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RELATIONSHIPS BETWEEN ROAD ACCIDENTS AND HOURLY TRAFFIC FLOW—II

PROBABILISTIC APPROACH

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Abstract—This is a continuation of the investigation into relationships between accident rate and hourly traffic flow as outlined in Part I of the research. The underlying study attempts to determine appropriate models for single- and multi-vehicle accident rates in conjunction with free-flow and congested-flow conditions. For the free-flow data, the total accident rate-hourly flow curve follows the U-shaped configuration. This form is the result of a convex downward and a convex upward curve for single- and multi-vehicle accidents, respectively. For the congested-flow data (characterized by multi-vehicle accidents), the accident rate increases sharply with hourly flow. The models are applied to probabilistic aspects with consideration of a fitted, generalized (hourly flow dependent) headway model. The headway model represents the probability that two vehicles which are, even instantaneously, under a car-following mode are in a potentially hazardous situation. The approach investigated is believed to provide an essential input for both simulation studies and theoretical models of road traffic accidents.

1. INTRODUCTION

This is the second part of a continuing study to explore the interrelationship between road accidents and hourly traffic flow. In the first part [Ceder and Livneh, 1981], the entire research based on nationwide data, is presented in four phases, with phase IV exhibited by this paper. The availability of data makes it possible to further separate the consideration of traffic flow components according to the following chain: ADT → Hourly Flow → Hourly Free- and Congested-Flow. The latter separation, which is outlined here, may be approached from both deterministic and probabilistic viewpoints. The probabilistic aspects, which are emphasized in this work, are both vindicated and based on determined deterministic relationships between an accident measure and the hourly (free/congested) flow.

2. DETERMINATION OF FREE-FLOW AND CONGESTED-FLOW CONDITIONS

At any instant, the driver of an automobile is confronted with a plethora of visual, aural, vestibular and pressure stimuli. Though motion of his vehicle causes continuous changes in these stimuli, the driver cannot, and need not, evaluate each change in each stimulus dimension. Of major interest are those stimuli which create a potential hazardous situation. One of the known analytical tools used to assess the safety of individual cars is the car-following theory[Edie, 1974]. This theory assumes that a driver will react to a stimulus generated by the nearest forward vehicle. The uniqueness of this theory is that it provides a bridge between the motion of individual cars and the entire traffic flow[Ceder, 1979]. However, the car-following rules are applicable only to those vehicles which are under the influence of other vehicles in the stream—behaviour particularly noticeable under congested-flow conditions.

Consequently, it is probable that as the traffic flow becomes more congested, the vehicles are more constrained and can hardly perform a desirable manoeuvre. Such situations have a direct bearing on rear-end and chain collisions, as they result from an inappropriate time lag of a following car response to a disturbance caused by the vehicle ahead. This argument leads to the separation of free-flow and congested-flow conditions, since they have different effects upon single- and multi-vehicle accidents. While free-flow conditions are characterized by both single- and multi-vehicle accidents, congested conditions are particularly characterized by multi-vehicle accidents.

It is difficult to distinguish between free-flow and congested-flow periods when considering only the hourly flow variable. Even in a well defined free-flow condition (say, from the traffic flow theory[Edie, 1974]), one can observe a platoon of vehicles moving under a congested mode; and vice versa in a congested-flow condition. Therefore, a safety-based criterion is established: The *congested* hourly flow periods are determined by those periods in which 95% or more of the overall accidents are multi-vehicle accidents, and furthermore, the proportion of rear-end collisions is 85% or more of total accidents. The remaining flow periods are considered free-flow conditions. As indicated in the next paragraph, this criterion determines the boundary of flow rate (on an hourly basis period) between free and congested modes.

