Accid. Anal. & Prev., Vol. 14, No. 1, pp. 19-34, 1982
Printed in Great Britain.

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0001-4575/82/010019-16$03.00/0

RELATIONSHIPS BETWEEN ROAD ACCIDENTS AND HOURLY TRAFFIC FLOW—I

ANALYSES AND INTERPRETATION

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(Received 15 April 1980)

Abstract—This research extends the investigation of the relationships between measures of accidents and traffic flow, and considers the hourly flow instead of the average daily traffic (ADT), which has already been reported. The findings of this study serve as a basis for further clarification of the interactions between various levels of traffic flow and road accidents. Eight four-lane road sections were studied during an 8-year period, providing adequate data based on carefully predefined criteria. Power functions are fitted and classified according to: (1) time-sequence analysis for each roadway section; and (2) cross sectional analysis on a one year basis. The results are presented, separately for multi and single vehicle accidents, in a matrix-format. A linear dependency was observed between the power and the logarithm of the multiple constant. This was done in a similar fashion to the previously reported study of the relationships between road accidents and ADT. The results for each type of analysis and type of accident are discussed, and three examples of a practical application are given.

1. INTRODUCTION

This research was conducted as part of the establishment of the safety evaluation procedures for the analysis and interpretation of road accidents in Israel. The format of the entire research, based on gathering nationwide data, is shown in Fig. 1. The basis for such research is the availability of an adequate data bank consisting of effective reporting, storage, retrieval and compilation systems. The study was conducted in four major phases, as indicated in Fig. 1: (1) phase I investigates the relationships (power functions) between two measures of total accidents (density and rate), and average daily traffic (ADT) on four-lane and two-lane interurban road sections; (2) phase II extends the investigation of phase I for the four-lane sections through separate consideration of single and multi-vehicle accidents; (3) phase III examines deterministic relationships between two weighted accident measures and the hourly traffic flow for each type of accident; and (4) phase IV attempts to go thoroughly into the relationships between measures of accidents and hourly traffic flow by separating the traffic stream into free-flow and congested-flow modes, and by interpreting the results in a probabilistic manner.

Phases I and II were previously reported by Ceder and Livneh [1978]. Phases III and IV are described sequentially in this (Part I) and in the following (Part II) papers.

In the past, some aspect of road geometry has been identified as a dominant factor in accident causation at a given location. Thereafter, an attempt was made to interpret the frequency and number of accidents using the ADT value as additional information to the road geometry. Nevertheless, when eliminating considerations of geometry, the ADT by itself cannot be used to explain the overall interaction between traffic flow characteristics and accidents. For that purpose, one should approach the actual traffic flow observed at the time of the accident. In addition, the level of risk associated with traffic flow can be determined only on the basis of smaller time intervals than daily periods. This work attempts to clarify and improve the understanding of the relationships between measures of accidents and hourly traffic flow, which is more fundamental than the accident/ADT relationship. The analysis and interpretation are performed in a similar fashion to that outlined in phases I and II of the study.

2. SOME PREVIOUS STUDIES

The common measure of accidents considered in relation to hourly traffic flow, q, is the "accident rate" A_r. This rate is usually based on the number of accidents per year per million (or 10^6) motor vehicle-kilometers (or miles); that is, the annual number of accidents based on the annual amount of exposure. Several forms of relationship between A_r and q have been

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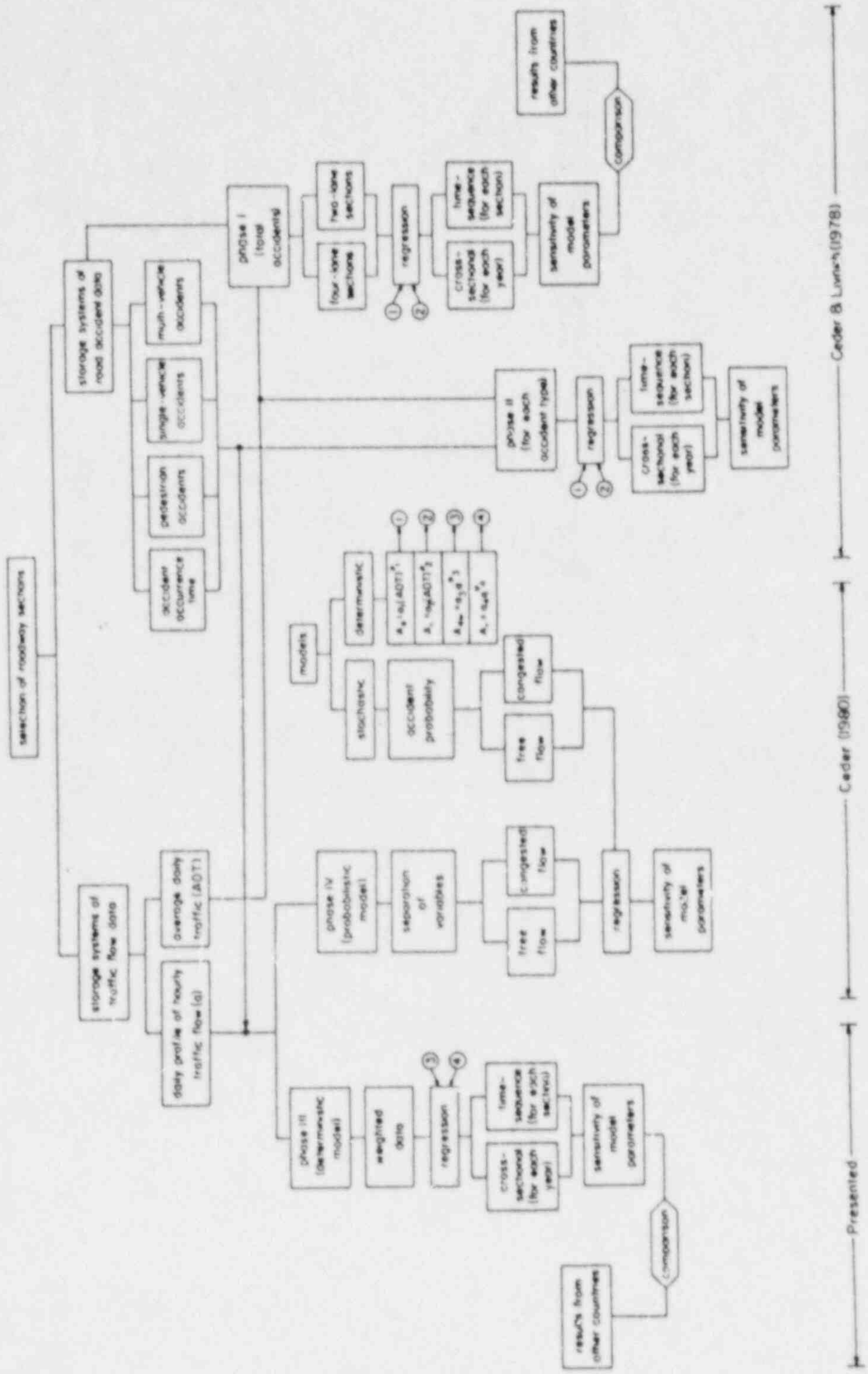

Fig. 1. The overall research plan.

found in the literature. The variety of $A_r - q$ dependency is probably due to different types of accidents, ranges of flows in the analysis, and road designs.

