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53 traffic control and operations
55 traffic measurements

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FOREWORD

The strong and widespread interest in procedures by which road and street capacity and safety can be measured or predicted continues to lead researchers into various perspectives of this multifaceted area. Planners, designers, and operators of highway facilities will find information of value in the papers constituting this RECORD.

Recognizing that the procedure in the 1965 Highway Capacity Manual may yield inadequate results in the analysis of weaving sections, Roess, McShane, and Pignataro studied the effects that lane configuration can have on operations. The procedure being developed in their work, the inclusion of lane configuration as a primary element, will be useful in later modification of the HCM procedure. In this paper, the authors illustrate the use of their proposed modified procedure and suggest that it will ultimately enable the designer to ensure balanced operation of the lanes in the weaving area.

Fellinghauer and Berry used time-lapse photography to gather data for evaluation of the effects of opposing flow on left-turn reduction factors in computing capacity of two-lane approaches (no left-turn lanes) at two-phase signalized intersections. They conclude that left turns have a much greater reducing effect than that given in the HCM. Some comparisons with the Australian method are also presented.

The effects of automobile-utility trailer combinations on capacity and safety led Siria and Deen to the work reported in the final paper. Their conclusions indicated among other things that these combinations are involved in accidents more than their proportionate numbers would indicate, that their accidents are related to wind forces created by passing vehicles or cross section design or to weaving, and that the automobile equivalency factor for these combinations is approximately equal to that for trucks.

CONFIGURATION, DESIGN, AND ANALYSIS OF WEAVING SECTIONS

Roger P. Roess, William R. McShane, and Louis J. Pignataro,
Polytechnic Institute of New York

The paper discusses and illustrates the effect that lane configuration can have on weaving-section operations, specifically, how lane utilization may be substantially controlled by configuration. Data from the 1963 urban weaving-area capacity study are used to quantify these effects. Further illustration is given through the use of lane-transition matrices to model gross weaving movements under various configurations. The paper concludes that the weaving procedure in the Highway Capacity Manual leads to inadequate results because of the failure to differentiate between weaving and nonweaving lane requirements. That procedure is modified to take this into account and to avoid the design of sections with a proper total number of lanes. The modified procedure yields a configuration that prohibits proportional use of those lanes by weaving and nonweaving traffic.

•EXISTING PROCEDURES for the design and analysis of weaving sections as specified in the Highway Capacity Manual (1) have been shown to be inaccurate in the prediction of level of service (2). Some of the inaccuracies can be traced to ambiguities in the specification of service standards and to the basic inaccuracy of the k-factor equivalence expansion mechansim of the HCM weaving procedure (1, ch. 7; 3, 4).

Another, possibly more basic cause of the observed inaccuracies lies in the fact that lane configuration is not considered as a parameter in the design-analysis procedure.

CONFIGURATION AS A FACTOR IN WEAVING DESIGN AND ANALYSIS

Lane configuration is a tremendously important factor that can have a drastic effect on lane utilization by component flows, the number of lane changes required to complete a weaving movement, relative speeds of component flows, and other operational factors.

Leisch (5) demonstrated the benefits of certain configurations in meeting varied or changing demand patterns. Leisch also showed how configuration changes can reduce total lane changing in a weaving section (2). The effect of configuration improvements is also adequately demonstrated in the results of the Ward-Fairmount weaving study (6), in which increased vehicle speeds for similar volumes were accomplished largely through configuration changes. (The Ward-Fairmount study is treated later in this paper.)

Of principal concern to the designer or analyst is the effect of configuration on actual or potential lane utilization. The HCM procedure results in a computation for N, the number of total lanes in the weaving section. No explicit distinction is made between lanes needed for weaving and those for outer flows. Figure 1 shows how lane configuration can substantially limit the number of lanes used by weaving flows, regardless of the total number of lanes provided. Three weaving sections of equal length are shown, and configuration differences are seen to drastically affect potential lane utilization.

Publication of this paper sponsored by Committee on Highway Capacity and Quality of Service.

A ramp-weave section is a weaving section formed by consecutive on- and off-ramps joined by an auxiliary lane. A major weave section is one in which three or more entry and exit legs have two or more lanes, forming either a major fork, a major merge point, or both.

All weaving movements in a ramp-weave section must take place in shoulder and auxiliary lanes (Fig. 1a). Secondary lane-changing movements are possible from the center lane; the extent to which the center lane may be used for secondary lane changing is primarily related to the length provided. Based on these considerations, it is seen that weaving vehicles could occupy at best two full lanes. Partial occupation of the center lane would be more than offset by through vehicles using the shoulder lane, as indicated by the HCM. Therefore, although it seems possible to have weaving vehicles occupy two full lanes, a reasonable maximum of one full lane plus a substantial proportion of a second might be more appropriate.

The major weave section shown in Figure 1b is similar to a ramp-weave section, and, again, weaving movements are restricted primarily to two lanes, although secondary lane movements may take place from either of two outside lanes. Once again, it appears feasible for weaving vehicles to occupy two full lanes, perhaps a little more, depending on the extent of the outer flows.

The geometry of a major weave section, however, can be slightly altered to effect a significant change in possible lane use, as shown in Figure 1c.

In this configuration, a vehicle can make a weaving movement without changing lanes. Weaving movements can be made with a single lane change (as is usually the case) from either of two additional lanes. Therefore, with this configuration it is feasible for weaving vehicles to occupy three full lanes and possibly part of another. In addition, it would be expected that a weaving movement requiring no lane change would be more efficient than one requiring a lane change.

The configurations shown in Figure 1 indicate the possible effect of lane configuration on effective utility of weaving section lanes. The HCM methodology does not consider this vital aspect. It is apparent that simply providing the proper total number of lanes is not sufficient to guarantee the predicted operating characteristics. Lane arrangement may be such that the use of the lanes by weaving and nonweaving flows is not in proportion to the relative flows, resulting in part of the roadway being underutilized while another portion is overutilized.

Because lane arrangement depends heavily on the design of entry and exit legs, any design procedure should seriously consider this element as an integral part of the weaving area. In many cases it might be preferable to add lanes to exit or entry roadways rather than completely reconstruct a poorly operating weaving area.

The effects of lane configuration may be demonstrated and supported by (a) examination of flow speeds and lane utilization from a study of urban weaving areas and (b) use of lane-change transition matrices.

Relative Speeds and Lane Use

A 1963 study of urban weaving-area capacity (2) conducted by the Bureau of Public Roads showed that, in most cases, the speed of weaving vehicles and the speed of nonweaving vehicles are within 5 mph of each other. Table 1 gives the speeds of weaving and nonweaving vehicles obtained in the study. This is to be reasonably expected, for, in many weaving situations, weaving and nonweaving vehicles must share the same lanes. This has the effect of creating more or less uniform flow throughout the section.

In some cases, however, the roadway is wide enough to allow weaving and nonweaving flows to substantially isolate. In such instances, the effect of weaving flows on nonweaving flows is minimal, and large differences in speed might be observed. As indicated in Table 1, such differences most often occur on ramp-weave facilities with auxiliary lanes, where nonweaving vehicles may separate to the outer lanes. The geometry and lane configuration of a ramp-weave site restrict weaving vehicles to primarily the shoulder and auxiliary lanes. On major weave facilities, however, weaving flows tend to be dominant. If multilane entry and exit legs exist, weaving vehicles may occupy the major portion of the roadway. The higher speeds of nonweaving vehicles in the ramp-weave case indicate that weaving flows would expand into the outer lanes if the lane configuration and length permitted it. In terms of balanced use of roadway space, such situations indicate an underutilization of outer lanes and congestion in weaving lanes.

It appears that, in the cases of wide speed differentials, weaving vehicles are restricted to the limited portions of the roadway as a result of lane configuration. This can be verified by considering the relative number of lanes that are occupied by weaving and nonweaving vehicles in each experiment.

The appropriate service volume for each experiment is determined from HCM Table 9.1 on the basis of observed speeds and is adjusted by using standard correction factors for lane widths and percentage of trucks. The volume of nonweaving vehicles is then divided by this adjusted service volume to obtain an estimate of the number of lanes occupied by nonweaving vehicles. The number of weaving lanes is then the total number of lanes minus the number of nonweaving lanes. These computations are given in Table 2. Note that in no instance does the number of lanes occupied by weaving vehicles in a ramp-weave section exceed 2.0 (maximum of 1.75). In several major weaves, however, more than 2.0 lanes were occupied. The distinctive lane configuration of ramp-weave sections restricts all weaving movements to essentially two lanes: the shoulder lane and the auxiliary lane. Major weave sections, which have widely variable configurations, permit easy use of more than two lanes by weaving traffic.

It must be admitted that the data in Table 2 are not conclusive, if considered alone. The major weave sections generally have higher weaving volumes than the ramp-weave sections shown and would normally be expected to have a larger portion of the roadway taken up by weaving vehicles. However, the results given in Table 1 indicate that wide speed differentials between weaving and outer flows most often occur on ramp-weave sections. This clearly indicates that additional roadway space was available to weaving vehicles had some exterior constraint not prevented them from making use of it. Although such speed differentials also occur on major weave facilities, they occur less frequently and the differences tend to be smaller.

Note that all of the major weaves of Tables 1 and 2 are of the type shown in Figure 1c. These observed results strongly support the lane utilization effects of configuration hypothesized herein.

Lane-Change Transition Matrices and Configuration

The effects of configuration on weaving-area performance may also be analytically demonstrated by using a matrix of lane-changing probabilities similar to those developed by Worrall and colleagues (7, 8) and Drew (9). The following is a brief development of the lane-changing matrix presented by Worrall, Bullen, and Gur (8).

Assume that there is a probability $p(r)_{ij}$ of a vehicle making a lane change in the rth segment of length L from lane i to lane j, where L is defined such that only one lane change per segment is possible. The following m × m matrix of values describing the probability of all lane-changing movements in segment r may be constructed:

$$P(r) = \begin{bmatrix} p(r)_{11} & p(r)_{12} & \dots & p(r)_{1m} \\ p(r)_{21} & p(r)_{22} & \dots & p(r)_{2m} \\ \vdots & & & \\ p(r)_{m1} & p(r)_{m2} & \dots & p(r)_{mm} \end{bmatrix}$$

Figure 2 shows the physical interpretation of $p(r)_{ij}$. If the probable lane distribution of vehicles at the entrance of segment r is described by a vector

$$A = [a_1 \ a_2 \ a_3 \ \dots \ a_m]$$

and the probable lane distribution at the end segment r is described by another vector

$$B = [b_1 \ b_2 \ b_3 \ \dots \ b_m]$$

Figure 1. Weaving movements in (a) a ramp-weave section and (b) and (c) two major weave section configurations.

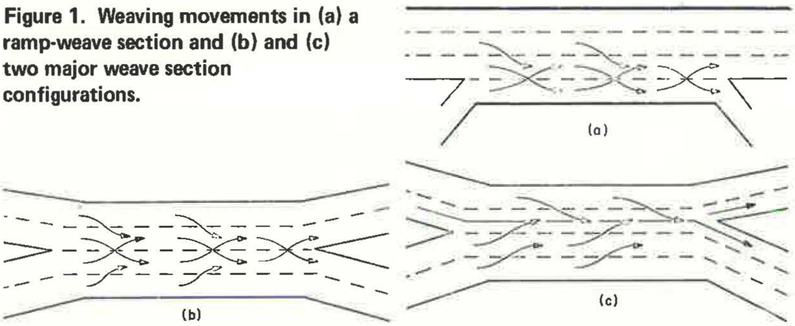


Table 1. Relation of space mean speed of nonweaving vehicle to weaving vehicle speed.

Type	>5 mph Less	±5 mph	5 to 10 mph More	10 to 15 mph More	>15 mph More
Ramp weave with auxiliary lane	1	10	0	2	4
Major weave and collector-distributor	2	17	4	1	0
All	3	27	4	3	4

Table 2. Number of freeway lanes occupied by weaving and nonweaving vehicles.

