PEDESTRIANS' AND CYCLISTS' EFFECT ON THE CAPACITY OF THE RIGHT TURN MOVEMENT AT SIGNALISED INTERSECTIONS: AN EMPIRICAL PILOT STUDY

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General Background

The use of bicycles as a means of transport in journey-to-work trips is widespread in North West Europe. In countries such as The Netherlands, Germany and Denmark there is a tradition for research concerning bicycle traffic.

For reasons of safety and convenience special cycle tracks alongside vehicle lanes are often provided where possible. Where such tracks exist through signalized intersections cyclists create problems for right turning vehicles in the same manner as do pedestrians using a pedestrian crossing. Cyclists are often much more numerous than pedestrians along main arterials. Therefore, there is a need for a method for estimating the effect on capacity of the volume of cycle traffic. Only a few methods exist (e.g. Tepley (1990)).

Scope of this study.

The idea of this study was to carry out field observations at a small number of signalised intersections, from these observations firstly to estimate a regression model of delays as a function of flows, secondly to calibrate a simulation model and finally to use the simulation model to obtain an overview of the problem through a large number of - simulated - observations.

Four signalised intersection were selected in the central part of the Copenhagen region. At each intersection one approach was chosen for observations. In the approaches the lanes under study are exclusive right turning lanes. The geometric designs were of the general type shown in fig 1. Turning cars must give way to both cyclists and pedestrians.

In the actual cases the two crossings are moved a few meters away from the intersection in order to provide some waiting positions for the right turning cars. In some cases the cycle track is carried round the bend for right turning cyclists.

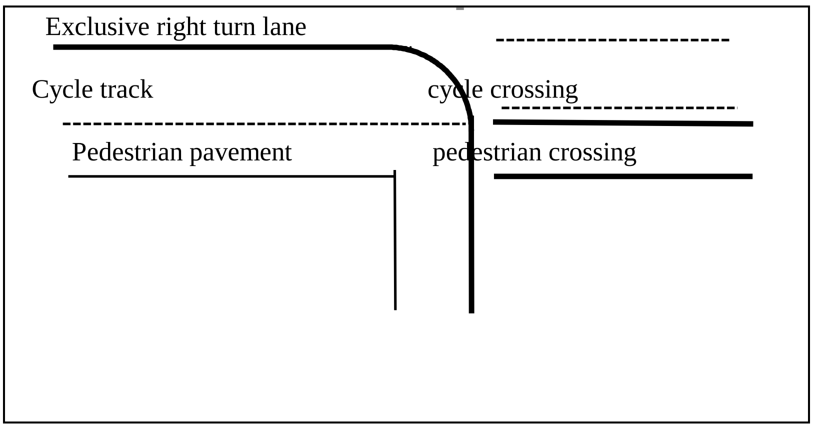

Figure. 1: Right turn arrangement at intersections under study

The idea was then to observe the right turning car flow related to the flows of cyclists and pedestrians. When the signal turns green the right turning car traffic must give way to the group of cyclists and pedestrians who have been accumulated during the red phase, here called “the red phase group”.

When “the red phase group” has passed there will still be a presumably random flow of cyclists and pedestrians for which the car drivers must give way.

Finally, in order to model right turn capacity it is necessary to have estimates of time headways in an uninterrupted car flow. This may also be measured directly in the field. However, data for cases with uninterrupted flow are rather limited at these intersections.
In short: In this project the purpose is

1. to develop a model for the delay of the first turning vehicle caused by “the red phase group”

2. to develop a model for the discharge rate of a queue of motor vehicles in the right turn lane after “the red phase group” has passed as a function of the cyclist and pedestrian flows

3. to develop an estimate of the effects of cyclists and pedestrians on the right turning car capacity at signalised intersections.

The simulation model used.

The creation of a microscopic traffic simulation model is a complicated task. A program developed in the Netherlands - FLEXSYT - has been found useful. The owner of the program - The Dutch Ministry of Transport - kindly offers the program free of charge for research purposes.

In this application of the program the necessary input is

a fairly detailed description of the geometric design
flow data for cars, cyclists and pedestrians
traffic signal settings
critical time gap for unsignalised conflicts - only the car vs. cyclists and pedestrians conflicts in this case
the location of detectors for signals or for counts and measurements

The output of the program is normally the average time spent in the system between certain detectors, average delays etc. Also flows in and out of the system are reported.

Analytical approach used to model the effects of light traffic on right turn capacity for cars

The traffic flow in a typical urban signalised intersection in Denmark comprises not only motor traffic but also light traffic such as pedestrians and cyclist. When turning motor traffic and conflicting pedestrian and cyclist flows have the same green period, the tuning motor traffic is by Danish law obliged to give way to the conflicting pedestrian and cyclist flows. The presence of the conflicting pedestrian and cyclist flows thus slow down the turn movement and thus reduce the capacity of the turning vehicle movement in the intersection.

Until now there does not exist any methods in the Danish Guidelines which enables the practitioner to evaluate the pedestrians and cyclists effect on the capacity of the turning movements in signalised intersections. The aim of this research is to set up a preliminary method to that end. However, the focus is confined to a method which addresses the conflicting light traffic’s capacity reducing effect on the right turn movement under non saturated conditions in urban signalised intersections.

A non saturated condition is in this research defined as a condition where it is possible for the light traffic, arriving in the light traffic green period and in the preceding light traffic red period, to depart in the light traffic green period. This confinement of the research is reasonable, since the magnitude of light traffic in Danish signalised intersections will very seldom (if at all) result in saturated or over saturated conditions. This is also believed to hold true for the traffic conditions in Danish signalised intersections in the for see able future.

For the model approach the following special parameters were used:

- *the green queue* (*GrQ*) is defined as the queue of right turning vehicles in the exclusive right turn lane at the start of this movement's green period,
- *the green lag* (*GrL*) is the time interval between the start of the conflicting light traffic's green period and the right turn movements green period. There are two green lags, the green lag between the right turn movement and conflicting pedestrians and the green lag between the right turn movement and the conflicting cyclists. The green lag is only defined for situations where the light traffic's green periods start before or simultaneously with the right turn movement's green period. In the former case it is positive and zero in the latter case.
- the *red phase group* is defined as the number cyclists and the pedestrians accumulated during the red period plus the cyclists and pedestrians who arrive at the intersection in the green period while the front vehicle of the green queue, vehicle 1, is waiting to depart.
- the *simultaneously green period* (*SGr*) is defined as the time after the departure of vehicle 1 where the conflicting light traffic and the right turn movement have green simultaneously.

