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On the Validity of Some Traffic Engineering Folklore

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1 INTRODUCTION

This paper presents commentary on various points listed in an article titled "Traffic Engineering Folklore" by Taylor, Bennett and Ogden (1996), which were presented as "a summary of quick and easy approximations to traffic related problems". This *folklore* was based on an Australia-wide survey of traffic professionals in the 1980s. The total list consists of 149 points, covering a wide range of areas of interest to traffic engineering.

Such folklore consists of generalisations by its nature, and can be helpful if used with the understanding that various points may not be valid in specific situations, and some information may get outdated as traffic characteristics change in time. The authors advised the readers to "use the material as a good guide, and refer to the appropriate standards and manuals for final details".

This paper presents comments on selected points using information from recent research, and gives the results of tests of the validity of some points using detailed analytical and computer-based traffic modelling. The points covered relate to vehicle dimensions, capacities, vehicle and pedestrian speeds, signalised intersections and roundabouts. It is shown that most points considered are generally valid as approximations. Suggestions are made for revising the statements analysed, and additional statements are presented for consideration.

The grouping of discussion points follows approximately the same headings (topics) and refers to the numbers used in the paper by Taylor, et al. The statement from Taylor, et al. is quoted at the start of each section (or subsection), shown in italics.

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2 VEHICLE-RELATED DIMENSIONS

2.1 Car Length in Australia

In Australia, the 85th percentile car length is 4.7 m and the width is 1.9 m. (Taylor, et al, Point 6)

The statistics given in *Table 2.1* and data shown in *Figure 2.1* confirm the statement about the car length. These statistics are derived using individual vehicle data collected on the Eastern Freeway in Melbourne during a 4-hour survey of single-lane traffic (6.44 am to 10.43 am on 3 September 1998). Data were collected using a two-loop presence detection system. Detailed information on site characteristics, data collection and analysis methods can be found in Akçelik, Besley and Roper (1999).

In order to reduce possible bias in the measurement of vehicle lengths, statistics given in *Table 2.1* are based on data representing light vehicles with speed ≥ 80 km/h and vehicle length ≥ 3.0 m. Vehicles with length less than 3.0 m represented 2 % of original data. In order to use a subset of data for comparison of statistics, data representing vehicles travelling with headways ≥ 2.0 s were selected. The statistics for this subset given in *Table 2.1* are seen to be almost identical to statistics for the full data set. Data in *Figure 2.1* are for the selected subset. Further investigation with different subsets gave similar statistics.

Table 2.1

Vehicle length statistics for light vehicles (metres) based on data collected on the Eastern Freeway, Melbourne

	All data (4355 data points)	Data with headways ≥ 2.0 s (1665 data points)
Average	4.3	4.2
85th percentile	**4.7**	**4.7**
50th percentile	4.3	4.3
15th percentile	3.8	3.7
Minimum	3.0	3.0
Maximum	5.5	5.4

Statistics in this table are based on data representing vehicles with speeds ≥ 80 km/h and vehicle lengths ≥ 3.0 m.

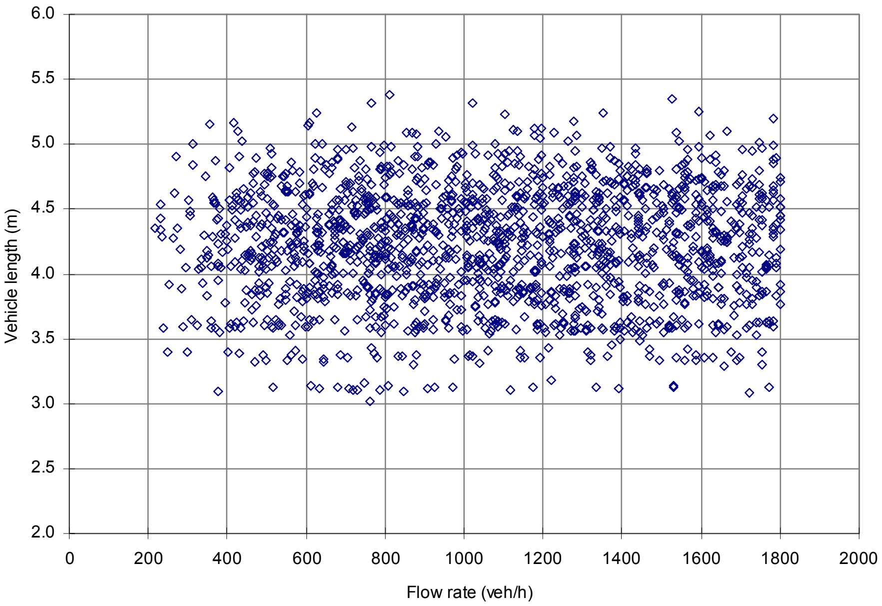

Figure 2.1 - Vehicle lengths for light vehicles measured on the Eastern Freeway, Melbourne (data for vehicles with headways ≥ 2.0 s)