Type	No.	1 Nonweaving Vol.	2 Weaving Vol.	3 Nonweaving SV	4 Total Lanes	5 = 1/3 Nonweaving Lanes	6 = 4 - 5 Weaving Lanes
Ramp weaves	3	3,986	1,098	1,765	4	2.25	1.75
	7	3,374	1,666	1,460	4	2.30	1.70
	8	3,157	1,775	1,265	4	2.49	1.51
	9	4,572	1,526	1,804	4	2.53	1.47
	11	5,008	1,354	1,485	5	3.54	0.55
	12	5,918	638	—[a]	5	—	—
	14	6,222	627	—[a]	5	—	—
	16	5,719	940	—[a]	5	—	—
	17	3,897	1,112	1,302	4	2.97	1.03
	18	2,487	951	1,085	4	2.45	1.55
	21	4,220	539	1,582	4	2.65	1.35
	28	5,096	1,366	1,455	5	3.50	1.50
	29	1,806	1,434	1,480	3	1.33	1.67
	30	2,030	1,108	—[a]	3	—	—
	32	3,902	1,300	—[a]	4	—	—
	33	6,133	1,252	1,582	5	3.92	1.08
	34	2,706	1,131	980	4	2.76	1.24
Major weaves	4	4,649	2,486	1,840	4	2.53	1.47
	13	4,555	2,974	—[a]	5	—	—
	23	3,478	2,502	1,570	5	2.20	2.80
	24	2,019	2,293	1,420	5	2.12	2.88
	49	2,933	2,166	—[a]	4	—	—
	50	2,814	2,238	—[a]	4	—	—
	51	1,913	1,678	1,470	4	1.30	2.70
	52	2,182	2,453	1,508	4	1.45	2.55
	53	792	1,823	1,400	4	0.57	3.43
	54	631	1,767	1,425	3	0.44	2.56
	60	2,384	2,859	1,718	—[b]	—	—
	61	2,170	1,970	—[a]	—[b]	—	—
	63	1,598	2,564	1,620	3	0.97	2.03
	64	3,100	3,014	—[a]	—[b]	—	—
	65	2,465	1,651	1,440	—[b]	—	—

[a]Level of service F prevails; service volume variable. [b]Not available.

then

$$B = A \times P(r)$$

where m = the number of lanes in the section under study.

The Worrall study concerned normal lane changing, but was adapted to fit other situations. It may be restated as lane changing dictated only by driver choice and not necessitated by the need to complete a merging, diverging, or weaving maneuver. Under the latter circumstances, lane changing may be considered a random event, P(r) should be the same for all r, and the lane distribution after a segment of highway of any given length can be stated as

$$B = A \times P^N$$

where N is the number of segments of length L in the section under consideration. Further, under stable flow, the number of lane changes from i to j will be equal to the number of lane changes from j to i, and the lane distribution will remain constant.

It should be noted that the segment length L, which permits only a single lane change, may vary based on speed and possibly volume factors. This is a refinement not addressed by Worrall. Limited data collected by Worrall suggested that a segment length of 250 ft satisfactorily met all requirements under all observed flow conditions.

Transition matrices may be applied to weaving or ramp situations by the separate consideration of flow components and the appropriate use of absorbing elements in the array. An absorbing element is one that does not allow transitions out of the lane. This is convenient and allows the prediction of lane changes only in the direction of interest (weaving vehicles are assumed not to make any reverse lane changes). Separate lane-changing matrices are used for each component flow.

Consider, for example, the four-lane weaving section of Figure 3. It is possible to consider separate lane-changing matrices for weaving movements AY and BX. Absorbing states in the two matrices are set such that only lane changes in the direction of the weave are permitted. This simplifying assumption is well-founded in observed lane-changing behavior.

It is assumed that the values for p_{ij} are constant. Although it might be expected that p varies depending on conditions such as relative volumes and segment of the weaving section, an investigation into the nature of these probabilities (2) showed that p does indeed tend to be constant.

The lane-changing matrices for movements BX and AY are given by

$$P_{BX} = \begin{bmatrix} 1 & 0 & 0 & 0 \\ p & (1-p) & 0 & 0 \\ 0 & p & (1-p) & 0 \\ 0 & 0 & p & (1-p) \end{bmatrix}$$

and

$$P_{AY} = \begin{bmatrix} (1-p) & p & 0 & 0 \\ 0 & (1-p) & p & 0 \\ 0 & 0 & (1-p) & p \\ 0 & 0 & 0 & 1 \end{bmatrix}$$

If vectors α and β are defined as the distribution of weaving vehicles at the entrance and exit of the weaving section respectively, then

$$\alpha_{BX} = [0 \quad 0 \quad \alpha_3 \quad \alpha_4]$$

$$\alpha_{AY} = [\alpha_1 \;\; \alpha_2 \;\; 0 \;\; 0]$$

$$\beta = [\beta_1 \;\; \beta_2 \;\; \beta_3 \;\; \beta_4]$$

It is seen that

$$\beta = \alpha \times P^N$$

The vector β includes the possibility of unsuccessful weaving movements. For example, for movement BX, $\beta_1 + \beta_2$ represents the percentage of successful weaves, that is, those that completed lane changes into one of the lanes of their desired exit leg. $\beta_3 + \beta_4$ represents the percentage of unsuccessful weaves. The unsuccessful weave will most likely force its way into the proper lane at the last moment, creating a serious traffic disturbance. It may be argued, therefore, that the total number of unsuccessful or "forced" weaves is an indicator of the quality of service being provided.

For the purpose of illustration, four configurations of a four-lane weaving section of constant length are shown in Figure 4. This procedure may be applied to other configurations, with some simplifying assumptions:

1. Weaving vehicles entering on a given leg will be evenly distributed among the lanes of that leg;
2. Illustrative computations will assume that movement BX is of primary importance;
3. The section under consideration will be 1,500 ft long, an average length for such configurations;
4. The unit segment length L will be 250 ft, the length determined by Worrall for most freeway conditions (N = 6); and
5. The value of p will be varied, and comparative results will be studied.

Now, the four configurations of Figure 4 may be compared.

For the configuration shown in Figure 4a,

$$\alpha_{BX} = [0 \;\; 0 \;\; 0.5 \;\; 0.5]$$

$$\beta_{BX} = [\beta_1 \;\; \beta_2 \;\; \beta_3 \;\; \beta_4]$$

The probability of a successful weave $PR_{BX} = \beta_1 + \beta_2$ or $1 - (\beta_3 + \beta_4)$.

For the configuration shown in Figure 4b,

$$\alpha_{BX} = [0 \;\; 0 \;\; 0.5 \;\; 0.5]$$

$$\beta_{BX} = [\beta_1 \;\; \beta_2 \;\; \beta_3 \;\; \beta_4]$$

$$PR_{BX} = \beta_1 + \beta_2 + \beta_3 \text{ or } 1 - \beta_4$$

For the configuration shown in Figure 4c,

$$\alpha_{BX} = [0 \;\; 0.33 \;\; 0.33 \;\; 0.33]$$

$$\beta_{BX} = [\beta_1 \;\; \beta_2 \;\; \beta_3 \;\; \beta_4]$$

$$PR_{BX} = \beta_1 + \beta_2 \text{ or } 1 - (\beta_3 + \beta_4)$$

For the configuration shown in Figure 4d,

$$\alpha_{BX} = [0 \;\; 0.33 \;\; 0.33 \;\; 0.33]$$

$$\beta_{BX} = [\beta_1 \;\; \beta_2 \;\; \beta_3 \;\; \beta_4]$$

$$PR_{BX} = \beta_1 + \beta_2 + \beta_3 \text{ or } 1 - \beta_4$$

In general, if $\alpha = [\alpha_1 \ \alpha_2 \ \alpha_3 \ \alpha_4]$ and P^N_{BX} is computed, then

$$[\beta_1 \ \beta_2 \ \beta_3 \ \beta_4] = [\alpha_1 \ \alpha_2 \ \alpha_3 \ \alpha_4] \begin{bmatrix} 1 & 0 & 0 & 0 \\ 1 - (1 - p)^N & (1 - p)^N & 0 & 0 \\ R_1 & R_2 & (1 - p)^N & 0 \\ R_3 & R_4 & N_p(1 - p)^{N-1} & (1 - p)^N \end{bmatrix}$$

Computing the elements β_1, β_2, β_3, and β_4 and substituting in the equations for PR_{BX} yield the results given below. These are plotted for various values of p as shown in Figure 5.

Configuration	PR_{BX}
A	$1 - (1 - p)^6 - 0.5\ [6p\ (1 - p)^5]$
B	$1 - 0.5\ (1 - p)^6$
C	$1 - 0.66\ (1 - p)^6 - 0.33\ [6p\ (1 - p)^5]$
D	$1 - 0.33\ (1 - p)^6$

For a given value of p, it is apparent that the most efficient configuration for movement BX is D, followed by B, C, and A in that order. This was to be expected. Note that configuration D has two lanes in which weaving movements may take place without a lane change, thus providing two through lanes for weaving vehicles. Both configurations B and C provide one through lane for weaving vehicles, B by splitting a lane at the diverge and C by combining one at the merge. Because the merging maneuver entails greater friction than the diverge maneuver, B would be expected to be more efficient, although the analysis does not take this factor into account. The configuration shown in Figure 4a, which requires a lane change for every weaving movement, is the least efficient.

These results confirm and reinforce the hypothesis on lane utilization presented herein. Configuration D allows a greater portion of its width to be used by weaving vehicles than each of the other configurations, with B and C allowing greater utilization than A.

It is clear that the advantage of one configuration over the other decreases as p increases. Configuration, then, becomes increasingly important as p decreases.

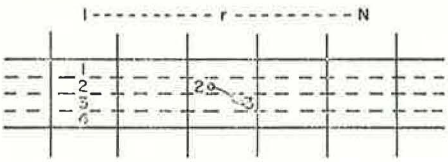
Conclusions on Configuration

Lane configuration has been shown to be a factor of great importance in weaving-area operations, particularly influencing lane utilization. The hypothesized restriction effects have been illustrated and verified by observations of actual data and by analysis.

As a result, several major conclusions regarding lane configuration may be offered.

1. Certain configurations restrict weaving vehicles to a portion of the roadway. Lanes beyond this portion may be used only by nonweaving vehicles, and width improvements will not affect weaving flows. It is therefore imperative that any design analysis procedure differentiate between weaving and nonweaving lane requirements, permitting a design reflecting the proper relative positioning of each.
2. Lane configuration is greatly affected by entry and exit lane arrangements. Therefore, design of these elements should be included in any design procedure.
3. The large difference in weaving and nonweaving vehicle speeds observed on many ramp-weave sections reflects the restrictive nature of this type of configuration and results in underutilization of outer lanes and congestion in weaving lanes.
4. There appear to be a maximum number of lanes that can be used by weaving vehicles for any given configuration. A summary of apparent maximum values of lane utilization is given in Table 3.
5. Configurations allowing some weaving movements to take place without a lane change will operate more efficiently than configurations that require lane changes for a weaving movement, given similar volumes and equal lengths and total number of lanes.

Figure 2. Lane-changing probability.



P(r)$_{23}$ illustrated

Figure 3. Four-lane weaving section.

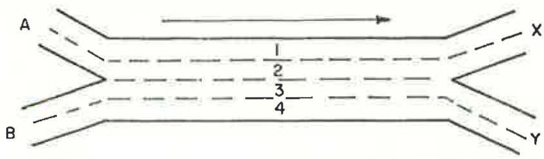


Figure 4. Alternative weaving configurations.

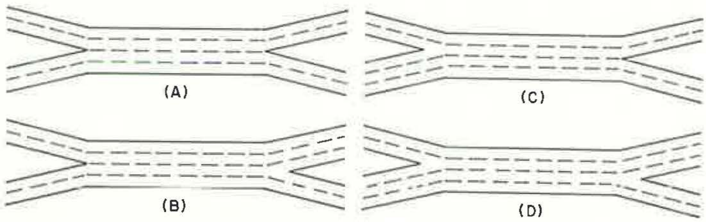
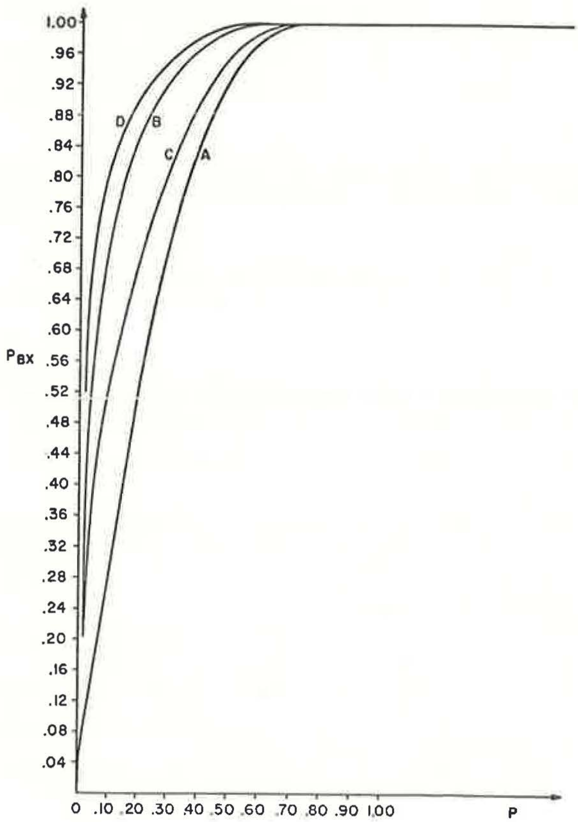


Figure 5. Comparison of configurations for various values of P for movement BX.