A closer look at the departure of right turners from an exclusive right turn lane will reveal the following when the movement is hampered by conflicting light traffic:

1. When the signal for the right turn movement turns green vehicle 1 gives way to the red phase group. The departure of vehicle 1 is thus delayed and the extent of the delay depends on the size of the green lag and on the size of the red phase group.
2. When vehicle 1 has departed some or all of the following right turning vehicles in the green queue will depart in the right turn movement's remaining green period. Depending on the signal settings the right turn movement's remaining green period can be split up into two parts. The simultaneously green period and a part where only the right turn movement has green, that is a part where there is no conflicting light traffic. During the simultaneous green period the presence of conflicting light traffic will hamper the departure of the right turning vehicles.

If the there had not been any conflicting pedestrians or cyclists the capacity of the exclusive right turn lane could be calculated as:

$$n = \frac{Gr}{T_H\ O} \tag{1}$$

where:

n = the capacity of the exclusive right turn lane in passenger car equivalents per second.
T_H = the average headway between right turning passenger cars when not obstructed by conflicting pedestrians and cyclists.
Gr = the right turn movement's green period in seconds.
O = the signalised intersection's cycle time in seconds.

The presence of conflicting light traffic will delay vehicle 1 at the front of the green queue and hamper the departure of the right turning vehicles in the simultaneously green phase. One way of taking these effects into account would firstly be to regard the delay of vehicle 1 as an extension of the right turn movement's red period. This is reasonable since no right turning vehicles are able to depart while vehicle 1 is delayed. The green period Gr in equation (1) could thus be reduced by the average number of seconds, T_D, vehicle

1 is delayed. The average delay , T_D , will depend on the average size of the red phase group. The average size of the red phase group in the non saturated condition depends on:

the cyclist green lag;
the pedestrian green lag;
the length of cyclists' red period;
the length of the pedestrians' red period;
the arrival characteristics of the conflicting cyclists;
the arrival characteristics of the conflicting pedestrians; and
the interaction between the right turning vehicle and the conflicting light traffic.

Secondly, the effect of the conflicting light traffic on the car departure rate during the simultaneously green period, could be modelled by letting the size of the headway depend on the arrival characteristics of the conflicting pedestrians and cyclists.

So, if the remaining green period is identical to the simultaneously green period the presence of conflicting light traffic would change equation 1 to equation 2 below.

$$n = \frac{Gr - T_D}{O\, T_H(\ )} \tag{2}$$

If the right turn movement's green period extents beyond the simultaneously green period the capacity of the exclusive right turn lane could be calculated as:

$$n = \frac{1}{O}\left(\frac{Gr - T_D - SG}{T_H} + \frac{SGr}{T_H(\ )}\right) \tag{3}$$

where:

n	= the capacity of the exclusive right turn lane in passenger car units per second.
T_H	= the average headway between right turning passenger cars when no light traffic is present (sec/pcu).
$T_H(\ )$	= the average headway between right turning passenger cars when the right turn is hampered by conflicting light traffic.
Gr	= the right turn movement's green period in seconds.
SGr	= the simultaneously green period in seconds.
T_D	= the average delay of vehicle 1.
O	= the signalised intersection's cycle time in seconds.

In the next section it is described how the observations necessary for the modelling of T_D and $T_H(\ )$ have been collected.

Collection of empirical data

The approach used by Tepley (1990) can be regarded as a so called empirical approach. What data to collect depends to a large extent on whether the approach is an empirical or a theoretical one. In this research it was chosen to use an empirical approach in setting up the models of T_D and $T_H(\ )$. The empirical approach was chosen in an attempt to circumvent the inherent complexities in the modelling of the interaction between right turning vehicles and conflicting light traffic.

In this approach the data collection procedure consisted of videotaping the interaction between right turning vehicles and conflicting light traffic in a selected number of urban signalised intersections. The following signalised intersections where chosen for the analysis:

Dronning Louises Bro / Frederiksborggade, where the exclusive right turn lane on the approach on Dronning Louises Bro was studied.
Smallegade / Gl. Kongevej/Falkonér Allé, where the exclusive right turn lane on the Falkonér Allé approach was studied.
Jagtvej / Ågade / Falkonér Allé, where the exclusive right turn lane on the Falkonér Allé approach was studied
Nørrebrogade / Jagtvej, where the exclusive right turn lane on the southern approach of Nørrebrogade was studied.

From each of these signalised intersections 4 hours of videotaping were recorded. Depending on the sizes of the flows of conflicting light traffic and right turn car traffic the 4 hours contained either the morning period (that is, the morning peak hour and the hours before and after it) or the afternoon period.

Collecting data on vehicle 1's delay
In order to collect data to model T_D it was first necessary to define which moments in time started and terminated the delay endured by vehicle 1. In this research the start of the delay was defined as the moment the right turn movement's signal turned green. The termination of the delay was defined as the moment where the front of vehicle 1 crossed the boundary line *L* illustrated in figure 2. The boundary line was only marked on the monitor screen and not physically in any of the 4 intersections.

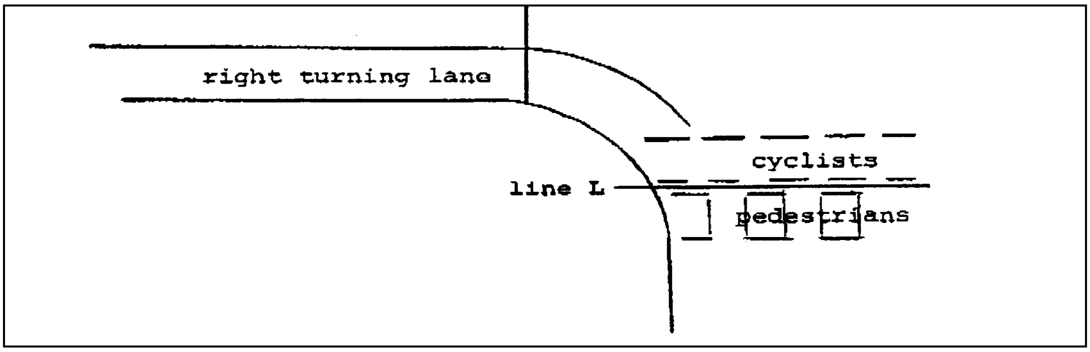

Figure 2. The boundary line L determining the end of vehicle 1's delay.