2.2 Storage Length for Cars

Usually allow 6 to 7 m per vehicle storage length for a queue of cars, but sometimes 8 m is used in turning bays. (Taylor, et al, Point 7)

Queue storage lengths (jam spacings) for light and heavy vehicles as well as various related queue discharge parameters determined from comprehensive surveys conducted during 1996 to 1998 at 18 signalised intersection lanes in Melbourne and Sydney (Akçelik, Besley and Roper 1999) are given in *Table 2.2*. Parameters other than jam spacings are presented for the purpose of discussions in other sections of the paper. The symbols used in *Table 2.2* are:

L_{hjLV}	:	jam spacing for light vehicles (m)
L_{hjHV}	:	jam spacing for heavy vehicles (m)
L_{sj}	:	jam space (gap) length calculated from $L_{sj} = L_{hjLV} - L_v$ using an average light vehicle length of $L_v = 4.4$ m
v_n	:	maximum queue discharge speed (km/h)
q_n	:	maximum queue discharge flow rate (veh/h)
h_n	:	minimum queue discharge headway (s)
L_{hn}	:	spacing (m) at maximum queue discharge flow speed
v_x	:	average queue clearance wave speed (km/h)
t_x	:	average departure response time, i.e. the response time for the next vehicle in the queue to start moving (s)
v_f	:	free-flow speed (km/h)
v_n/v_f	:	ratio of the maximum queue discharge speed to the free-flow speed

Table 2.2

Queue discharge parameters at signalised intersections: Results for 18 survey sites in Melbourne and Sydney

Site	Location	L_{hjLV} (m)	L_{hjHV} (m)	L_{sj} (m)	v_n (km/h)	q_n (veh/h)	h_n (s)	L_{hn} (m)	v_x (km/h)	t_x (s)	v_f (km/h)	v_n/v_f
Average values												
Right-turn (isolated)		6.4	9.7	2.0	24.5	2033	1.77	12.0	27.3	0.84	65	0.38
Through (isolated)		6.9	11.3	2.5	45.1	2086	1.73	21.6	21.3	1.17	69	0.65
Through (paired int.)		na	na	2.6	30.9	1958	1.84	15.8	24.7	1.02	67	0.46
All Through sites		7.0	11.3	2.6	42.1	2057	1.75	20.4	21.7	1.15	69	0.61
Right-turn (isolated) sites with arrow control ("protected")												
TCS163	SYD	6.0	8.6	1.6	24.7	2098	1.72	11.8	25.7	0.84	60	0.41
TCS610	SYD	5.9	7.9	1.5	21.7	1966	1.83	11.0	24.9	0.85	60	0.36
TCS121	MEL	6.6	12.1	2.2	24.4	1948	1.85	12.5	27.2	0.87	70	0.35
TCS335	MEL	6.9	10.1	2.5	27.1	2133	1.69	12.7	32.2	0.77	70	0.39
Through (isolated) sites												
TCS1081	SYD	6.8	8.7	2.4	39.5	1790	2.01	22.1	17.6	1.39	60	0.66
TCS413	SYD	6.8	11.0	2.4	33.2	1801	2.00	18.4	19.4	1.26	60	0.55
TCS511	SYD	6.6	11.9	2.2	52.8	2283	1.58	23.1	21.1	1.13	70	0.75
TCS3196	MEL	7.0	13.1	2.6	31.7	1892	1.90	16.8	22.7	1.11	60	0.53
TCS4273 *	MEL	7.3	13.6	2.9	36.4	1938	1.86	18.8	23.1	1.14	60	0.61
TCS849	MEL	6.9	8.7	2.5	46.4	1999	1.80	23.2	19.6	1.27	70	0.66
TCS456	MEL	7.0	12.2	2.6	53.8	2422	1.49	22.2	24.8	1.02	80	0.67
Mel1	MEL	na	na	2.6	56.1	2558	1.41	21.9	26.3	0.96	80	0.70
Mel3	MEL	na	na	2.6	46.2	2423	1.49	19.1	26.8	0.94	80	0.58
Mel4	MEL	na	na	2.6	47.9	2217	1.62	21.6	23.0	1.10	60	0.80
Mel7	MEL	7.2	na	2.6	52.4	1968	1.83	26.6	18.7	1.35	80	0.66
Through (paired intersection) sites												
Mel2	MEL	na	na	2.6	30.9	1982	1.816	15.6	25.2	1.00	60	0.52
Mel5	MEL	na	na	2.6	27.1	1804	1.995	15.0	23.6	1.07	60	0.45
Mel6	MEL	na	na	2.6	34.6	2112	1.705	16.4	25.8	0.98	80	0.43

SYD = Sydney, MEL = Melbourne

na : jam spacing data not available for some Melbourne sites

* 6 per cent uphill grade

The statistics given in *Table 2.2* show that queue discharge characteristics for through and right-turn movements differ significantly, and there are also significant differences between through lanes at isolated sites and paired intersection sites. The range of light vehicle jam spacings observed for all sites is 5.9 m to 7.3 m which supports the first part of *Point 7*.