This criterion enables one to disregard single-vehicle accidents (skids, roll-over, running-off-the-road) in congested-flow conditions, and to consider primarily rear-end collisions (in comparison to head-on and angle collisions). The criterion has been applied to three four-lane, divided roadway sections (13-3, 13-5 and 13-6) mentioned in the first part of this study[Ceder and Livneh, 1980]. It turns out that only for $q \geq 1600$ veh/hr per direction of travel (two lanes) this criterion is satisfied. These congested hourly flow periods were observed only on one roadway section (13-6). It is interesting to note that the criterion under consideration seems also to be adequate for high traffic flows on four-lane autobahns in Germany[Leutzbach *et al.*, 1970; Brilon, 1976].

The data considered in this paper are based on 8-year daylight (excluding night) accidents and hourly flow information for the above-mentioned three roadway sections. Certainly, for the three sections no attempt is made to differentiate between time-sequence and cross-sectional analyses, as in Part I of this work, but rather to emphasize free-flow and congested-flow conditions. The variable q is analyzed for free-flow periods, with 100 veh/hr intervals within the range $0 < q < 1600$; and for congested flow periods, with 50 veh/hr intervals within the range $1600 \leq q \leq 1900$ (the maximum measured hourly flow on section 13-6 is 1900 veh/hr).

3. FREE-FLOW AND CONGESTED-FLOW MODELS

Under free-flow conditions, both single- and multi-vehicle accidents occur where the proportion of single-vehicle accidents is decreasing with q. Based on the data exhibited in Part I of this work, it can be surmised that on four-lane divided roadways, the majority of multi-vehicle accidents are rear-end collisions over all q ranges. It is, therefore, reasonable to claim that a potential multi-vehicle accident is associated with those traffic situations in which $h < T$, where h = headway in seconds (front bumper to front bumper) and T = a time lag of the driver-vehicle system in seconds (sufficient time in order to completely perceive, interpret, decide and act, and for the vehicle to respond).

The time lag is an essential variable both in the event of an emergency deceleration (to avoid rear-end collision), and in the event of a risky manoeuvre (to avoid angle collision). On four-lane divided roadways, head-on collisions rarely occur, and can be disregarded. Hence, under free-flow conditions, the probability of a multi-vehicle accident is particularly dependent on the interaction of two events, A and B:

Event A: $h < T$ in a car-following mode;

Event B: a risky situation (e.g. the leading car performs a hazardous manoeuvre, or the driver of the following car drastically reduces his attention, or an external factor interferes with one of the vehicles).

According to the multiplicative law of probability:

$$P\begin{Bmatrix}\text{multi-veh}\\ \text{accident}\end{Bmatrix} = P(A \cap B) = p(A) \cdot p(B|A). \tag{1}$$

The measure $A_r(q)$ which is the accident rate (acc/10^6 veh-km) is explained in Part I of this work. This measure, divided by 10^6, can be used as a probability measure for an accident in each veh-km within the interval $q \pm \Delta q$. It is certain that the event A is dependent on the hourly flow q and therefore, $p(A) = p(h < T|q)$. The expression $p(B|A)$ in eqn (1) depends also on q and it is represented by a power function. These interpretations lead, from eqn (1), to:

$$A'_{rF}(q) = p(h < T|q) \cdot \alpha_1 q^{\beta_1} \tag{2}$$

where $A'_{rF}(q) \cdot 10^{-6}$ is the (probability) measure for a multi-vehicle accident in each veh-km and $\alpha_1 q^{\beta_1} \cdot 10^{-6}$ represents the probability of a risky situation within the flow $q \pm \Delta q$ such that

$h < T$; α_1, β_1 are constants and certainly all the probability expressions are constrained so as to be less than 1.

Another approach in the interpretation of $p(B|A)$ is through the definition of $A_{d*}(q)$, from Part I of this study, which is the accident density (acc/10^3 km) per one hour exposure of traffic flows within the interval $q \pm \Delta q$. Under free flow of q veh/hr, the number of headways which are potential for accidents is $q \cdot p(h < T|q)$. Assuming a steady flow for one hour:

$$A_{d*}(q) = q \cdot p(h < T|q) \cdot \gamma(q)$$

where it is clear that $\gamma(q) \cdot 10^{-3}$ is the probability for being in a risky situation when $h < T$. Since $A_{d*}(q) \cdot 10^{-3}/q$ is the number of accidents per veh-km, it turns out that $\gamma(q) = p(B|A)$.