In each intersection the boundary line was located such that once the front of vehicle 1 had crossed the line the vehicle would complete its right turn and not halt to let conflicting light traffic pass it. The location of the fictive boundary line thus varied from intersection to intersection due to the different geometric lay outs of the intersections.

The procedure above, to determine vehicle 1's delay automatically, introduces so called geometric delay into the delay measurement. This geometric delay will be present irrespective of whether vehicle 1 is delayed by conflicting light traffic or not.

It is only the delay caused by the conflicting light traffic which is of interest, not the delay generated by the geometric layout of the intersection. Therefore, the delay measured by the outlined procedure had to be adjusted downwards with an amount equal to the geometric delay. The average, T_G, of the geometric delays measured in the intersection was used to adjust downwards the delay measurements stemming from that particular intersection.

So, for each intersection the following observations were made:

1. the time interval, t_{V1}, from the right turn signal turns green to the moment where the front of vehicle 1 crossed the boundary line. The time interval was only measured in cycles where there, at the start of green period, was a green queue present and a red phase group to delay vehicle 1.

2. the geometric delay. The geometric delay was only measured in cycles where there was a green queue present at the start of the right turn green period, but no pedestrians or cyclists to delay the departure of vehicle 1.

3. the number of cyclist and pedestrian arrivals in each of the cycles mentioned above. The pedestrians and cyclists counted were pedestrians using the zebra crossing and the cyclists using the cyclist path to the right of the exclusive right turn lane. The pedestrians counted on the zebra crossing walked in both directions. Cyclist who got off their bike at the intersection were counted as pedestrians.

Collecting data on the light traffic's effect on the queue headways
the following observations were made in order to model the conflicting light traffic's effect on the average headway T_H:

1. the headways between right turning passenger cars from the green queue whose departure in the green period were not delayed. These measurements were made in order to assess the headway, TH, between non impeded right turning passenger cars.

   Only green queues without heavy vehicles were considered applicable for the measurements. The heavy vehicles are taken into account by means of evaluating flows in pcu's.

2. the time interval, tVH, between vehicle 1's crossing of the boundary line and the crossing of the boundary line of the last passenger car in the green queue to depart in the studied green period.

   The number of passenger cars from the green queue departed in the studied green period.

Data on light traffic arrivals and summaries of collected data
All measurements mentioned above were not necessarily made in consecutive cycles. Whether data was retrieved from a given cycle depended on whether there was a green queue present in the cycle or not.

For the model approach it is necessary also to have an idea of the pedestrians' and cyclists' arrival distributions. Why this is so is discussed later. To use light traffic arrivals from non consecutive cycles as the basis for deriving the arrival distributions of the light traffic may result in incorrect distributions. Therefore, in order to get a more correct picture of the arrival distributions, pedestrian and cyclist arrivals were registered in each consecutive cycle in the peak hour of each intersection. These observations from consecutive cycles were used to derive the cyclist and pedestrian arrival distributions.

The table below provides a short summary of the signal settings, traffic flows and the number of observations retrieved from each studied intersection.

Table 1: Signal settings	O (sec.)	R_c (sec.)	GrL_c (sec.)	R_p (sec.)	GrL_p (sec.)
Nørrebrogade / Jagtvej	60	38	2	38	2
	80	48	2	48	2
Dronning Louises Bro/ Frederiksborggade.	60	44	0	45	0
	80	64	2	61	6
Jagtvej/ Ågade	60	42	0	38	0
	100	74	0	70	0
Smallegade/ Gl. Kongevej	60	40	0	44	4
Total					

Table 2: Observations	*O (sec.)*	*Number of obs. on T_G*	*T_G (sec.)*	*Number of obs. on T_H*	*T_H (sec.)*	*Number of obs. on T_D*
Nørrebrogade / Jagtvej	60	?	1.9	98	2.5	73
	80					37
Dronning Louises Bro/ Frederiksborggade.	60	108	1.9	360	1.9	54
	80					54
Jagtvej/ Ågade	60	30	2.5	58	2.7	32
	100					50
Smallegade/ Gl. Kongevej	60	40	2.4	71	2.5	40
Total / Over all average				587	2.2	340

The measured average geometric delays in the 4 intersections were 1.9 seconds at the Nørrebrogade/Jagtvej intersection, 1.9 seconds at the Dronning Louises Bro/Frederiksborggade intersection, 2.5 seconds at the Jagtvej/Ågade intersection and finally 2.4 seconds at the Smallegade/Gl. Kongevej intersection. The average of the measured headway not affected by conflicting light traffic was measured.

The data analysis

<u>The model of vehicle 1's delay</u>.

The delay of vehicle 1 depends on the size of the red phase group. The size of the red phase group depends on the conflicting pedestrian and cyclist arrival rates, that is C and P, the pedestrian and cyclist green lags GrL_p and GrL_c and the pedestrian and cyclist red periods R_P and R_C. The basic structure of the model used to calculate vehicle 1's delay should contain the mentioned variables, that is TD should be expressed as:

$$T_D = \mathrm{F}(GrL_p, GrL_c, R_p, R_c, C, P) \tag{4}$$

Apart from the mentioned variables the model should also have the property that T_D is zero when C and P are zero. As a consequence it was decided to use the following model structure in the regression analysis:

$$T_D = (R_c - b_1 GrL_c)\left[\sum_{i=1}^{m} k_i C^i\right] + (R_p - b_2 GrL_p)\left[\sum_{i=1}^{q} l_i P^i\right] + sPC \tag{5}$$

b_1, b_2, k_i, l_i and s in equation (5) are constants with values found through the regression analysis. Likewise, the regression analysis determines m and q.