The second part of *Point 7* is not supported since, in fact, the jam spacings observed for right-turn lanes (short lanes) are in the range 5.9 to 6.9 m (average 6.4 m), shorter than those observed for through lanes (range 6.8 to 7.3 m, average 7.0 m). It is also interesting to note that the jam spacings for Sydney sites (5.9 to 6.8 m) are smaller than those for Melbourne sites (6.6 to 7.3 m), probably due to higher traffic pressure levels.

As discussed in *Section 6*, jam spacing is an important parameter at traffic signals. For example, lower jam spacings and lower departure response times (higher queue clearance speeds) at right-turn sites help to achieve low queue discharge headways, therefore high maximum (saturation) flow rates.

It is also interesting to note that the values of parameters given in *Table 2.2* for Australian conditions are consistent with those reported for US and European conditions. For example, Bonneson (1992) reported a jam spacing of L_{hjlv} = 7.9 m/veh measured under US conditions, possibly representing larger cars in the USA. He also reported start response times of 1.0 to 1.3 s (t_x in the range 0.8 to 1.4 s in *Table 2.2*), and a response wave speed of 28.5 km/h (v_x in the range 18 to 32 km/h in *Table 2.2*). Niittymäki and Pursula (1997) reported start response times (t_x) in the range 0.9 to 1.0 s, and queue discharge headways in the range 1.7 and 2.0 s observed in Finland (h_n in the range 1.4 to 2.0 s in *Table 2.2*).

Using an average light vehicle length of 4.4 m, the jam spacings given in *Table 2.2* are seen to imply gap lengths (distances between vehicles in the queue) in the range 1.5 to 2.9 m.

Table 2.2 also shows that jam spacings for heavy vehicles at signalised intersections are 7.9 to 13.6 m, with similar characteristics as light vehicle jam spacings (shorter for right-turn lanes, etc).

In conclusion, it is suggested that *Point 7* could be modified as follows:

> " Usually allow 6 to 7 m per vehicle storage length for a queue of cars (light vehicles) and 9 to 14 m for a queue of heavy vehicles. It may be appropriate to use 7.0 m per car (11.5 m for heavy vehicles) for through traffic lanes, and 6.5 m per car (10.0 m for heavy vehicles) for turning lanes. "

3 CAPACITIES

3.1 Uninterrupted Flows

For uninterrupted flow under ideal conditions, capacity is between 1800-2200 veh/h/lane. (Taylor, et al, Point 25)

In terms of daily flow, the two-way capacity of a 4-lane, 6-lane or 8-lane freeway can be estimated as 2000-2200 veh/h per lane. (Taylor, et al, Point 27)

As discussed in Akçelik, Besley and Roper (1999), the maximum queue discharge flow rate at signals could be used to represent the capacity of uninterrupted flows. Based on *Table 2.2,* uninterrupted stream capacities in the range 1800 to 2300 veh/h could be used (excluding higher maximum flow rates in the range 2400 to 2550 veh/h observed at several sites).

Akçelik, Roper and Besley (1999) reported a maximum flow rate of 2500 veh/h/lane for the Eastern Freeway in Melbourne based on 5-minute aggregation period, and 2300 veh/h based on 15-minute aggregation period. The latter is more appropriate for capacity analysis purposes, and the value of 2300 veh/h is consistent with the value suggested by the US Highway Capacity Manual (TRB 1998, Chapter 3).

It may therefore be concluded that *Points 25 and 27* quoted above are reasonable statements although it is seen that current maximum flow rates (capacities) are higher than those suggested in these statements.

In conclusion, it is suggested that *Points 25 and 27* could be modified as follows (combined):

> " For uninterrupted flows under ideal conditions, capacity is between 1800-2300 veh/h/lane for arterial roads, and 2100-2500 veh/h/lane for freeways. "

3.2 Interrupted Flows

The saturation flow for a through lane on an approach lane to a signalised intersection is 1,800-2,200 veh/h of green time. (Taylor, et al, Point 30)

Based on the survey results summarised in *Table 2.2*, it would be more appropriate to restate *Point 30* as:

> " The saturation flow for a through or right-turn (arrow-controlled) lane on an approach lane to a signalised intersection is 1,800-2,400 veh/h of green time. An ideal (basic) saturation flow of 1950 to 2100 through car units per hour per lane is appropriate for general use, subject to adjustments for site-specific factors that reduce saturation flows. "

The typical capacity of a through lane at a signalised intersection, where the intersecting roads have approximately equal flows, is between (0.45 x 1800 =) 800 veh/h and (0.45 x 2000 =) 900 veh/h. (Taylor, et al, Point 31)

This statement applies to a simple two-phase system, and assumes that 10 per cent of the signal cycle is lost due to intergreen times, which is a reasonable assumption (e.g cycle time = 120 s, intergreen time per phase change= 6 s). Therefore, a total of 90 per cent of the hour is available as green time for the two conflicting through movements (45 per cent each due to "approximately equal flows"). Capacity is calculated as the percentage of total green time available multiplied by the saturation flow (1800 and 2000 veh/h). Although, in relation to *Point 30*, increased saturation flows are suggested above, *Point 31* could be accepted without change since it is relevant to a two-phase system appropriate for small intersections where saturation flows are likely to have lower values.

