Transportation Research Procedia

Volume 15, 2016, Pages 87–99

ISEHP 2016. International Symposium on Enhancing Highway Performance

Variability of Capacity and Traffic Performance at Urban and Rural Signalised Intersections

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Abstract

The paper presents the issue of short time variability (in signal cycles) of the capacity values of signalised intersections and its variation depending on the intersection's location (urban and rural). The specificity of the shape, traffic load and operation of signalised urban and rural intersections was the basis for analyses. This study was carried out in the period 2008 - 2012 at over 70 intersections. The analysis of headways between queued vehicles (passenger cars) on the stop lines helped determine the saturation flows in the individual signal cycles. The authors characterised statistically their variability and identified differences between urban and rural intersections. The saturation flows at the latter location had considerably lower values, causing worse traffic conditions at similar saturation flows. They also showed the impact of the number and location of lanes used by vehicles travelling straight, of the intensity of development of the rural intersection's surroundings and of weather conditions on the level of saturation flow. They presented regression models estimating saturation flows. The paper also presents an analysis of the reliability of the entry to a signalized intersection and recommends the adoption of reliability levels, which would allow determining the degree of acceptance (the percentage of drivers) of traffic performance depending on the degree of congestion and of control parameters.

Keywords: saturation flow variations; capacity; signalised intersection; reliability.

1 Introduction

The location of intersections in areas of different forms of development and unique characteristics of traffic and traffic control related to the function of the area entail different needs and types of driver behaviour. Significant differences have been noticed in the functioning of urban and rural signalised intersections. The increasing use in Poland of traffic signals to control traffic at rural intersections has inspired the authors to conduct empirical research in period 2008 - 2012 into the efficiency of both urban and rural intersections and potential determinants of their functioning (Chodur & Others, 2012).

[*] Masterminded EasyChair and created the first stable version of this document

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Selection and peer-review under responsibility of the Scientific Programme Committee of ISEHP 2016

doi:10.1016/j.trpro.2016.06.008

The study focused on intersections' geometric features and features of vehicle and pedestrian traffic. The measurements at intersections was conducted with the use of video recording technology. During the assessment of intersection efficiency, it is important to estimate the intersection's entry capacity, and in the case of signalised intersections the estimation involves the determination of saturation flow. This paper is devoted mainly to the characterisation of this important feature, especially its variability between intersection locations, different weather conditions and times. It also describes road managers; and users; acceptance of difficult conditions in states of traffic overload, and the authors refer to the reliability of the functioning of the intersection entry in urban and rural areas.

2 Specificity of urban and rural intersections

Signalised intersections on rural roads have their own specificity, including:

- high-speed traffic on the main road, far exceeding the speed limit in built-up areas, high imbalance between the congestion on the main lane (national road) and on the lower class road.
- small pedestrian traffic, mostly connected with public transport stops, which demands a phase that allows pedestrians to safely cross the road with the way of way, which has a significant share in the signal cycle that is not always sufficient to accommodate the demand flow on the side road,
- significant share of heavy vehicles in traffic flows on the main road, including trucks with trailers,
- lack of intensive land development in the intersection's surroundings, which allows you to adjust the geometry of the entry to the needs of traffic and control. This characteristic is also typical of urban intersections, especially those on the outskirts, along arteries with speed limits which are higher than that in built-up areas and light pedestrian traffic.

2.1 Geometry and traffic organisation

There is a little difference between the design of road intersections in built-up and rural areas: at rural intersections more emphasis is put on dynamics, and at urban intersections on the efficiency of service of different road users. Operating speed on sections of urban roads, especially national, typically exceeds 80 km/h and if there are clear reasons for traffic at the intersection to be controlled by a set of traffic signals, then the speed limit on entry should be, for operational reasons (adequate reaction to the change of signals) reduced, typically to 70 km/h (Tracz & Others, 2001). Often the main, indeed sometimes the only reason, for control of traffic by means of traffic signals at rural intersections is the need to improve road safety. Another important consideration involves ensuring the efficiency of the intersection by affording priority to vehicles moving along the national road.

