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## Comparison of Observed and Estimated Shared Left-Turn Lane Capacity at Signalized Intersections.

El-Sayed Shawaly

*Civil Engineering Department., Faculty of Engineering., El- Mansoura University., Mansoura., Egypt.*

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COMPARISON OF OBSERVED AND ESTIMATED SHARED  
LEFT-TURN LANE CAPACITY AT SIGNALIZED INTERSECTIONS

BY

DR \ EL-SAYED ABDEL-AZIM MOHAMED SHAWALY

CIVIL ENGINEERING DEPARTMENT, EL-MANSOURA UNIVERSITY, EGYPT

مقارنة بين قيم سعة حارة الدوران للسيارات الناتجة من القياسات

الميدانية والمحسوبة من المعادلات النظرية عند التقاطعات التي

تعمل بنظام الاشارات الضوئية

ملخص البحث :

يهدف البحث الى دراسة تأثير نسبة تواجد مركبات تتجه للسيارة ( أو لليمين فسي المملكة المتحدة ) مع تلك المركبات المتجمعة للأمام على سعة الحارة المشتركة في التقاطعات التي تعمل بنظام اشارات المرور الضوئية - وقد وجد أن هناك عدة عوامل موروثة وهندسية تؤثر في سعة مثل هذه الحارات المشتركة منها كثافة حجم المرور القادم من الاتجاه المقابل وسعة تخزين مثل تلك المركبات المتجهة للسيارة وأرضية الاشارة الضوئية ، ولقد اختيرت عدة تقاطعات وتم تجميع معلومات وقياسات ميدانية ومقارنة النتائج الميدانية مع تلك التي تم حسابها من المعادلات النظرية المختلفة والتي تعتمد على عدة فروق ولها حدود عند التطبيق أمكن التوصل الى أنسب هذه المعادلات التي يمكن استخدامها في التنبؤ بالسعة الفعلية للحارة ذات المرور المشترك.

[ ABSTRACT ]

Turning movements at signal-controlled intersections considerably affect the approaches' capacities. Capacity of shared left-turn lane (i.e. where straight-ahead and left-turning vehicles share the same lane width) with opposing flow is greatly influenced by a number of traffic and geometric parameters at signal-controlled junctions. This is particularly important for normal two-phase controlled intersection where all left-turners (or right-turners in U.K.) have to face conflict with the opposing flow since both of them have to share the same green phase.

Different models have been developed recently for capacity calculation of shared left-turn lanes. These models are based on diversity of assumptions and each has some limitations when applied in practice. The main objective of this paper is to check these models and compare their results with capacities obtained from field observations. At last, conclusions regarding their accuracy and to what extent they can represent the actual capacity of shared left-turn lanes are presented.

[1] INTRODUCTION

It is of great importance for the practicing traffic engineers to have the ability of predicting the approach capacity. The most common and traditional methods for predicting the capacity of lanes of opposed turning movements (e.g. left-turn when assuming the right hand passing rule on road or vice versa) by means of either a through car equivalence coefficient [1] or a composite saturation flow [2].

Although for those lanes having high proportion of left-turners it is recommended to dedicate a separate lane in order to segregate the conflicting traffic movements, but due to the limited road widths at some intersections number of traffic movements have to share the same lane width and the same green stage with the opposing flow [3]. Therefore it is very important and recommended to be able to estimate quantitatively the effect of turning

movements on shared lane capacity at signal-controlled intersections. In case of dedicated left-turn lane the through traffic can proceed continue without interruption or much delay being held by left-turning vehicles right-turning in the U.K). On the other hand, the shared left-turn lane has negative side effect since the overall intersection capacity decreases and total delay increases because of the portion of the cycle time taken to accommodate the opposed movement and lost time due to extra intergreen periods [3].

Several theoretical models have been developed which can be used for calculating the capacity of shared left-turn lanes at signalized intersections. These theoretical models take account of the different factors affecting the lane capacity such as storage room capacity, intensity of opposing traffic, and percentage of left-turning vehicles on the subject lane. In fact ignoring these factors in the derivation of the traditional methods usually resulted in underestimating the capacity of shared left-turn lanes.