The idea behind the basic structure in eq. (5) is that vehicle 1's delay is triggered by the pedestrians and cyclists accumulated during the light traffic red phase period and the following green lag. The accumulated number of cyclists can be calculated as the number of cyclist arrivals in the cyclist red period and green lag minus the number of cyclist departures in the cyclist green lag. Likewise for the number of accumulated pedestrians. The accumulated number of cyclists can be expressed analytically as:

$$\text{Accum. cyclists} = (R_c + GrL_c)\cdot C - \text{const.}\cdot GrL_c \cdot C = (R_c - b_1 GrL_c)\cdot C \tag{6}$$

The accumulated cyclists (eq. (6)) cause delay to vehicle 1 which, for reasons of simplicity, could be represented by multiplying the number of accumulated cyclists with a factor k1. During this delay

$a_1(R_c+b_1GrL_c)C$, where the accumulated cyclists depart, new cyclists arrive at the intersection. The average number of arriving new cyclists could be calculated as $k_1\,(Rc+b_1GrL_c)C\cdot C$, and the delay they cause on vehicle 1 could again be represented by a factor times $k_1(R_c+b1GrL_c)C\cdot C$. That is, the extra delay $k_2\,(R_c+b_1GrL_c)C^2$. If the argument is continued then during the delay $k_2\,(R_c+b_1GrL_c)C^2$ new cyclists will arrive and they again cause extra delay which could represented by $k3\,(R_c+b_1GrL_c)C^3$ and so on. To assess the delay caused by the arriving cyclists it is now only necessary to add up the cyclists' delay contributions just outlined. This results in the summation expression for the delay caused by cyclists only in eq. (5). The summation expression for the pedestrians' separate contribution to vehicle 1's delay in eq. (5) is based on the same reasoning as the one just described for the cyclists.

Clearly, the idea behind the model structure is a simplification of reality. In the reality vehicle 1 will at some stage either find a time gap between arriving cyclists (and pedestrians) in which to depart, or force its right turn movement through the conflicting flow of cyclists (and pedestrians). The end of the process is represented by m and q in the equation which again are determined by the regression analysis.

Since the pedestrians and cyclists typically depart simultaneously it could be expected that the model should also contain an element which depends on both the cyclist and the pedestrian arrival rates. This is the reason for the presence of the last term sPC in eq. (5). Intuition suggests that s would be negative because while vehicle 1 is waiting for slow pedestrians to pass fast cyclists could pass as well without causing vehicle 1 extra delay. It was found, however, that the interaction between the cyclists and the pedestrians had, apparently, no significant effect on vehicle 1's delay.

Between a number of candidate models the model below was selected for the practical estimation of vehicle 1's delay. The model was selected because of its relative simplicity and because it was one of the models which provides the best fit to the data; that is the model explains some 51% of the variance in the data.

The analytical expression of this model is:

$$T_D(C_O,P_O) = (R_c - 6.6GrL_c)\left(2.6C_O - 10.4C_O^{\,2} + 13.5C_O^{\,3}\right) + 0.9(R_p - 2.9GrL_p)P_O \qquad (7)$$

C_O = the actual cyclist arrival rate in cycle O (cyclist/second).
P_O = the actual pedestrian arrival rate in cycle O (cyclist/second).
R_c = the cyclist red period in seconds.
R_p = the pedestrian red period in seconds.
GrL_c = the cyclist green lag in seconds.
GrL_p = the pedestrian green lag in seconds.

Regression models work to some extent like a "black box". One is therefore well advised in being careful with the interpretation of the estimated parameters in the models. However, it is interesting to note that b_1 (=6.6) is larger than b_2 (= 2.9) in eq. 7. This suggests that the green lag is more effective in decreasing the cyclists' effect on vehicle 1's delay than decreasing the effect due to the pedestrians. An explanation for this relationship may be that pedestrians typically cross the zebra crossing, and thus the vehicle right turn path, in both directions while the cyclists typically only crosses the vehicle right turn path in one direction. Because of pedestrians crossing from the far side of the zebra crossing fewer pedestrians are free of the vehicle right turn path when the right turn signal turns green. This renders the pedestrian green lag less effective in reducing vehicle 1's delay compared to a situation where all pedestrians (like the cyclists) crosses the vehicle right turn path from the near side of the zebra crossing. The near side of the zebra crossing is the side closest to the right turning vehicles path of way.

Practitioners will normally only have an idea of the average cyclist and pedestrian arrival rates C and P at the signalised intersection under evaluation. It is therefor necessary to adjust eq. (7) accordingly so that T_D becomes a function of the average light traffic arrival rates and not the actual light traffic arrival rates C_O and P_O in a cycle.

Adjustment with regard to the light traffic's arrival distributions

The transformation of eq. (7) can be done by use of the pedestrian and cyclist arrival distributions. C_O and P_O can be written as C_O*/O and P_O*/O where C_O* and P_O* are the actual number of cyclist and pedestrian arrivals in a cycle of a length of O seconds. If $\mathrm{P_c}\{\}$ is the distribution of the number of cyclist arrivals in a cycle and $\mathrm{P_p}\{\}$ is the distribution of the number of pedestrian arrivals in a cycle then the transformation of eq. (7) will look like:

$$T_D(\mathrm{C,P}) = \sum_{i=0}^{\infty} \mathrm{P_c}\{\mathrm{C}_O{}^* = \mathrm{i}\}(R_c - 6.6GrL_c)\left(\frac{2.6C_O{}^*}{O} - \frac{10.4(C_O{}^*)^2}{O^2} + \frac{13.5(C_O{}^*)^3}{O^3}\right) + \sum_{\mathrm{j}=0}^{\infty} \frac{\mathrm{P_p}\{P_O{}^* = \mathrm{j}\}\ 0.9(R_p - 2.9GrL_p)P_O{}^*}{O} \qquad (8)$$

$$= (R_c - 6.6GrL_c)\ (2.6C - 10.4\mathrm{E}\{(C_O{}^*)^2\}/O^2 + 13.5\mathrm{E}\{(C_O{}^*)^3\}/O^3\ ) + 0.9(R_p - 2.9GrL_p)P$$

