TRAVEL TIME MODELS

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INTRODUCTION

The estimation of travel time is really quite simple until one attempts to actually calculate a consistent value under varying road conditions. The reason for this is that no single satisfactory theory of traffic flow is available. There are many theories but most are hard presses to actually represent what is being observed on modern highways. Also the estimation of travel time for analytical purposes is the summation of link travel times through networks made up of; freeways, two-lane highways and intersections that may or may not have traffic signals. Added to this is the complexity of; significant geometric changes, heavy vehicles, congestion and urban rural travel. This paper outlines the current problem and suggest a possible remedy.

1. TRAVEL TIME MODELS

The travel time for a journey may be summarized by the equation;

$$T = \frac{d}{V(Q)} + D(Q) \tag{1}$$

where T = journey time
d = distance travelled
V = average running speed
D = stopped delay
Q = a flow dependent variable

Wardrop (1952) found that, for central London, the journey speeds were fairly symmetric and had a coefficient of variation about 25 percent less than journey times which appear more log normal. Using speed flow data for central London, he developed the empirical relationship:

$$v_r = \min 31 - \begin{bmatrix} \dfrac{q + 430}{3(w-6)} & , mph \\ 24 & , mph \end{bmatrix} \tag{2}$$

where v_r = space mean running speed, mph
q = total flow, veh/h
w = road width, feet

Belcher (1990) has reported on an extension of Wardrop's work for central London that gave average daily journey speed (0700 to 1900 hours) of:

$$V = 23\left[1 - \frac{q}{C}\right]^{1/2} \tag{3}$$

where V = average network journey speed, mph
q = observed flow, pcu/h
C = ultimate capacity, pcu/h

Vehicle speeds in central London have slowly but erratically decreased from 12.1 mph in 1968-70 to 11.0 mph in 1986-88. The outer London roads' speed ranged from 24.6 mph in 1968-70 to 22.7 in 1983-85. The vehicle "crawl" speed appears to be about 11 mph. Equation 3 resembles a steady state model of journey time.

Another way to estimate time travelling along a route is to idealize the situation as a long queuing problem. The relationship has been used in transportation planning study travel time flow models for Toronto and Brisbane as reported in Blunden (1971). The function form is:

$$t = t_o\left[\frac{1-(1-j)\frac{q}{s}}{1-\frac{q}{s}}\right] \tag{4}$$

where t = travel time per unit distance
t_o = free flow travel time
q = flow
s = saturation (capacity) flow
j = level of service factor related to parking, intersection, etc.

When $j = 1$ the result is a single channel queuing model for exponential arrival and service times. Typical values for t_o, s and j as reported by Blunden (1971) are given in Table 1.

The simplified version of this last equation is the U.S. Bureau of Public Roads travel time model which is:

$$t = t_o \left[1 + 0.15 \left(\frac{q}{c} \right)^4 \right] \tag{5}$$

Smeed (1967) proposed the following model for average journey time within the CBD:

$$\bar{t} = \frac{t}{2} + \frac{\left(7.409 A^{1/2}\right) 10^{-6}}{\left[1 - \frac{n}{33} t A V_2 \right]^{1/3}} \tag{6}$$

where: $\bar{t}$ = average journey time measured from the time the first vehicle enters the CBD
- t = period over which entries to the CBD are spread
- n = number of vehicles entering CBD during t
- A = area of CBD in feet, squared, and
- f = fraction of CBD area devoted to roads

Table 1

PARAMETER VALUES FOR TRAVEL TIME MODEL

Condition	t_o, min/mile	j	S veh/h
Motorway	0.8 - 1.0	0 - 0.2	2000
Urban Arterial	1.5 - 2.0	0.4 - 0.6	1800
Collector Road	2.0 - 3.0	1.0 - 1.5	1800 total
Arterial, Brisbane	1.55	0.43	1820
London			
75 ft wide	2.3	0.295	3977
52 ft	2.3	0.300	2756
37 ft	2.3	0.767	2419
32 ft	2.4	1.095	1929
22 ft	2.7	1.405	1227

The actual calculation of travel time can be quite complicated. An example of complexity is the instructions in the AASHTO(1977) Red Book on how to avoid the journey time calculation with congestion queues because it is very complex. Not only is the process complicated, the interactions of various components of the road are not well understood. Ideally, the analyst wants a closed-form mathematical model that will give consistent unbiased answers. The development of procedures to estimate journey time have gone from simple mathematical models to complex computer simulations. The validity of many of the models must be questioned. For example the journey time between two bridges in the city of Vancouver has remained constant for the last forty years even though the number of traffic signals and the volume of traffic has increased greatly.