Use of Configuration in Weaving Design-Analysis

A procedure for design and analysis of weaving sections based on the configuration concepts presented here and on a new algorithm relating the principal weaving parameters of weaving volume, weaving ratio, and others to the design parameters of length and width (for weaving vehicles) is being developed at this writing (2). However, the results and concepts presented can be used to modify current HCM procedures to produce both results.

Principally, the HCM equation for width should be separated into its component parts. Thus,

$$N = \frac{V_T + (k - 1)\, V_{w2}}{SV}$$

becomes

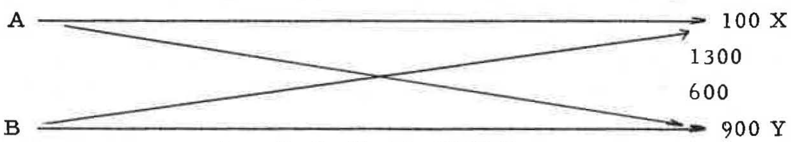
$$N_{01} = \frac{V_{01}}{SV}$$

$$N_w = \frac{V_{w1} + k\, V_{w2}}{SV}$$

$$N_{02} = \frac{V_{02}}{SV}$$

In design, this procedure gives the opportunity to examine what type of configuration is needed for a particular section and to determine the total number of lanes.

If the number of lanes required just for weaving vehicles is 2.3, a configuration requiring all weaving vehicles to make a lane change will not be adequate. Table 4 shows that such configurations limit weaving vehicles to 2.0 lanes at best. Thus, a configuration allowing the major weaving movement to proceed without a lane change is needed, and a lane dividing to two lanes at the diverge or two lanes merging to one at the merge point will be used, preferably the former. Consider the following design example:



Length limited to 2000 ft.

Let A = 1 lane, leg B = 2 lanes, leg X = 1 lane, and leg Y = 2 lanes. These may be subject to change.

Design for level of service C with a peak-hour factor = 0.91. All volumes are in passenger car equivalents.

From HCM Table 9.1, SV = 2,750/2 = 1,350 passenger cars/hour/lane. For level of service C, quality of flow = II or III (HCM Table 7.3). From HCM Figure 7.4, for V_{we} = 2,200, L = 2,000, and k = 2.95, quality of flow = III (acceptable). Now,

$$V_{01} = 100/1{,}350 = 0.07$$

$$V_w = \frac{1{,}200 + 2.95\ (600)}{1{,}350} = 2.20$$

$$V_{02} = 900/1{,}350 = \underline{0.67}$$

$$2.94 \text{ or 3 lanes}$$

The use of the HCM equation directly would have merely yielded the total of three lanes, and a weaving section would have been designed (Fig. 6). However, if the requirement that $V_w = 2.2$ is considered, this configuration, which requires that all weaving vehicles execute a lane change, will not be adequate. Weaving vehicles will be restricted to less than 2 lanes, most likely 1.8 or so inasmuch as this is really a left-hand ramp-weave (Table 4), and will experience less than level of service C. The major outer flow will experience a better level of service. This design would produce an unbalanced operation.

If the configuration is altered by adding a lane to exit X, an adequate design with the same N and L is achieved (Fig. 7). Obviously, the ability to add a lane at the exit must be weighed against the necessity to drop the lane later. However, often downstream ramps or volume fluctuations make the addition and later dropping of a lane feasible. It may be economically more feasible than providing a full interchange where a weaving design may be otherwise inadequate.

The Ward-Fairmount Study: A Case Study

The Ward-Fairmount study (6) graphically illustrated the effect of lane configuration on weaving-area operations. The section of I-8 in San Diego between Ward and Fairmount Avenues habitually experienced level of service F breakdown flow. Through two successive improvements—(a) adding a lane to the off-ramp at Fairmount Avenue, thereby creating a through lane for one weaving flow, and (b) breaking up the on-ramp into two successive on-ramps—improvements in flow were accomplished.

Although the total length of the weaving section was also increased, a major portion of the improvement in conditions was attributable to the configurational changes made.

The configurations, as well as flows, for the before, intermediate, and after conditions are shown in Figure 8. Travel times, delays, and level of service are also included.

Both the HCM and the analysis procedure being developed as part of the weaving-area operations study were used to predict the study results. As previously noted, this latter procedure is based on the configuration concepts presented herein. Approximate procedures for the after conditions are used, inasmuch as flows are not broken down by on-ramp for the multiple-weave case.

It is interesting to note that the HCM weaving procedure, when applied to the Ward-Fairmount study, predicts level of service D or E for both the before and intermediate configurations, and level of service E for the after configuration. This is clearly in conflict with the actual levels of service F, E, and D observed. The HCM procedure does not permit proper analysis of essentially unbalanced conditions, where, because of configurational constraints on lane use, weaving flows may be congested while outer flows move freely.

The procedure based on configuration and a new algorithm permits such an analysis. When applied to the Ward-Fairmount case (4), the before configuration was shown to have level of service F for weaving flows, while outer flows operated at level of service C. The intermediate condition was shown to provide balanced level of service E operation for all flows because a through lane was provided for weaving. An approximate analysis provided estimates of slightly improved balanced operation in the after case, but still in level of service E. These results clearly reflect the actual conditions observed.

SUMMARY AND RECOMMENDATIONS

The central theme of this paper has been the overwhelming importance of lane configuration as a weaving design-analysis parameter.

Data have been analyzed to show the effect of this element on the use of roadway width by weaving vehicles, particularly how poor design of lane configuration may restrict weaving vehicles to small portions of the roadway. Analytic treatment demonstrated how configurations may be altered to improve the performance of a weaving section wtihout changing length or width.

This has resulted in the development of a design-analysis procedure for weaving sections incorporating configuration as a primary element as part of the weaving-area operations study. The procedure uses new equations for weaving requirements. Because of their interim nature, they are not published here.

Table 3. Maximum utilization factors.

Configuration	Maximum No. of Lanes Occupied by Weaving Vehicles	
	Theoretical	Observed
Ramp weave	2.00	1.75
Major weave with no weaving movements without lane change	2.00+	–
Major weave with at least one weaving movement without a lane change	3.00+	3.43

Figure 6. Weaving section design using HCM equations.

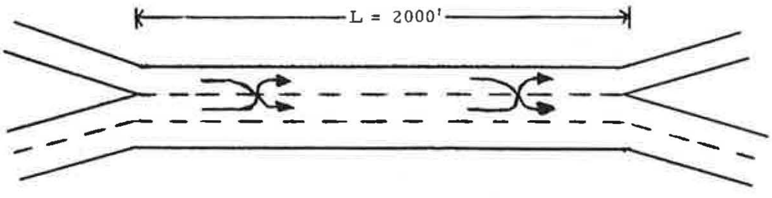


Figure 7. Weaving section design using modified HCM equations.

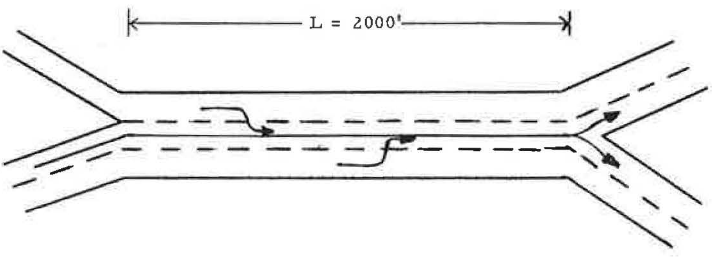
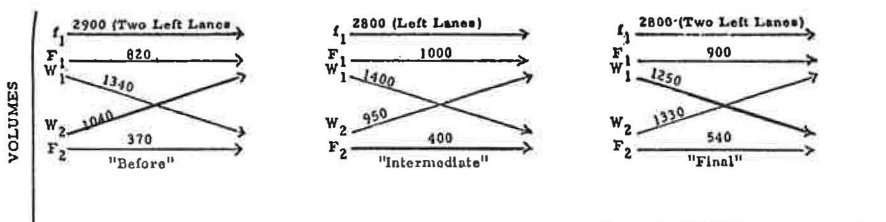


Figure 8. Before, intermediate, and final data for Ward-Fairmount study.



QUALITY OF FLOW

	TRAVEL TIMES			AVERAGE NO. OF VEHICLES IN QUEUE	DELAY		AVERAGE SPEED (MPH)	OVERALL LEVEL OF SERVICE
	Average (seconds)	Standard Deviation (seconds)	Improvement %		Average (seconds)	Reduction %		
"Before"	63	10	--	15	19	--	28	F
"Intermediate"	54	11	14	10	10	47	33	E
"Final"	44	2	19	2	0	100	41	D

COMPARATIVE RESULTS

(Travel through 2642' study section during peak hour)

The principles presented, however, have been used to modify the existing HCM procedure to include the element of lane configuration, and the use of the modified procedure has been illustrated.

It should be remembered that internal inaccuracies of the HCM procedure are not corrected by this modified procedure, although improved design due to the inclusion of lane configuration as a design element will result.

The procedure being developed under the weaving-area operation study for design-analysis of weaving sections permits a sensitivity to factors not included by existing methods. It will be possible, with consideration of lane configuration, to predict and understand the basic causes of unbalanced roadway use and the markedly different operating conditions for the several flow components. More importantly, such designs can be avoided. Lane configuration may be utilized to accomplish more efficient use of roadway space and balanced operation in which all flows experience similar operating conditions.

ACKNOWLEDGMENT

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EFFECTS OF OPPOSING FLOW ON LEFT-TURN REDUCTION FACTORS AT TWO-PHASE SIGNALIZED INTERSECTIONS

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Donald S. Berry, Northwestern University

This paper reports results of a field study to evaluate the effects of opposing flow on left-turn reduction factors in computing capacity of two-lane approaches to two-phase signalized intersections with no separate left-turn lanes. Time-lapse photography was used to gather data on the approaching and opposing volumes and the delaying effects of left turners. Reduction factors were computed for different numbers of left turns per cycle by using regression analysis. The two variables that explained most of the variance were the ratio of the opposing flow in the curb lane to its capacity and the ratio of the arrival time of the first left-turning car to the green-plus-yellow time. Results indicate that, when the opposing curb lane is loaded, left turns have a much greater reducing effect than that given in the Highway Capacity Manual. The capacity-reducing effect of a single left-turning vehicle is greater than the incremental reduction effect of the second, third, and fourth left turners in a signal phase.

• THE CAPACITY of intersections controlled by two-phase traffic signals is adversely affected by left-turning vehicles, especially when the opposing traffic volume is high. Reduction factors for left turns, as given in the Highway Capacity Manual (1) for the basic case of no separate left-turn lanes or signals, do not reflect the effects of different levels of opposing traffic volume.

The purpose of this study was to evaluate the effects of opposing flow on left-turn reduction factors and capacity for two-phase signalized intersections with no separate left-turn lanes.

STUDY SITE

The field study was conducted at a suburban intersection in Skokie, Illinois (Fig. 1). The traffic being studied entered on the two two-lane approaches of Golf Road during the same signal phase, with green times varying from 20 to 60 s, as controlled by a fully traffic-actuated controller. Parking was prohibited, and interference from pedestrians and buses was negligible. Right turns were not permitted on red. Golf Road carries predominantly commuter traffic and has very few commercial vehicles in the evening peak hours. The speed limit is 40 mph (64 km/h). Volumes per cycle arriving from the west vary considerably because of the presence of an eight-phase fully traffic-actuated signal at Waukegan and Golf Roads 0.35 mile (0.56 km) west of the Gross Point intersection. This resulted in considerable variation in opposing volumes for different cycles. In the evening peak hours, right turns average 2.8 percent and left turns about 10 percent.

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DATA COLLECTION

Time-lapse photography was used to record entering and opposing volumes on Golf Road and the delaying action of left-turning vehicles, as affected by their arrival times. The films were taken in evening peak hours in February and March 1973; some films were taken in 1970 (7). Film speeds of 60 and 100 frames/min were used. A total of 364 cycles were analyzed, with analysis confined to the green-plus-yellow intervals for Golf Road. Filming was done only on clear days and under dry pavement conditions.

Separate field measurements were also made by cycle for entering volumes and headways in lane 1 (the curb lane) for both approaches on Golf Road by using hand tally counters and stopwatches. These counts, made only for loaded portions of cycles in evening peak hours, provided data on starting time delays, entering headways, and percentages of right-turning vehicles and permitted computation of capacity of these lanes for use in computing opposing volume-capacity (v/c) ratios for opposing lane 1 volumes. Results given in Table 1 are for through-car equivalents and conform to values obtained in earlier studies in the area (9).

DATA PROCESSING

A modified 16-mm Kodak Analyst projector was used in projecting the films taken. Traffic volumes for each lane for each direction were counted, and delays for individual left-turn movements were recorded. Only phases having at least one left-turning vehicle were analyzed. The data were analyzed by converting volumes to equivalent through-car units (TCUs), with commercial vehicles considered as equivalent to 2.0 TCUs and right-turning cars equivalent to 1.25 TCUs, as used in the Australian method of computing capacity (4). Delays due to left turns were determined from the film frame numbers recorded when the rear wheels of through and left-turning cars crossed the reference lines shown in Figure 1. All times as recorded in frame numbers were punched on computer cards for analysis.