For single-vehicle accidents under free-flow conditions, one cannot assume a clear cut intersection between events. Equation (3) represents the power function for these accidents:

$$A''_{rF} = \alpha_2 q^{\beta_2} \tag{3}$$

where $A''_{rF} \cdot 10^{-6}$ is the (probability) measure for single-vehicle accidents in each veh-km and α_2, β_2 are constants. The total measure for free-flow accidents is the summation of eqns (2) and (3):

$$A_{rF} = p(h < T|q)\alpha_1 q^{\beta_1} + \alpha_2 q^{\beta_2}. \tag{4}$$

For the complimentary congested-flow conditions, a simple power function is selected as a model (due to already determined criterion, mentioned in the previous section):

$$A_{rC} = \alpha_3 q^{\beta_3} \tag{5}$$

where $A_{rC} \cdot 10^{-6}$ is the measure for multi-vehicle accidents in each veh-km and α_3, β_3 are constants.

4. GENERALIZED HEADWAY MODELS

The first expression in eqn (4) includes the headway probability distribution which basically is determined by measurements of headways between successive vehicles in a single-lane stream. While substantial literature has developed regarding the mathematical description of this distribution [Edie, 1974], only a few studies are concerned with model parametrization in respect to different q values.

The search for a generalized headway model in terms of q dependency has led to three different models—each constructed with two components associated primarily with free and constrained vehicles. The first, reported by Grecco and Sword [1968], is an empirically-based model which considers Schuhl's distribution (best fitted distribution among: Schuhl, Gamma, Erlang and Pearson type III). Their results, from measurements on four-lane divided roadways, include an hourly flow variable on a per lane basis, q_1, and takes the form:

$$p(h \geq t) = 115 \cdot 10^{-5} \cdot q_1 \cdot e^{(1-t)/2.5} + (1 - 115 \cdot 10^{-5} q_1)\, e^{t/(0.0122 q_1 - 24)} \tag{6}$$

where $t \geq 1$ second and the parameters were derived according to the range $0 < q_1 \leq 700$ veh/hr.

The second study, reported by Dawson and Chimini [1968], describes what they call the hyperlang headway model. Their model is a linear combination of a translated exponential function and a translated Erlang function:

$$p(h \geq t) = \alpha_1 e^{(\delta_1 - t)/(\gamma_1 - \delta_1)} + \alpha_2 e^{k(\delta_2 - t)/(\gamma_2 - \delta_2)} \sum_{x=0}^{k-1} \frac{\left(k \dfrac{t - \delta_2}{\gamma_2 - \delta_2}\right)^x}{x!} \tag{7}$$

where δ_1, δ_2 are the minimum headways and γ_1, γ_2 are average headways for free and constrained vehicles, respectively; k is an index that indicates the degree of nonrandomness in the constrained headway distribution; and α_1, α_2 denote the proportion of free and constrained vehicles, respectively ($\alpha_1 + \alpha_2 = 1$). The parameters of eqn (7) were evaluated for one-lane flows (on a four-lane divided roadway) ranging from 158 ($k = 1$) to 957 ($k = 6$) veh/hr. Though eqn (7) does not include the hourly flow variable, the adjustable parameters for nine different flow levels provide sufficiently adequate data for this work.

