Journey speed. The final formula for the journey speed \( v \) in central London in 1962 is therefore given by

\[
1 - \frac{v}{28 - 0.0058q} = \frac{1}{1 + \frac{0.0057}{28 - 0.0058q}}
\]

Up-dating the formula

Thomson\(^{5}\) has shown that there is evidence for increases in the capacity of streets in central London. In order to bring the London curve up to date, it is necessary to make some allowance for this. This is most easily done by ensuring that the curve passes through the observed value for 1966. It will do so if the capacity of the intersections is assumed to have increased by 6 per cent, from 2,610 to 2,770 p.c.u./h, so that the coefficient of \( q \) in the second term of the formula becomes 0.071. The resulting curve is shown in Fig. 3, together with Thomson’s observed values for 1966, which show reasonable agreement with the curve. As Thomson pointed out in the same paper the peak figure for 1962 was somewhat exceptional, and the assumption made in the earlier paper \(^{5}\) that the capacity was substantially higher during the peak hour than at other times can be discarded. Thomson timed a linear relation between journey speed and flow to his data, but the curve shown in Fig. 4 agrees better with the observations and, more importantly, with known information on the absolute capacity of intersections.

Generalisation of the formula

If \( v \) is to be used in urban areas other than central London, it is necessary to calculate the parameters involved. The factors representing running speed can be generalised by first of all writing \( \left( \frac{v}{w} \right)^{2} = \frac{1}{w} \) in the form

\[
1 - \frac{v}{28 - 0.0058q} = \frac{1}{1 + \frac{0.0057}{28 - 0.0058q}}
\]

The second term in (3) represents delay at intersections has already been put in the general form \( F_{b} \) (1/q)\( \Delta \). In Fig. 3, London there are about five controlled intersections per mile. We may assume that the average proportion of effective green time, \( \lambda \), is 0.45 approximately (on the basis of a cycle time of 60 sec and a 27-sec effective green time, on average).

The capacity of the intersections is proportional to the average stop line width (provided this is greater than about 10 ft); we suppose this width to be proportional to the average carriageway width \( w \).

Thus the second term in (3) should be of the general form \( F_{b} \) (1/q)\( \Delta \) where \( F_{b} \) is a constant. When \( w = 42 \text{ ft} \) and \( \lambda = 0.45 \) we should have \( F_{b} = 0.36 \text{ ft} \), i.e. \( x = 147 \text{ ft} \). This gives

\[
F_{b} = \left( \frac{1}{q} \right) \Delta \text{ ft} = 0.36 \text{ ft}
\]

where \( v \) = journey speed (m.p.h.), \( q \) = total flow on road, both directions (p.c.u./h), \( \lambda \) = average carriageway width in feet (in the case of one-way streets, \( \lambda \) is in the case of two-way streets, \( \lambda \) is in the first term should be replaced by \( \lambda \)).

\[
f = \text{number of controlled intersections per mile}
\]

\[
\gamma = \text{propotion of effective green time at intersections}
\]

It should be noted that if the width of the intersections differs appreciably from that of the roads between, the value of \( w \) used in the second term should be that for intersections.

We shall now consider the effects of varying the parameters in the above equation.

The effect of varying flow. Figure 5 shows how journey speed varies with flow in a typical case with \( v = 31 - 0.8 \times 42 \) and \( \lambda = 0.45 \) for various road widths. The curves are plotted in terms of q/w because this quantity, the flow per unit width of carriageway or traffic intensity, is an important item determining the speed.

The effect of varying width. Figure 5 also shows that for a given value of traffic intensity the journey speed increases with width, although the effect is small if the width is greater than 40 ft. For a given flow q, of course, increasing the carriageway width has a considerable effect on journey speed. For example, if the width is 10 m, an increase of 50 per cent in width will increase the journey speed to about 15 m/h.

\[
f = \text{Effect of varying the number of intersections per mile. Figure 6 shows the effect of varying the number of controlled intersections per mile on the journey speed/flow relation in a typical case, with carriageway width \( w = 40 \text{ ft} \) and \( \lambda = 0.45 \). It is interesting to note that, over a considerable range of values of traffic intensity q/w, the curves in both Figs 5 and 6 are approximately parallel.

The effect of varying the proportion of green time at intersections. In practice, the proportion of effective green time is likely to be about 0.45, as for central London, because the major intersections will be more frequent will tend to be symmetrical and have values of \( \lambda \) of this order. However, there may be exceptional cases where \( \lambda \) has an effectively higher value, because the intersection approaches have been widened (or effectively widened by stricter control of parking or the intersections). Alternatively the value of \( \lambda \) may be reduced by the presence of long intergreen periods (e.g. for pedestrians) or of many signals with more than two phases. Figure 7 has therefore
been plotted to show the effect of varying λ on the journey speed/density ratio in a typical case. It will be seen that this ratio is high in w and χ, but the effect of moving the curve approximately parallel to itself is small. The effect of varying λ on the journey speed/density ratio is small.

The effect of varying λ on the journey speed/density ratio is small.