CAPACITY REDUCTION FACTORS

The capacity of the approach with no left turns, in TCUs, was considered to be double the through-car capacity (TRUCAP) of lane 1, as determined from the field stopwatch study. The possible capacity (POCAP) of the center lane (lane 2) when left turners were present was defined as the number of left-turning cars that turned during the cycle plus the number of through cars that could have entered during the remaining green time of the cycle while the lane was not blocked by left turners.

The left-turn reduction factor (REFA) was defined as

$$\text{REFA} = \frac{\text{TRUCAP (lane 1)} + \text{POCAP (lane 2)}}{2\ \text{(TRUCAP)}}$$

REGRESSION ANALYSIS

To calculate the capacity and the left-turn reduction factors for an approach with varying opposing flows and left-turn movements, we used the BMDO2R computer program for a stepwise multiple regression analysis. In the regression analyses, the only cycles analyzed were those in which there was at least one left-turning vehicle in the approach being studied. Corrections were made later for the proportion of cycles having no left-turning vehicles (see Table 3).

Many variables were examined in the regression analysis. Those that remained after preliminary runs include the following:

1. Volume-capacity ratio of equivalent through cars in lane 1, opposing (VITRCO);
2. Volume-capacity ratio of equivalent through cars in lane 1, approaching (VITRCA);
3. Ratio of arrival time of the first left-turning car in lane 2, approaching, to the green-plus-yellow time of the phase (RAT); and
4. Number of left-turning vehicles, approaching, in the cycle (ALTA).

Regression analyses were made with data grouped according to light, medium, and heavy opposing flows (v/c ratios for lane 1, opposing, of 0 to 0.5, 0.5 to 0.75, and 0.75 to 1.00). The coefficients for computing REFA for cycles in each of the three groups of v/c ratios of lane 1, opposing, are given in Table 2.

The multiple correlation coefficients obtained were between 0.60 and 0.70 for the three stratified groups of opposing v/c ratios for lane 1. Inasmuch as many factors influencing the intersection performance are difficult to control, such as gap acceptance of left-turn vehicles and the arrival patterns of opposing queues, which were not considered in this study, the results obtained can be considered reasonably accurate.

Figure 2 shows an example of the extent of scatter for data points for a subgrouping of v/c ratios of lane 1, opposing, of 0.6 (0.55 to 0.65). Each data point represents one cycle and has an RAT value noted beside the point. The scattering of points along each vertical line reflects in part the effects of different RATs.

Of the variables considered in the regression analysis, RAT and VITRCO explained most of the variance of the reduction of capacity. ALTA makes only a minor additional contribution to the variance explained by the regression analysis, whereas the opposing left-turn movement and the opposing through-car traffic in the center lane were found to be covered by other variables and of little added value in predicting the reduction of capacity.

The importance of the ratio of arrival time of an approaching left turner to the green-plus-amber time on capacity can easily be verified by observing the traffic performance of a similar intersection. When no left-turn vehicles arrive in the center lane at the beginning of green, drivers of through cars in lane 2 approaching the intersection will usually use the center lane until the lane is blocked by an approaching vehicle desiring to make a left turn. If a left-turn car arrives at the beginning of the green period or has been left over from a previous cycle, through-car drivers will shift to lane 1 to avoid being trapped in the center lane, thereby leaving the lane primarily to a few left-turn vehicles.

The higher the opposing traffic flow is, the longer the delay experienced by left-turning cars will be, which will result in fewer through cars being able to use the center lane. This is evident from computations of distribution of volumes by lane. The ratios of approaching traffic volumes using the curb lane versus the center lane were observed as 1.53, 4.0, and 5.85 for opposing v/c ratios in lane 1 of 0 to 0.50, 0.51 to 0.75, 0.76 to 1.00. This explains why the through traffic in the opposing center lane is of little importance in predicting the reduction of capacity when there is high opposing traffic flow with some left turns. Most of the opposing through traffic used the curb lane.

MODIFICATION OF RESULTS OF REGRESSION ANALYSIS

In the regression analysis, only cycles having at least one left-turn car per green-time phase were included. Therefore, adjustments were made for the regression equations to consider the percentage of cycles having 0 left-turn cars for given average numbers of left-turn cars per cycle.

The frequency of cycles with different numbers of left turns per cycle was examined. The arrival pattern was found to follow the Poisson distribution (Fig. 3). The Poisson distribution thus was used to compute the proportion of cycles with 0 left turns (approaching) for different average numbers of left turners per cycle.

For cycles with 0 left turns (approaching), it is assumed that the capacity of lane 2 (approaching) is the same as for lane 1 approaching. The modified reduction factors for different average numbers of left-turning cars per cycle are given in Table 3.

These equations in Table 3 are being studied further to determine whether it is proper to assume that the capacity of the approach in cycles with no left turns is double that of lane 1. Preliminary studies of cycles with no left turns show that drivers still tend to queue in the right lane to avoid the possible effect of a left turner. This effect seems to vary with the average percentage of left turns. Full capacity of lane 2 appears to be attained only by prohibiting left turns.

Figure 4 shows the modified reduction factors for an opposing v/c ratio in the curb lane of 0.6 for various values of ALTA and RAT. This figure also shows a dashed line

Figure 1. Study site.

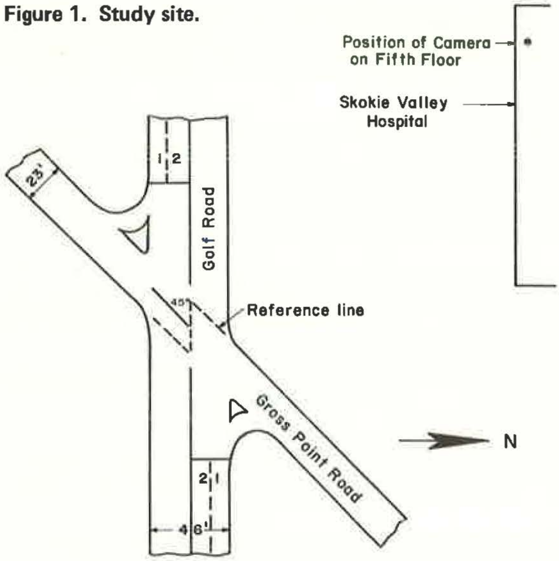


Table 1. Lane 1 discharge data for Golf Road for loaded portions of cycles.

Direction	Number of Cycles	Average Starting Time Delay (s)	Average Headway (s)
Eastbound	41	3.10	2.09
Westbound	29	3.05	2.08

Table 2. Coefficients from multiple regression for computing REFA.

	v/c for Lane 1, Opposing		
Variable	0 to 0.50	0.50 to 0.75	0.75 to 1.00
Constant	0.92948	0.97550	0.76049
VITRCO	-0.38648	-0.54134	-0.20251
RAT	0.28638	0.37936	0.31750
ALTA	-0.00564	-0.00770	0.00162
VITRCA	0.02973	0.05052	-0.01417

Figure 2. Regression results for v/c = 0.6 in opposing lane 1.

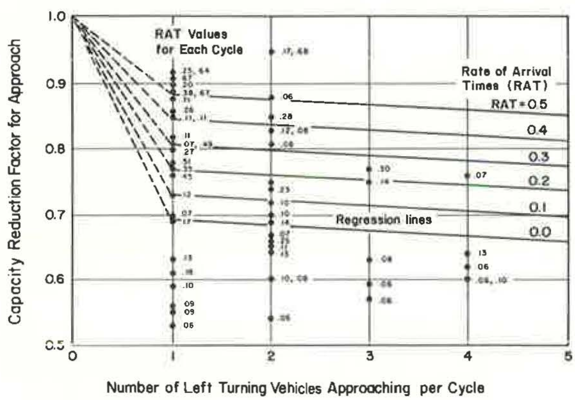


Figure 3. Observed distribution versus Poisson distribution.

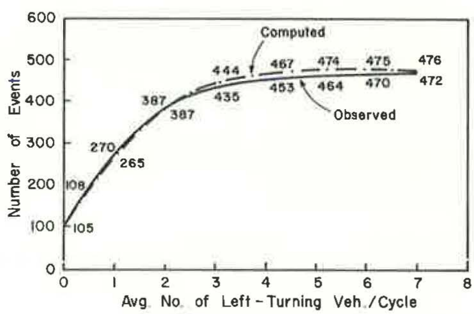
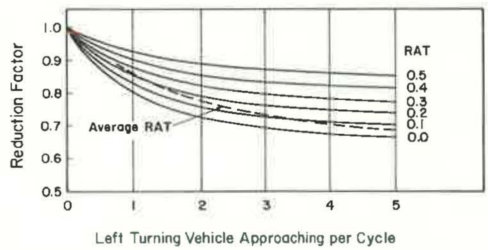


Table 3. REFA modified to include cycles with 0 left turns.

Average No. of Left-Turning Cars per Cycle	Probability of Cycles With 0 Left Turns	Modified Reduction Factor
1	0.368	0.368 + 0.632 × REFA[a] 1
2	0.135	0.135 + 0.865 × REFA 2
3	0.049	0.049 + 0.951 × REFA 3
4	0.018	0.018 + 0.982 × REFA 4
5	0.007	0.007 + 0.993 × REFA 5
6	0.002	0.002 + 0.998 × REFA 6
7	0.001	0.001 + 0.999 × REFA 7

[a]Reduction factor as obtained by the regression analysis, Table 2.

Figure 4. Reduction factors in relation to arrival times for v/c = 0.6 for opposing lane 1.



representing the average arrival times of the first left-turn vehicle for one, two, three, and four left turners per cycle.

Figure 5 shows the overall effects of left-turn vehicles on capacity, considering the observed mean values of the ratio of arrival rate for various average numbers of left-turn vehicles for the entire range of v/c ratios of opposing lane 1. This figure presents reduction factors that could be applied in capacity computations for percentages of left-turn vehicles, which correspond to the average number of left turners per cycle.

COMPARISON OF RESULTS

Left-turn reduction factors were calculated for varying conditions of opposing volumes and compared with those computed according to the Australian method and by the Highway Capacity Manual, as shown in Table 4. The results indicate that the values obtained in this study are somewhat lower than the values obtained by the Australian method for high v/c ratios in lane 1, opposing. The values are higher than those obtained by Lombaard (see Appendix). These differences warrant further study.

CONCLUSIONS

The following conclusions apply primarily to two-lane approaches to signalized intersections having conditions similar to the study site (two-phase signals, no separate turning lanes, no parking interference, and left turns from each of the opposing approaches in most cycles).

1. The v/c ratio of opposing traffic in the curb lane has a very significant effect on the capacity-reducing effects of left turns.
2. The time of arrival (RAT) of the first left-turning car (expressed as a ratio of arrival time to the green-plus-yellow phase length) also significantly affects the capacity-reducing effects of left turns (Fig. 4). RAT is related to the number of left turns per cycle and to whether any left-turn vehicles are held over from the previous cycle.
3. The capacity-reducing effect of a single left-turning vehicle in a cycle is greater than the incremental reduction effect of the second, third, or fourth left-turning vehicle in a cycle. In contrast, for the Australian method, the through-car equivalents for left-turn reduction factors are constant for different numbers of left-turning vehicles per cycle.
4. Left-turn reduction factors for 10 percent left turns, as computed in this study, appear to be affected slightly more by opposing flow than are left-turn reduction factors computed by the Australian method. At high opposing flows, the Highway Capacity Manual procedure underestimates the reducing effects of left turns.
5. The numbers of left turns and through cars in the opposing center lane do not contribute to the regression equation. Their effects, however, are reflected in the

Figure 5. Effects of left turns on capacity of a two-lane signalized approach.

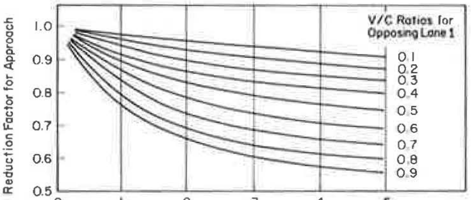


Table 4. Comparison of REFAs by Australian method, present study, and Highway Capacity Manual.

		Opposing Flow, vph		
Method	Factor	400	800	1,000
Australian	E_{LT}	2.1	3.1	3.7
	Capacity, vph	1,946	1,782	1,700
	Left turns, vph	195	178	170
	REFA	0.90	0.83	0.79
Present study, opposing lane 1	Capacity, vph	1,080	1,080	1,080
	Volume, vph	240	560	750[a]
	v/c ratio	0.22	0.52	0.69
	REFA	0.93	0.84	0.77
Highway Capacity Manual	REFA	0.91	0.91	0.91

Note: Data for two-lane approaches, saturation flow of 3,600 vphg, 10 percent left turns, two-phase signal with 60-s cycle and G/C = 0.60, no left-turn lane, and no parking or standing.