The third work, recently reported by Wasielewski[1979], is based on the semi-Poisson headway distribution model. The estimate of the total headway probability density function, $\hat{f}(t)$, is given by:

$$\hat{f}(t) = \Phi\hat{g}(t) + A\lambda\, e^{-\lambda t} \int_0^t \hat{g}(u)\, du, \tag{8}$$

with

$$\Phi = 1 - A\lambda \int_0^\infty e^{-\lambda t} \int_0^t \hat{g}(u)\, du\, dt$$

where Φ is the proportion of following (constrained) vehicles; $\hat{g}(t)$ is an estimate for $g(t)$ and the latter is the probability density function of the constrained vehicles; A and λ are parameters which are evaluated from the observed data (in those situations in which the vehicles are not under the car-following mode). The findings of Wasielewski introduce the function $\hat{g}(t)$ and indicate that no significant disagreement is found between $\hat{f}(t)$ and the observed total headway probability density function; also, it is interesting to note that the flow dependence is considered only through the parameters A and λ. These findings are based on 42,000 observed headways regarding 12 groups of hourly flows ranging from 922 to 1985 veh/hr per lane on a six-lane divided roadway.

The above three reviewed models are used here to evaluate the expression $p(h < T|q)$ in eqn (4). The value of T defined in the previous section is considered as 2 sec. Generally, T is a distributed variable and ranges from 0.5 to even 4.0 sec, depending on the complexity of the driving situation[Greenshields, 1965]. Let us recall that $h < T$ is considered as a *potential* situation for a multi-vehicle accident where for $h \geq T$, it is improbable that such an accident will occur. The value of $T = 2$ is substituted in eqns (6) and (7), and in the numerical integration of eqn (8), in order to obtain the function $p(h < 2|q_1) = 1 - p(h \geq 2|q_1)$, where q_1 = the flow on a per lane basis.

The results are demonstrated in Fig. 1. In the upper part of this figure, the results of each model are exhibited separately where the flow levels correspond to the based-data of each model. In the lower part of Fig. 1, a regression line based on a power function is indicated for all the models' results, namely:

$$p(h < 2|q_1) = 0.011 q_1^{0.472} \tag{9}$$

with standard error (SE) of 0.014 probability units. It is rather interesting to note that the regression line for the hyperlang and empirical-based models only, in which $q \leq 957$ veh/hr, is almost like eqn (9). That is, $p(h < 2|q_1) = 0.011 q_1^{0.478}$ with SE = 0.024. The interpretation of the latter result is that extrapolation of the first two models fits very well an independent model which is calibrated with data characterized by the range $922 \leq q_1 \leq 1985$ veh/hr. This finding supports and strengthens the generality of eqn (9).

5. REGRESSION RESULTS AND DISCUSSION

The selected data are described in Section 2 with the separation criterion between hourly free-flow and congested-flow periods. The fitted power function to multi-vehicle accident data under free-flow conditions is:

$$A'_{rF}(q) = 4.3 \cdot 10^{-5} \cdot q^{1.32} \tag{10}$$

with SE = 0.19 (acc/10^6 veh-km). The breakdown of eqn (10) in accordance with eqns (2) and