Belmont [1953], found for two-lane sections that A_r (during daylight only) increases almost linearly with q, whereas Smeed [1955], has shown that A_r for total accidents has a small variation for different annual q values. Nonetheless, Smeed pointed out that A_r values for single-vehicle accidents have a tendency to decrease with the increase of q, and multi-vehicle accidents show the opposite tendency.

Leutzbach [1966] and Gwynn [1967], have concluded for four-lane divided sections that a U-shaped dependency exists between A_r (for total accidents) and q, where the minimum A_r values are obtained for q values between approx. 600 to 1300 veh/hr per two lanes. Thereafter, Baker and Gwynn [1968], noted that A_r (total accidents) increases rapidly below $q = 550$ veh/hr per two lanes, but has little variation beyond this flow value.

Pfundt [1969], has compared three types of day and night accidents: rear-end collisions due to blocked lane(s); rear-end collisions due to slow and disabled vehicles; and single-vehicle accidents due to loss of control. For the first type of accident, A_r tends to have a convex upward curve with q (particularly at night); for the remaining two types, the curves are convex downward. In a different study, Leutzbach *et al.* [1970], have shown that on a four-lane Autobahn section in Germany, the resultant U-shaped curve in the $A_r - q$ plane is mainly attributed to rear-end accidents, whereas q values have little effect upon the A_r values of single-vehicle accidents. Chapman [1971], analyzed accident and flow data from England, and generally agree with Pfundt's findings.

Recently, Brilon [1976], in a study of 8 four-lane sections on German Autobahns, found similar results (U-shaped curves) to those reported by Leutzbach *et al.* [1970]. In addition, Brilon hypothesizes that the minimum A_r value is obtained for the most frequent range of q. This hypothesis is examined, among other analyses, in a following section. It should be emphasized that all the above mentioned studies consider the relationship between A_r and q only on the basis of roadway classification (cross sectional analysis).

3. DATA CLASSIFICATION AND ANALYSIS ORIENTATION

The data were selected, based on carefully defined criteria, with the aid of the data bank of the Israel Central Bureau of Statistics. As shown in Fig. 1, the data include (i) fatal and injury accidents on 4-lane interurban road sections, including the type and hour of day of the accidents; and (ii) hourly traffic flow: a daily profile (accomplished by fixed counters), by hour of day, for each road section. The overall data were gathered for an 8-year period (1967–75), excluding the war year 1973. In order to eliminate any undesirable and/or unknown influence of external parameters, four major criteria were imposed as follows:

(1) The roadway sections exclude geometric design elements which disturb the traffic flow (steep grades, curves, roadside obstacles, etc.), and are isolated from interactions, entries and exits (to eliminate the influence of cross-traffic);

(2) No changes were made in the roadway section characteristics of section length, pavement width, and shoulder width during the period 1967–75 (to ensure a comparable base for the data);

(3) There were different daily profiles for q: two daily peaks, one peak or no peak flow (to establish generality); and

(4) There were similar daily profiles for q, excluding weekends, for each roadway section during the period of 1967–75 (to ensure steady travel characteristics).

In order to clarify criterion (4), an example of average daily q profiles of a roadway section is exhibited in Fig. 2. The left illustration in Fig. 2 shows that the general tendency of each year-profile is eminently preserved. However, there are changes in the levels of each year-profile as the result of increasing ADT values with time. The latter effect on daily q profiles can be eliminated, to some extent, by the use of normalized q values. That is, taking $\rho = q/q_m$ instead of q, where $0 < \rho \leq 1.0$ and q_m is the maximum q value, demonstrates the similarity between the average daily q value, as shown on the right illustration in Fig. 2. It is worth noting that criterion (4) has also been applied to an examination of four seasonal average daily q profiles for each year, to ensure the steady characteristics of the travel pattern on each selected roadway

Fig. 1. The overall research plan.

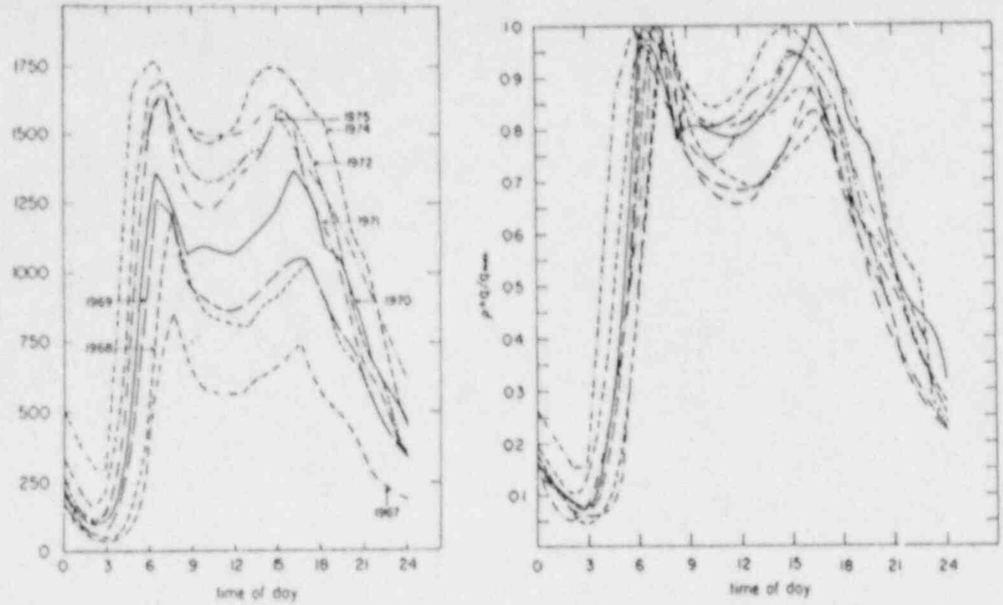

Fig. 2. An example (Roadway 13, section 6), of daily profiles of q and ρ (normalized values) for an 8-year period.

4(2)=8?

section. By the above described procedure, a total of 8 four-lane divided roadway sections were selected for study.

The accident and q data collected from the 8 roadway sections during the 8-year period were analyzed on a separate basis (described in detail by Ceder and Livneh, 1978), as follows: (i) *time-sequence analysis* of data for specific roadway section over an extended period of time (8 years); (ii) *cross-sectional analysis* of data for a given period of time (one year) for a group of roadway sections (8 sections) having the same roadway classification.