In 2008 – 2012, identification tests were carried out in several Polish provinces (40 intersections in rural areas and 31 in urban areas) in order to determine the real differences between solutions at urban and rural signalised intersections that could affect drivers' behaviour. The following is a comparison of the two kinds of intersections in terms of several selected features:

- in terms of the angle of the intersection of the roads axis, rural intersections reveal deviations higher than 10 ° from the right angle more often than urban intersections (Fig. 1).
- All the analysed intersection were channelized, with channelization of priority entries being more frequent. This is particularly significant at rural intersections because of the visibility of these intersections. The channelization of lower class road entries to rural intersection is predominantly of the "small drop" island type (60%).

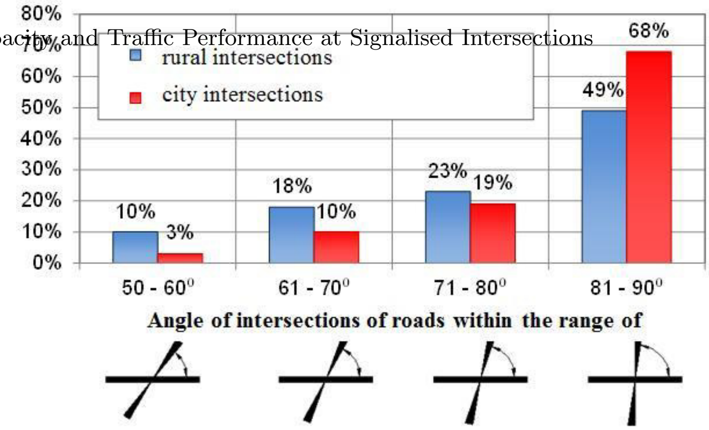

Figure 1: Comparison of the angle of intersections of road axes at urban and rural road intersections

- An additional left turn lane at entries of the main road is equally common at both urban and rural intersections. In rural areas, unsignalised intersections typically feature one entry lane of the lower class road. Whenever this solution existed at an intersection before installation of traffic signals, then it usually was retained after signals installation.
- At rural intersections, due to higher operating speeds, the slow-down stretch is longer, and in urban areas the accumulation section is longer due to longer queues.
- Pavements at main road entries to rural intersections are in most cases connected with a bus stop, which is usually located at the exit of the main road.
- The main road entries of rural intersections predominantly have one pedestrian crossing, and lower class road entries equally often have either no pedestrian crossing or one crossing.
- The smaller surface of rural intersections (Fig. 2) is mainly due to the lack of development of lower class road entries. Stop lines are not placed any nearer to the intersection even if there are no pedestrian crossings as visibility of traffic signals must be strictly assured.

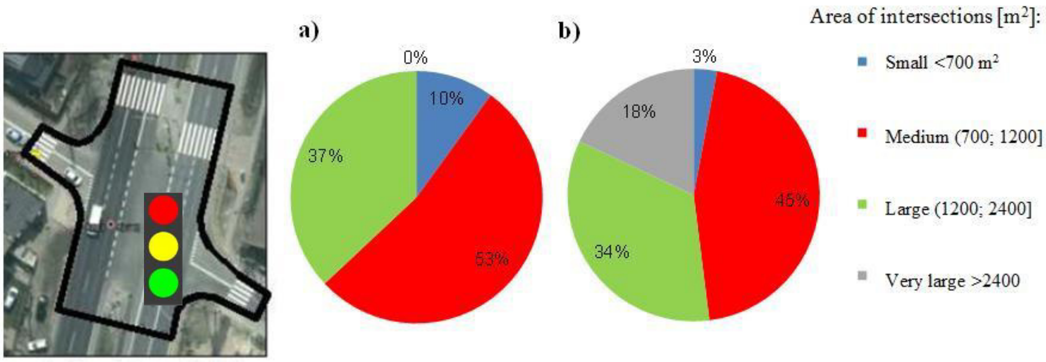

Figure 2: Classification of intersections in rural areas (a) and in built-up areas (b) in terms of their interior area

Visibility requirements before the end of the queue and the visibility of the signals at the approach to the intersection are also an essential consideration at rural intersections (Journal of Laws from 2004, No. 220, TD 2004).