In this paper, the problem addressed is to what extent these theoretical models are capable of predicting the right capacity of shared left-turn lanes. Therefore data collection on a number of sites have been carried out and then comparison was made between the observed values and the estimated values resulted from using these models. Finally statistically analysis was carried out on both estimated and observed results in order to know which of these model gives close agreement with the field observations.

## (2) Theoretical Models For Shared Left-Turn Lane Capacity

Traditional methods account for left-turners by means of a weighting factors to convert them into equivalent through car unit (T.C.U). These methods ignore the possibility of having number of spaces where left-turning vehicles can be stored without impeding the following straight-ahead vehicles. Also they do not take account of the distribution of the left-turners within the vehicle queue and the gap acceptance characteristics at such situation when left-turners wait for acceptable gap and then clear the intersection.

Recently, some models have been developed for shared left-turn lane capacity prediction which take account of storage room capacity and other geometric and traffic parameters. From practical experience and limited field measurements it was concluded that a primary source of error in applying the common methods was inadequate handling of conflicts between left-turning vehicles and opposing vehicles on two-way streets. These recent models are expected to give better results than other common methods. In this study number of recent models have been considered and their results were compared to field observations.

Firstly, Bang [2] approached the aspect of capacity calculation in the Swedish Capacity Manual in which he defined the saturation flow "s" as the highest stable flow in veh./hour of green (vphg) during existing condition. He developed the following two equations.

$$N_k = P \sum_{j=1}^{N-1} jq^j + Nq \quad (2-1)$$

$$N_k = p^2 \sum_{j=1}^{N-2} q^j j(j+1) + pq \sum_{j=1}^{N-1} N(N-1) + Nq \quad (2-2)$$

where :  $N_k$  is the number of through (or right-turning) vehicles that can be discharged from the lane during the saturated portion ( $G_k$ ) of the opposing flow green period,

$N$  is the maximum number of vehicles that can be discharged during green period ( $g$ ) (i.e during the saturated plus the unsaturated

parts of the opposing stream green period). This is equal to  $(Gx)$  assuming no blocking occurs, and

$p$  and  $q$  are respectively the proportions of left turners and through traffic in the stream under consideration.

Equation (2-1) yields the number of straight-through vehicles that can pass the intersection before the first left-turner arrives and blocks the lane. Equation (2-2) accounts for one left-turner queuing without blocking the lane.

This method seems to have two drawbacks, namely :-

- i) It accounts only for a storage room capacity of either zero or one within the intersection without blocking the lane.
- ii) During the unsaturated green period for the opposing stream the capacity is computed as if only left-turners exist [4] since only the number of gaps accepted is evaluated using the following equation :-

$$S_u = \left[ q_1 \frac{e^{-\alpha q_1}}{1 - e^{-\beta q_1}} \right] \quad (2-3)$$

where  $S_u$  the saturation flow during the unsaturated green period of the opposing stream,

$q_1$  is the opposing movement flow rate (veh./sec)

$\alpha$  is the accepted critical gap (sec),

$\beta$  is the minimum departure headway for opposed turners (sec).

Secondly, the Highway Capacity Manual (H.C.M 1985 [5]) has benefited from the previous work (Bang 1978 and Akcelik 1981) and gave an adjustment factor  $f_m$  for a shared lane capacity calculation. The procedures for calculating shared left-turn lane capacity are illustrated in detail in H.C.M 1985. Because the computational process is reasonably complex, a special work sheet for its calculation can be found in ref.[5]. The following equation gives only the adjustment factor  $f_m$ .

$$f_m = \frac{G_f}{G} + \frac{G_u}{G} \left[ \frac{1}{1 + P_L (E_L - 1)} \right] + \frac{2}{G} [1 + P_L] \quad (2-4)$$