[a]Assumes that the opposing volume distribution by lane is 75/25; this depends on the percentage of opposing left turns and the v/c ratio of approaching lane 1.

v/c ratio of the opposing lane 1 flow, which is higher when opposing vehicles are diverted to lane 1 because the center lane is blocked by left turners.

6. The center lane can carry a full load of traffic only if left turns are prohibited. When left turns are allowed and more than half the cycles have at least one left-turning vehicle, drivers shy away from the center lane, and its output with 0 left turns may be much lower than the curb lane.

Further study is needed of how to adjust for cycles with 0 left turns, for different average percentages of left turns. Study of procedures for estimating the proportion of opposing traffic in lane 1, for different percentages of left turns in lane 2, opposing, and for different v/c ratios of lane 1, approaching, also is needed.

This approach to left-turn reduction factors should be studied for additional intersections, including those controlled by pretimed signals.

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APPENDIX

EARLIER RESULTS BY LOMBAARD

Lombaard (7) studied left-turn reduction factors at six two-lane approaches on Golf Road in 1970, utilizing time-lapse photography. He undertook regression analyses, using as a variable the v/c ratio of opposing traffic in the two lanes. Data were combined for those approaches in which the downstream signal was more than 0.70 mile (1.3 km) away and for those with short opposing distances (when the downstream signal was only 0.35 mile (0.6 km) away, as at the Gross Point Road intersection).

His results identified the problem of the changing distribution of flow in lane 1 and the center lane, as the v/c ratio increased for opposing flow. He found ratios of lane 1/lane 2 volumes of 2.2 for v/c ratios, opposing, of 0 to 0.50, 4.28 for v/c of 0.51 to 0.75, and 6.13 for v/c of 0.76 to 1.00. Left-turn movements declined from 11.0 to 7.7 to 6.8 percent for the three v/c stratifications.

Lombaard's reduction factors for data aggregated by cycle yielded mean left-turn reduction factors of 0.82, 0.65, and 0.62 for short opposing distances, for the three opposing-flow v/c ratio groupings of 0.0 to 0.5, 0.51 to 0.75, and 0.76 to 1.00. He concluded that, with at least one left-turning vehicle per cycle, the opposing v/c ratio affects the left-turn reduction factor more than the percentage of left-turning vehicles in the approach. He also concluded that when opposing left-turn movements are encountered, iterative or trial-and-error solutions are needed to deal with the effects of left turns on the opposing v/c ratios.

AUTOMOBILE-UTILITY TRAILER COMBINATIONS ON RURAL HIGHWAYS IN KENTUCKY

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An analysis of accident records indicated that automobile-utility trailer (AUT) combinations are involved in a disproportionately high number of traffic mishaps. Examination of the history of accidents involving AUT vehicles indicated that differential crosswinds and unanticipated driving maneuvers contribute to the driver's loss of control. AUT combinations contributed to the fatigue loss in pavement life approximately 50 percent as much as single-unit, two-axle, six-tire trucks (per vehicle). In general, AUT vehicles constituted approximately 3 percent of the total traffic stream. Analysis of speed distributions indicated an equivalency factor for AUT combinations equal to that for trucks for similar roadway types and topographical conditions.

•THE Kentucky Bureau of Highways recently completed several studies characterizing traffic on highways within the state. The first of these studies (1) established a methodology for predicting the vehicular composition of the traffic stream as related to significant local variables. A methodology was needed to increase the accuracy of predictions of cumulative equivalent axle loads (EALs). The validity of the proposed procedure depends on the accuracy of vehicle classification and loadometer data used as inputs. A second study (2) was conducted to enhance the validity of the predictive technique of the first by providing data on the lateral distribution of traffic on four- and six-lane limited-access facilities. An analysis of loadometer and classification data of traffic using bridges spanning the Ohio River from Kentucky resulted in a proposed methodology (3, 4, 5, 6) by which the fatigue life of a bridge could be evaluated.

Present methods of classifying vehicle types do not segregate automobile-utility trailer (AUT) combinations. Traffic classification counts merely denote an AUT combination as a passenger car. If a trailer is being pulled by a pickup truck, the combination is recorded as a single-unit, two-axle, four-tire truck. In compliance with this practice, previous studies of traffic characteristics (1, 2, 3, 4) made no special notation of these vehicles. However, a surprisingly large number of automobiles pulling utility trailers were noted by the data collectors. Preliminary observations indicated that during peak periods of traffic flow up to 10 percent of the total traffic stream was AUT combinations.

The present study, therefore, was conceived with the following objectives:

1. To establish the presence of AUT combinations on certain rural Kentucky highways,
2. To ascertain the effect of AUT combinations on capacity (level of service) for various highway types and various dissimilar highway sections (in terms of number of equivalent automobiles),
3. To provide a basic data bank for denoting quantitative trends for this vehicle type in the future,
4. To examine the advisability of counting AUT combinations separately in classification studies,

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5. To consider the effect that AUT axle loads have on the total equivalent axle load accumulation, and
6. To investigate accidents involving AUT vehicles.

ACCIDENT DATA AND ANALYSIS

Preliminary comparisons of accident involvement rates of AUT combinations to percentages of this vehicle type in the traffic stream revealed a glaring disproportionality (Table 1). These data were obtained from toll road records (7) and from available accident reports. Inasmuch as these figures are valuable only for intuitive purposes, it was anticipated that a detailed analysis of accident records would provide additional information.

Extensive accident records (1965-68) of Kentucky highways were available for analysis. The geographical distribution of roadways investigated is shown in Figure 1.

Initially, AUT accident trends were compared with those of accidents in general. The procedure involved examination of all single-vehicle accidents, accidents involving AUT combinations, single-vehicle accidents involving AUT combinations, and traffic volumes by means of a graphical representation of trends by hour of day, day of week, and month of year. Typical distributions of total traffic volume, total accident occurrence, and total AUT accidents are shown in Figure 2. There was no marked difference (except for the smoothness of the curves as a function of sample size) between hourly distributions of AUT accidents relative to traffic volume distribution and hourly distributions of all accidents. The same was true for single-vehicle accidents. During daylight hours, there was a greater percentage of single-vehicle AUT accidents than single-vehicle accidents; at night the opposite trends were evident. It was hypothesized that these trends were caused by the lower volume of AUT traffic at night.

Typical accident and traffic volume distributions by day of the week are shown in Figure 3. Again, similarities were apparent. However, accident and volume distributions of AUT traffic showed marked differences. Tuesday was the lightest day for AUT traffic, yet Tuesday was the third highest day for AUT accident occurrence. A similar situation existed for Friday, whereas for Saturday the opposite was true. Thus AUT traffic and AUT accidents cannot be said to coincide to the degree that was exhibited for all traffic and all accidents. For all single-vehicle accidents, similarities with volume distributions were again evident. Once again, the condition of more accidents than volume for AUT single-vehicle accidents prevailed for Tuesday and Friday; the opposite held true for Monday and Saturday. It may be concluded from these observations that the distribution by day of the week of all accidents, both single-vehicle and total, was not identical to that of similarly classed AUT accidents.

Distributions of accidents and volumes by month are shown in Figure 4. Generally, AUT accidents illustrated the same trends as all accidents. There were, however, some notable exceptions. The percentage of AUT accidents increased markedly in April, whereas the percentage of all accidents dropped significantly. The trends then coincided until October, when AUT accidents rose noticeably over a rather exaggerated September low. At the same time, all accidents decreased slightly from September to October. Again, in November, the percentage of AUT accidents dropped perceptibly while the percentage of accidents in general increased slightly. If we discount exaggerations (again probably caused by small sample sizes), trends in single-vehicle accident and single-vehicle AUT accident distributions seemed to follow similar patterns with the exception of the previously noted differences for October and November. Volume of AUT traffic (as a monthly percentage of the yearly total) increased significantly during the summer months; a corresponding increase in accident proportions was not observed. A relatively high percentage of AUT accident occurrence during December and January was countered by the lowest number of AUT vehicles during these 2 months. This suggests that AUT accidents, like accidents in general, correlate rather highly with periods of inclement weather and reduced visibility. The distribution of single-vehicle AUT accidents shows similar features to all AUT accidents, but the increase in summer accidents corresponding to high summer volumes was more noticeable.

Another manner in which accidents involving AUT combinations can be compared

Table 1. Accident involvement of AUT combinations during 1967 and 1968.

Road	Accidents Involving Combinations (percent)	Combinations in Traffic Stream (percent)	Ratio
Bluegrass Parkway	8.92	2.96	3.01
Kentucky Turnpike	5.72	2.66	2.15
Mountain Parkway	1.54	1.27	1.21
West Kentucky Parkway	4.24	3.85	1.10
US-41	1.23	2.02	0.61
US-27	5.26	1.12	4.70
US-60	2.86	1.26	2.26
I-64	4.47	1.16	3.85
I-65	10.51	2.80	3.75
I-75	8.38	4.21	1.99

Figure 1. Geographical distribution of roadways investigated.

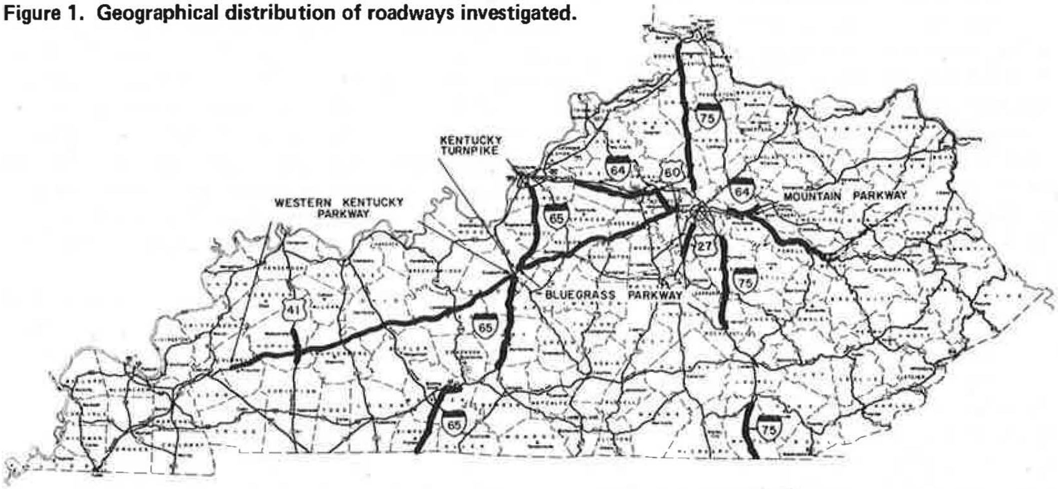
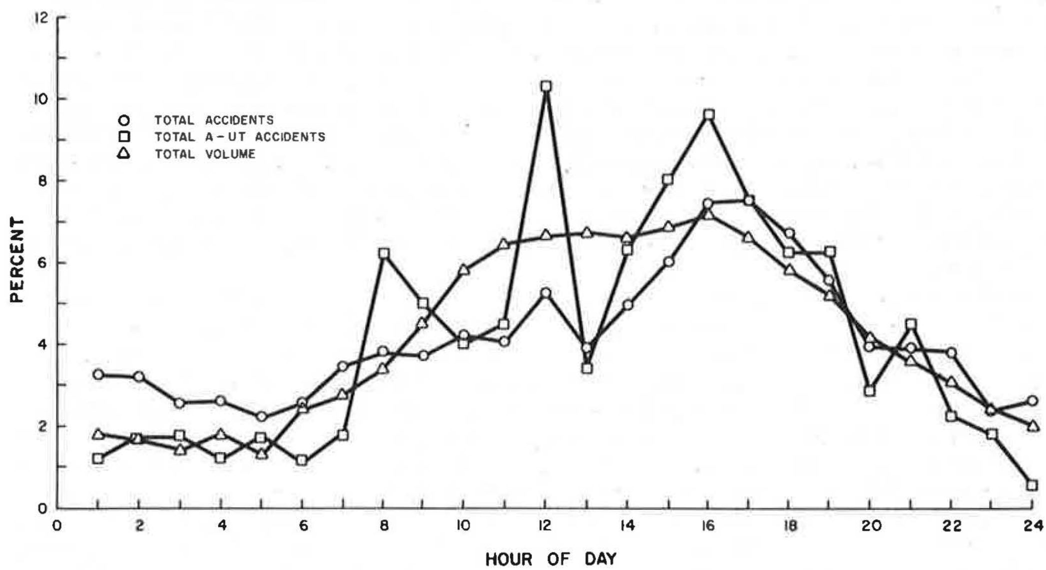


Figure 2. Accident and volume distributions by hour of day for I-75 in Scott County.



with other types of accidents is by distribution in space. It was hypothesized that any location at which the number of AUT accidents was much greater than that of accidents in general could be analyzed for possible contributing factors. A typical spatial distribution of accident occurrences is shown in Figure 5. The methodology to select sites for detailed investigation initially identified all locations at which at least two AUT accidents had been reported. Judgment was then used to ascertain whether the number of AUT accidents represented a disproportionate percentage (60 percent or more) of the total number of accidents reported at that location. It was decided that, although specific accident records at each site could provide insight into probable causes of the problem, they would be best used as a supplement to on-site investigations.