This approach enable, in essence, the consideration of dynamic and environmental effects inherent in the relationship between measures of accidents and measures of traffic flow. In the next two paragraphs the differences between a consideraion of ADT and q values with respect to the time-sequence and cross sectional analyses are explained.

In the time-sequence analysis (for a specific roadway section), each ADT value refers to a different measure of accidents, due to their possible mutual changes over the considered period of time; hence, they constitute, say, eight data points for the 8-year period. The time-sequence analysis with q is based *for each year* on a number of data points (dependent on the considered q range and intervals), whereas only one data point is presented when considering the accident/ADT relationship. The ADT value increases, usually, with time, and this increase is reflected in the time-sequence analysis. The q values, however, could have only an upper bound, and therefore, it is perhaps only possible to notice the changes with time for high q values. Nevertheless, the time dependency, in the time-sequence analysis, for the relationship between measures of accidents and q, is relatively negligible, due to overlap among the q ranges of the considered years.

In the cross sectional analysis (for a given year), each ADT value refers to a different measure of accidents, due to the various roadway sections; hence, they constitute, say, eight data points for 8 roadway sections. Similar to the explanations for the time-sequence analysis, when considering q values, one obtains several data points *for each road section* rather than a single data point (for the accident/ADT relationship).

To summarize, for the time-sequence analysis, the relationship between measures of accidents and q emphasizes the uniqueness of each roadway section, while for the cross sectional analysis, the uniqueness of each year (or other given period) is emphasized.

4. MEASURES AND MODELS

The data were gathered, basically, by matching each type of accident with the average q value at the time of the accident. The q value is considered with respect to the interval $q \pm \Delta q$,

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where $\Delta q = 50$ veh/hr per direction of travel (two lanes). That is, each q range was divided into intervals of 100 veh/hr on a two-lane basis.

The measures of accidents were carefully defined and selected as follows: $i \Rightarrow$ denotes either a particular year (for the time-sequence analysis) or a particular roadway section (for the cross sectional analysis).

$$A_d(q) = \sum_i \frac{N_i(q)}{L_i} = \begin{Bmatrix} \text{accident density (acc/km)} \\ \text{for the interval } q \pm \Delta q \end{Bmatrix}$$

where L_i = the length, in kilometres, of roadway section i; in the cross sectional analysis i denotes a section and L_i is a variable, while in the time-sequence analysis i denotes a year and then $L_i = L$, i.e. has a constant value; and $N_i(q)$ = total annual number of accidents (of a given type) which occurred during the five work days of each week for the interval $q \pm \Delta q$.

Another component which should be taken into account is the exposure time of each $q \pm \Delta q$ interval. Hence:

$$T(q) = \sum_j \sum_i t_{ij}(q) = \begin{Bmatrix} \text{the annual exposure time of} \\ \text{traffic flows within the interval } q \pm \Delta q \end{Bmatrix}$$

where $t_{ij}(q)$ = the daily exposure time for the interval $q \pm \Delta q$ at the jth day, $j = 1, 2, \ldots, 261$ (excluding weekends).

An example of $T(q)$ distribution is shown in Fig. 3, where the upper illustration is for the time-sequence analysis (one section, over an 8 year period), and the lower illustration is for the cross sectional analysis (one year for 8 sections).

As the consequence of the above definitions, two accident measures were selected for this research:

$$A_{dw}(q) = \frac{A_d(q) \cdot 10^3}{T(q)} \tag{1}$$

$$A_r(q) = \frac{A_{dw}(q) \cdot 10^6}{q} \tag{2}$$

where $A_{dw}(q)$ = weighted accident (acc/10^3 km·hr), which means the accident density (acc/10^3 km) per *one hour exposure* of traffic flows within the interval $q \pm \Delta q$; and $A_r(q)$ = accident rate (acc/10^6 veh-km).

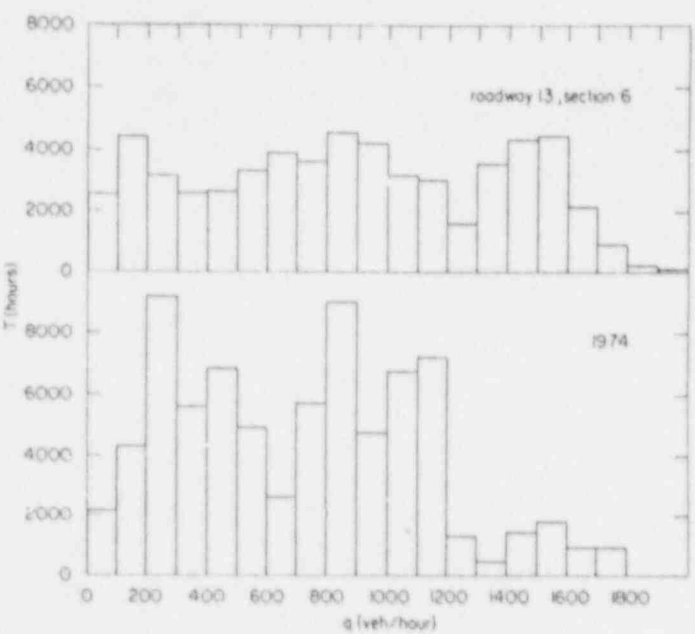

Fig. 3. An example of $T(q)$ distributions for the time-sequence (upper part) and cross sectional (lower part) analyses.

The models which were found appropriate for the analyses made are power functions (same as the models in phases I and II, see Fig. 1):

$$A_{dw}(q) = a_3 q^{p_3} \tag{3}$$
$$A_r(q) = a_4 q^{p_4} \tag{4}$$

where a_3, a_4 and p_3, p_4 are constant parameters which are determined by a linear regression technique. Note that the powers p_3 and p_4 determined the functional tendency (certainly for positive a_3, a_4 values):

$p_3, p_4 > 1 \Rightarrow$ convex upward;
$p_3, p_4 = 1 \Rightarrow$ linear upward;
$1 > p_3, p_4 > 0 \Rightarrow$ concave upward;
$p_3, p_4 = 0 \Rightarrow$ constant linear; and
$0 > p_3, p_4 \Rightarrow$ convex downward.