The calculation of $\mathrm{E}\{(C_O*)^2\}$ and $\mathrm{E}\{(C_O*)^3\}$ in eq. (8) depends on the type of distribution characterising the number of cyclist arrivals in a cycle. Analysis of the collected data on the number of cyclist arrivals in the cycle have shown that the cyclist arrivals follow a Negative Binomial distribution with variances between 1.02 and 4 times the mean. The Negative Binomial distribution gives rise to the problem that practitioners normally do not know the variance of the number of cyclist arrivals per cycle. They would therefore not be able to calculate $\mathrm{E}\{(C_O*)^2\}$ and $\mathrm{E}\{(C_O*)^3\}$ directly. One way of circumventing this problem would be to approximate the Negative Binomial distribution with a Poisson distribution where the mean is identical to the variance. Under the present circumstances it is believed that the Poisson approximation is an acceptable solution to the problem. This belief is mainly due to:

- At present it is not known whether the Negative Binomial distribution, with variances between 1.02 and 4 times the mean, is a good representative of the cyclist arrival distributions in the majority of urban signalised intersections. The 1.02 suggests that at some Danish signalised intersections the arrival distribution will be very close to a Poisson distribution.
- Eq. (7) is "only" able to explain 51% of the variance in the data. This, apparently, leaves room for larger discrepancies between actual observed average delay and the average delay estimated with eq. (8) even when the true cyclist arrival distribution at the intersection is used in the calculation.

In the Poisson distribution $\mathrm{E}\{(C_O*)^2\}$ is equal to $\mathrm{E}\{C_O*\}$ (1+ $\mathrm{E}\{C_O*\}$) and $\mathrm{E}\{(C_O*)^3\}$ is equal to $\mathrm{E}\{C_O*\}$ ($\mathrm{E}^2\{C_O*\}$+3 $\mathrm{E}\{C_O*\}$ + 1). Inserting these two expressions in eq. (9) leads to:

$$T_D = (R_c - 6.6GrL_c)\ (2.6C - 10.4C^2 + 13.5C^3 - 10.4C/O + 13.5\ (C/O^2 + C^2/O)) + 0.9(R_p - 2.9GrL_p)P \qquad (9)$$

C = the average cyclist arrival rate (cyclist/second).
P = the average pedestrian arrival rate (cyclist/second).
0 = the length of the cycle in seconds.
R_c = the cyclist red period in seconds.
R_p = the pedestrian red period in seconds.
GrL_c = the cyclist green lag in seconds.
GrL_p = the pedestrian green lag in seconds.

Figure 3 illustrates eq. (9)'s estimates on vehicle 1's average delay C is equal to P, $GrL_c = GrL_p = 0$ seconds , $R_c = R_p = 40$ seconds, and $O = 60$ seconds.

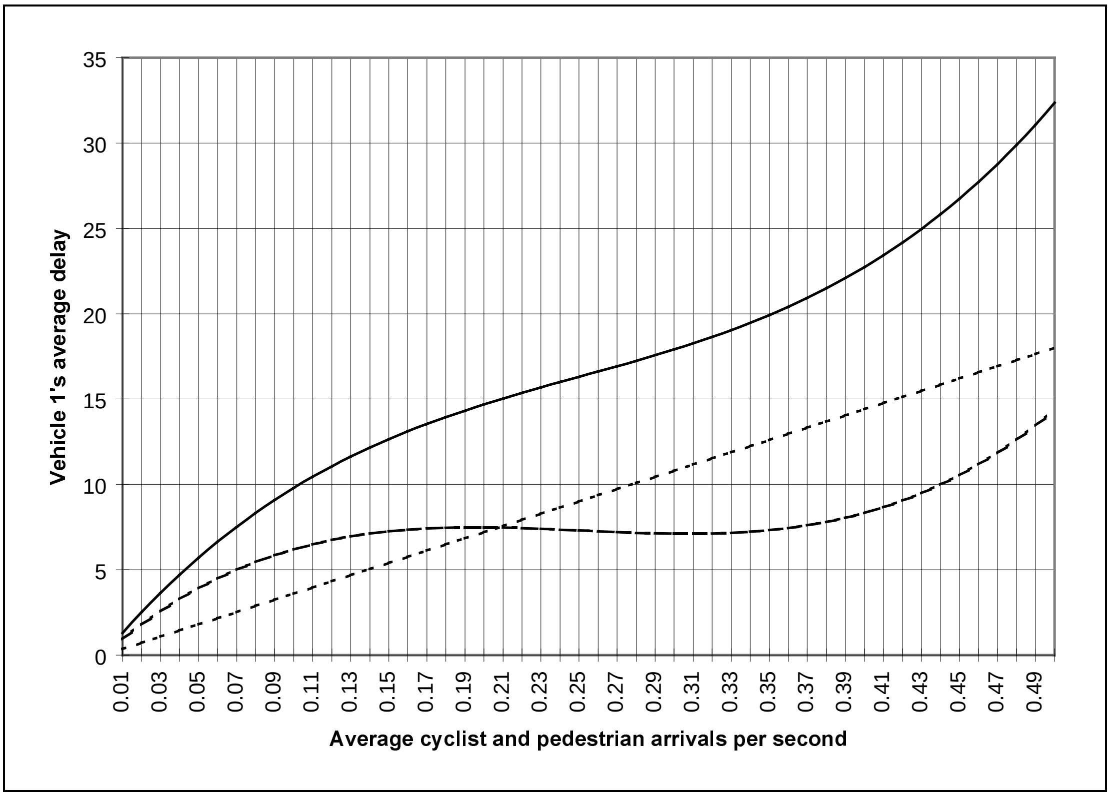

Figure 3. Estimated average delay with equation (9).

The non dotted line is vehicle 1's average delay estimated by eq. (9). The linear dotted line represents vehicle 1's average delay caused by pedestrians. The dotted curve in the figure represents the cyclists' contribution to vehicle 1's average delay.

