15. 8
Total	45, 208	44. 9	25, 727	61. 2	19, 481	33. 5
Nondrivers:						
Male	13, 220	13. 1	4, 341	10. 1	8, 879	15. 1
Female	42, 078	42. 0	12, 098	28. 7	29, 980	51. 4
Total	55, 298	55. 1	16, 439	38. 8	38, 859	66. 5

majority of the trips, it was given as the primary mode of travel. It was possible with these data to note the extent to which taxicabs were used as feeders to other local modes. The modes of travel used in connection with taxicab trips are given in Table 9.

For the large majority of the taxicab trips (86.1 percent), the taxicab was the primary mode of travel used. In the remainder of the taxicab trips it was used as a feeder or distributor at ends of a commuter railroad or rapid transit trip. The taxicab was almost never used in conjunction with an automobile trip or a bus trip.

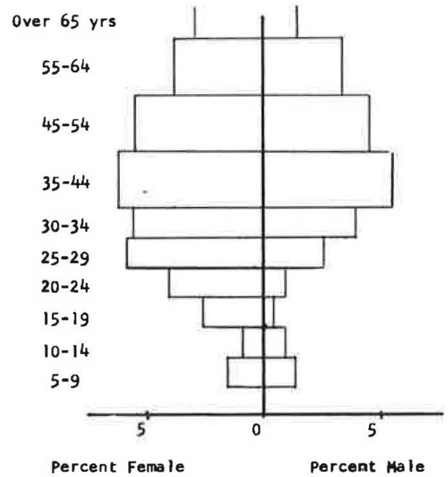

Figure 7. Age-Sex diagram for taxicab users, 1956.

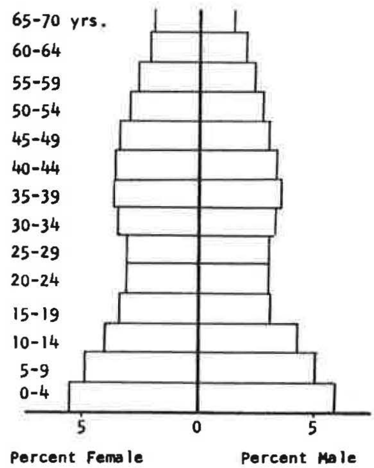

Figure 8. Age-Sex diagram for the Chicago SMSA (1960 Census).

CHARACTERISTICS OF THE TAXICAB USERS

Age-Sex

The age-sex diagram of taxicab users shown in Figure 7 is shaped quite differently than Figure 8, which is the diagram for the Chicago SMSA (1960 census). Most of the taxi riding was done by people in middle age groups. More than three-fourths (78.6 percent) of all taxicab riding was done by persons 30 years old or older, while this age group constituted only 51.8 percent of the population (1960). The difference was even more pronounced in the trips destined to the central area. In this area, the largest proportion of the trips (28.3 percent) were made by the 35-44 year old age group. Age-sex diagrams for trip makers destined to the central and non-central areas are given in Figures 9 and 10.

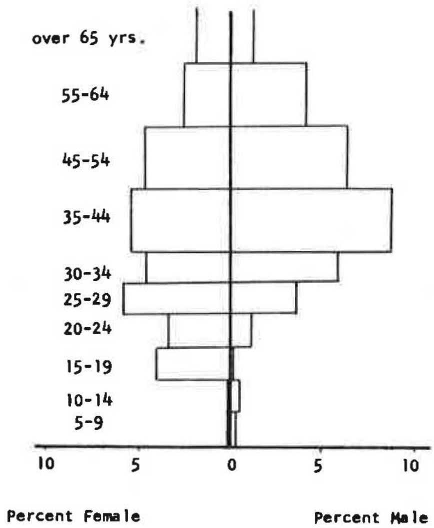

Figure 9. Age-Sex diagram for taxicab users destined to central area.

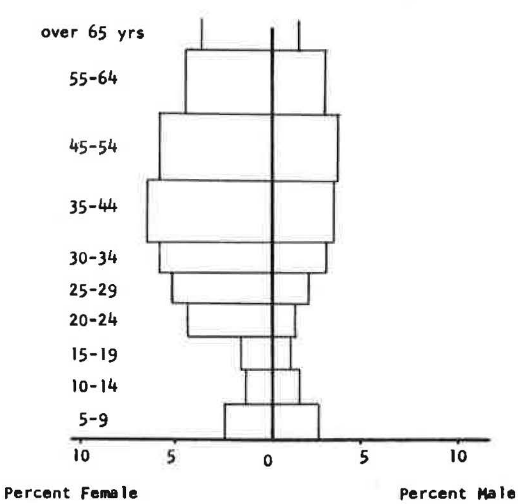

Figure 10. Age-Sex diagram for taxicab users destined to non-central area.