The models represented by eqns (3) and (4) are fitted separately for single- and multi-vehicle accidents. The sum of these models for each category of analysis (time-sequence and cross sectional), might reveal the possible dependency between the total accidents and q, and might also yield the earlier mentioned U-shaped function. If the latter is the result, then one can arrive at equations satisfying optimum conditions, i.e. $dA_{dw}/dq = 0$ or $dA_r/dq = 0$, which yield the following optimum parameters:

$$q_{do} = \left(\frac{-p_3'' a_3''}{p_3' a_3'}\right)^{1/(p_3' - p_3'')} \qquad q_{ro} = \left(\frac{-p_4'' a_4''}{p_4' a_4'}\right)^{1/(p_4' - p_4'')} \tag{5}$$

where the prime represents the parameters of eqns (3) and (4) for single-vehicle accidents and the double prime for multi-vehicle accidents. The substitution of q_{do} and q_{ro} (provided that each is a positive value) in eqns (3) and (4), respectively, gives accordingly the optimum weighted accident density measure, A_{dwo}, and the optimum accident rate measure A_{ro}.

5. RESULTS AND FINDINGS

The results of the time-sequence and cross sectional analyses are summarized in Tables 1 and 2, respectively. These regression results, by accident type, refer to eqns (3)–(5), and are accompanied by the standard error values (SE_{dw} and SE_r in units of A_{dw} and A_r, respectively).

The results given in Tables 1 and 2 are shown in Figs. 4 and 5 for multi and single-vehicle accidents, respectively; also an attempt is made in Fig. 6 to show the summation of the curves. Note that in Figs. 4–6 all the curves are within the actual q range. From these results and analyses, four major findings are identified:

(a) *For the multi-vehicle accident models* all $p_3 > 0$ with a convexity tendency for the time-sequence models ($1 < p_3 < 2$), and mixed functional tendency for the cross sectional models. While $A_{dw}(q)$ is always increasing with q, the $A_r(q)$ is either increasing or slightly decreasing with $q(-0.37 < p_4 < 3.82)$. There are two interesting observations: (i) three out of four time-sequence models in which $A_r(q)$ does not increase sharply with q are characterized by low upper bound value of the q range; and (ii) the two cross sectional models obtained for two years before the energy crisis (1971–72), indicate a lower safety level than the two models after the energy crisis (1974–75), where all four models have similar q ranges.

(b) *For the single-vehicle accident models* all $p_4 < 0$ indicating convex downward curves in the $A_r - q$ plane. On the other hand, there is a mixed functional tendency for the $A_{dw} - q$ relationship. The comparison between single—and multi-accident models demonstrates less curve-dispersion of the former for both the time-sequence and cross sectional analyses.

(c) *For the summation of the single- and multi-vehicle accident models* half of the $A_r(q)$ models are characterized by U-shaped curves, and the remaining half by convex downward curves. Three out of four of the $A_r(q)$ models which do not have U-shaped curves are indicated

Table 1. Regression results of the time-sequence analysis

Roadway number	Section number	Accident type	$A_{dw} = a_3 q^{p_3}$					$A_r = a_4 q^{p_4}$				
			a_3	p_3	SE_{dw}	q_{do}	A_{dwo}	a_4	p_4	SE_r	q_{ro}	A_{ro}
1	9	s*	20.0	−0.89	0.235	263.24	$2.8 \cdot 10^{-4}$	20.31	−2.56	0.43	435.54	0.81
		m**	$1.29 \cdot 10^{-5}$	1.59	0.266			$2.9 \cdot 10^{-9}$	2.90	1.05		
1	10	s	550.0	−1.41	0.356	413.16	$1.9 \cdot 10^{-4}$	$35.07 \cdot 10^{6}$	−3.04	1.92	539.41	0.31
		m	$2.04 \cdot 10^{-6}$	1.88	0.433			$5.07 \cdot 10^{-12}$	3.82	1.85		
11	8	s	0.03	0.34	0.362	–	–	37.02	−0.67	0.54	–	–
		m	$5 \cdot 10^{-4}$	1.16	1.130			2.72	−0.09	1.79		
11	9	s	0.27	−0.17	0.103	–	–	24.75	−0.73	0.18	–	–
		m	$2.2 \cdot 10^{-4}$	1.19	1.930			4.15	−0.37	0.23		
13	3	s	0.17	−0.12	0.802	–	–	9.81	−0.57	0.61	–	–
		m	$4.86 \cdot 10^{-3}$	0.72	0.401			4.80	−0.28	0.80		
13	5	s	120.0	−1.00	0.667	252.31	$7.7 \cdot 10^{-4}$	$3.86 \cdot 10^{9}$	−3.77	1.48	484.20	0.70
		m	$3.33 \cdot 10^{-5}$	1.64	1.460			$2.14 \cdot 10^{-8}$	2.71	1.40		
13	6	s	1.0	−0.36	0.160	114.00	$1.8 \cdot 10^{-4}$	$5.14 \cdot 10^{4}$	−2.09	1.15	424.23	0.33
		m	$1.37 \cdot 10^{-4}$	1.26	0.733			$5 \cdot 10^{-7}$	2.10	1.21		
21	4	s	10.0	−0.66	0.567	112.91	$7.3 \cdot 10^{-4}$	$6.07 \cdot 10^{6}$	2.99	1.78	–	–
		m	$2 \cdot 10^{-3}$	1.05	0.201			2.18	0.03	0.36		

* single-vehicle accidents

** multi-vehicle accidents

Table 2. Regression results of the cross sectional analysis

Year	Accident type	$A_{dw} = a_3q^{P_3}$					$A_r = a_4q^{P_4}$				
		a_3	P_3	SE_{dw}	q_{do}	A_{dwo}	a_4	P_4	SE_r	q_{ro}	A_{ro}
1967	s*	$6.33\cdot10$[illegible]	0.56	0.146	-	-	7.68	-0.46	0.36	250.98	0.67
	m**	$6.5\cdot10^{-6}$	1.75	0.167			$5.8\cdot10^{-11}$	3.77	2.54		
1968	s	$1.62\cdot10^{-2}$	0.31	0.023	-	-	25.30	-0.76	0.26	-	-
	m	$3.7\cdot10^{-3}$	0.62	0.267			0.83	-0.08	0.44		
1969	s	$3.13\cdot10^{-4}$	0.94	0.133	-	-	0.37	-0.06	0.22	-	-
	m	$4.66\cdot10^{-5}$	1.41	0.167			0.07	0.35	0.37		
1970	s	$3.33\cdot10^{-2}$	0.16	0.102	-	-	1151.52	-1.54	0.80	338.82	1.17
	m	$2.98\cdot10^{-4}$	1.21	0.567			0.28	0.22	0.80		
1971	s	10.0	-0.74	0.403	271.02	$2.7\cdot10^{-4}$	$5.33\cdot10^{7}$	-3.29	0.26	449.39	0.25
	m	$4.16\cdot10^{-5}$	1.41	0.562			$2.33\cdot10^{-7}$	2.19	0.89		
1972	s	$9.66\cdot10^{-3}$	0.44	0.213	-	-	15.12	-0.63	0.38	-	-
	m	$3.36\cdot10^{-3}$	0.83	0.833			3.81	-0.18	0.93		
1974	s	0.10	0.01	0.333	-	-	$2.25\cdot10^{5}$	-2.22	0.36	-	-
	m	$6.1\cdot10^{-3}$	0.69	0.623			5.75	-0.30	0.93		
1975	s	3.43	-0.58	0.202	204.37	$2.4\cdot10^{-4}$	$1.43\cdot10^{6}$	-2.69	0.26	929.07	0.46
	m	$2.1\cdot10^{-4}$	1.12	0.803			0.24	0.09	0.83		