The capacity of a signalised intersection can be quickly estimated by assuming 30 signal cycles per hour and that each vehicle takes 2 seconds to pass through it.
(Taylor, et al, Point 122)

Point 122 does not specify enough information as it is also necessary to know the green time. With the assumptions implied by the statement, and if the green time (G) is known (example G = 50 s):

Capacity per cycle = G / 2 (example: = 50 / 2 = 25 veh/cycle)
Capacity per hour = number of cycles in the hour x capacity per cycle
= 30 x capacity per cycle = 30 G / 2 = 15 G (example: 15 x 50 = 750 veh/h)

These assumptions are equivalent to the use of a cycle time of 120 s (= 3600 / 30), a saturation flow rate of 1800 veh/h (= 3600 / 2), and assuming that the effective green time is equivalent to the displayed green time (g = G). Thus:

Capacity = (g / c) s =(G / 120) 1800 = 15 G as above.

Thus, a simpler statement based on the same assumptions is:

> " The capacity of a signalised intersection can be quickly estimated as 15 times the green time for each traffic lane".

A better statement reflecting typical saturation flows would be as follows (for driving on the left-hand side of the road):

> " The capacity of a signalised intersection can be quickly estimated as 15 times the green time for each through traffic lane, 17 times the green time for each right-turn traffic lane (green arrow), 13 times the green time for each left-turn traffic lane, and 5 times the green time for filter right-turn lanes".

The factors given in this statement are based on saturation flow rates of 1800 veh/h, 2040 veh/h, 1560 veh/h and 600 veh/h, respectively.

3.3 Gap Acceptance and Roundabouts

In a simple gap acceptance situation with single-lane minor flow, capacity is achieved with the sum of the major and minor flows being approximately 1,400 veh/h for balanced flows and 1,600 veh/h for unbalanced flow. (Taylor, et al, Point 34)

The capacity of a two-lane roundabout is approximately equal to that of a two-lane signalised intersection. (Taylor, et al, Point 125)

These are interesting points to be tested using aaSIDRA (Akcelik and Associates 2000a) and reported in a future edition of this paper.

4 TRAFFIC FLOW CHARACTERISTICS

4.1 Volume Ratios

On urban arterials, peak hour/24 hour volume ratios are 10 per cent for uncongested conditions and 7-9 per cent for congested conditions. (Taylor, et al, Point 40)

Interestingly, it has been found through the use of the Annual Sums facility for various intersection cases in aaSIDRA 1.0 (Akcelik and Associates 2000a) that this rule also applies to various other statistics such as total operating cost (vehicle operating cost plus value of time), total fuel consumption, total CO_2 emission, etc. Thus, peak-hour to 24-hour ratios for such statistics are approximately 10 per cent, and therefore the total values of such statistics per day can be calculated as about 10 times the peak hour values.

The statement could be modified as follows:

> " On urban arterials, peak-hour to 24-hour ratios of vehicle volumes as well as such statistics as total operating cost (vehicle operating cost plus value of time), total fuel consumption, total CO_2 emission, etc. are approximately 10 per cent. Thus, the total values of such statistics per day can be calculated as about 10 times the peak hour values, and the total annual values can be estimated roughly as 3650 times the peak hour values . "

4.2 Speeds

Operating speeds depend on drivers' perceived appropriateness of the speed zone and are commonly 10 km/h more than the speed limit in 60, 70, 80 and 100 km/h zones. (Taylor, et al, Point 49)

Speed limits often are set at the 85th percentile operating speed. (Taylor, et al, Point 50)

These statements can be compared against the freeway speed data given in *Table 4.1* and shown in *Figure 4.1* based on Akçelik, Roper and Besley (1999). The data in this table indicate that average speed is equal to the speed limit of 100 km/h, and the 85th percentile speed is around 108 km/h. Thus, for the freeway environment with a 100 km/h speed limit, the above statements do not seem valid: average speed would have been 110 km/h based on *Point 49*, or the speed limit would have been set as 110 km/h according to *Point 50*.

It is also interesting to note the interdependence of the speed limit and operating speed in these two statements!

Table 4.1

Speed statistics for light vehicles (km/h) based on data collected on the Eastern Freeway, Melbourne

	All data (4355 data points)	Data with headways ≥ 2.0 s (1665 data points)
Average	100	101
85th percentile	107	109
50th percentile	100	102
15th percentile	92	94
Minimum	80	80
Maximum	132	132

Statistics in this table are based on data representing vehicles with speeds ≥ 80 km/h and vehicle lengths ≥ 3.0 m.