One location was situated on a relatively steep vertical downgrade in relation to several relatively deep rock cuts. Crosswind conditions created by such cuts have been recognized to contribute to accidents. It was hypothesized that crosswinds would affect AUT vehicles more than they would automobiles because of the increased surface area on which wind forces could act. Sudden steering reactions required when a vehicle is subjected to differential crosswind could add to the already difficult task of controlling an AUT combination. Two other locations were similar to the first. At these sites, however, steep grades reduced the speeds of AUT combinations, inducing other vehicles to overtake and pass. The passing of a vehicle also creates a wind loading on both the passing and passed vehicle. Thus these particular accident sites indicated that at least some AUT accidents occur at locations where cuts induce crosswinds or where steep grades lead to wind currents from passing vehicles. These wind factors may be sufficient to affect AUT vehicles although they may not necessarily affect other traffic to such a negative extent.

Another site involving a disproportionate number of AUT combination accidents was a section of six-lane Interstate roadway (three lanes in each direction) with relatively high traffic volumes. Informational signs announcing exit ramps and availability of gas, food, and lodging may precipitate weaving by all traffic and especially AUT traffic. There was also a median crossover at this site; a waiting vehicle within the crossover could induce erratic maneuvers within the traffic stream and thus indirectly create a traffic conflict or a collision. Therefore, the high rate of AUT accidents at this site was probably induced by weaving maneuvers performed during high traffic volume conditions. At another site, the only indicative factor was a blank blue sign panel that previously was lettered REST AREA 2 MILES. It was not known whether the sign message appeared at this site, but there is no subsequent rest area to warrant such a message. Had this sign been erected with such a message, weaving would have been induced. There do not appear to be any contributing conditions, other than some advanced directional signing and the overpass of a county road with its concomitant bridge piers. At a final location, nothing notable in the way of signing appeared in the southbound lanes, but, in the northbound direction, several sign panels preceding an exit (EXIT 1 MILE, GAS-FOOD-LODGING) seemed to present a situation that could induce weaving. In addition, a combination of the cut profile and tree patterns adjacent to the roadway created a situation where wind could be a problem. There was also a crossover located in the area. Specific accident records did not indicate this crossover to be a problem. The primary problem at this site appeared to be a combination of wind and weaving.

A general purview of records of accidents involving AUT combinations seemed to indicate that the primary sources of trouble were trailer hitches becoming loosened while vehicles were in motion and a general loss of control of the AUT combination. There was nothing to indicate that loss of control could be solely attributed to conditions of wet weather. Situations seemed to indicate that more often loss of driver control resulted from wind gusts created by roadway topography or overtaking vehicles. Such situations are difficult if not impossible to correct through modification of the roadway. The apparent difficulty lies with the vehicle itself and not with any roadway disparity. Of course, roadway situations in deep cuts and steep grades, which may contribute to a wind problem, are the result of a desire for economic optimality. Possible elimination or reduction of such situations is necessarily a trade-off against the economic toll of accidents induced by such features. The important factor is that these situations can present problems and may be genuine causes of accidents.

Figure 3. Accident and volume distributions by day of week for I-75 in Scott County.

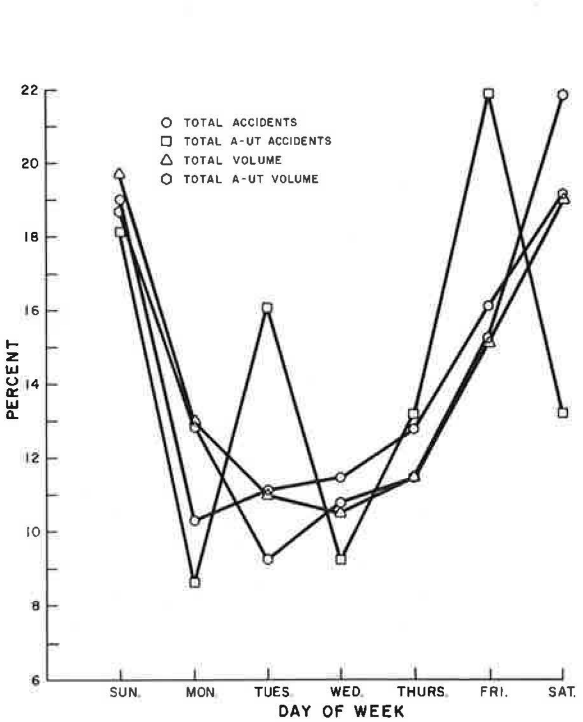


Figure 4. Accident and volume distributions by month of year for I-75.

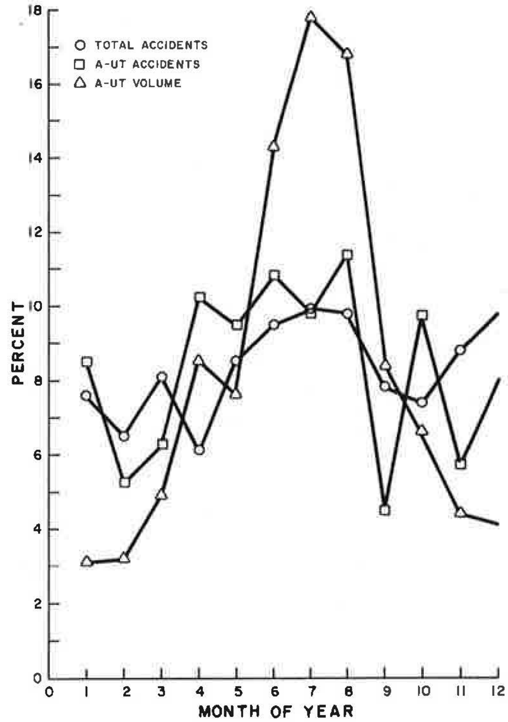
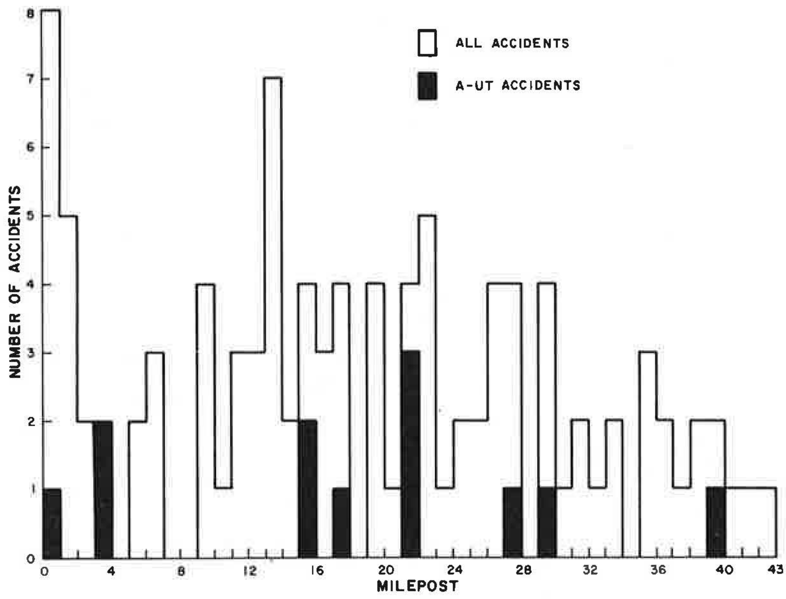


Figure 5. Spatial distribution of accidents on the Bluegrass Parkway.



As a final step in the accident analysis, frequency rates of AUT accidents were compared with the rates of all accidents. The common denominator of this analysis was the accident rate per 100 million vehicle-miles. Reliable measures of such rates were obtained by analyzing accident records, ADT values, and roadway lengths for all accidents. Similarly, rates were computed for AUT accidents by using the number of AUT accidents, the appropriate roadway length, and the volume of AUT traffic. AUT volumes were computed by using data obtained from traffic classification counts and by expanding this information with proper expansion factors. Using the volume of AUT combinations was thought to be a more legitimate procedure than using total volumes and AUT accidents.

Results of the analysis for 10 sections of roadway are shown in Figure 6. The four toll roads are four-lane limited-access highways with attendant toll facilities. US-41 and the three Interstate roadways are four-lane limited-access highways with no toll facilities. US-27 is a two-lane rural highway, and US-60 is a four-lane, no-toll, no access control facility. For the toll roads, the ratio of AUT rates to total accident rates had an unweighted mean value of 0.97. This was markedly different from the unweighted mean value (3.32) for the four toll-free, four-lane, limited-access facilities. This disparity could not be related with any statistical significance to traffic volume, median design, or accidents that occurred on toll facilities. Likewise, no correlation could be established with percentage of AUT vehicles in the traffic stream. Consideration of density did not offer a solution. Finally, this situation was judged to be the result of data sample size. A closer examination of Figure 5 reveals several peculiarities that could most aptly be related to sample size. For instance, the two-lane section of US-27 had the lowest accident rate of all roads considered. This did not conform to intuitive reasoning, for US-27 carried a relatively dense traffic stream. Furthermore, many AUT accident rates were based on a single AUT accident. Undoubtedly, larger accident sample sizes would provide better indications. In general, however, it can still be said that the frequency of AUT accidents was greater than that of accidents involving automobiles alone. The unweighted combination of the statistics shown in Figure 5 indicated that AUT accidents occur 2.35 times more than all accidents.

ANALYSIS OF WEIGHT DATA

To test the hypothesis that the AUT combinations contribute significantly more to accumulated equivalent axle loads on a pavement structure than standard automobiles, we proposed obtaining sample weights of AUT vehicles. No records were available of any previous loadometer data on AUT combinations in Kentucky. A literature search did not reveal any data acquired elsewhere. Principal determinants in selecting weighing sites were compatibility with accident data and availability of facilities for weighing vehicles. Extensive accident records were available for rural, limited-access facilities in the state—both toll roads and Interstate highways. Permanent loadometer stations had been constructed in conjunction with several Interstate facilities, and three of these installations were in operation. The I-75 weigh station was located in Scott County, the I-64 station in Shelby County, and the I-65 station in Hardin County.

Weighing operations were conducted only during the 16-hour period between 6 a.m. and 10 p.m. during the summer of 1970. AUT traffic between 10 p.m. and 6 a.m. did not appear to warrant the inclusion of this time period in the weighing operations. This decision was justified by the number of AUT vehicles finally weighed on I-65 and I-75 (114 and 202 respectively). Thus, a statistically large sample of vehicles in each direction of travel was weighed. However, only 49 vehicles were weighed on I-64. Of these, 21 were eastbound vehicles and 28 were westbound. The relatively smaller number of vehicles weighed was partially attributable to the small daily traffic volumes on I-64 and because of less responsiveness on the part of AUT combination drivers to enter the weigh station area. For each set of data, representing each AUT combination weighed, axle loads, axle spacings, direction of travel, roadway name, and type of trailer being pulled were recorded.

It was desirable to separate the trailers into distinguishable categories so as to evaluate trends for given trailer types. However, it was realized that to obtain statistically significant sample sizes there was a certain practical limit to the number of categories that could be used. As the number of categories increased, the size of each data subset necessarily decreased. Thus, it was decided to categorize the vehicles into three to six classes. A pilot study of vehicle classification was conducted before any data were collected for use in the study to establish procedures and determine classifications of trailers to be used in the actual data collection process.

With the exception of the relatively small amount of data acquired at the I-64 weigh station, the 16-hour weighing period provided statistically sufficient data sample sizes. At the I-64 weigh station, the gross number of vehicles weighed (49) was a significant sample size, but subdivisions of the data into smaller groupings reduced the size of samples below that generally regarded as being statistically large (i.e., 30).

The relationship between vehicle load and contribution to fatigue, whether the fatigue being considered involves structural metallic materials (as in bridge members) or asphaltic or portland cement concrete pavement substances, can best be analyzed by consideration of discrete loading distributions. The initial phase of weight data analysis was to calculate values of selected characteristics. Results of this analysis are given in Tables 2 and 3.

Because the principal intended use of the axle weight data was their application to pavement design techniques, decomposition of these data into subsets of vehicle type, road name, and direction of travel was a necessity if trends peculiar to a certain subset were to be identified (1). However, if certain subsets could be examined with extraneous variables eliminated, the analysis could pinpoint more accurately the source of these trends. To determine whether certain aspects of data subsets were combinable required that appropriate statistical tests be used to examine the equality of means and variances: the Smith-Satterthwaite t-test for equality of means and the F-test for equality of variances. Each of these statistical analyses was performed at the 95 percent level of confidence, with the $\alpha = 0.05$ region divided into two tails.