2. TRAFFIC FLOW MODELS

The difficulties of dealing with congestion by analytically correct techniques appears to be forcing much of the recent interest in traffic flow models and travel delay. Central to most travel time estimates is some observed value of speed. Speeds on North American highways have changed dramatically as have the influencing factors. There is no agreement on either an "optimal" or "safe" speed. It would appear that the only speed that can be calculated from first principles is the "economic speed", a speed at which no one drives. The observed driving speed is generally beyond the speed limit for the majority of drivers and follows a normal distribution.

The speed flow relationships of the 1985 HCM have been shown to be wrong by many authors. That the model was incorrect was known as early as the 1970's and reported by Yager (1971) for an urban expressway in California. Studies in the U.K. by Duncan (1974) found that for motorways the speed flow was constant at about 95 km/h out to about 73 percent of capacity, then the speed dropped to 80 km/h. A revised model, known as the JHK model is a better representation of field observations of U.S. traffic, see Chin and May (1991) who recommend the JHK model for the basic section of a freeway.

Most of the studies have avoided dealing with the issue of congestion flow modelling. Allen, Hall and Gunter (1985) have shown that care must be used when collecting data since it is quite easy to mix up free flow and congested data. Congestion flow models have been proposed by Heidemann (1987) for Germany autobahns and Koshi et al (1983) for Japanese freeways.

The estimation of travel speed through weaving and ramp sections seem to be inconclusive and erratic. The two approaches to estimate speed are; estimate speed for each lane and then use a weighted average value, or average all the data over all the lanes.

The estimated capacity conditions and speed-flow relationships proposed by various authors are given in Table 2.

Table 2

ESTIMATED FREEWAY CAPACITY CONDITIONS

Source	Section	Flow pcphpl	V_c, Speed km/h	V_f-V_c km/h
Greenshields (1933)	Highway	2350	35	35
1965 HCM	Basic	2000	45	40
1985 HCM	Basic	2000	50	40-50
JHK	Basic	2200	88	8
Hall & Gunter (1986)	Basic	2200	80	
Chin & May (1991)	Tunnel	2300	80	16
Cassidy May (1991)	Weaving	2200	77	20
Heidemann (1987)	Rural	1900	80	32
Koshi et al (1983)	Basic	2000	50	10

The ideal capacity of the basic section of freeway is about 2200 to 2300 pcphpl. This is an increase from 2000 in the 1985 Highway Capacity Manual but lower than the 2400 suggested by some authors. The influence of flow on speed ranges from a high of 2.25 (km/h)/100 vehicles/h in the 1985 HCM, through about 0.70 for a tunnel to a low of 0.36 for the JHK curve. The change from the 1985 HCM to the JHC curve is almost one order of magnitude.

The capacity of a two-lane rural highway is more varied. The observed flow range is between 2200 to 2800 pcph for two way travel. The actual maximum number is difficult to estimate and is a complex interaction of vehicle flow, trucks passing opportunities, etc. The traditional speed-density and speed-flow ideas are best represented by the U.S. work of Greenshields (1933) and U.K. studies by Duncan (1974). Hoban (1987) has incorporated these ideas along with a crawl speed for beyond capacity conditions into the World Bank's HDM III speed-flow model. The recent data collected by Yager (1983), Morrall and Wearner (1986), and Robichaud et al (1991) does not entirely support the classical ideas, see Figure 1.