* single-vehicle accidents

** multi-vehicle accidents

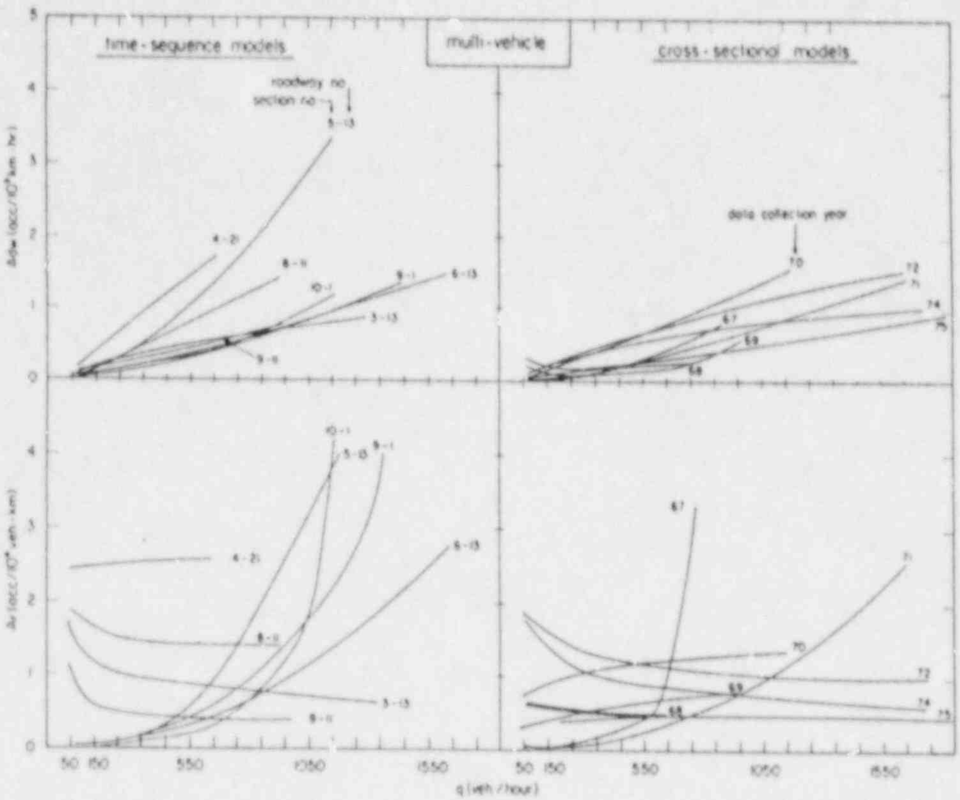

Fig. 4. The results of the time-sequence and the cross-sectional models for multi-vehicle accidents.

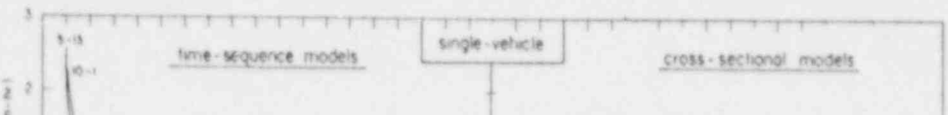

Fig. 5. The results of the time-sequence and the cross-sectional models for single-vehicle accidents.

* single-vehicle accidents
** multi-vehicle accidents

by upper bound on the q range below 1000 veh/hr per two lanes. This might be (one of) the reasons for this observation. Nevertheless, the optimum average q_{ro} value for the minimum $A_r(q)$ value is 500 veh/hr for those U-shaped models. This optimum value results from opposing tendencies of multi-vehicle accident models (A_r increasing with q) and single-vehicle accident models (A_r decreasing with q). The average q_{ro} value is below those (600–1300 veh/hr) which were determined by Leutzbach [1966] and Gwynn [1967], probably due to a higher upper bound on the q range (about 3000 veh/hr) than that indicated in Fig. 8.

(d) The hypothesis made by Brilon [1976], that the minimum A_r value (for total accidents) is obtained for the most frequent range of q, is not strengthened by our data. In fact, the opposite is the case, since out of sixteen data sets of $T(q)$ distributions, none agrees with Brilon's hypothesis.

6. MATRIX REPRESENTATION OF THE RESULTS

Following a similar method to that outlined by Ceder and Livneh [1978], (phases I and II indicated in Fig. 1), the results are plotted on the matrix: the y-axis is the logarithm scale of a_3 or a_4, and the x-axis represents the scale for p_1 or p_2, respectively.