2.2 Features of road traffic and road user behaviour

The basic characteristics of traffic at the intersection include primarily traffic flow and its qualities, such as the structure of traffic directions and vehicle type and its variability over time, speed at the approach to the intersection. Studies conducted at rural intersections (Chodur & Others, 2012) reveal significant differences in demand flows. The aggregate flow at intersections ranged from very low at less than 500 P/h to significant at approx. 3000 P/h. Based on the proportions of demand flow, the studied intersections can be divided into two types, the first being a combination of roads with heavy traffic in both directions (intersection of national roads and intersection of a national road and a provincial road) and the second being an intersection of a national road and lower class roads. The latter type reveals significant differences in demand flows on the two directions and it is not infrequent that the flow on the entry from the lower class road does not exceed 10% of the traffic at the intersection. You can notice marked differences in the geometry of these two kinds of intersections in terms of the number of lanes and type of islands channelling the traffic inside the intersection.

A commonly used characteristic describing the range of demand flow variation during a 60-minute time interval based on 15-minute traffic volume values is the indicator of non-uniformity of peak hour factor k_{15}. For the analysed intersections, this indicator values exceeded 0.90, and the average value stood as 0.95. In practical analysis, given the absence of data on the variability of traffic flow within a 60-minute period, one could assume k_{15} to range between 0.92 – 0.98 (Chodur, 2012) in order to determine the traffic volume during a 60-minute period ($Q_o=Q/k_{15}$) for rural intersections. Variability of 5-minute intensity during a sixty minute time period rather does not assume the parabolic profile typical of short and manifest peak time periods of urban traffic.

The share of heavy vehicle traffic on the entries to urban intersections ranged from 7.7% to 31.0%. The average u_c value was approx. 21%. The variation in the share of heavy vehicle traffic on each lane of dual carriageway sections is significant. The u_c on the left lane rarely exceeded 15%, whereas on the right lane it even exceeded 50%. On average, left lane u_c stood at 11.4%, compared to right lane u_c at 40.1%

2.3 Control

The advisability of the use of traffic signals at rural intersections is mostly underscored by safety considerations and less frequently by flow efficiency considerations. The analysis conducted for the studied intersections showed that despite the slight demand flow at lower class road entries to intersections with intensive traffic on the main road (especially on a multilane road), installation of traffic signals is justified for reasons of both safety and traffic efficiency. Without traffic signals, vehicles negotiating the intersection from the minor entry would sustain unacceptable delays. Calculations indicate that on some of the intersections, traffic performance for lower road class entries was very good or good (delay did not exceed 15 or 30 s/P) and the degree of capacity utilization was slight (2% - 40% (Chodur & Others, 2012)). In terms of efficiency, these intersections could successfully function without traffic signals.

Determining the signal settings - the primary phases set-up (implemented in the state of continual detector activation) is a complex issue, whose aims is to ensure capacity and acceptable traffic performance. At high speeds at the entry to the intersection, the configuring of the detection system is important in terms of both security and traffic conditions. At intersections covered by the research, in most cases (77% of crossings), traffic signals of three basic phases were deployed, while allowing a conflict of flows during the phase of servicing of the secondary direction. In addition to the basic phases, use is also made of optional phases which are implemented to accommodate vehicle service needs on individual lanes and pedestrians at pedestrian crossings.

Traffic signal setting parameters – cycle and green phase lengths are determined mainly to ensure appropriate traffic performance. At rural intersections, it is important to find a balance between favouring privileged traffic on the national road and the service needs of pedestrians crossing the national road (they often require a long green signal) and forbearing from extending the waiting time for a green signal for vehicles serviced in other phases of traffic. The maximum program is laid out for situations when there are continuous excitations of detectors in all groups of signal.

The comparison of calculated intergreen times at rural intersections reveals that the calculation method mandated for Polish roads has led to the adoption of shorter intergreen times than in the US, Germany and France. In particular, there are noticeable differences in the length of the yellow signal. The constant, three-second length of the yellow signal is ill suited to high vehicle speeds. The typical speed limit of 70 km/h at the intersection entry does not guarantee the vehicle will stop safely within the 3 seconds of the yellow signal. Analyses taking into account the existence of feedback between the length of the yellow signal and driver behaviour show that lengthening the yellow signal to 4 seconds would help reduce the number of vehicles entering the intersection during the red signal by 33%.