The speed-flow and speed-density relationship has changed as shown in Figure 2. There is enough evidence in the literature to suggest the following. First, speed is very insensitive to flow out to a value approaching capacity. At capacity, the speed smartly moves to a crawl and also the highway has a flow somewhat less than capacity. Second, speed and density may form a concave relationship. The actual relationship may be either the bilinear model suggested by Mohr (1983) or the more elliptical one proposed by Heidemann (1987).

Figure 1

SPEED FLOW OBSERVATIONS

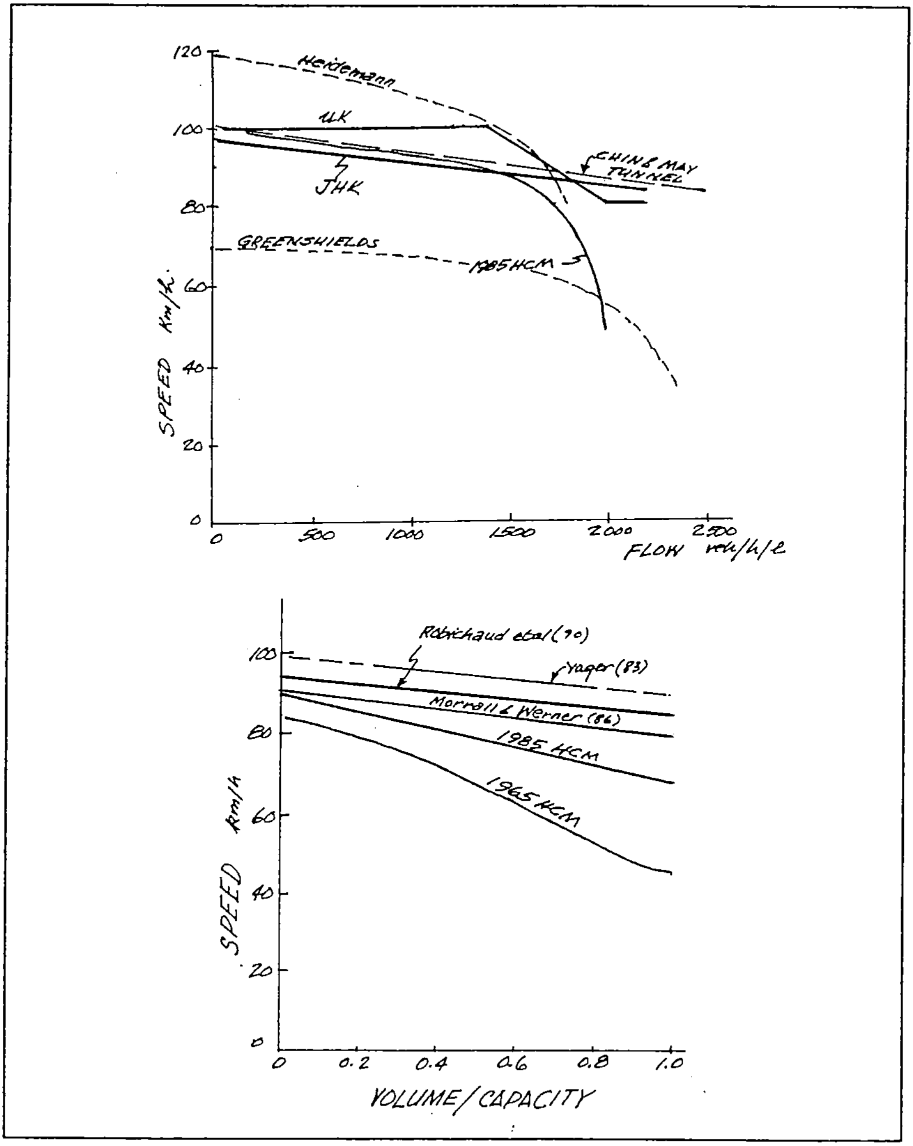
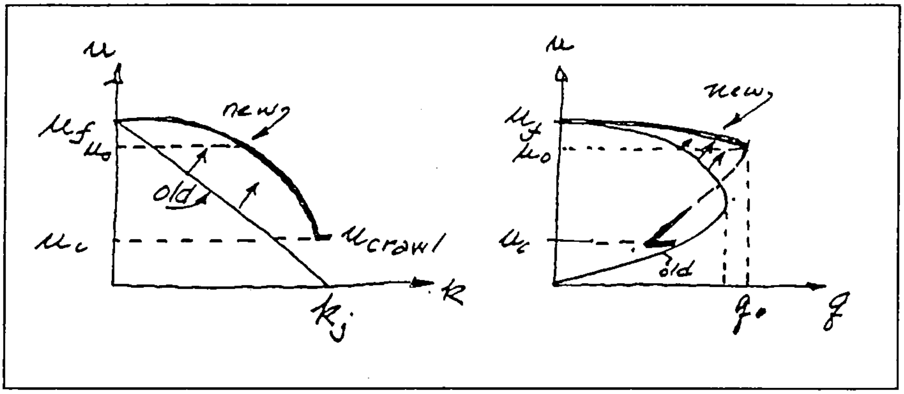