$G$  is the effective green time in seconds

$G_f$  is the duration of the green time phase during which through vehicles may move in the shared lane until the first left-turning vehicle arrives and blocks those behind it. This means a storage room capacity of zero. This is given by the following equation:-

$$G_f = \frac{2P_T}{P_L} \left[ 1 - P_T^{0.5g} \right] \quad (2-5)$$

where :-  $P_L$  is the left-turners proportion in the shared lane,

$P_T$  is the through traffic proportion in the shared lane,

$E_L$  is the approximate through vehicle equivalent for an opposed left-turner and is given by the following equation :-

$$E_L = \frac{1800}{1400 - V} \quad (2-6)$$

$V_o$  is the opposing flow rate ( $V_o \leq 1399$ )

$$q_q = G - G_u \quad (2-7)$$

$G_u$  is the unsaturated part of the green stage and is given by :-

$$G_u = \begin{cases} \frac{G - C Y_o}{1 - Y_o} & \text{if } Y_o \leq G/C \\ 0 & \text{if } Y_o \geq G/C \end{cases} \quad (2-8)$$

$C$  is the cycle time in seconds

$Y_o$  is the flow ratio for the opposing approach

Thirdly, a semi-empirical formula based on results obtained by computer simulation and calibration using field data, the Transport and Road Research Laboratory (T.R.R.L) has proposed a model which is well documented in the R.R.67 [6]. This model has been developed by Kimber, McDonald and Hounsell (1986) at Southampton University. They assumed a shifted negative exponential distribution for the opposing flow arrival pattern and a lognormal distribution of gap acceptance. The storage capacity for the left-turners within the intersection varied from zero to three spaces. The semi-empirical model arrived at is given by the following series of equations:-

$$S = S_g + S_c \quad (2-9)$$

where :-  $S_g = (S_o - 230) / (1 + (T-1)f) \quad (2-10)$

$$S_c = P(1 + N_s) (fX_o)^{0.2} 3600/\lambda_c \quad (2-11)$$

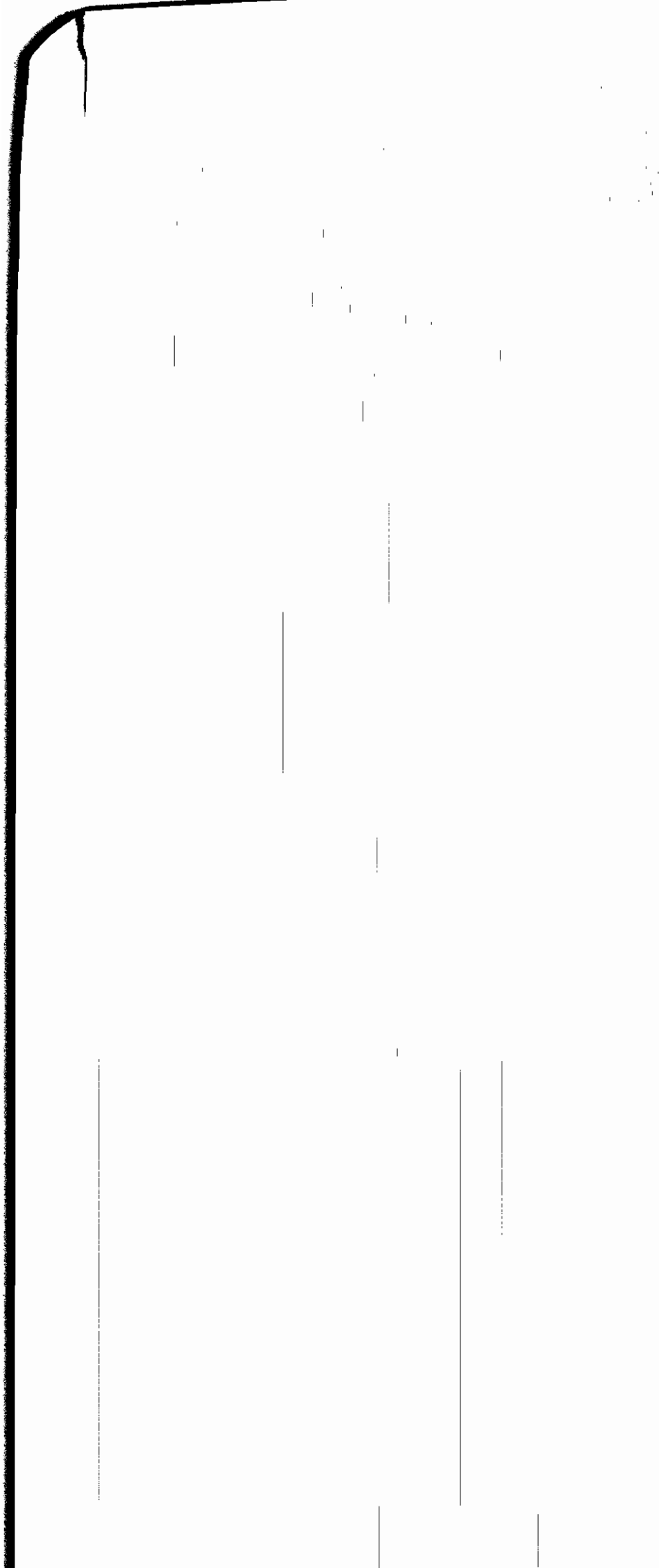
$$T = 1 + 1.5/r + t_1/t_2 \quad (2-12)$$