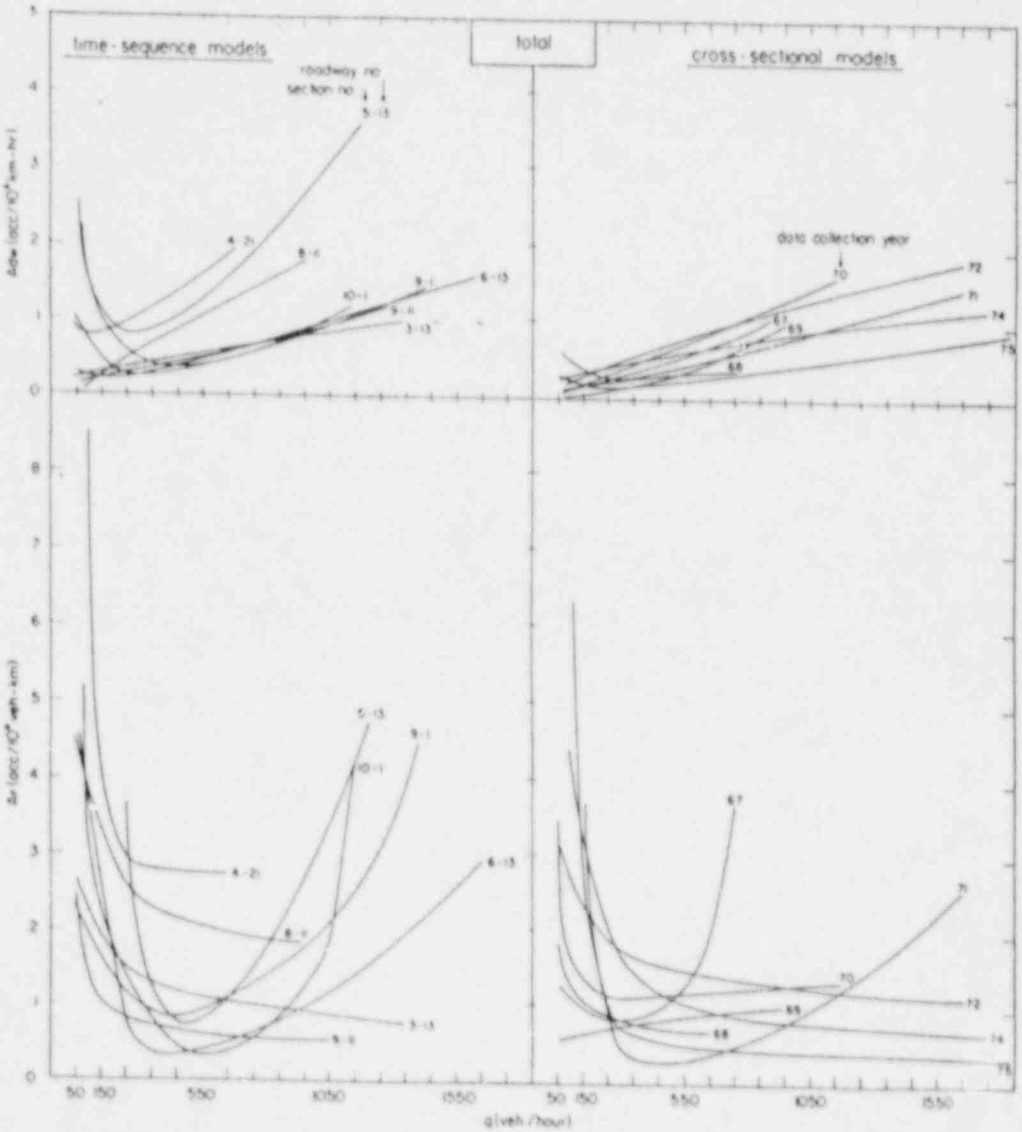

Fig. 6. The results (summation) of both multi- and single-vehicle accidents.

The log a_3 values versus p_3 values are illustrated in Fig. 7, and the log a_4 values versus p_4 values are illustrated in Fig. 8. From these figures, one can eminently observe the linear dependency that exists between the variables. Consequently a linear regression procedure was applied to each set of results. The fitted models are shown in Figs. 7 and 8 and are indicated in Table 3 with their coefficient of determination r^2. In addition, the F statistic is used to examine the possibility of combining the linear models of both the time-sequence and cross sectional analysis. This examination leads, indeed, to a common model through a three-stage test: (1) between the variances; (2) between α_1's or β_1's; and (3) between α_o's or β_o's (see Table 3). It was found that for both the $A_{dw}(q)$ and $A_r(q)$ models, there is no significant difference between the linear models at the 95% level. The common models are specified in Table 3, and are shown on the left part of Figs. 7 and 8 with respect to each type of accident. The remaining parts of these figures illustrate, for each type of accident, separate time-sequence and cross sectional models. Some of the findings mentioned in the previous section are clearly and systematically demonstrated in this matrix representation.

Each linear model shown in Table 3 represents a family of curves which intersect at a unique point. For example, this point in the $A_r - q$ plane (A_r^*, q_r^*) is obtained by:

$$A_r^* = a_4(q_r^*)^{p_4},$$

$$\log A_r^* = \log a_4 + p_4 \log (q_r^*)$$

and therefore, $\log A_r^* = \beta_o$, $\log (q_r^*) = -\beta_1$, and similarly, $\log A_{dw}^* = \alpha_o$, $\log (q_d^*) = -\alpha_1$. These intersecting points are symbolized with an asterisk in Table 3.

7. EXAMPLES OF A PRACTICAL APPLICATION

Knowledge of the proper relationship (and limitations involved) between A_{dw} or A_r and q is important from various aspects: traffic planning, design, operation and research. This section, however, introduces examples of only one practical application. That is, the evaluation of the safety level either before and after implementation of a roadway improvement project, or after short-term operation of a new roadway section. This practical application is discussed also in phases I and II (see Fig. 1), in view of the relationship between measures of accidents and ADT.

Example 1. It is assumed that in section No. 6 of roadway No. 13, a safety improvement was carried out in January, 1976. The data collected after one year are:

		$A_d(q)$		$A_{dw}(q)$		$A_r(q)$	
ranges of q	$T(q)$	multi-veh.	single-vel.	multi-veh.	single-veh.	multi-veh.	single-veh.
0–500	1044	0.114	1.699	0.109	0.163	0.436	0.652
500–1000	261	0.100	0.120	0.333	0.460	0.511	0.613
1000–1500	522	0.548	0.175	1.050	0.335	0.840	0.268
1500–2000	4437	4.138	0.106	0.933	0.024	0.533	0.014

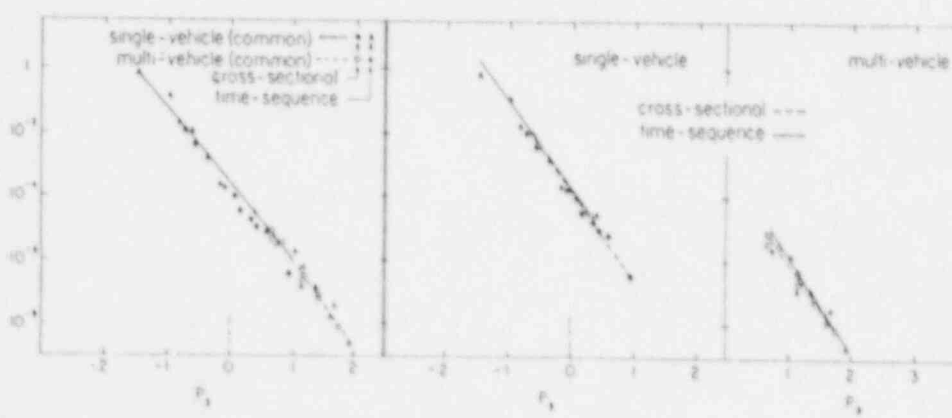

Fig. 7. Matrix representation of the model's parameters a_3 and p_3, where the left part distinguishes the results by accident type, and the remaining part—by both accident type and type of analysis.