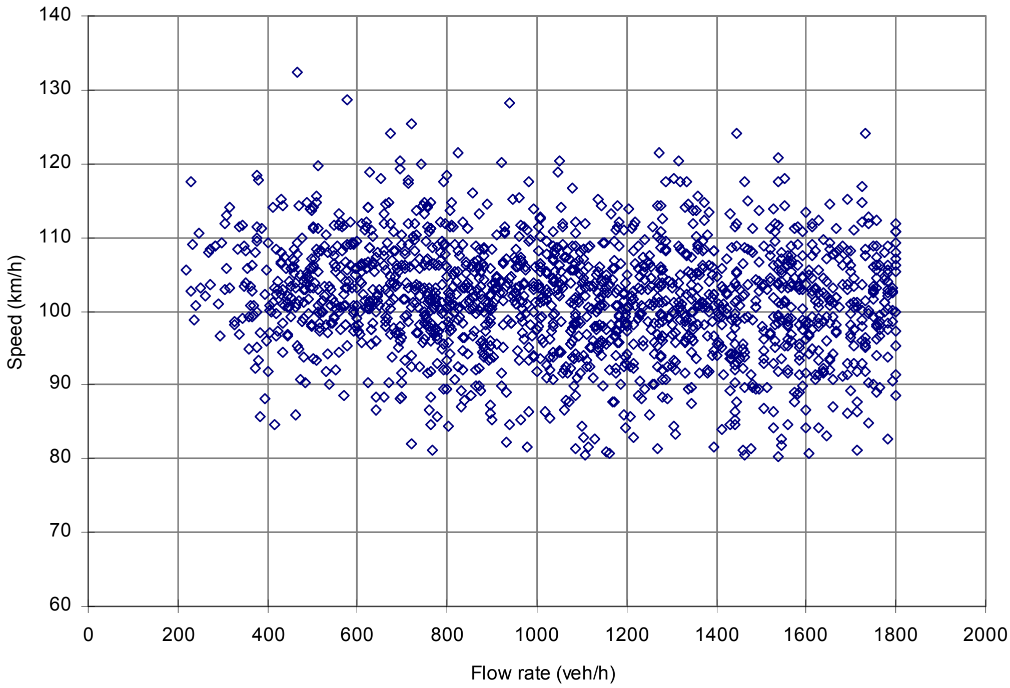

Figure 4.1 - Light vehicle speeds measured on the Eastern Freeway, Melbourne (data for vehicles with headways ≥ 2.0 s)

5 PEDESTRIANS

The average walking speed of pedestrians is 4 to 5 km/h, although the elderly often walk much slower. (Taylor, et al, Point 96)

The speeds in this statement correspond to 1.1 to 1.4 m/s. Recent surveys at mid-block signalised pedestrian crossings in Melbourne (Hill and Seidel 2000; Akcelik and Associates 2000b) show that *Point 96* is an acceptable statement considering pedestrians (see *Table 5.1* and *Figure 5.1*). A comparison of the results of Melbourne survey with published data is given by Hill and Seidel (2000).

The Melbourne study considered "pedestrians with walking difficulty" irrespective of their age. This group included elderly persons, people with physical disability, and parent pushing a pram and paying attention to a young child walking alongside. It is interesting to note that, as seen form *Table 5.1* and *Figure 5.1*, the 15th percentile speed has the following characteristics:

(i) for pedestrians with walking difficulty, it is very close to the design speed of 1.0 m/s recommended by AUSTROADS (1993, 1995) for accommodating slow pedestrians, and

(ii) for all pedestrians (including pedestrians with walking difficulty), it is very close to the general design speeds of 1.2 m/s recommended by AUSTROADS (1993, 1995).

This indicates that the use of the *15th percentile speed* for all pedestrians would be an appropriate crossing speed for signal timing purposes.

A useful rule based on the Melbourne study results is that the *15th percentile* crossing speed can be determined as 85 % of the *average* crossing speed, e.g. 0.85 x 1.4 m/s = 1.2 m/s.

Table 5.1

Crossing speeds of pedestrians with and without walking difficulty (overall speed across entire crossing for all sites combined, m/s)

	Average speed	Standard deviation	15th percentile	30th percentile	50th percentile	70th percentile	85th percentile
Pedestrians with walking difficulty	1.29	0.28	1.00	1.15	1.31	1.41	1.52
Pedestrians without walking difficulty	1.45	0.22	1.23	1.34	1.44	1.54	1.66
All pedestrians	1.42	0.24	1.18	1.31	1.42	1.52	1.65