3 Study of the vehicle service process

The main parameter which impacts on lane capacity at signalised intersection entries is saturation flow. According to the Polish method (Chodur & Others, 2004), saturation flow *S* is the maximum traffic volume that can cross the stop line of the lane or lane group at existing traffic and road conditions during one hour of effective green signal. The saturation flow value is specified in the actual vehicles or passenger car per hour of green signal. It is assumed in Polish conditions that the base saturation flow value of a non-collision direction on a lane on which there is no other collision direction is 1900 pcu/h and 1700 pcu/h if the lane is also used by a direction with a collision direction. Values considered as base are reduced to real life conditions after taking into account a range of influences, including lane width, radius of the turn, share of heavy vehicles, longitudinal inclination and lane location.

The value of instantaneous saturation flow corresponds to the maximum departure of vehicles from the queue on the lane during the green signal (Gaca & Others, 2008). Recent studies (Ostrowski, 2013, Chodur & Others, 2011) of base saturation flow values focus on analyses of variability of departure intensity during the middle interval of the green signal for pairs of passenger vehicles. The saturation flow in a single signal cycle can be determined from the quotient $3600/\bar{t}_{n,s}$, where $\bar{t}_{n,s}$ is the average saturation headway between pairs of passenger vehicles (Fig. 3).

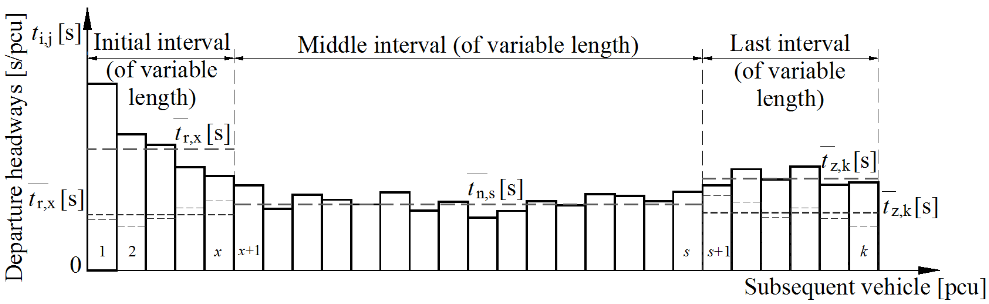

Figure 3: Service process model during a signal cycle (Chodur & Others, 2012, Ostrowski, 2014)

In papers (Chodur & Others, 2012, Ostrowski, 2014) the authors introduced a variable number of rejected *f* headways between vehicles during the initial interval of the green signal, and the last interval (Fig. 3) taking into account existing weather conditions. The beginning and end of the middle interval was determined on the basis of comparative analysis of adjacent headways between successive vehicles crossing the stop line by using *t*-Student parametric test at the significance level of $\alpha = 0.05$. Between 2 to 8 initial headways between vehicles were rejected. Empirical studies of variability of headways between vehicles and of saturation flows were conducted both in urban (Ostrowski, 2013, Chodur & Others, 2011) and rural (Chodur & Others, 2012) areas. Selected test results are shown in Fig. 4 and 5 for the through direction from research sites with a 1 x 2 and 2 x 2 lane cross-section.

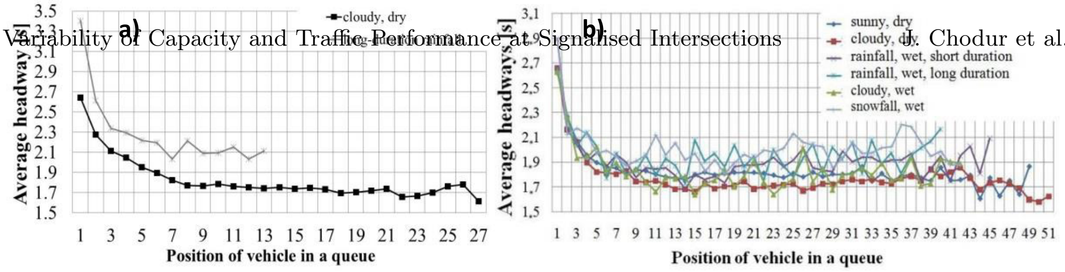

Figure 4: Headways between successive vehicles crossing the stop lines at the green signal in different weather conditions at a) one-lane b) two-lane urban intersections, from the inner lane. Sample sizes range from 30 to 1250 (Ostrowski, 2013)