TABLE 11

TAXICAB PERSON TRIPS BY OCCUPATIONAL GROUPS

Occupation Group	Taxicab Trips with Destinations in:						Percent (exclusive of housewives)	Proportion of Population
	Central Area		Non-Central Area		All Areas			
	Trips	Percent	Trips	Percent	Trips	Percent		
Professional and technical	8,749	20.7	6,855	11.8	15,604	15.6	28.2	12.1
Farmers	0	0.0	0	0.0	0	0.0	X	X
Managers, etc.	7,425	17.6	3,712	6.4	11,237	11.1	20.2	9.0
Clerical	3,796	9.0	4,981	8.5	8,777	8.8	15.8	20.5
Sales workers	5,235	12.5	2,734	4.7	7,969	7.9	14.4	7.9
Operatives	1,143	2.7	2,000	3.4	3,143	3.1	5.7	20.3
Private household workers	97	0.2	918	1.6	1,015	1.0	1.8	1.4
Craftsmen, foremen	935	2.2	977	1.7	1,912	1.9	3.5	15.4
Service workers	1,015	2.4	3,924	6.7	4,939	4.9	9.0	8.8
Laborers	153	0.4	570	1.0	723	0.7	1.3	4.6
Housewives	13,586	32.3	31,607	54.2	45,193	45.0	X	X
Total	42,166	100.0	58,340	100.0	100,506	100.0	100.0	100.0

There appeared to be a fairly high rate of usage of taxicabs by women for trips destined to the non-central area. Those trips destined to the central area were split nearly evenly between the sexes, while 67.1 percent of the trips to the non-central area were made by women (Table 10).

By looking at the difference in the number of trips made by drivers and nondrivers (another subclassification), more information may be found about the type of person that makes a trip by taxicab. The portion of taxicab trips that are made by persons holding a driver's license (55.1 percent) is somewhat less than the percent of the population (61.1 percent) which does not possess a driver's license (2, p. 120). There is, however, a substantial difference between the central and non-central area destined trip makers in this regard. In the central area, 38.8 percent of the trip makers are nondrivers, while in the non-central area, 66.5 percent of the trips are made by nondrivers. A large portion of the nondrivers are women (74.8 percent of the nondrivers). The portion of all taxicab trips made by women nondrivers is quite large at 42.0 percent.

Differences by Race

Nearly all (90 percent) taxicab travel was done by the white portion of the population. In the central area, 95.6 percent of the taxicab trips were made by whites and in the non-central area, 85.9 percent of the trips were made by whites. According to the 1960 census, 17.2 percent of the population of the Chicago SMSA and 23.6 percent of the population of the city was non-white (3 p. 128). Taking this as a rough estimate of income levels, a greater use of taxicabs by higher income groups is indicated.

Occupation of Taxicab Users

The portions of trips made by persons of various occupational groups is given in Table 11. Nearly half of all taxicab trips are made by housewives and the remainder of the trips are made primarily by white collar occupation groups. The comparison of the employment groups of taxicab users (not including housewives) with the employment groups of all the people in the Chicago SMSA (3) shows that higher proportions of taxicab users are in the white collar groups and smaller proportions are in the skilled labor group. The implications of this may be twofold. Either these people require taxicab trips because of the nature of their jobs, or because of their higher incomes they can better afford to use taxicabs. The occupation of the trip makers differed with those destined to the central area and those destined to the non-central area. Higher proportions of the trips destined to the central area were made by sales workers, managers, professionals, and technicians and smaller portions were made by housewives than for trips to the non-central area. The type of activity in the central area and an income effect seem to operate to cause this distribution of trips by occupational groups.

CONCLUSIONS

The analysis has shown the characteristics of taxicab travel to be quite different from the characteristics of all travel. Taxicab trips tend to be highly oriented to the central area, and used to a large extent for non-work trips. The trips tend to be quite short and to occur fairly evenly over the time of day. Outside the central area, taxicabs seemed to be used largely by non-drivers, housewives and persons of high income groups. Restrictive entrance into the taxicab business appears to be the limiting factor in taxicab usage. Unless problems in regulation are resolved any extensions of taxicab or similar service may be very difficult to bring about.

ACKNOWLEDGMENTS

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A Longitudinal Analysis of Household Travel

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ABRIDGMENT

The analysis of household travel behavior may be approached from a variety of different perspectives. One may, for example, as in a standard trip-generation study, simply consider the frequency of trip-making to and from the home to be a simple linear function of a household's socioeconomic and locational characteristics. Alternatively, attention may be directed toward the factors influencing individual mode-choice decisions, the level of family expenditures on transportation or the interpretation of travel as an aspect of derived demand.

This paper is concerned with a slightly less conventional form of analysis, based upon the interpretation of trip-making as an explicitly auto-correlated, time-dependent process. The analytic methodology was developed initially in response to a study of traffic induction and diversion. Its application in this context is discussed briefly, together with a selection of results from a pilot empirical study and some of the implications which the analysis bears for the study of travel demand.*

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