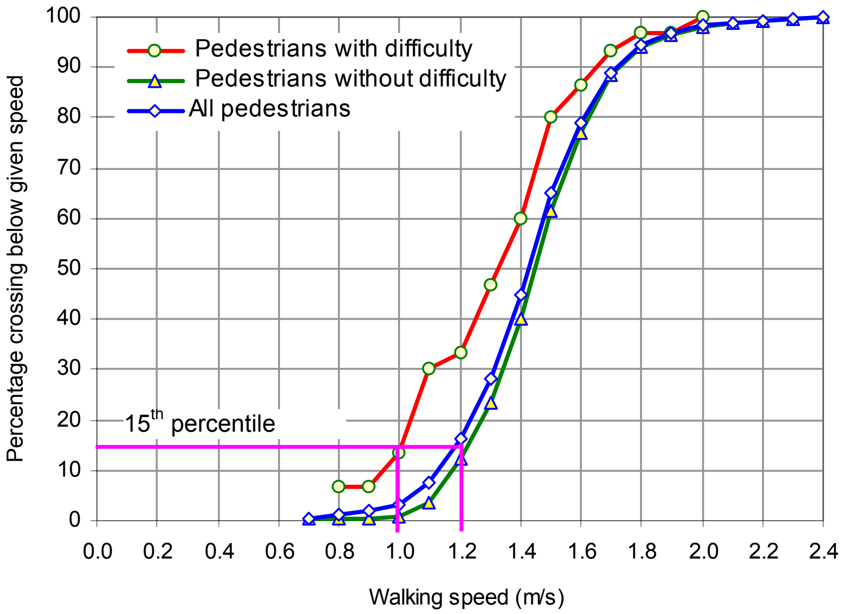

Figure 5.1 - Crossing speeds for pedestrians with and without crossing difficulties, and for all pedestrians (all sites combined)

6 SIGNALISED INTERSECTIONS

The lost time for a signalised intersection can be initially estimated as 5 s/phase. (Taylor, et al, Point 118)

The lost time for a traffic movement can be calculated as "the starting intergreen time less start loss plus end gain time" (Akçelik 1981). Analysis in Akçelik, Besley and Roper (1999) shows that start loss and end gain values depend on the method used to define and measure the saturation flow rate. Using the standard methods (Akçelik 1981, TRB 1998), the maximum queue discharge rates given in *Table 2.2* can be taken as saturation flow rates, and the start loss and end gain values found for the data given in *Table 2.2* (average values) are as follows:

Through traffic movements: start loss = 2.4 s, end gain = 2.6 s
Right-turn movements (arrow-controlled): start loss = 1.7 s, end gain = 2.7 s

Assuming an intergreen time of 6 s (4 s yellow plus 2 s all-red), which is now a more common practice in Australia, the lost times are found as:

Through traffic movements: 5.8 s
Right-turn movements (arrow-controlled): 5.0 s

Furthermore, the total lost time for the intersection is the sum of critical movement lost times (not phase times), and does not include phase intergreens if a critical movement runs in several phases continuously. Therefore, *Point 118* could be restated as:

> " The lost time for a signalised intersection can be estimated as sum of critical movement lost times using 6 s for through movements and 5 s for right-turn movements (arrow-controlled). "

A signalised intersection is uncongested if the degree of saturation is below 0.8, congested for values about 0.8 and saturated for values about 0.95. (Taylor, et al, Point 120)

This point uses the *practical capacity* measure to define *congestion*. The term congestion is given many different meanings by different professionals, and by users of the road system. It is clearer to use the term *oversaturation* instead of congestion for technical purposes, define it on the basis of full capacity, and use practical capacity to define a simple performance target as it is intended for. The aaSIDRA system of different practical degrees of saturation for different intersection types is appropriate (Akcelik & Associates 2000a). Therefore it is suggested that *Point 120* is restated as:

> " A signalised intersection is *undersaturated* if the degree of saturation (the ratio of demand flow to capacity) is below 1.0, and *oversaturated* if the degree of saturation is above 1.0. The *practical degree of saturation* can be used as a simple method to define an acceptable performance target, and allow for a safety margin for capacity estimation errors. Appropriate practical degrees of saturation for different intersection types are 0.90 for signalised intersections, 0.85 for roundabouts and 0.80 for sign-controlled intersections. Smaller values are chosen for unsignalised intersections due to higher levels of variability in traffic performance, and higher rate of deterioration as the demand flows approach capacity at these intersection types. "

A signalised intersection is at capacity (and saturated) when through movement queues consistently fail to clear. (Taylor, et al, Point 121)

This point is generally applicable to all intersection types (interrupted flow conditions), and is helpful towards a simple method for measuring capacity. This point also relates to the problem of volume counts at the stop line representing the capacity not the demand flow rates. In view of this, *Point 121* could be stated as:

> " The demand flow rate at an intersection lane is at or above capacity (saturated) when the queues consistently fail to clear. The capacity can be measured by simply counting the number of vehicles departing from the queue under such saturated conditions. Therefore, volume counts at the stop line fail to measure demand flow rates, which should be determined by counting traffic arriving at the back of the queue. "

The nth vehicle in a queue of through vehicles at a signalised intersection normally starts to move n seconds after the start of green. It follows from this and from the fact that vehicles pass through the intersection at approximately 2-second headway that the last vehicle to leave in a saturated phase is that one which starts to move half-way through the green time. (Taylor, et al, Point 123)

This point can be explained with the help of *Figure 6.1* based on Akçelik, Besley and Roper (1999). The statement *"The nth vehicle in a queue of through vehicles at a signalised intersection normally starts to move n seconds after the start of green."* suggests a queue departure response time of t_x = 1.0 s, and assumes that the reaction time for the first vehicle is the same as the queue response time ($t_r = t_x$). Data given in *Table 2.2* shows that t_x values are in the range 0.8 to 1.4 s for through and right-turn traffic lanes, and the average value for through traffic lanes is t_x = 1.15 s, which is close to the assumption made in *Point 123*.