Intuition suggests that the higher the cyclist arrival rate the higher is the average delay endured by vehicle 1. In figure 3 it seen that this is, apparently, not the case, since the cyclists' contribution to the average delay is more or less constant for arrival rates between 0.19 and 0.37 cyclist per second. A similar picture is obtained if eq. (7) is used to model vehicle 1's average delay as a function of the actual pedestrian and cyclists arrivals. So, the shape of the curve representing cyclists effect on the delay is not alone due to the Poisson approximation.

An explanation for the above result may be that drivers and cyclists change their driving and cycling behaviour as the cyclist arrival rate increases. The larger the cyclist arrival rate the more willing are drivers to depart in small time gaps between cyclists or to force their way through the cyclist (and pedestrian) flow. This change of behaviour in conjunction with increasing traffic flows has, amongst other sources, been reported in capacity studies of unsignalised intersections by Troutbeck (1990). In his research of the entry lane capacity at Australian roundabouts he found that drivers in the entry lane, giving way to circulating traffic, accepted smaller gaps between circulating vehicles as the flow in the circulation traffic increased.

The suggested change in the cyclist driver behaviour is hypothesised to be an increase in aggressive driving by the cyclist. That is, when cyclist arrival rates are high cyclists will be more inclined to use the full width of the cyclist path during the cyclist green period, thus increasing the cyclist discharge rate (the change from non aggressive to aggressive cyclist behaviour increases the cyclist discharge rate, measured in number of cyclist discharged per metre width of the cyclist path per second, from, say, 1 to1.5 or even 2, because cyclists make better use of the cyclist path). The shift from non aggressive to aggressive cyclist behaviour could result in more or less identical average delays for cyclist arrival rates within a certain range. For cyclist arrival rates well below the range the cyclists are non aggressive and delay increases with increasing cyclist arrival rates. For arrival rates above the range the average delay increases with increasing arrival rates because the sheer size of the arrival rates outweighs the effect of the cyclists' aggressive driving.

If the change of behaviour is the explanation then one could ask why a similar structure has not been observed in the pedestrians' contribution to vehicle 1's average delay ? At present it is difficult to answer that question. There is, of course, the possibility that the reason why is the model's inadequacy to model the pedestrians' effect on the average delay correctly. Only further research will be able to answer that question. Further research will also be able to elucidate whether the model provides an adequate description of cyclists' effect in general on vehicle 1's delay at urban signalised intersections.

Modelling the light traffic's effect on the headways

To elucidate the pedestrians' and cyclists' effect on the headway between departing vehicles from the green queue it was decided do let the effect be represented by a factor which is to be multiplied with the non disturbed average headway $T_{H.}$ That is, in the regression analysis the following model was used to find the model which best fitted the data:

$$t_{VH} = y\ T_H\ F(C_O, P_O) \tag{10}$$

t_{VH} = the time interval giving evidence on the cyclists' and pedestrians effect on the headway.

y = the number of passenger cars from the green queue which departed during the t_{VH} seconds of the green period.

T_H = the average headway between right turning passenger cars not hampered by conflicting light traffic during the right turn. Based on data from the 4 studied intersection T_H is 2.15 seconds.

$F(C_O,P_O)$ in eq. (10) is the factor which should, in theory, depend on the actual pedestrian and cyclist arrival rates in the cycle. $F(C_O,P_O)$ was modelled such that when C_O and P_O are zero $F(C_O,P_O)$ is 1.0. With this property models were set up which at maximum could explain no more than 30% of the variation in the data. One reason for this poor result could be the mentioned model constraint ($F(0,0)$ = 1.0). Table 3 shows the models which provide the best fit when this constraint is relaxed, i.e. the regression analysis determines the value of $F(0,0)$.

Table 3. Fitted models [$F(0,0)$ = constant] :	Explained variance
$T_H()$ = 2.15*1.5 = 3.2	30.1%
$T_H()$ = 2.15*1.9 (1-1.6C_O - 0.4P_O)	35.7%
$T_H()$ = 2.15*2.1(1-2.1C_O +1.9 C_O^2 - 1.4P_O +4.4P_O^2)	36.2%

Even when the property is relaxed it seen from table 3 that the models do not provide a very good fit to the data. It is also seen, when comparing the light traffic dependent models in table 3 with the model $T_H()$ = 1.5*2.15 = 3.2 seconds, that the relationship between the light traffic arrival rates and the headways is very weak. The light traffic arrival rates are, apparently, only able to explain some 6% of the variation in the headway data.

Figure 4 and 5 shows the observed headways plotted against the observed cyclist arrival rates and the pedestrian arrival rates. It seen that the majority of all the observations have low cyclist and pedestrian arrival rates (the arrival rates in figure 4 and 5 are based on the number of arrivals in the cycle period, not on the number of arrivals during, say, an hour).