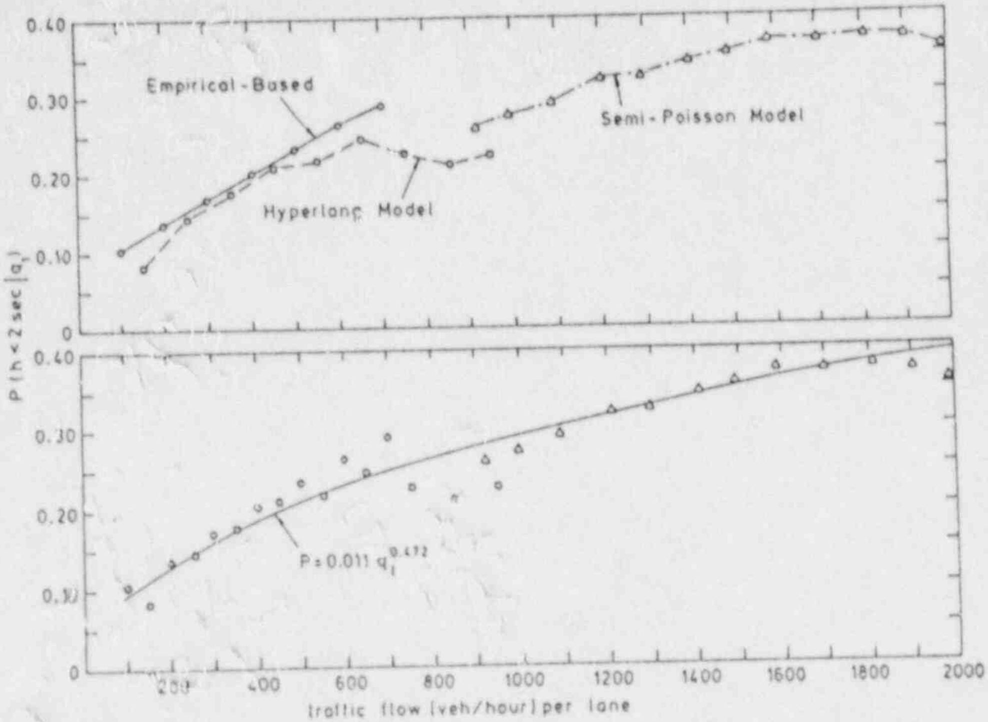

Fig. 1. The probability of headway less than 2 sec according to the empirical-based results of Grecco and Sword[1968], the Hyperlang Model of Dawson and Chimini[1968], and the semi-Poisson Model of Wasielewski[1979]; in the lower figure, a regression model is shown for the three sets of results.

(9), reveals that $\alpha_1 = 5.42 \cdot 10^{-3}$ and $\beta_1 = 0.848$: This breakdown presumes that eqn (9) can be applied to two-lane flows by considering separately each lane behaviour, i.e. $p(h < 2|q)$ is based on eqn (9) with $q_1 = \frac{1}{2} q$. Consequently, α_1 and β_1 of eqn (2) determine the expression $p(B|A)$ belong to eqn (1).

For single-vehicle accidents, the following formula is obtained through regression:

$$A''_{rF}(q) = 232.27 \cdot q^{-1.15} \tag{11}$$

with SE = 0.34 (acc/10^6 veh-km). Equation (11) is associated with eqn (3) and eqn (4) is fulfilled through the summation of eqns (10) and (11), i.e. $A_{rF} = A'_{rF} + A''_{rF}$ which is the total measure for free-flow accidents. The left side of Fig. 2 illustrates both the data points and the free-flow model.

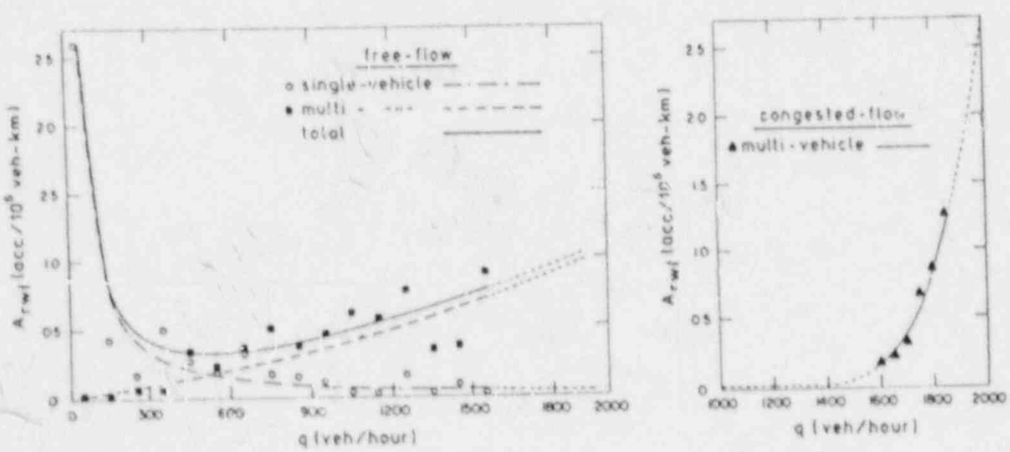

Fig. 2. The data and regression models for free-flow and congested-flow conditions.