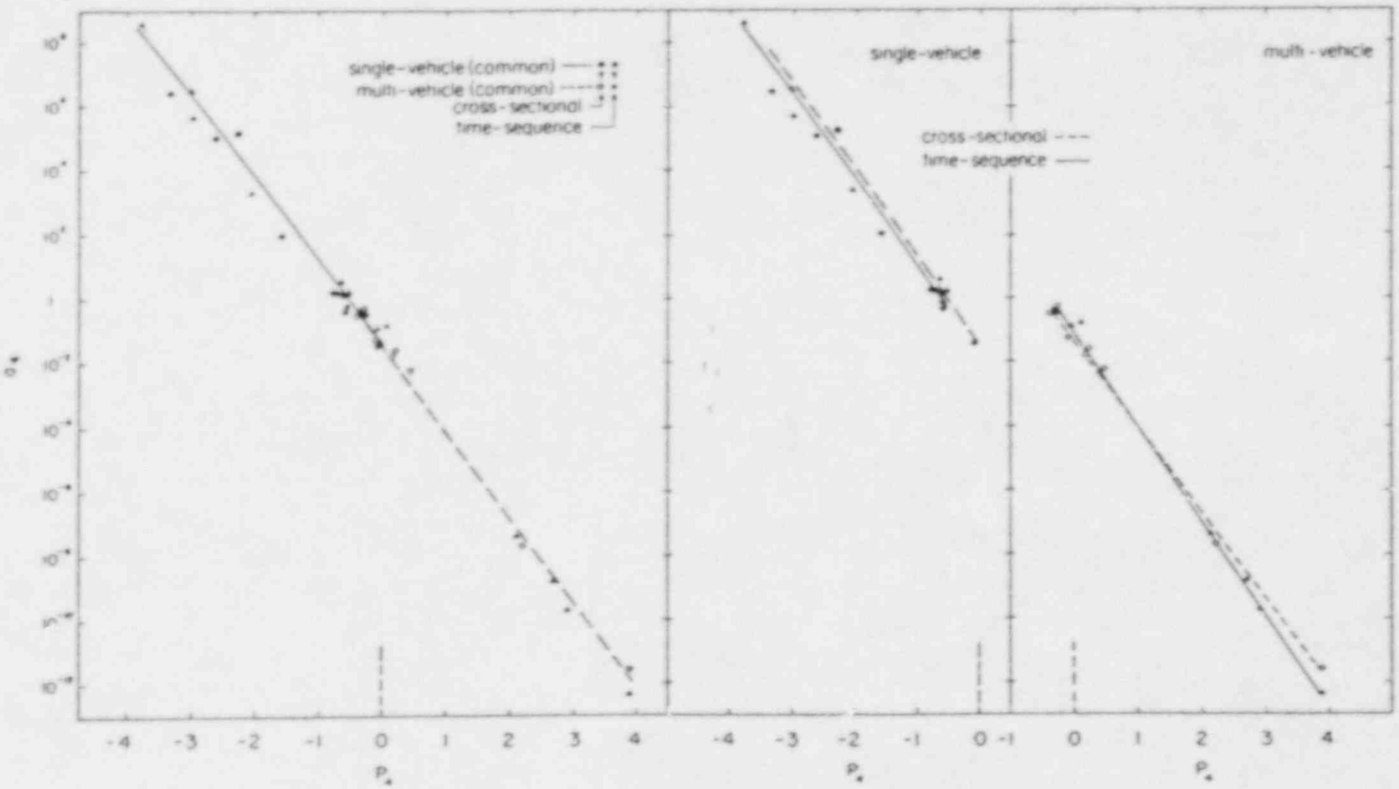

Fig. 8. Matrix representation of the model's parameters a_* and p_*, where the left part distinguishes the results by accident type, and the remaining part—by both accident type and type of analysis.

Table 3. Results of the linear dependency between a_3 and P_3 and a_4 and P_4

model	parameters	multi-vehicle time-sequence	multi-vehicle cross-sectional	multi-vehicle common	single-vehicle time-sequence	single-vehicle cross-sectional	single-vehicle common
$\log a_3 = a_0 + a_1 P_3$	a_0	0.40	−0.47	−0.24	−0.34	−0.94	−0.76
	a_1	−3.06	−2.7	−2.73	−2.11	−2.57	−2.52
	r^2	0.98	0.99	0.99	0.98	0.99	0.99
	P_3 lowest	0.72	0.62	0.62	−1.71	−0.74	−1.71
	P_3 highest	1.88	1.75	1.88	0.34	0.94	0.94
	A_{dw}*	2.51	0.36	0.57	0.46	0.11	0.17
	q_d*	1148.15	501.19	537.03	128.82	371.54	331.13
$\log a_4 = \beta_0 + \beta_1 P_4$	β_0	−0.01	−0.16	−0.10	−0.35	−0.43	−0.35
	β_1	−2.92	−2.75	−2.85	−2.54	−2.81	−2.64
	r^2	0.99	0.99	0.99	0.94	0.97	0.98
	P_4 lowest	−0.37	−0.30	−0.37	−3.77	−3.29	−3.77
	P_4 highest	3.82	3.77	3.82	−0.56	−0.06	−0.06
	A_r*	0.98	0.69	0.79	0.44	0.37	0.45
	q_r*	831.76	562.34	707.95	346.74	645.65	36.52

with suitable units to those indicated for the models. The question is, whether the level of safety improved, and to what magnitude. According to this research approach, the results derived by the time-sequence analysis can be applied to this example. Hence, the evaluation procedure is based on the results indicated in Table 1 for roadway No. 13, section No. 6 for each q range along with a confidence interval. Since the power functions are intrinsically linear (can be expressed by natural logarithms, in a linear form), 95% confidence limit can be found for the new data (after the improvement), $\ln(A_r)_{new}$ or $\ln(A_{dw})_{new}$ in the transformed plane according to:

$$\pm t(n-2, 0.975) \cdot s \cdot \left\{1 + \frac{1}{n} + \frac{[\ln(A_r)_{new} - \overline{(\ln A_r)}]^2}{\sum_i [\ln(A_r)_i - \overline{(\ln A_r)}]^2}\right\}^{1/2}$$

for a two-sided 95% level t-test using the t-table with $(n-2)$ degrees of freedom, and where n is the number of data points, s is the standard error for either $\ln A_r$ or $\ln A_{dw}$, and $\overline{(\ln A_r)}$ or $\overline{(\ln A_{dw})}$ is the mean value of $\ln A_r$ or $\ln A_{dw}$, respectively. Note that this confidence limit is based on the assumption that the residuals in the transformed scale are normally distributed with mean zero and constant variance. If, for example, the confidence interval is $\pm SE_{dw}$ and $\pm SE_r$ for $A_{dw}(q)$ and $A_r(q)$, respectively, then by substituting the data in the models, the following results are obtained:

q	$A_{dw}(q)$ multi-veh.	safety change	single veh.	safety change	$A_r(q)$ multi-veh.	safety change	single veh.	safety change
250	0.140	**	0.137	***	0.054	**	0.500	***
750	0.558	**	0.092	****	0.545	**	0.050	***
1250	1.062	**	0.077	****	1.594	**	0.017	***
1750	1.622	**	0.068	**	3.231	*	0.009	***

*Significant improvement
**an improvement, but not significant
***a deterioration, but not significant
****significant deterioration

Fig. 8. Matrix representation of the model's parameters a_4 and p_4, where the left part distinguishes the results by accident type, and the remaining part—by both accident type and type of analysis.