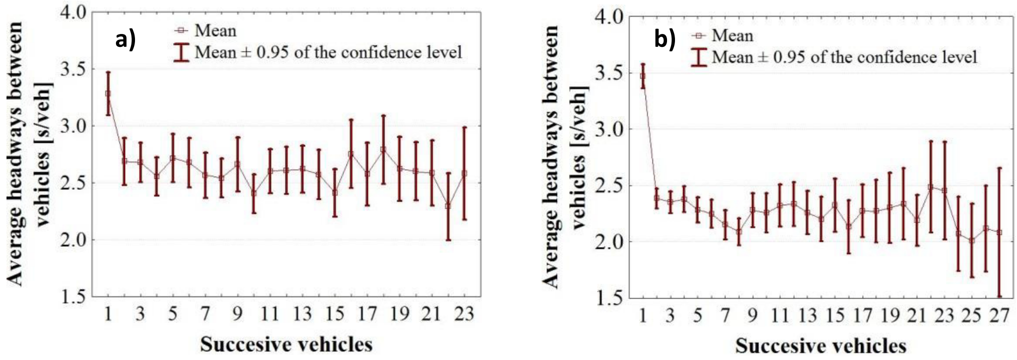

Figure 5: Headways between successive vehicles crossing the stop line at the green signal at rural intersections a) with one lane for the through direction, b) two lanes for the through direction from the inner lane. Sample sizes range from 28 to 482 (Chodur & Others, 2012)

The studies have shown that the average headways between vehicles in the queue crossing the stop line are markedly greater on entries to rural intersections, and the dispersion of their instantaneous values is similar at urban and rural intersections.

Converting the average headways between vehicles to saturation flows, the authors achieved relatively low values in adverse weather conditions in the cities and in favourable weather conditions outside the city at intersections on national roads. The average values differ from recommendations given in the Polish method (Chodur & Others, 2004), which are mainly tailored to urban conditions. It was noted that:

- for urban intersections, the mean base of saturation flows S_w is ranges from 1670 pcu/h during snowfalls and prolonged rainfalls to 2170 pcu/h in favourable weather conditions (cloudy weather and dry road),
- for rural intersections, the mean base of saturation flows S_w is: for one lane - 1428 pcu/h, for two lanes with a through direction, it is 1582 pcu/h for the inner lane and 1466 pcu/h for the outer lane, respectively.

Selected research sites for saturation flows in rural areas (Chodur & Others, 2012) revealed flows causing substantial queues of vehicles at the red signal. The sites were divided into study groups on the grounds of the degree of urbanisation, which was mainly due to the nature and function of the surroundings and its activities as well as to the distance to adjacent signalised intersections. Fig. 6 shows the variability of saturation flows for intersections with one and two lanes for through traffic, divided into lanes and degrees of urbanisation. The authors fitted theoretical distributions to empirical data. Normal distribution, whose fit with the empirical distribution was checked using the Kolmogorov-Smirnov test at the significance level α = 0.05, was the best theoretical distribution. The normalcy of distributions was also confirmed using the Shapiro-Wilk test.