$$t_1 = 12 X_o^2 / (1 + 0.6(1-f)N_s) \quad (2-13)$$

$$t_2 = 1 - (fX_o)^2 \quad (2-14)$$

The notations for the above equations being as follows:-

- $S$  saturation flow (pcu/hr)
- $S_g$  saturation flow in lanes of opposed mixed turning traffic during the effective green period (pcu/hr)
- $S_c$  saturation flow in lanes of opposed mixed turning traffic after the effective green period (pcu/hr or veh./hr according to utilisation)
- $S_o$  saturation flow per lane for opposite entry (pcu/hr)
- $T$  T.C.U (through car unit) factor of a turning vehicle in a lane of mixed turning traffic. Each turning vehicle is equivalent to "T" straight-ahead vehicles.
- $f$  proportion of turning vehicles in a lane
- $P = 1 + \sum (\alpha_i - 1) P_i \quad (2-15)$
- $\alpha_i$  is the pcu value of vehicle type  $i$
- $P_i$  is the proportion of vehicles of type  $i$  in stream
- $N_s$  number of storage spaces available inside the intersection which



left-turners can use without blocking following straight ahead vehicles.

- $X_0$  degree of saturation of opposing arm (i.e the ratio of the flow on the opposing arm to the capacity of that arm).
- $\lambda$  proportion of the cycle which is effectively green for the phase under consideration
- $c$  cycle time in seconds
- $r$  radius of curvature of vehicle path (metres)

Fourthly, and more recently Brahim [7] proposed a new model in which the conflict occurs at the shared left-turn lane can be broken down into three basic consecutive stages. These stages are represented graphically in Fig.(1). Number of assumptions have been made concerning the derivation of this model and can be referred to in ref.[3]. The stages of this model and its equations are as given below :-

- 1) The number of the straight ahead vehicles "D" passing through the intersection before the blocking occurs

$$D = \sum_{i=0}^k p^i (1-p)^{(N-i)} (N-i) N C_i + p^{(k+1)} \sum_{j=k}^{N-1} (j-k) (1-p)^{(j-k)} C_k \quad (2-16)$$

- 2) Also the number of vehicles waiting to turn left ,W, by the end of the discharge of the opposing queue is given by the following equation :-

$$W = \sum_{i=0}^k p^i (1-p)^{(N-i)} N C_i + (k+1) p^{(k+1)} \sum_{j=k}^{N-1} (1-p)^{(j-k)} C_k \quad (2-17)$$