Figure 2

CHANGING SPEED-FLOW-DENSITY



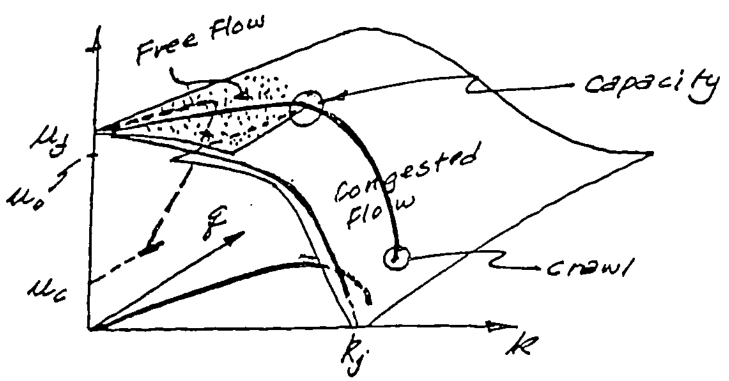
An isometric view of the "catastrophic" model of speed, flow and occupancy is shown in Figure 3. The ideas behind this construction are to be found in Navin and Hall (1989). The surface curve casts a shadow on the other planes that form the traditional u-q, u-k, q-k, curves. The surface has an upper and lower sheet. The upper sheet (shaded) represents free flow conditions and is essentially a linear relationship between speed-flow-density. According to observations, as capacity is approached, it is possible for the speed to change while flow and density remain constant. The data supports the idea of an unstable area beyond capacity and a crawl condition where speed drops to the lowest surface and flow is somewhat less than capacity.

The influence of geometric constraints such as curves and grades have been well researched by Wardrop (1963), McLean (1976), and Branac (1990). The speed estimates do appear to be reasonably robust. The influence of trucks, when on grades, has also gained increased attention as their numbers have increased, see Khan et al (1990). Trucks and recreational vehicle impact on a grade is now estimated by computer simulation but there is a reasonable consistency of the observed data.

Finally, much of the early data being used in the basic theories come from the 1950's and 1960's. Motor cars as a device have changed greatly and are much more responsive to the driver. While this observed fact has not been integrated into the speed models, its influence on traffic stream characteristics is only being observed in sufficient detail to suport the proposed changes.