A rather arbitrary method was necessarily chosen to evaluate results of the statistical comparisons. Four criteria were established. The first was the acceptable statistical combination of three of the four axle loads. The second was the acceptability of combining gross loads. The third examined the combinability of two of the three axle loadings. Statistical lumping of the wheel base was the final criterion. If three of the four criteria were satisfied, it was deemed sufficient evidence of the combinability of the statistical parameter under study. As a result of these tests, the only data lumping deemed proper was that of I-64 eastbound with I-64 westbound and that of I-65 northbound with I-65 southbound.

Pavement design philosophies embody a concept of failure by fatigue in both flexible and rigid pavements and recognize the fatigue-contributing equivalence of a certain number of passages of a standard axle load to a single passage of another weight. The passage of a sufficiently heavy axle contributes to a reduction in the remaining fatigue life. Thus, any unanticipated increase in the number of axle loads from any traffic source could theoretically decrease the useful life of the pavement. Because AUT combinations are categorized merely as automobiles in traffic classification studies, trailer axles are not included in pavement design analyses. If the trailer axles should prove to be relatively heavy, then the damage to the pavement could be significant. When both car axles and trailer axles are considered in a cumulative fatigue analysis for flexible pavement design, the additional EALs accumulated for a 20-year design period for a roadway with significant AUT traffic was approximately 5 percent. AUT combinations contributed to the fatigue loss in pavement life about 50 percent as much as single-unit, two-axle, six-tire trucks (per vehicle).

TRAFFIC COUNTS

Locations for classification studies were restricted from both the aspect of compatibility with available accident records and facilities available for loadometer studies and the aspect of congruity with radar speed study information. Visual classification surveys were conducted in the vicinity of the three loadometer stations. Additional sites were selected to provide data from different classes of roads. One site was

Figure 6. Summary of accident rates.

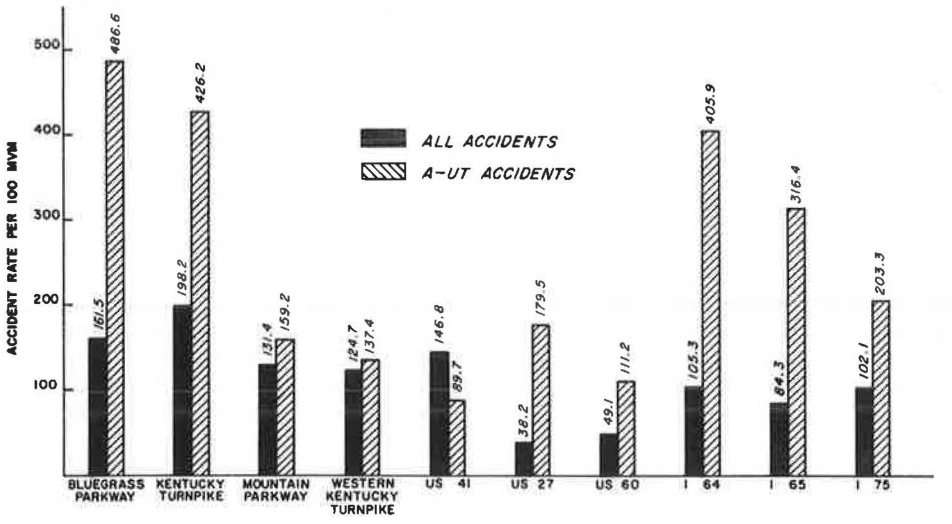


Table 2. Summary of AUT axle weights and spacings by roadway.

	I-75		I-75 Northbound		I-75 Southbound		I-65		I-65 Northbound		I-65 Southbound		I-64		I-64 Eastbound		I-64 Westbound	
Item	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.
1st axle weight (lbm)	2,270	76	2,233	465	2,303	400	2,338	341	2,360	339	2,325	344	2,258	351	2,301	94	2,224	388
2nd axle weight (lbm)	2,697	112	2,605	308	2,778	337	2,771	177	2,725	588	2,800	499	2,579	485	2,549	455	2,603	515
3rd axle weight (lbm)	1,798	164	1,871	214	1,733	250	1,893	352	1,730	993	1,995	524	1,693	894	1,742	1,165	1,654	820
4th axle weight (lbm)	1,730	642	2,360	781	1,507	419	2,088	669	2,028	867	2,112	617	1,610	750	1,100		1,695	783
Gross load (lbm)	6,949	222	6,856	148	7,032	262	7,325	314	7,041	934	7,502	507	6,756	824	6,643	1,571	6,845	1,573
1st axle spacing (ft)	9.9	0.6	9.8	0.7	9.9	0.5	10.0	0.6	9.9	0.6	10.0	0.6	10.0	0.5	10.1	0.4	10.0	0.6
2nd axle spacing (ft)	14.4	2.6	14.0	2.4	14.7	2.8	14.3	2.8	14.0	2.7	14.5	2.9	13.9	2.4	13.6	2.3	14.1	2.5
3rd axle spacing (ft)	2.5	0.4	2.4	0.6	2.5	0.4	3.2	1.6	3.1	0.2	3.3	2.0	2.8	0.2	2.8	0.4	2.8	0.1
Wheel base (ft)	24.6	3.2	23.9	3.0	25.1	3.4	24.7	3.5	24.2	3.3	25.1	3.6	24.4	3.1	24.0	2.4	24.7	3.3

Note: 1 lbm = 0.45 kg; 1 ft = 0.3 m.

Table 3. Summary of AUT axle weights and spacings by trailer type.

	All Data		One-Axle Trailers		Two-Axle Trailers		Boat Trailers		House Trailers		U-Haul Types of Trailers		Others	
Item	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.
1st axle weight (lbm)	2,290	118	2,269	98	2,417	337	2,357	521	2,459	407	2,194	349	2,193	334
2nd axle weight (lbm)	2,704	89	2,657	136	3,014	661	2,788	371	2,781	336	2,538	505	2,713	170
3rd axle weight (lbm)	1,814	117	1,791	56	1,878	649	1,530	704	2,906	394	1,483	454	1,366	298
4th axle weight (lbm)	1,847	681			1,847	681	1,483	418	2,518	520	1,807	647	1,756	733
Gross load (lbm)	7,041	88	6,713	123	9,156	915	6,992	665	8,412	456	6,453	464	6,439	306
1st axle spacing (ft)	9.9	0.6	9.9	0.6	10.3	0.4	10.0	0.9	10.1	0.4	9.8	0.6	9.9	0.5
2nd axle spacing (ft)	14.3	2.7	14.0	2.4	16.1	3.5	17.1	2.1	15.7	2.4	12.6	1.4	13.0	2.0
3rd axle spacing (ft)	2.9	1.1			2.9	1.1	2.3	0.3	2.9	0.1	2.9	0.1	3.3	1.9
Wheel base (ft)	24.6	3.3	23.8	2.6	29.3	3.3	27.7	2.8	26.2	3.1	22.8	2.3	23.3	2.6

Note: 1 lbm = 0.45 kg; 1 ft = 0.3 m.

located on US-41 in Hopkins County. Other locations selected were on US-27 in Jessamine County and on US-60 in Woodford County. It was believed that these six classification study locations, combined with information available from four toll roads, would provide necessary classification information for purposes of this study.

At each site, there was a physical limitation on the number of varying types of information that could be obtained for each count. Some information desired included the lane distribution of total traffic and of AUT traffic and information on whether automobiles had trailer hitches. During any one count period, distribution of traffic by lane or the separation of those vehicles having trailer hitches could be recorded, but not both. A count of cars with trailer hitches was an indicator of the potential of AUT combinations on the roadway. At sites on I-64, I-65, US-27, US-60, and US-41 and the short count on I-75, data concerning trailer hitches were recorded. For the week-long count on I-75, where determination of the presence of a trailer hitch during darkness was difficult, we decided to record the lane distribution of automobiles and of AUT combinations.

A long count (a staggered, week-long study that included each hour of the week) was conducted on I-75 in Scott County. Personnel limitations precluded a 24-hour per day, 7-day continuous count. The remaining counts, which were short, were conducted at locations on I-65, I-64, US-27, US-60, and US-41. The short counts were of 12-hour duration from 8:00 a.m. to 8:00 p.m. These data were supplemented by toll receipts.

Before the classification information was obtained, a method to classify trailer types was chosen. An investigation of the licensing procedure in Kentucky indicated that only "house trailers" and the general class of "trailers" were licensed; a better stratification of trailer type information was needed. During initial counts, data collectors observed that an unusually large number of miscellaneous trailers that could be classified separately as campers were being recorded.

Stratification of trailers by axle configuration was included because this is the type of information needed in an analysis of the effect of axle loads on the pavement. A systematic presentation of loadometer data would of necessity include those types of data needed for the computation of the average numbers of axles in various subsets. Distinction was made between those trailers having two axles closely spaced in tandem and those spaced similar to standard automobiles.

Table 4 gives the average percentages of vehicle types for each of the six roadways at which classification information was obtained. This table also presents a weighted (by volume) average of all data and of data acquired at four-lane, controlled-access facilities. It can be seen that AUT vehicles ranged from 1.12 percent of total traffic on US-27 to 4.24 percent on I-75; the weighted mean value was 2.47 percent on all roads and 3.0 percent on four-lane, controlled-access highways. Thus, the total weighted percentage of recreational vehicles on all roads was 3.48 percent and on all four-lane, limited-access facilities 4.11 percent. The range was a low of 1.75 percent on I-64 to a high of 5.56 percent on I-75.

From data obtained for I-75, it was possible to determine the distribution of vehicle types by hour of day (Table 5) and day of week (Table 6). An analysis of the percentage of AUT traffic as a function of hour of day indicates a good correlation with traffic volume. A similar attempt to relate percentage of AUT vehicles to daily volumes did not produce any significant correlation. It was hypothesized that correlation with volume was significant when day of the week could be incorporated into the percentages, but, when percentage as a function of volume is stratified by day of the week, no correlation was evident. The real meaning of this correlation was not that there was a causative relationship between AUT traffic and traffic volumes but that the increase in AUT traffic during certain periods of time was proportionately greater than the increase in traffic in general. It was obvious this was true for certain days of the week, and the data seem to indicate that this was also true for certain hours of the day.

An analysis was also performed to test the directional equality of vehicle percentages and volume percentages. At the 95 percent level of significance, the percentages of the four vehicle types and of volume were not significantly different by direction of travel.

Furthermore, the following analysis was made of the percentage of non-AUT vehicles that had a trailer hitch:

Road	Vehicles With Trailer Hitches (percent)	Road	Vehicles With Trailer Hitches (percent)
US-41	9.68	I-65	8.16
US-27	11.31	I-64	7.22

The mean percentage of such vehicles was 9.09, and the standard deviation was 1.79. There was no statistically significant difference in the percentages of non-AUT vehicles with trailer hitches. The percentage of this type of vehicle indicated a potential for as much as 10 to 12 percent of the total traffic being AUT vehicles.

Analysis of the percentage of AUT vehicles in the shoulder lane of traffic revealed an unweighted mean percentage of 90.49 when the data were stratified by hour and 88.68 percent when categorized by day. Examination of the hourly percentages revealed that, except for the period between 4 a.m. and 5 a.m. when every AUT vehicle was traveling in the shoulder lane, no particular hour had a statistically significant percentage differential. Similar analysis of percentages by day revealed no significant deviation. It may be concluded that approximately 90 percent of AUT combinations travel in the shoulder lane. Hourly distributions of the percentages of AUT vehicles in the shoulder lane are given in Table 7. Daily distributions were as follows:

Day	Percent	Day	Percent
Sunday	87.39	Thursday	88.45
Monday	90.93	Friday	86.37
Tuesday	91.08	Saturday	89.66
Wednesday	86.90		

The mean was 88.68, and the standard deviation was 1.91. The t-test indicated that Monday, Tuesday, Wednesday, and Friday have significantly different percentages.

The final analysis of traffic classification data was a summary of trailer types. A matrix of five trailer types and three axle configurations was used (Table 8). The distribution of trailer types is dominated by camper trailers; each of the other four trailer types shares an approximately equal percentage of the total. Nearly four-fifths of all trailers had one axle, and less than 1 percent had three axles. Camper trailers were the dominant type of one-axle and three-axle trailers but were the least dominant two-axle trailer. House trailers were the least prevalent one-axle trailer. With the exception of miscellaneous trailer types, house trailers were also the most prevalent two-axle trailer. There were no three-axle boat or U-Haul trailers observed. The largest single trailer type was the one-axle camper trailer.