Therefore the total number of vehicles crossing the stopline T ( $T = D+W$ ) during g is obtained by summing up the above two equations (i.e. equations (2-16) and (2-17)).

$$T = \sum_{i=0}^k p^i (1-p)^{(N-i)} N C_i + p^{(k+1)} \sum_{j=k}^{N-1} (j+1) (1-p)^{(j-k)} C_k \quad (2-18)$$

where

$$N C_i = \frac{N!}{i!(N-i)!} \quad \text{and} \quad C_k = \frac{j!}{k!(j-k)!}$$

It is apparent that this model can be used for any storage capacity (i.e any value for k) and in all cases,  $D = (1-p)T$  and  $W = p T$ .

- 3) After the opposing queue has dissipated, gaps are available during the second stage of conflict for any waiting left-turning vehicle to cross and clear the junction. Assuming  $\alpha$  and  $\beta$  as previously defined in equation (2-3), the discharge rate of the waiting left-turners can be calculated using equation (2-3). Subsequently the average time interval between successive left-turning vehicles is therefore  $1/S_u$  and during that time the expected number of vehicles able to cross the stopline from the subject lane is determined from equation (2-3) with  $k=0$ . In the case of  $N = 0.5/S_u$  (assuming one vehicle every 2 seconds across the stopline) the resultant number "U" of left-turning vehicles through gaps and also straight-ahead vehicles during the second stage of the conflict is therefore given by the equation below:-

$$U = (g - g_s) S_u [1 - (1-p)^{0.5/S_u}] / p \quad (2-19)$$

With regard to the intergreen time period, the waiting left-turning vehicles have already been included in  $T$  in equation (2-18) and all these are assumed to turn left during the intergreen period. The total number of vehicles entering the intersection per cycle is therefore  $T + U$ , which when multiplied by the number of cycles per hour gives the shared lane capacity.

### [3] Data Collection and Analysis

Number of sites of different layouts and geometric characteristics has been chosen for field observations in the Sheffield urban area. The main conditions upon which the decision of retaining a site were based on: varying proportions of left-turners (i.e. right-turning in U.K) in the opposed movements and different storage room capacities ( $K=0, 1$  and  $2$ ) for opposed left-turners at different sites. Five approaches have been retained for field observations at four junctions. These junctions are shown in Fig.(2) through Fig (5). Two approaches were two-lane entries with the off-side lane either exclusively dedicated to right-turning traffic (Penistone Road Fig. (3) ) or shared with straight-ahead traffic (Greenland Road Fig (4) ). The other three approaches were only one lane entry with different storage room capacities depending on the entry width and the flare at the stopline. The proportions of the right-turners varies from 8.6 % to 48.3 % on different occasions.

Concerning the signal settings at these sites they were operating on fixed cycle time bases except only one site which was operating on fixed time with an early-cut-off phase that occurred as the demand for left-turn appreciably increased.

The author has carried out some other data using the videotape recording technique in order to supplement the available data in the department during his stay in the U.K. The equipment used in data collection consisted of the NV-MS Panasonic Camcorder sited on a tripod. The Camcorder was positioned on the roof of an adjacent building which is overlooking the junction under consideration.

From the films taken, thirty five recordings gave acceptable sample of real time not less than 30 minutes. These films were analysed in the traffic laboratory of the Civil and Structural Engineering in the University of Sheffield. Counts of vehicles types and their turning movements (i.e. straight-through or right and left-turn) were recorded. Also, the time the vehicles crossed the stopline were registered for each cycle by the aid of the time-base written on the filmed tapes. Different categories of vehicles were converted to p c u's using the RR 67 equivalent factors (Kimber, Mc Donald and Hounsell [6] ). It should be emphasized that the same conversion factors have used throughout this study.

It was necessary also to measure  $\alpha$  and  $\beta$  values in order to calculate the predicted saturation flows. Therefore direct measurements were carried out during the data analysis from the filmed tapes to obtain the mean values of both  $\alpha$  and  $\beta$  for substitution in the predictive models.