Point 123 further assumes a saturation headway of h_n = 2.0 s corresponding to a saturation flow rate of 1800 veh/h whereas the average saturation headway for through lanes in *Table 2.2* is h_n = 1.75 s representing larger saturation flows (over 2000 veh/h) observed in more recent times).

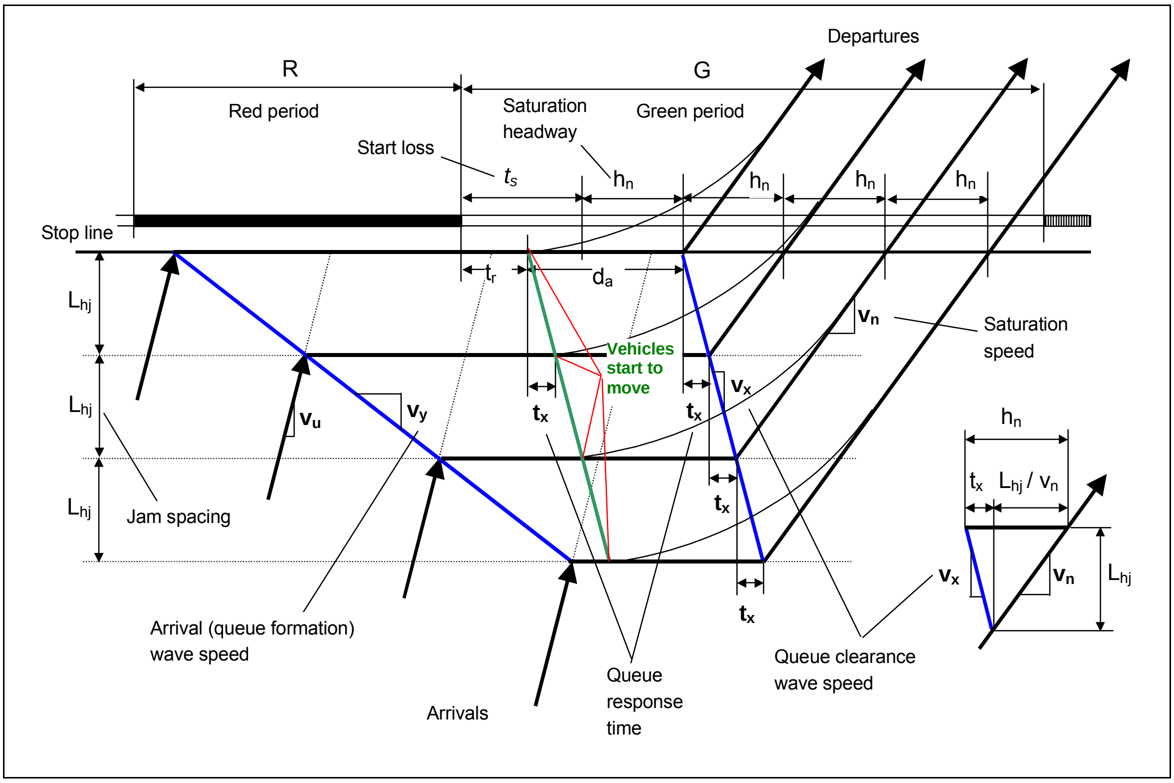

Figure 6.1 - Arrival and departure characteristics at signalised intersections

Ignoring departures during the yellow time, the last vehicle (say Nth vehicle) in a saturated phase with a green time of G seconds will leave after G = t_s + N h_n seconds, where t_s is the start loss. Thus, N = (G - t_s) / h_n vehicles will depart during G. According to *Point 123*, Nth vehicle starts moving at time G / 2. Based on the discussion at the start of this section, let us use a start loss value of t_s =2.4 s, and using h_n = 2 s, N = (G - 2.4) / 2 = 0.5 G - 1.2 vehicles. Using t_x = 1.0 s, Nth vehicle would start moving at time N t_x = N = 0.5 G - 1.2 seconds since the start of green period.

For example, for G = 50 s, N = 24 vehicles, and the 24th vehicle would start to move at time 24 s since the start of green period (approximately half-way).

However, using the data given in *Table 2.2* for through traffic lanes (h_n = 1.75 s and t_x = 1.15 s), N = (G - 2.4) / 1.75 = 0.57 G - 1.4 vehicles would depart during a fully saturated green period, and the Nth (last) vehicle that departs during this period would start moving at time N t_x = 1.15 N = 0.66 G - 1.6 seconds since the start of green period. For the same example as above, for G = 50 s, N = 28 vehicles, and the 28th vehicle would start to move at time 31 s since the start of green period (6 s later than half-way).