There was one roadway section, I-75, at which the classification study extended to each hour of the week. It was hypothesized that a calculation could be made to determine the percentage of daily AUT traffic that occurs during each hour of the day and this information could be used to expand a 12-hour count to a full day's count. Similar calculations could then be made for day of the week. Information available from toll road collections could then be used to project the data from the month in which it was taken to the entire year.

There were several assumptions implicit in this numerical manipulation. The distribution by hour of the day was lumped for all days of the week. Therefore, the assumption was that the distribution does not vary within the week. There are several obvious instances in which this assumption is not valid. However, in general, it was

Table 4. Percentage distribution of vehicle types.

Road	Automobiles	AUTs	Campers	Trucks	ADT (vpd)
I-75	85.21	4.21	1.32	9.23	22,988
I-64	80.90	1.16	0.59	18.53	10,586
I-65	77.85	2.80	1.13	18.22	9,860
US-27	90.24	1.12	0.72	7.92	9,740
US-41	79.43	2.02	1.14	17.41	8,510
US-60	86.29	1.26	0.83	11.62	12,000
Weighted avg					
All data	83.59	2.47	1.01	12.93	12,281
Four-lane	81.72	3.00	1.11	14.17	12,986

Table 5. Percentage distribution of vehicle types on I-75 by hour of day.

Hour	Automobiles	AUTs	Campers	Trucks	Average Volume (vehicles/hour)
Midnight to 1	73.72	2.26	1.60	22.02	418
1 to 2	72.96	3.03	1.65	22.36	364
2 to 3	71.87	2.90	1.54	23.69	315
3 to 4	75.97	2.80	0.98	20.25	424
4 to 5	76.19	2.99	1.29	19.53	320
5 to 6	83.36	3.26	0.94	12.44	561
6 to 7	82.75	3.62	1.27	12.36	631
7 to 8	85.22	3.64	1.24	9.90	785
8 to 9	85.77	4.19	1.22	8.82	1,043
9 to 10	86.59	4.70	0.98	7.73	1,334
10 to 11	87.37	4.99	0.99	6.65	1,481
11 to noon	87.12	4.95	1.22	6.71	1,528
Noon to 1	87.64	4.68	1.12	6.56	1,526
1 to 2	87.29	4.89	1.27	6.55	1,517
2 to 3	87.91	4.86	1.33	5.90	1,583
3 to 4	87.19	4.59	2.13	6.09	1,639
4 to 5	88.10	3.93	1.44	6.53	1,513
5 to 6	88.09	4.19	1.28	6.44	1,316
6 to 7	87.11	3.87	1.46	7.56	1,186
7 to 8	84.94	4.07	1.47	9.52	951
8 to 9	81.93	5.35	1.16	11.56	824
9 to 10	83.18	3.34	1.30	12.18	693
10 to 11	80.23	2.92	1.41	15.44	557
11 to midnight	78.69	3.14	1.26	16.91	411

Table 6. Percentage distribution of vehicle types on I-75 by day of week.

Day	Automobiles	AUTs	Campers	Trucks	Volume (vpd)
Sunday	90.21	3.98	1.20	4.61	32,080
Monday	84.99	4.23	1.26	9.52	20,878
Tuesday	81.99	3.57	1.14	13.30	17,589
Wednesday	79.33	4.35	1.13	15.19	16,842
Thursday	80.60	4.24	1.56	13.60	18,369
Friday	85.34	4.18	1.20	9.28	24,589
Saturday	87.92	4.87	1.64	5.57	39,569

Table 7. AUT traffic in shoulder lane by hour.

Hour	Percent	Hour	Percent
0-1	94.87	12-13	87.80
1-2	98.70	13-14	89.21
2-3	93.75	14-15	86.99
3-4	96.39	15-16	86.53
4-5	100.00	16-17	85.58
5-6	95.31	17-18	88.60
6-7	93.12	18-19	85.67
7-8	84.50	19-20	87.82
8-9	92.81	20-21	84.47
9-10	87.47	21-22	89.51
10-11	88.20	22-23	93.86
11-12	89.22	23-24	91.43

Note: Mean = 90.49; standard deviation = 4.44; and largest deviation from mean is not significantly large.

felt that the hypothesis was true. Similarly, the assumption was implicit that the week during which the classification study was conducted was typical of every week of the year. Finally, the assumption was also made that the years for which toll data were acquired were typical. In addition, the assumption was implicit that distributions by hour and by day on I-75 were typical of other roads.

Table 9 gives the percentages of AUT vehicles of the total volume for each hour of the day. It can be seen that the percentage occurring between 7 p.m. and 8 p.m. exceeds that occurring between 7 a.m. and 8 a.m. and that the percentage occurring between 8 a.m. and 9 a.m. and that occurring between 8 p.m. and 9 p.m. were not significantly different. Therefore, it can be concluded that the 8 to 8 shift for the 12-hour count was preferable to a 7 to 7 shift. The percentage of daily AUT vehicles counted between 8 a.m. and 8 p.m. was 77.31 and was distributed as follows:

Day	Percentage of Total	Day	Percentage of Total
Sunday	18.74	Thursday	11.43
Monday	12.94	Friday	15.07
Tuesday	9.21	Saturday	21.86
Wednesday	10.75		

A similar distribution by month of the year is as follows:

Month	Percentage of Total	Month	Percentage of Total
January	3.08	July	17.74
February	3.18	August	16.76
March	4.87	September	8.43
April	8.52	October	6.69
May	7.76	November	4.50
June	14.39	December	4.08

SPOT SPEEDS

The final phase of the study was to determine various spot-speed parameters for different vehicle types. This information could be used to determine AUT combination equivalency factors to be used in capacity analyses. Furthermore, because accident potential on high-speed facilities increases as speed differentials increase, an analysis of speed differential trends might yield a correlation with accident records.

The choice of locations for spot-speed studies was made in conjunction with appropriate criteria for other phases of the study. Specific criteria considered especially relevant to the collection of spot-speed information were relatively straight and level sections of roadway and appropriate possibilities for concealment of measuring apparatus. The requirement that the roadway section be relatively straight and level was derived from the assumption that the most important aspect to be considered is the relative speed between AUT combinations and automobiles, not the absolute speed of either.

At least 3 hours of data in each direction were obtained for each road. Spot speed was recorded for as many vehicles as possible. However, only the first vehicle of a platoon was recorded since this vehicle was the speed determinator of the queue. This limited the data that could be obtained on the two-lane roadway, US-27; however, the greater volume and multilane aspects of the other roads eased the effects of this restriction. Speeds were obtained for automobiles, AUT vehicles, and trucks.

A statistical analysis of speed data indicated a statistically significant difference between AUT combination and automobile speeds at each of the six test sites. Cumulative speed distributions were arrayed according to 85th, 50th, and 15th percentile for automobiles, trucks, and AUT combinations for the six roadway sections. Use of the 85th percentile is consistent with the normal practice used to establish speed limits and gauge the normal running speed of the traffic stream. The 50th percentile is the median speed, a common measure of central tendency. The 15th percentile is used as a lower base for running-speed calculations, sometimes used as the speed below which allowance should not be made in the design of speed-influenced elements. It is also an appropriate statistical symmetry for the 85th-percentile speed.

Based on a symmetry analysis, i.e., a comparison of the difference between the 85th-percentile level and the 50th-percentile level with the difference between the 50th percentile and the 15th percentile, it can be said that automobiles were relatively symmetrical in their speed distribution and exhibited a slightly greater tendency toward dispersion at lower speeds. Trucks were not greatly skewed in their distribution, yet they exhibited a marked trend toward greater variance at lower speeds—more so than automobiles. The speed of AUT vehicles exhibited the greatest variance in distribution in either direction, undoubtedly because of a smaller sample size. However, when the mean difference between upper and lower differentials was computed, the AUT distribution was more heavily skewed downward than the distribution of either automobiles or trucks. By inference, the lower half of the AUT speed distribution was more widely variant than those for automobiles or trucks, indicating that the lower half of the speed range was more extended for AUT combinations.

Equivalency factors can be computed to a remarkable degree of accuracy from speed distributions (8). The process used here to compute equivalency factors for AUT combinations was to compare speed distributions of automobiles, trucks, and AUT combinations. Using established factors for trucks as a base and mean ratio between truck-auto differences and AUT-auto differences as a multiplier, a related figure for AUT combinations was calculated.

Speed-differential ratios for five percentile levels are listed for each road in Table 10. It can be seen that the mean on each of these roads was close to unity. Therefore, the automobile equivalency factor for AUT combinations is essentially the same as the factor for trucks.

SUMMARY AND CONCLUSIONS

The purpose of this discussion has been to consider the influence of AUT combinations on several aspects of highway design and operation. The accident history of these vehicles, the influence of their axle weights on pavement design, the relative proportions of these vehicles in the traffic stream, and the relative speed distributions of these vehicles and other vehicle types are factors that have never before been considered. The purpose of this discussion was not to provide an exhaustive treatise on any of these subject areas but merely to consider all four areas from a general viewpoint.

The following conclusions can be drawn from the results of the study:

1. Accidents involving AUT combinations are disproportionately greater than the prevalence of these vehicles in the traffic stream;
2. Although the size of the data sample was small, several types of locations that seemed to be problem areas for AUT accidents were pin-pointed;
3. Indications at these locations were that AUT accidents are related to wind forces created by either passing maneuvers or cross-sectional configurations or to weaving;
4. Trailer axles, though generally heavier than automobile axles, are relatively light;
5. When both car axles and trailer axles are considered in a cumulative fatigue analysis for flexible pavement design, the additional EALs accumulated for a roadway with significant AUT percentage is approximately 5 percent;
6. Four-fifths of the AUT combinations on the road are one-axle trailers;
7. The camper trailer is the most common type of trailer;
8. The speed distribution of AUT combinations closely resembles that of trucks; and

Table 8. Trailer type percentages.

Trailer Type	Item	One-Axle	Two-Axle	Three-Axle	Summation
House	Mean	11.59	4.28	0.14	16.01
	Std. Dev.	6.04	3.60	0.40	8.80
Boat	Mean	17.54	2.31	0.00	19.85
	Std. Dev.	12.62	3.30	0.00	15.54
U-Haul	Mean	14.57	4.25	0.00	18.82
	Std. Dev.	8.42	4.95	0.00	11.82
Camper	Mean	23.89	0.59	0.53	25.00
	Std. Dev	15.00	0.98	0.03	13.07
Other	Mean	13.01	7.05	0.26	20.32
	Std. Dev.	6.06	4.00	0.73	9.25
Summation	Mean	80.60	18.47	0.93	100.00
	Std. Dev.	8.28	7.99	1.43	

Table 9. Hourly distribution of AUT traffic.

Hour	Percentage of Total	Hour	Percentage of Total
Midnight to 1	1.14	Noon to 1	7.36
1 to 2	1.13	1 to 2	7.61
2 to 3	0.94	2 to 3	7.88
3 to 4	1.22	3 to 4	7.73
4 to 5	0.98	4 to 5	6.10
5 to 6	1.88	5 to 6	5.66
6 to 7	2.35	6 to 7	4.71
7 to 8	2.93	7 to 8	3.98
8 to 9	4.49	8 to 9	4.53
9 to 10	6.44	9 to 10	2.38
10 to 11	7.59	10 to 11	1.67
11 to noon	7.76	11 to 12	1.54

Table 10. Spot speeds and ratios.

		Spot Speeds at Selected Percentiles (mph)					
Road	Vehicle	15	30	50	70	85	Mean
US-27	Automobiles	42	44	48	52	56	
	Trucks	34	39	43	45	50	
	AUTs	38	41	43	45	49	
	Ratio[a]	0.89	0.95	1.00	1.00	1.02	0.97
US-41	Automobiles	56	59	63	66	69	
	Trucks	52	55	58	60	64	
	AUTs	44	52	57	59	61	
	Ratio[a]	1.18	1.06	1.02	1.02	1.05	1.07
US-60	Automobiles	53	57	59	64	65	
	Trucks	47	51	55	58	60	
	AUTs	44	50	53	56	60	
	Ratio[a]	1.04	1.02	1.04	1.04	1.00	1.03
I-65	Automobiles	58	61	64	67	70	
	Trucks	54	56	59	61	63	
	AUTs	50	53	55	60	65	
	Ratio[a]	1.08	1.06	1.07	1.02	0.97	1.04
I-75	Automobiles	61	64	66	69	72	
	Trucks	52	55	58	60	62	
	AUTs	52	55	58	61	65	
	Ratio[a]	1.00	1.00	1.00	0.98	0.95	0.99
I-64	Automobiles	59	62	65	66	70	
	Trucks	54	58	60	63	65	
	AUTs	54	56	58	60	64	
	Ratio[a]	1.00	1.04	1.03	1.05	1.02	1.03

Note: 1 mph = 1.6 km/h.

[a]Ratio of truck spot speed to AUT spot speed.

9. The automobile equivalency factor for AUT combinations is approximately equal to that for trucks.

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