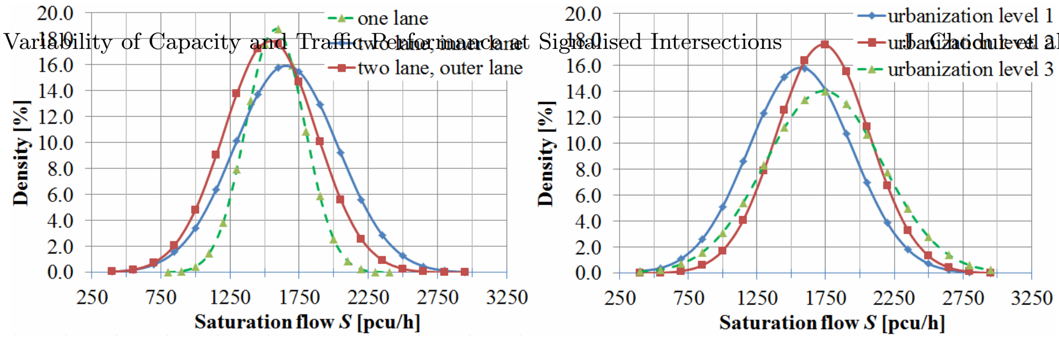

Figure 6: Density functions for saturation flow S [pcu/h] at rural intersection entries (Chodur & Others, 2012)

Variability analysis of saturation flows was also carried out in cities with less than 200 000 residents. The analysis indicates that in Poland, as in Canada, (Canada, 2008), the variability of base values of saturation flows is meaningful for analysis. The average value of base saturation flow of 1750 pcu/hz as revealed by the study was significantly lower than the one currently assumed in method (Chodur & Others, 2004) (1900 pcu/h) and occurring in larger cities (Ostrowski, 2013, Chodur & Others, 2011). Below, Fig. 7 summarizes and compares the values of saturation flows at urban and rural intersections. The lower values of saturation flows on the through lanes of rural intersections result from drivers' less aggressive driving and observance of the larger headways between vehicles. The spread of instantaneous values of saturation flows at both intersection locations is similar. Like fluctuations in demand flows, it causes variable traffic performance in subsequent signal cycles.

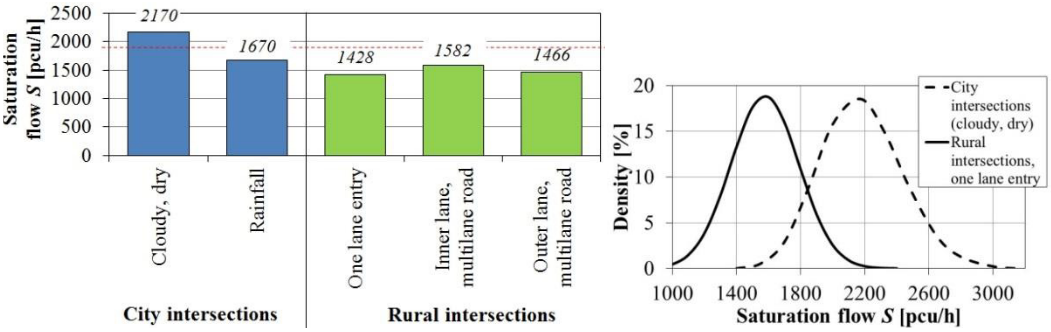

Figure 7: Comparison of empirical values of saturation flows determined for urban and rural conditions

3.1 Mathematical models of base values of saturation flows

The Polish method (Chodur & Others, 2004), and other foreign methods (Canada, 2008, Akcelik, 1989, HBS 2001, HCM 2010) are all based on a fixed base value of saturation flow, which is then adapted to the local conditions by introducing correction coefficients. The above analysis shows that the base value is not fixed, and its volatility is of a deterministic and random nature. Mathematical models based on the results of empirical analysis used to determine base values of saturation flows can be found in papers (Chodur & Others, 2012, Ostrowski, 2013, Perez-Cartagena & Tarko, 2005). The models designated for urban and rural intersections have been collected and compared in Tables 1 and 2 below.