The optimum conditions, by differentiating, yield $q_{oF} = 503$ veh/hr, which is similar to that found in Part I of the work (for data composed also of night accidents and without separating free and congested conditions).

For congested flow conditions, eqn (5) through regression, takes the form:

$$A_{rC} = 7.21 \cdot 10^{-48} \cdot q^{14.46} \tag{12}$$

with SE = 0.06 (acc/10^6 veh-km), and is demonstrated on the right side of Fig. 2. Despite the small number of data points, it is possible to observe a sharp increase in A_{rC} as q increases.

In traffic flow theories [Edie, 1974], a congested-flow behaviour refers particularly to a low, slow and congested stream of vehicles. Under these conditions, the time headway (not the spacing between vehicles) is usually higher than that observed under high flow levels and therefore, the probability of collisions is reduced. Perhaps this explanation can cast light on the results in Fig. 2 which show a diminishing tendency of A_{rC} as q decreases. A study on the attentional demands of drivers [Ceder, 1977], also indicates the increase in collision risk under peak flow conditions. This study, based on a driver's uncertainty model, shows that under peak flow conditions (small spacing with relatively high speed), drivers tend to absorb information incompletely. This mode is characterized as overload attention. The latter might explain the relatively high probability of being involved in a collision at such flow conditions.

Generally, traffic engineers attempt to manage traffic at high flows in order to enable movement of as many units of car as possible in a unit of time. Their belief in a productivity measure such as this results in neglect of the safety component, which is clearly indicated in Fig. 2. It is desirable to approach a weighting objective function which will balance increased savings in travel time with an increased accident rate as the flow level increases.

6. PROBABILISTIC ASPECTS

This section further examines probabilistic interpretations of the accident measures. These aspects are an essential input for both simulation studies and theoretical models of traffic accidents.

Equation (1) considers the intersection between the two events A and B. While event A has been widely investigated, event B is a complex one and depends on the driver population, human factors and other elements which can hardly (if at all) be predicted. An attempt is made here to examine event B given that event A occurs, based on the investigated data. The component $p(B|A)$ in eqn (1) takes the form:

$$p(B|A) = 5.42 \cdot 10^{-9} \cdot q^{0.848} \tag{13}$$

which is the probability of being in a risky situation given that the headway (between two vehicles in a single lane) is less than two seconds. For example, if one counts 491 vehicles in a single lane during one hour, one can expect to observe (or measure) 100 out of 490 headways to be characterized by $h < 2$ sec (using eqn 9). The probability, for those vehicles involved in these 100 headways, of being in a risky situation for one kilometer of driving is $1.87 \cdot 10^{-6}$ (substituting $q = 982$ in eqn 13). In other words, this is the probability that the situation becomes an actual accident from a potential accident.

Two additional probabilistic aspects which can be derived from the results of this study are:

(1) determination of the number of kilometers with an hourly flow q, for a given probability such that (at least) one accident will occur.

(2) determination of the number of hours with an hourly flow q, for a given probability such that (at least) one accident will occur.

For both aspects, the determined quantity (kms or hrs) does not necessarily maintain the continuity property (e.g. for the first aspect it gives the number of kms exposed to q in one year for a given probability).

In fact, for both aspects repeated independent trials (Bernoulli trials) are performed. Considering the first aspect, one inspects whether or not (at least) one accident will occur at each veh-km under the flow q, presuming independence between each two inspections. For large numbers of veh-kms the description of the first aspect approaches the normal distribution