### [4] Observed Versus Predicted Results

Since Bang [2] clearly stated that the Swedish model was designed for storage room capacities of either zero or one, therefore it can not be used in calculating the saturation flows of some approaches where  $k$  is more than one. Although the H.C.M. 85 model has been deduced considering that on the arrival of the first left-turning vehicle, the lane is blocked for those vehicles wanting to proceed straight-ahead, it did not restrict the use of the formulae but on the contrary recommended them for all situations. Therefore it has been decided to use all above stated theoretical models (except Bang's) for



calculations and to test their results against the observed values.

Tables (1), (2), (3) and (4) present the data summary for the studied sites as well as the observed and calculated shared lane saturation flows for different storage room capacities of zero, 1 and 2 respectively. Also figures (3), (4) and (5) show observed against calculated saturation flows when H.C.M 85, RR 67 and Brahimi and Ashworth's models respectively are used. Each group of points represent site with specific storage room size. It is worth noting that each point on these plottings represents the mean value for the whole period of observation (up to 74 minutes on some occasions). It is clear from these figures to what extent the theoretical models can predict the actual observed values of shared lane capacities. It is apparent from these figures that both H.C.M. 85 and RR 67 models seem to underestimate the shared left-turn lane capacities, in particular, for those sites of lower capacities. While on the other hand Brahimi and Ashworth's model does give good agreement with observed saturation flows for this range of capacity. Nevertheless, noticeable differences can be seen between observed and predicted saturation flows using the same model for higher values (e.g saturation flows  $\geq 1500$  pcu/hr).

#### (5) Statistical Analysis of Results

In order to test the validity of the theoretical models previously stated in Section [2], one needs to verify their calculated results against actually observed values of saturation flows collected in the course of this study. The paired "t" test was performed to check whether the differences obtained between observed and calculated values for saturation flows are significant or not. Calculations necessary for this statistic test were carried out and included in table (5) for 5% and 1% level of significance. The results of the test showed that the null hypothesis is rejected at 5% level for all models predictions while it is accepted for only two models namely H.C.M. 85 and Brahimi and Ashworth's models when 1% level of significance was used.

#### [6] Conclusions and Recommendations

In relation to the agreement between the observed and predicted saturation flows of shared left-turn lane at signal-controlled intersections, the following conclusions can be drawn :-

- 1) Both H.C.M. 85 and RR 67 models seem to underestimate the saturation flows when comparison was made with the field observation undertaken at three different sites in Sheffield urban area.
- 2) Statistical analysis indicated that the theoretical model proposed by Brahimi and Ashworth gives fair agreement with the observed saturation flows. Although the statistical analysis showed that at 1% level of significance the null hypothesis is accepted, nevertheless, the differences found between observed and predicted values from all models considered in this study are not too different from each other.
- 3) In order to give an answer to the question of which of these models gives the most reliable estimate of the capacity of shared left-turn lane, it is necessary to carry out much more extensive observations at several sites of various storage sizes. Alternatively a simulation study can be performed though the theoretical models can be tested over a wider range of parameters.

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Table (1) Data Summary on Turning Movements and Observed and Calculated Shared Lane Capacity for Cricket Inn Road/ Bernard Road Junction.

( k = 0 )

date & time of observn	signals settings C / G	right turn percentage %	observed sat. flow pcu/hr	calculated sat. flow H.C.M	calculated sat. flow R.R.67	calclate sat.flow Eq. 2-18
24/5/88pm	60/9+11	13.4	1840	1691	1477	1756
	60/8+10	10.4	1945	1746	1737	1860
25/5/88pm	60/9+11	11.9	1818	1707	1544	1798
	60/8+10	11.3	1863	1728	1560	1820
27/5/88pm	60/9+11	11.5	1855	1725	1560	1812

Table (2) Data Summary on Turning Movements and Observed and Calculated Shared Lane Capacity for Penistone Road / Rutland Road Junction.