Models	Measurement site	Multiple regression equation	Variable characteristics					
			identifier	mean	min	max	σ	ν
I	One lane $f_4 \leq 10\%$	$S_w = 1689 + 82 \cdot f_1 + 5 \cdot f_2 - 175 \cdot \delta_2 - 243 \cdot \delta_4 \pm 160$ ($N = 2834$) $R = 0.78$ $R^2 = 0.61$	S_w [pcu/h]	1907	887	2679	255.19	0.13
			f_1 [s]	1.97	-3.9	8.3	1.82	0.93
			f_2 [pcu/cycle]	24	5	65	10.62	0.45
II	**a)** Left and right lane when $f_{4,L} \leq 10\%$ $f_{4,R} \leq 10\%$	$S_w = 1249 + 91 \cdot f_1 + 20 \cdot f_2 - 138 \cdot \delta_2 + 85 \cdot \delta_7 \pm 150$ ($N = 522$) $R = 0.71$ $R^2 = 0.50$	S_w [pcu/h]	1899	1354	2459	213.12	0.11
			f_1 [s]	0.78	-1.7	5.0	1.04	1.33
			f_2 [pcu/cycle]	27	13	50	4.74	0.17
	b) Left and right lane when $f_{4,L} \leq 10\%$ $f_{4,R} > 10\%$	$S_w = 1810 + 68 \cdot f_1 + 2 \cdot f_3 - 502 \cdot f_4 - 113 \cdot \delta_2 - 213 \cdot \delta_3 \pm 176$ ($N = 1819$) $R = 0.67$ $R^2 = 0.45$	S_w [pcu/h]	1912	1060	2580	238.76	0.12
			f_1 [s]	1.43	-5.0	7.7	1.86	1.29
			f_3 [pcu/cycle]	28	0	83	14.19	0.50
			f_4 [-]	0.047	0	0.46	0.08	1.73
III	Left, middle and right lane when $f_{4,L} \leq 10\%$ $f_{4,M} \leq 10\%$ $f_{4,R} \leq 10\%$	$S_w = 1577 + 83 \cdot f_1 + 9 \cdot f_3 - 196 \cdot \delta_2 - 73 \cdot \delta_1 - 84 \cdot \delta_5 + 165 \cdot \delta_8 \pm 166$ ($N = 6042$) $R = 0.72$ $R^2 = 0.52$	S_w [pcu/h]	1964	1014	2842	240.60	0.12
			f_1 [s]	1.62	-5.9	8.0	1.75	1.08
			f_3 [pcu/cycle]	19	0	47	4.18	0.22

Table 1: Regression models for basic saturation flow S_w in through traffic at city intersection entries.

Symbols used in table 2: f_1 – delay during the initial interval; f_2 and f_3 – queue at the beginning of the green signal and demand flow; f_4 – share of heavy vehicles; δ_i (0 or 1): δ_1, δ_2, δ_3 – short-duration rainfall, long-duration rainfall and snowfall (wet); area of intersection [m^2]: δ_4 – small < 1200, δ_5 – medium (1200; 2400>, δ_6 – large >2400; δ_7 – left lane (inner lane); δ_8 – afternoon peak; N – sample size.

No.	f_4 [%]	Saturation flow models S_w [pcu/h]	For
1 lane	-	$1258.98 - 0.74 \cdot G \cdot \delta_9 + 159.63 \cdot f_1 - 222.44 \cdot f_4 \cdot f_5$	$f_5 > 0$, $f_1 > 0$
		$1482.32 - 1.08 \cdot G \cdot \delta_9 + 144.53 \cdot f_1 - 126.38 \cdot f_4 \cdot f_5$	$f_5 < 0$, $f_1 > 0$
2 lanes	<10	$1211.77 + 61.67 \cdot f_1 + 83.55 \cdot \delta_7$	1*
		$1700.62 + 49.16 \cdot f_1 + 49.42 \cdot \delta_7$	2&3*
	10÷25	$1440.75 + 198.95 \cdot \delta_7 - 1001.84 \cdot f_4 \cdot f_5$	1*, $f_5 > 0$
		$1812.46 + 123.06 \cdot f_1 + 220.25 \cdot \delta_7 - 1666.63 \cdot f_4 \cdot f_5$	2&3*, $f_5 > 0$
	≥25	$1237.09 + 164.57 \cdot f_1 - 117.16 \cdot f_4 \cdot f_5$	$f_5 > 0$
		$1563.62 + 139.92 \cdot f_1 - 765.86 \cdot f_4 \cdot f_5$	$f_5 < 0$

* level of urbanisation

Table 2: Regression models for basic saturation flow S_w for through traffic at rural intersection entries.