Figure 3

PROPOSED SPEED-FLOW-DENSITY RELATIONSHIP



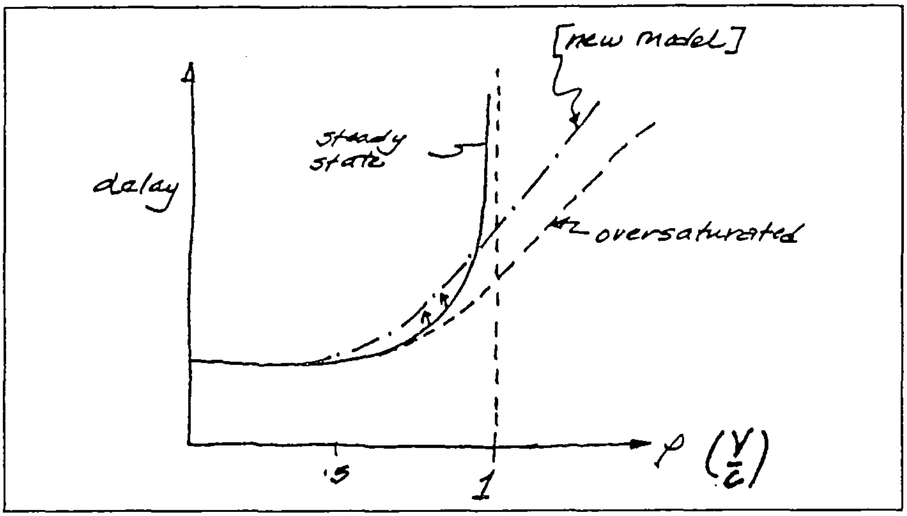
3. INTERSECTION DELAY MODELS

Considerable progress has been made in the analytical techniques to evaluate delay at signalized intersections. The work by Webster (1958) of the U.K., the 1985 HCM, and Alcelik (1988), of Australia has been shown by Burrow (1989) of the U.K. to be special cases of a more general detailed delay equation. All the models give acceptable answers to a saturation of 0.50, see Figure 4. The delay functions in the 1985 HCM and the Australian model have difficulty in the range from 0.70 to 0.80. Burrows' detailed model corrects these problems by a more precise consideration of traffic peaking characteristics. There is still considerable work to be completed before any of these models can be used to represent congested conditions.

The classic papers on unsignalized intersection delay were written by Wardrop (1952), Tanner (1951), and Webster (1958). The delay at uncontrolled (non-signalized) intersections is computationally very elegant but the results are suspect particularly at high volumes. Some researchers would go so far as to question the validity of the negative exponential gap acceptance assumption.

Figure 4

DELAY TIME AT A SIGNALIZED INTERSECTION



4. IMPACT ON TRAVEL TIME MODELS

This summary has outlined a few of the new ideas that are coming into the traffic flow and intersection delay literature. The impact of these changes on the results of benefit-cost analysis have yet to be assessed but one suspects they will be quite significant on travel economic time benefits. The impact on the time benefits may be only twenty percent rather than eighty percent.

Greenshields (1973) wrote an interesting article in which he was critical of using only time to measure the efficiency of traffic systems. He proposed a new measure that is claimed to include; cost, safety, and comfort. The quality of flow index is modelled after the Reynolds number in fluid flow and has the form:

$$Q = \frac{tsd}{L} \tag{7}$$

where:
- Q = quality index
- t = time
- s = change of speed
- d = change of direction
- L = distance

The smaller the value of Q Greenshields claims, "the cheaper, safer and more comfortable is the travel." The quality index should be used with other measures such as cost.

5. CONCLUSION

There is little doubt that automobiles and to some extent driving have improved over the last half century. These changes have been reflected in the relationship between speed, flow and density or occupancy. The end result is that where cars previously operated almost independently within a stream of traffic now they must be considered as platoons or as loosely connected trains of vehicles pulsing along a highway. Consequently vehicles travel very close together at high speeds and this combined with the shorter high performance vehicles has produced a higher volume at capacity and reduces influence of flow on speed.

The speed flow relationship is a fairly flat linear speed-flow out to capacity rather than a drop to a crawl speed and flow reduced from capacity. The speed-density model is either some bilinear relationship as suggested by Mohr (1983) or a more elliptical shape found by Heidemann (1987) in Germany. The ideal capacity is probably about 2400 vehicles per hour measured over 15 minute intervals and a reduction of speed being about 0.4 to 0.7 (km/h)/100 veh/h). The crawl speed is about 20 km/h.

The basic economic difficulty remains, if travel speed is insensitive to traffic volume then how does an analyst employ time to estimate traffic diversion. The economic justification of many new facilities rests upon as much as 80 percent of the benefits from time savings. The revised shape of the speed flow curve probably means the benefits will be reduced to only about 10 of the original estimate. There is obviously more involved in the time diversion then simply time difference. Greenshields' suggestion of a new measure is probably appropriate. Even given new measures to judge projects analysts will probably have to rely more and more upon detailed vehicle simulations to accurately evaluate future projects.

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