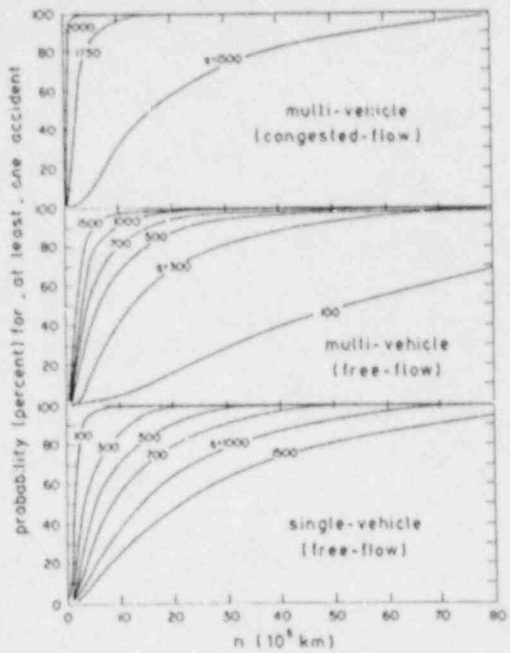

Fig. 3. The resultant relationship between the probability for at least one accident and n for various q(veh/hr) values.

(approximation to the binomial distribution). Also, since the analysis is not successive with respect to q, a vehicle involved in an accident is, theoretically, not excluded from further examinations (otherwise, the appropriate distribution is geometric rather than normal). Thus, the analysis of previous sections enables a definition:

$A_r(q) \cdot 10^{-6}$ = the probability of being involved in an accident in each veh-km within the flow range $q \pm \Delta q$ ($\Delta q = 100$ veh/hr)

X = number of accidents—normally distributed (μ_1, σ_1^2)

where

$$\mu_1 = n \cdot A_r(q) \cdot 10^{-6}; \quad \sigma_1^2 = n \cdot A_r(q) \cdot 10^{-6}[1 - A_r(q) \cdot 10^{-6}]$$

and n is the number of kilometers travelled by a vehicle at a given flow (within the range $q \pm \Delta q$). The investigated probability is $p(X \geq 1|q)$. Figure 3 illustrates this probability as a function on n for different flow levels, based on eqns (10)–(12). For example, at the probability of 90%, a flow of 1000 veh/hr needs to cover 40 million km so that (at least) one single-vehicle accident will occur, in comparison with 8.5 million km for a multi-vehicle accident. Figure 4 illustrates the functional dependency between n and q for three probability levels: 30, 70 and 95%. That is, both Figs. 3 and 4 demonstrate the resultant relationship between n, q and $p(X \geq 1|q)$—each in a different manner.

Similarly, for the second aspect:

$A_{dw}(q) = A_r(q) \cdot q \cdot 10^{-6}$ = the probability of being involved in an accident on any kilometer exposed to one hour of flow within the range $q + \Delta q$

y = number of accidents—normally distributed (μ_2, σ_2^2)

where

$$\mu_2 = m \cdot A_{dw}(q); \quad \sigma^2 = m_2 \cdot A_{dw}(q)[1 - A_{dw}(q)]$$

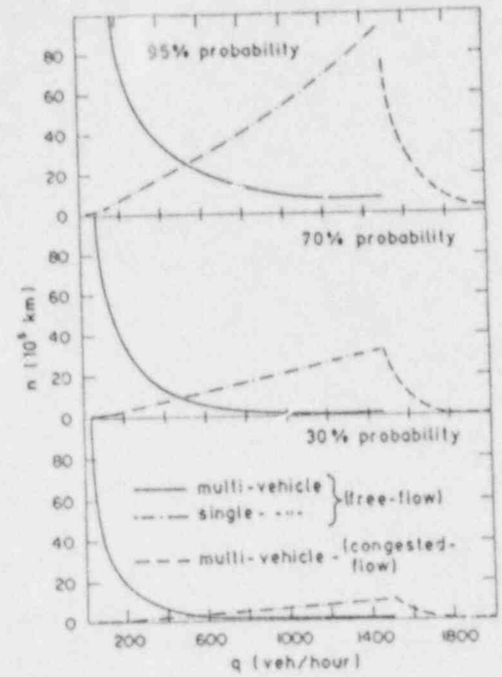

Fig. 4. The resultant relationship between n and q for three levels of probabilities (for at least one accident).

and m is the number of hours which experience a flow (within the range $q \pm \Delta q$) at a given kilometer. The investigated probability $p(y \geq 1|q)$ is shown continuously in Fig. 5, and for only three levels in Fig. 6.