Using the data given in *Table 2.2* for arrow-controlled right-turn movements (h_n = 1.77 s and t_x = 0.84 s), and the start loss value suggested at the start of this section (t_s = 1.7 s), N = (G - 1.7) / 1.77 = 0.56 G - 1.0 vehicles would depart during a fully saturated green period, and the Nth (last) vehicle that depart during this period would start moving at time N t_x = 0.84 N = 0.53 G - 0.8 seconds since the start of green period. For example, for G = 20 s, N = 10 vehicles, and the 10th vehicle would start to move at time 10 s since the start of green period (half-way). Thus, the rule is seen to apply to arrow-controlled right-turn movements as well.

It can be concluded that, *Point 123* is an acceptable general rule. It is suggested that it is stated as:

> " The nth vehicle in a queue of through or arrow-controlled right-turn vehicles at a signalised intersection starts to move approximately n seconds after the start of green. It follows from this and from the fact that vehicles pass through the intersection at approximately 2-second headway that the last vehicle to leave in a saturated green period is that one which starts to move approximately half-way through the green time. "

7 CONCLUDING REMARKS

The analyses presented in this paper show that most points considered are generally valid as approximations (as they are intended to be), but it would be useful to revise most points as suggested in this paper. One observation about the list given by Taylor, Bennett and Ogden (1996) is that the Environmental Considerations section refers to "environmental capacity" but does not discuss the subject of pollutant emissions by vehicle traffic, including greenhouse gas emissions.

In addition to those analysed, the following points are suggested for consideration.

Intersections

- " For signalised and unsignalised intersection lanes, the *cycle-average queue length*, which includes the intervals with no queued vehicles, can be calculated as the total flow rate times the average delay to all vehicles queued and unqueued. The *average back of queue*, which represents the maximum reach of the queue in an average cycle (signal cycle or gap cycle) is roughly more than twice the cycle-average queue length. "

Roundabouts

- " A four-way single-lane roundabout with 4-metre lanes, an inscribed diameter of 40 m, and equal entry flows on all approaches assuming cars only with 20 per cent left-turn, 60 per cent through and 20 per cent right-turn flow can carry about 2600 veh/h at 85 per cent of full capacity (*practical capacity*). Under similar assumptions, a two-lane roundabout with an inscribed diameter of 60 m can carry about 4400 veh/h, and a three-lane roundabout with an inscribed diameter of 80 m can carry about 6000 veh/h. "

- " *Unbalanced demand flows* cause reduced capacities at roundabouts, and may cause excessive queueing at the entry lane affected by them (often alleviated using metering signals). For example, an entry lane against a circulating flow rate of 1000 veh/h, which consists of 900 veh/h from Approach A and 100 veh/h from Approach B, has significantly less capacity than the same entry lane against the same circulating flow rate (1000 veh/h), which consists of 500 veh/h from Approach A and 500 veh/h from Approach B. "

Actuated Signals

The following interrelated points are suggested for actuated signals. These points are emphasised in aaSIDRA training workshops (Akcelik & Associates 2000a). Also see Akçelik (1997) on the effect of short lanes on signal cycle time.

- " Avoid the use of large *maximum green settings*. Always remember that *someone's green is someone else's red*. By extending the green time for a movement, you are extending the red time, therefore queue length, for all conflicting movements. In turn, these movements will need more green time to clear the queues. This will result in long cycle times, with performance deterioration (longer delays, longer queues) for all movements. Therefore, it is possible to improve the intersection performance by reducing the maximum green setting. "

- " The missing link in signal design can be found between the traffic design engineer, who designs the intersection geometry based on optimum cycle time assuming fixed-time signals (often a short cycle time), and the signal operations engineer, who sets actuated controller settings based on rules that have no relation to fixed-time signal optimisation (usually leading to long cycle times).

- " Contrary to the traditional traffic signal teaching, the capacity does not always keep increasing with increased cycle time. It is likely to decrease especially in the cases of short lanes (flares), filter (permitted) turns and shared lane blockages. In such cases, a larger number of signal cycles per hour (therefore a shorter cycle time) will result in substantial benefits in capacity and performance of the intersection. "

- " Pedestrians always benefit from short cycle times. An all-pedestrian phase will often increase the cycle time, and therefore increase the delays to vehicles as well as pedestrians. "

Environmental Considerations

- " Typical carbon dioxide (CO_2) gas emission rate for a car is 2.5 kg per litre of fuel used. Therefore, a car that has an average fuel consumption rate of 8 L/100 km, and travels 20,000 km per year, contributes 4 tonnes of CO_2 per year to the greenhouse gas emissions. One million such vehicles, representing 20,000 million vehicle-kilometres of travel per year, contribute 4 million tonnes of CO_2 emission per year. "

… and finally:

Microscopic Simulation

- " Seeing is believing, but do not always believe what you see in sophisticated animations. Question the methodology and data used in microscopic simulation as you would question the analytical models, since all models are built on assumptions that simplify the complex real-life traffic behaviour. "

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