(k = 1)

date & time of observn	signals setting C / G	right turn percentage %	observed sat. flow pcu/hr	calculated sat. flow H.C.M	calculated sat. flow RR 67	calculated sat.flow Eq. 2-18
27/6/86pm	74/41	100	378	283	208	299
30/6/86pm	73/40	100	357	276	209	291
30/6/86pm	84/48	100	384	317	205	338
30/6/86pm	56/30	100	389	328	265	340
30/6/86pm	90/51	100	350	225	195	324
30/6/86pm	75/42	100	287	214	184	224
30/6/86pm	78/43	100	345	253	193	268

Table (3) Data Summary on Turning Movements and Observed and Calculated Shared Lane Capacity for Greenland Road / Stanforth Road Junction.

(k = 1)

date & time of observn	signals setting S / G	right turn percentage %	observed sat. flow pcu/hr	calculated sat. flow H.C.M	calculated sat. flow R.R.67	calculated sat. flow Eq. 2-18
2/7/87pm	68/28	13.9	1614	1699	1667	1743
7/7/87pm	68/28	19.9	1789	1516	1446	1651
7/7/87pm	68/28	17.50	1360	1651	1613	1722
9/7/87pm	68/28	9.70	1317	1704	1658	1743
22/9/87pm	68/28	8.60	1582	1650	1595	1686
23/9/87pm	68/28	10.80	1475	1704	1667	1723
2/7/87pm	79/28	7.40	1762	1707	1669	1723
7/7/87am	79/28	48.20	671	816	711	1028
3/7/87pm	79/28	18.20	1403	1482	1419	1671
7/7/87pm	79/28	14.60	1590	1511	1453	1639
8/7/87pm	79/28	14.80	1516	1524	1467	1667
27/9/87pm	79/28	7.00	1784	1705	1666	1706
29/9/87pm	79/28	19.00	1527	1466	1401	1657

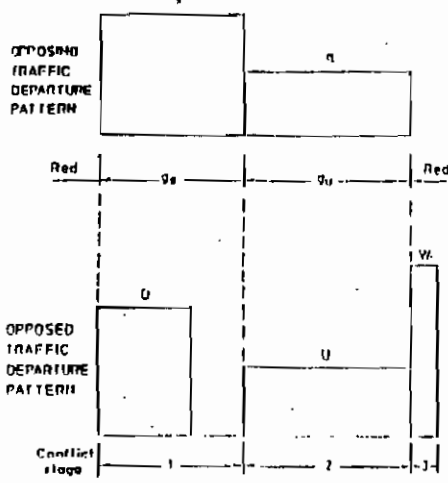
Table (4) Data Summary on Turning Movements and Observed and Calculated Shared Lane Capacity for Crookes Road / Witham Road Junction.

(k = 2)

date & time of observn	signals C / G	right turn percentage %	observed sat. flow pcu/hr	calculated sat. flow H.C.M	calculated sat. flow R.R.67	calculated sat. flow Eq. 2-18
2/2/90am	82/28	15.00	1572	1340	1430	1548
	82/28	19.00	1308	964	1054	1363
5/2/90am	82/28	13.00	1607	1446	1588	1673
	82/28	19.00	1188	933	1031	1326
15/2/90am	50/17	13.50	1461	1264	1707	1808
	50/17	25.00	1287	1167	1182	1366
16/2/90am	50/17	17.00	1584	1368	1692	1745
	50/17	22.00	1736	1118	1182	1398
16/2/90am	82/28	13.00	1596	1565	1460	1596
	82/28	12.50	1280	1378	1167	1387

Table (5) Paired "t" Test Summary on Differences Between Observed and Predicted Values of Saturation Flows

Cases Parameters	Observed & H C M	Observed & R R 67	Observed & Eq. 2-18
Sample size	35	35	35
Differences	-2316	3126	2217
$D^- = \frac{\text{Difference}}{n}$	-66.17	89.31	63.34
$S_{D^-}$	163.62	170.37	164.31
Degree of freedom $D \times n$	34	34	34
$t = \frac{D^-}{D}$	-2.39	3.10	2.28
$t_c$ at .05	$\pm 2.03$	$\pm 2.03$	$\pm 2.03$
$t_c$ at .01	$\pm 2.77$	$\pm 2.77$	$\pm 2.77$



Diagrams for left-turns: Left-turn departures only straight-ahead vehicles; Left-turn departures through gaps and other straight-ahead vehicles; Left-turn departures during the intersection period.