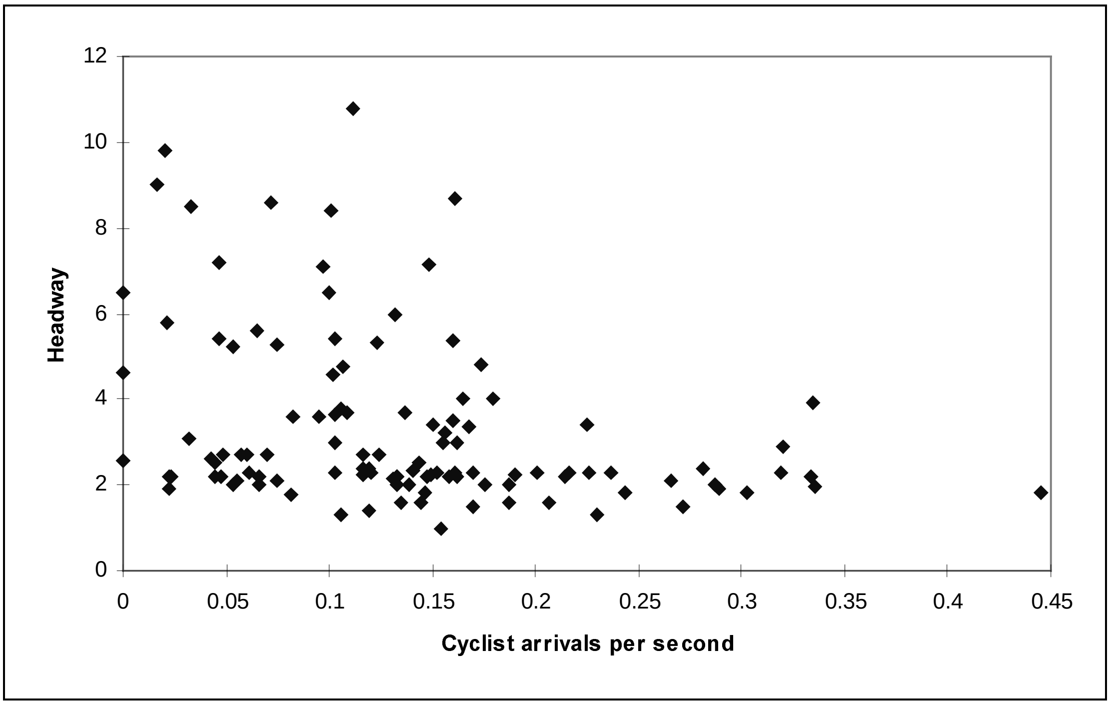

Figure 4.

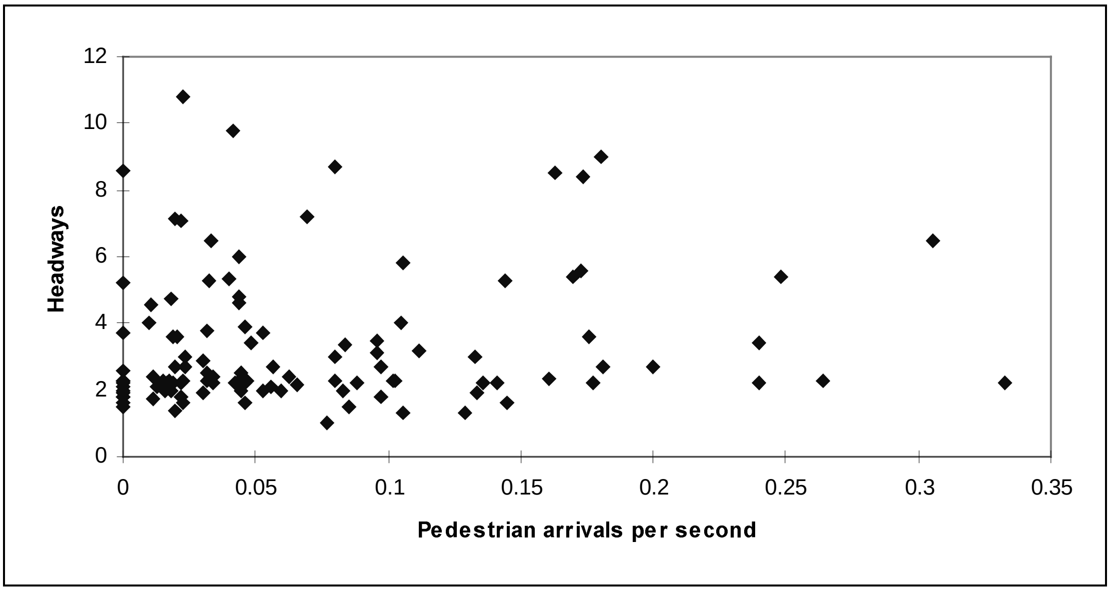

Figure 5

Apart from a small sample size (114 observations) the predominance of observations with low light traffic arrival rates could be one of the explanations for the weak relationship between headway and light traffic arrival rates. Maybe, a clearer picture would have been obtained if more observations with high light traffic arrival rates had been collected. The simulations give less variance. On the basis of the above results it could be argued that one should model the light traffic's effect on the headway by applying the factor 1.5.

It does not seem very reasonable to suggest a more complicated model if it does not yield over all better estimates on headways than the very simple model, T_H = 1.5 x 2.15 = 3.2 seconds.

Calibration of the simulation model from empirical data

The critical parameters in the simulation model are the values of the critical time gaps in the gap acceptance function when right-turning cars must give way to cyclists and pedestrians. These parameters define to what extent the right-turning cars are able to pass through the cyclist and pedestrian flows.

The empirical data from the intersections do not permit a direct measurement of the critical time gap in the car-cyclist or car-pedestrian gap acceptance process. The observations cannot distinguish between delays due to cyclists and to pedestrians. Therefore, the critical gap values cannot be found from the empirical data. Furthermore, it is quite likely that there are some differences in the critical gaps between the 4 intersection locations. And finally, the model does assume a gap acceptance process where the cars actually will give way to cyclists and pedestrians in all cases. This may not always happen in practice. The observations do indicate, though, that cars generally give way, as they should.

In the calibrations, observations are compared to simulation results. The calibration consists then mainly in testing how different values of the critical gaps for cars vs cyclists - T_C - and for cars vs pedestrians - T_P - can make simulation results coincide reasonably well with observations. Earlier empirical studies in Denmark - although not at signalised intersections - have indicated that the values of the critical gaps are speed dependent such that larger speeds of the priority road user mean larger gaps. It was assumed, therefore, that $T_C > T_P$. The numerical values were supposed to be 3 - 7 sec for T_C and 2 - 4 sec for T_P.

The calibrations were based on trial-and-error. For each location observations were selected where flows of cyclists and pedestrians were fairly high. Only observations of the delay of the first car were used for the calibrations. The calibrations were carried out by varying the two time gap values independently. Time gap values were selected in half-second intervals. For each pair of T_C - T_P values a simulation run was carried out. The pair of T_C - T_P values that made empirical data correspond best with the simulated values was chosen. The results of the calibrations are given in table 4.

Table 4.	Dr. Louises Bro	Smallegade	Jagtvej/Ågade	Nørrebrogade
T_C	4.5	6.5	6.5	6.5
T_P	3.0	4.0	4.0	3.0

Critical time gap values (seconds) found during simulation model calibration.

From these numerical values of critical time gaps it was decided to base the following example calculations on the time gap values T_C = 6.5 sec. and T_P = 3.5 sec.