In order to deal with continuous variation in p the following approximate formula may be used:

\[ p = 0.5 (1 - e^{-1.85}) \]

where \( w \) is the average length of the unconnected road measured from the stop line in feet and \( g \) is the effective green time in seconds. If the side road is less than 50 ft, the effective length of the side road is taken as 0.5 (value of \( w \)).

The speed of the formula for the second term of (4) should be reduced by 10 when there is a stop signal.

\[ v = \sqrt{\frac{(100 - w^2)}{(w^2 - 8)}} \]

Further information required is it most desirable that the formulae derived in this paper should be tested against data of speeds in actual towns. A small set of data is available for this purpose, but a great deal more are needed. It is considered that the formulae should be revised to give a more realistic account of road width, for example, or more data were available. Information included in the volume of traffic in passing cars per hour at representative points in the central area of a town and corresponding average speeds on sample journeys across the central area. Information on existing survey work and parking on the street would also be needed. Ideally, evening and weekend periods should be covered separately, and also periods during the working day and outside working hours. Any information of this character, however limited in scope, would be useful. The author would be grateful for offers of help by providing data, either already existing, or in the future.

APPENDIX

DELAY/FLOW CURVES FOR TRAFFIC SIGNALS

Fixed time signals, Webster's formula for delay at fixed time signals, can be put in the following form:

\[ d = \frac{1 - \lambda}{1 - \lambda - \frac{y}{w}} \]

\[ y = \frac{1}{1 - \lambda - \frac{y}{w}} \]

where \( w \) is the average delay (sec), \( y \) is the cycle time (sec), \( g \) is the effective green time (cycle), \( w \) is the saturation flow (veh/h), \( n \) is the number of vehicle arrivals per cycle, and \( \lambda \) is the degree of unsaturation.

The above formulae, for \( \lambda = 0 \) and \( \lambda = 0.5 \), are used to yield the usual range of the quantities concerned, and the results are shown in Fig. 1. Here \( c \) is plotted against \( y \), and it will be seen that the curves do not depart very far from a linear relation with \( y \) (i.e., flow with flow).

Vehicle-actuated signals, Webster's formula has shown that the average delay at a typical actuated signal can be deduced with reasonable accuracy by assuming that the cycle is the optimum one for fixed time signals (giving minimum delay). It is the applied rate of flow that is common to both cycles, though in the fixed time delay formula. He also shows that the optimum cycle for two-phase signals is approximately

\[ \text{Continued on page 539} \]

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Traffic Capacity of Rural Two-Lane Highways

by J. H. Fletcher, M.Sc., A.I.C.E., A.M.I.C.E., A.M.I.H.E.

Traffic Capacity of Rural Two-Lane Highways in Great Britain is directed to maintaining and progressively improving the extensive network of two-lane roads. In order to continue evaluating funds on the most beneficial improvements much more needs to be known about the effects of various types of improvement on traffic flow and safety.

From time to time theoretical capacities have been calculated for the question of traffic to be carried on two-lane roads. These have always proved unrealistic, because the calculated maximum capacities appear to be exceeded by free flow conditions without taking into account the effect of the geometry of the road /asphalted speed and flow. In practice the capacities quoted in successive Ministry of Transport memoranda have given a much lower value to the conditions on average roads, although improved two-lane roads may carry double the recommended volume before the full capacity of the road is utilized.

What ideal, the improvement of two-lane roads to fully continuous traffic under all conditions is not a practical objective because the money is not at present available to achieve this level of provision, nor can it be justified on economic grounds in relation to other requirements. A more realistic definition of the minimum level of provision on rural roads is therefore necessary.

Improvement of rural roads

On typical rural roads drivers appear to be willing to regulate their speed to suit the alignment of the road provided that the road is fairly uniform and smooth. Faster drivers are eliminated, but frustration arises when queue building occurs. This is almost entirely due to the conflict between different classes of traffic. The build-up of moving queues of cars and light traffic behind slower commercial vehicles does not appear to be acceptable. On all but motorways the 80 mph speed limit applies to all vehicles whilst their weight ratio is too low to permit speeds comparable with those of cars on motorways and dual carriageways.

Some relief on overtaking seems desirable before roads can be said to have reached capacity. Thus the capacity of a rural road can be defined as that flow at which queues or 'bunches' in the traffic stream accumulate more rapidly than they can be dispersed. At capacity cars will be able to travel at higher speeds than commercial vehicles although the amount of delay compared with free flowing conditions will be a function of both traffic flow and the overall alignment, width and visibility standards of the road. Where this flow is exceeded on any route, traffic will build up into merr or less continuous stream at speeds of the slower commercial vehicles, as in urban traffic.

Using this definition of capacity as a basis for improvement, the first objective in maximising capacity should be to ensure that those features of the alignment which slow down commercial vehicles are improved, thus reducing the build-up of queues. Consequently, road width and curvature units, for example, can be brought up to a national standard. Secondly, visibility must be adequate over a sufficient proportion of the road to permit the required amount of overtaking (Fig. 1). Curve and stopping sight distance should be adequate to allow cars to maintain

Fig. 1. A high capacity signalised road in Oxfordshire designed

Fig. 2. Two-lane road showing inadequate passing opportunities.

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