7. SUMMARY

This study which is the last, and phase IV of the entire research (shown schematically in the first figure in Ceder and Livneh, 1981), attempts primarily to consider accident data regarding

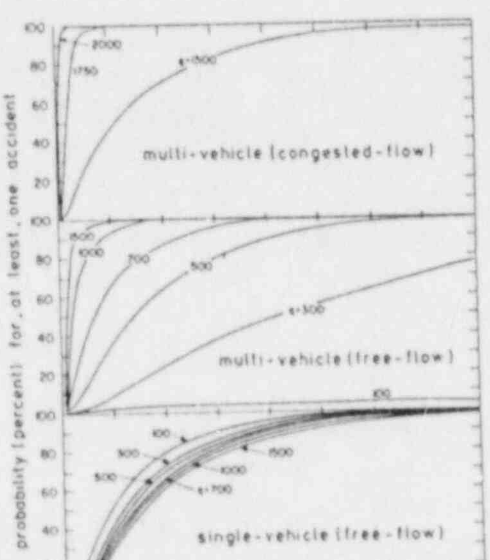

Fig. 5. The resultant relationship between the probability for at least one accident and m for various q (veh/hr) values.

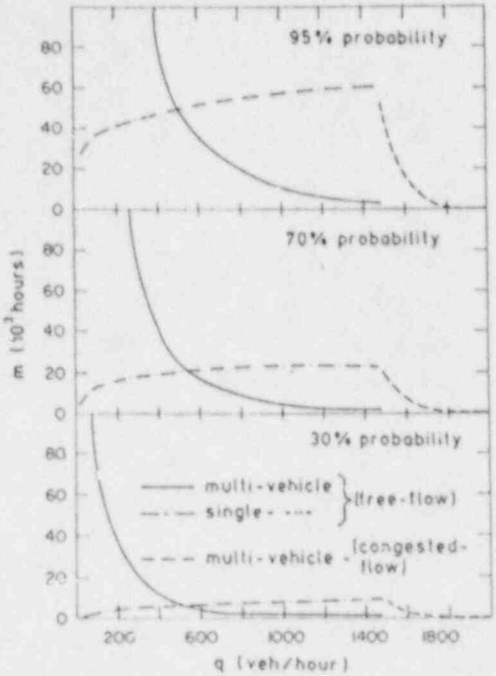

Fig. 6. The resultant relationship between m and q for three levels of probabilities (for at least one accident).

the separation between free-flow and congested-flow conditions. The adopted and developed probabilistic approach outlined in this paper is another way of tackling the determined relationships between the accident rate and the hourly traffic flow.

Based on a predetermined criterion, the congested-flow conditions are separated from the free-flow conditions, and subsequently applied and correlated to accident data in Fig. 2. For the free-flow data, the total accident rate curve follows the known U-shaped configuration with respect to the hourly flow, which is the result of combining a convex downward and a convex upward curve for single and multi-vehicle accidents, respectively. For congested-flow data, the (multi-vehicle) accident rate is sharply increased with hourly flow. This outcome suggests, from a safety viewpoint, avoidance of high flow levels in contrast to the general traffic engineers' desire to move as many cars as possible in a unit of time (i.e. to approach a capacity level). A balanced traffic productivity measure might then attempt to maintain the stream of vehicles (assuming it is under control) at a buffer point below its maximum range.

The probabilistic aspects utilize a generalized headway model fitted to three different models from the literature. This headway model is hourly flow dependent and it represents the probability that two vehicles which are, even instantaneously, under the car-following mode, are in a potentially hazardous situation. The remaining underlying probability aspects are based on the determined accident models for the free-flow and congested-flow conditions. It is believed that such models are essential input both for simulation studies and for theoretical models of road traffic accidents.

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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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