Extension of empirical data by means of simulation

After the simulation model has been “calibrated” against empirical data it is now possible to extend the range of values for cycle and pedestrian flows mainly by filling in combinations of flow data not covered in the field studies. All flow values for cycle traffic: 0, 300, 600, 900, 1200, and 1500 cyclists per hour are combined with the following values for pedestrian traffic: 0, 200, 400, 600, 800, and 1000 pedestrians per hour. Table 5 corresponds to equation (8). This table is not directly comparable with Figure 3, which illustrates equation (8). Figure 3 gives the total delay split into components due to cycles and pedestrians separately. This is not possible in the simulation model. Figure 3 also gives only values when cyclist and pedestrian flows are equal. The models fit reasonably well in terms of order of magnitude of delays.

Table 5	O = 80 s	Gr = 30 s					
Delay for first car		Cyclists per hour					
		0	300	600	900	1200	1500
Pedestrians per hour	0		5.7	11.3	17.4	21.9	26.2
	200	1.7	6.5	12.9	18.9	23.6	27.2
	400	3.6	8.3	14.5	21.1	25.0	28.5
	600	6.2	10.8	16.9	22.5	26.8	29.3
	800	9.1	13.6	19.7	25.0	27.8	30.3
	1000	13.9	17.8	22.9	26.7	29.6	31.2

Table 5 indicates a generally stronger effect of cyclists than of pedestrians, which is expected because of the higher value of the critical gap for cyclists. (In fact, the delay in seconds for the first car may be modeled as 1.9 per hundred cyclists plus 1.0 per hundred pedestrians for cycle traffic up to 900). But the S-shaped curve of fig. 3 does not correspond to table 5. It must be remembered that the S-shape might be due to changing value of the critical gap as suggested by Troutbeck (1990) while the simulation works with a constant critical gap.

Note that when *average* delays in the simulations are very close to or over the green period (30 sec. in table 5) traffic is probably not yet in a steady state but queues are still building up. Therefore, values in tables 5 - 6 for - say - C + P > 1800 are likely to be incorrect or at least unstable.

Table 6 gives results through simulation for headways in the remainder of the green period after first car.

Table 6	O = 80 s	Gr = 30 s					
Queue headways for followers		Cyclists per hour					
		0	300	600	900	1200	1500
Pedestrians per hour	0		2.9	3.8	3.9	3.5	2.3
	200	2.6	3.6	4.3	4.1	3.2	1.8
	400	3.4	4.4	4.7	4.0	2.9	1.1
	600	4.0	4.9	4.8	3.9	2.1	0.5
	800	4.9	5.4	4.8	3.0	1.5	Negative
	1000	5.3	5.3	4.0	2.3	0.3	Negative

The figures in table 6 are directly comparable to the results given in table 3. It appears that the headway for moderate flows of light traffic is somewhat higher than in the simplest model of table 3 and that the flow dependence is as should be expected. It should be noted that the values are at least in theory independent of the signal settings because they relate only to the random flow situation after the red phase group has passed.

A method to estimate the capacity effects of cyclists and pedestrians at signalised intersections

The idea in this method is to estimate the effect of cyclists and pedestrians in the case where the available green time is fully utilized considering the actual cyclist and pedestrian flow. The available green phase in seconds for the right turners is considered. From this time the average delay for the first vehicle given through equation (8) or a corresponding simulation (table 5) is subtracted. The remaining green period is divided by a figure taken from the equations of table 3 or the relevant figure of table 6.

Example: O = 80 sec, Gr = 30 sec, P = 400 ped./h, C = 600 cyc./h, T_C = 6.5 sec, T_P = 3.5 sec

From table 5: Delay for first car = 14.5 s.
Available time for followers: 30 - 14.5 = 15.5

From table 6: Expected headway = 4.7 s
Expected followers: 15.5/4.7 = 3.3 cars. Expected total per green period 1 + 3.3 = 4.3 cars per green period.
Capacity if P = C = 0: Average headway = 2.15 s/car; 30 s / 2.15 s/veh. = 14.0 cars per green period.
Loss of capacity due to pedestrians and cyclists: (14.0 - 4.3)/14.0 = 69 %.

Therefore, the effect of moderate flows of pedestrians and cyclists is substantial.

Conclusions

The aim of this pilot study has been to set up a preliminary model to elucidate the effects of pedestrians and cyclists on the right turn capacity for cars at signalised intersections. The modelling has been based on field observations in Denmark where heavy volumes of both pedestrians and cyclists may be observed.

The model is based on the idea that one model should be developed for the first vehicle due to the group of pedestrians and cyclists accumulated during red and another model be developed for all following right turning cars. So capacity calculations must be based on a pair of models.

The pair of models have been developed both analytically and by means of simulation. The analytical models are developed by means of non-linear regression while the simulation is based on a model calibrated on the same set of observations. The simulation model is based on gap acceptance theory.

The two pairs of models compare fairly well in terms of order of magnitude of the computed delays. However, the regression model for the first car shows a kind of S-shape for the delay as a function of flows while the simulation model is convex in both pedestrian and cyclist flows.

At present it is not clear which of the two curve shapes provide the best description of the light traffic's true effect on the first car. If the S-shape is true to reality then it could be explained by changes in the cyclists' driver behaviour, towards a more efficient use of the cyclist path, and changes in the car drivers' critical gap as cyclist and pedestrian flows increase. The hypothesised change in the critical gap was not allowed for in the simulation, where the critical gap was assumed constant irrespective of the size of the light traffic arrival rates. Similarly, the cyclist driving behaviour was also assumed the same in the simulation irrespective of the size of the cyclist arrival rates.

With regard to the second model - the duration of the following headways - both observations and simulations point to a fairly small effect of the cyclist and pedestrian flows. It may be reasonable to model this duration either as a constant times the duration of the average time headway for a queue of right turning cars when no cyclists or pedestrians are present or as a headway value taken from a simulation.

The model set up in this research provides only a preliminary insight into pedestrians' and cyclists' effect on the right turn capacity at Danish signalised intersections. More research is called for before any final conclusions can be reached on the model's general applicability in the practical capacity assessment of the right turn movement in Danish signalised intersections.

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