FIG (1) Shared Lane Opposed Left-Turn Model

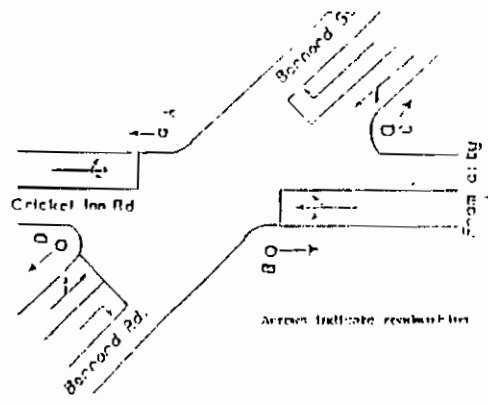


FIG (2) Cricket Inn Road/Bernard St. Junction

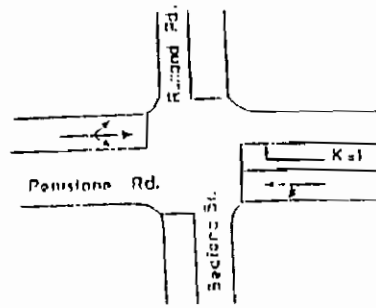


FIG (3) Penistone Road/Rutland Rd. Junction

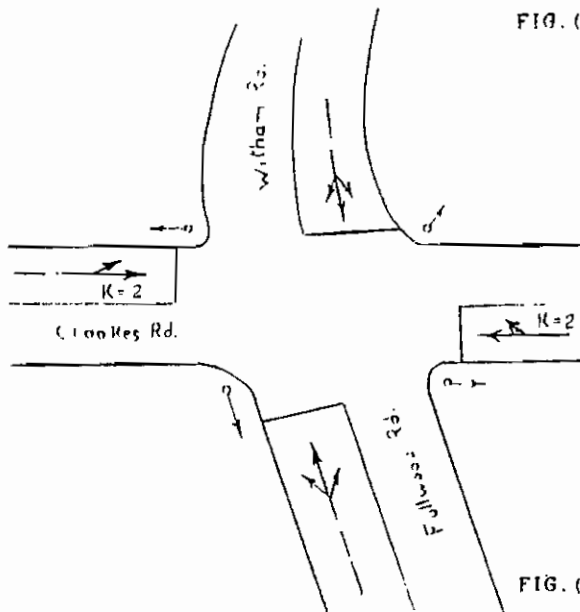


FIG (5) Crookes Road/Wither Rd. Junction

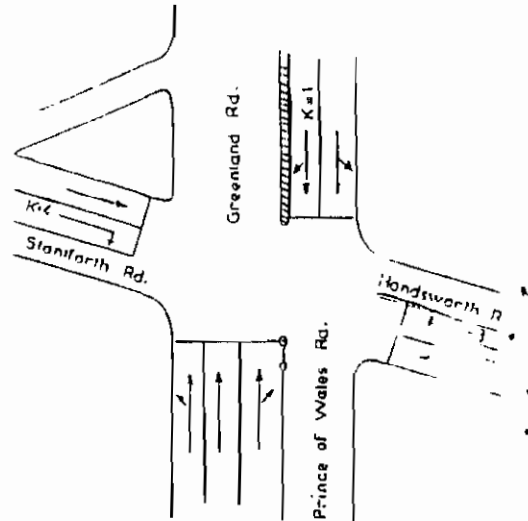


FIG (4) Greenland Road/Staniforth Rd. junction