The asterisks attempt to interpret the results with respect to the confidence interval. It is worth noting that the data could also be analyzed by the A_d(ADT) and A_r(ADT) models specified for the considered roadway section in Table 3 of Ceder and Livneh [1978]. In the latter case, the results indicate significant improvement for multi-vehicle accidents and significant deterioration for single-vehicle accidents. Certainly, the consideration of q instead of ADT determines, more specifically, the relative changes in the safety level after the improvement. Furthermore, the knowledge of the exposure time for each q range might lead to isolation of the problematic daily hours from a safety standpoint.

Example 2. If the data from *example* 1 are associated with a new four-lane section (also after one year of operation), then it is only possible to select an appropriate cross sectional model. That is, the time-sequence model cannot be applied due to lack of comparable basis and/or information. The selection of a cross sectional model might be based on two criteria: (i) that it includes the upper bound q range of the considered data; and (ii) that it reflects the environmental characteristics of the considered new roadway section (usually exhibited by the latest year model which satisfies criterion (i)). Consequently, the model selected is that of 1975 and, in a similar way to example 1, the results are obtained by substituting the data in the models indicated for 1975 in Table 2 (based on SE_{dw} and SE_r):

	$A_{dw}(q)$				$A_r(q)$			
q	multi-veh.	Safety change	Single-veh.	Safety change	multi-veh.	Safety change	Single-veh.	Safety change
250	0.102	***	0.139	***	0.394	***	0.507	***
750	0.349	***	0.074	****	0.435	***	0.026	****
1250	0.618	***	0.055	****	0.456	***	0.007	***
1750	0.900	***	0.045	**	0.467	***	0.003	***

*significant improvement
**an improvement, but not significant
***a deterioration, but not significant
****significant deterioration
} with respect to similar roadway sections which have the same characteristics

Perhaps the major finding is the significant deterioration in single-vehicle accidents at the mid-q range. When considering the A_d(ADT) and A_r(ADT) models in phase I, the results are a significant improvement in the safety level of the new roadway section for multi-vehicle accidents and an improvement, but not significant, for single-vehicle accidents. In essence, in this example, the ignorance of q produces a situation in which the relative safety deterioration at the mid-q range cannot be detected.

Example 3. An alternative means of estimating the $A_{dw}(q)$ and $A_r(q)$ models is use of the interrelationship between p_3 or p_4 and $\log a_3$ or $\log a_4$, respectively. In fact, for any given q, $T(q)$ and the number of accidents on a four-lane section, one can obtain a crude estimation of such models. For example, on a four-lane section, the hourly flow, during specific daily hours, increased from 500 to 800 veh/hr for two lanes (due to either a closure of a parallel road or by means of traffic direction). The question is whether the level of safety has been changed and to what magnitude. The A_{dw} and A_r values for $q = 500$ veh/hr and this q value are then substituted in the common linear models indicated in Table 3. This substitution determines the appropriate models to be used for the "after change" ($q = 800$ veh/hr) data. That is, $\bar{A}_r(q)$ for multi-vehicle accidents is obtained through determination of a_4 and p_4, based on the observed value $A_r = 1.10$ acc/10^6 veh-km:

$$\left.\begin{aligned} 1.1 &= a_4 \cdot 500^{p_4} \\ \log a_4 &= -0.1 - 2.85 p_4 \end{aligned}\right\} \begin{aligned} a_4 &= 370 \\ p_4 &= -0.936 \end{aligned}$$

and similarly, $\bar{A}_r(q)$ for single-vehicle accidents and $\bar{A}_{dw}(q)$ for multi- and single-vehicle accidents can be calculated. The complete "before and after" data and results are as follows:

Accident type	Situation average q	Before 500	After 800
multi-vehicle	A_{dw}	0.55	0.90
	a_3	0.011	—
	p_3	0.633	—
	$\overline{A_{dw}}$	—	0.74
single-vehicle	A_{dw}	0.20	0.20
	a_3	0.024	—
	p_3	0.341	—
	$\overline{A_{dw}}$	—	0.23
multi-vehicle	A_r	1.10	1.12
	a_4	370	—
	p_4	−0.936	—
	$\overline{A_r}$	—	0.71
single-vehicle	A_r	0.40	0.25
	a_4	62.55	—
	p_4	−0.813	—
	$\overline{A_r}$	—	0.27

where $\overline{A_r}$ and $\overline{A_{dw}}$ (The expected safety measures if the safety level remains the same) are the results of substituting $q = 800$ in the models. The comparison between A_{dw} and $\overline{A_{dw}}$ and between A_r and $\overline{A_r}$ for $q = 800$ reveals that: (i) single-vehicle accidents have almost not changed, though the absolute A_r value after the change decreased by 40%!; and, (ii) relative improvement is observed for the multi-vehicle accidents though the absolute A_{dw} value after the change increased by 60%! For such applications, it is also advisable to determine that the resultant p_3 and p_4 values are within or close to the indicated range in Table 3.

CONCLUDING REMARKS

This research attempts to find quantitative models (power functions) to represent the possible dependency between two measures of accidents and the hourly flow for eight interurban road sections during an 8-year period.

From these attempts to search for proper relationships between measures of accidents and the hourly flow, it is apparent that the technical procedure involves a combination of two primary types of analyses: time-sequence (for each roadway section) and cross sectional (on a one year basis). For each type of analysis, the total accidents are primarily separated into multi- and single-vehicle accidents. The latter separation enables one to: (1) distinguish accident costs for each type of accident and for each q range; (2) find the differential effects of traffic flow on each type of accident; and (3) perform a more reliable safety evaluation for "before and after" studies.

Phases I and II of the overall study, described in Fig. 1, are concerned with the influence of ADT on the measures of accidents. However, this consideration by itself cannot explain the interactions between road accidents and traffic flow, since it is only based on a daily average. The consideration of hourly flows provides a better understanding of these interactions. In addition, it is possible to move one step forward in order to further understand the accidents/traffic flow dependency by separating the hourly flow into free-flow and congested-flow. This separation into components of both types of traffic flow and type of accident will ultimately lead toward more accurate accident prediction based on the traffic flow. The following paper [Ceder, 1982], describes this further analysis (phase IV in Fig. 1), and attempts also to determine and compare the probabilities for each accident/flow type component.

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UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D. C. 20555

May 26, 1987

MEMORANDUM FOR: John Milligan
Technassociates

FROM: Emile L. Julian, Acting Chief
Docketing and Service Branch

SUBJECT: SEABROOK EXHIBITS

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All of the documents sent to TI for processing fall within the exempt classification.