The analyses show that the saturation flow value is significantly affected by the following: delay during the initial interval f_1, length of the green signal G, the presence of saturated cycles δ_9, longitudinal slope f_5, location of lane δ_7, "hidden" share of heavy vehicles in the traffic f_4 and others (table 2). Generally, the larger the percentage of heavy vehicles in the traffic, the lower the average values of the saturation flows. For the segmented multiple regression models the authors obtained high determination coefficients R^2 ranging between 0.68 and 0.86. Different models predicting saturation flows S_w were divided with respect to longitudinal slope (f_5<0 or f_5>0) and urbanization level (1 or 2 and 3).

You will notice that at larger headways at which queued vehicles pass the stop lines (including at entries to rural intersections, in adverse weather conditions) the number of initial headways rejected while determining saturation flows will be less. Average time lost during the signal phase has similar values, but the dispersion of time lost is much higher in favourable than in adverse weather conditions and at rural intersections. The findings may be used in research into saturation flows, through a rational determination of the middle, saturated interval of the green signal.

3.2 Impact of saturation flow on the performance of signalised urban and rural intersections

Saturation flows are the basis for selecting control parameters and the calculation of the capacity of the lane or a group of lanes (Chodur & Others, 2004). Based on the capacity of the lane or a group of lanes you can evaluate traffic performance, i.e. calculate the mean value of delay, length of residual and maximum queues and stops, and then assess the level of service (Chodur & Others, 2004). These measures constitute the basis for assessing the adequacy of the adopted signal program, but also the geometry and organisation of traffic at the intersection.

Failure to accurately determine saturation flows causes considerable underestimation of delays sustained by vehicles, particularly in the case of traffic flows approaching or exceeding the capacity of the analysed lane groups (Figure 8). In order to show the relationship between average delays and saturation flows, the authors conducted simulation studies (Ostrowski, 2013) at an assumed value of demand flow and variable saturation flow and at different control parameters. It was assumed that the value of demand flow arises from the assumption that for saturation flow S = 1800 pcu/h, the demand flow will approach lane capacity ($X = Q/C \approx 1.0$). Five distinct average values of saturation flows S [E/h] were determined: 1400, 1600, 1800, 2000 and 2200. The values assumed result from an empirically established range of variability of saturation flows and illustrate the impact of weather conditions and intersection location. In the case of saturation flows corresponding to the standard values, vehicle delays do not exceed the acceptable service levels as defined in (Chodur & Others, 2004). In the event that for the same operating conditions the intersection is located in rural areas, delays will increase several times because capacity will be exceeded.

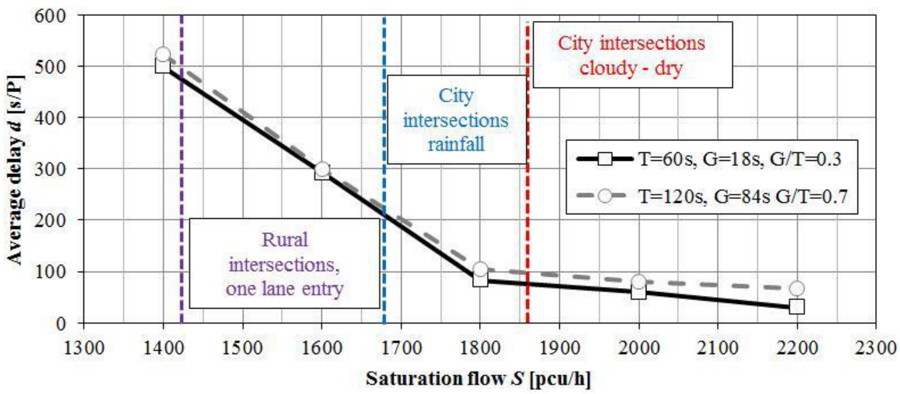

Figure 8: Relationship between average delay and saturation flow for selected control parameters (G/T = 0.3 and 0.7, T = 60 s and 120 s)

The saturation flows in the subsequent cycles are a random variable (number of vehicles which can be serviced during the green signal in a given cycle is subject to clearly visible fluctuations). Proper representation of the non-deterministic nature of the decongestioning of queues at entries to signalised intersections enables the assessment of the intersection in terms of reliability (loss and possible recovery of reliability). Research into and analysis of issues linking reliability and capacity and analyses of traffic performance are presented below and in papers (Ostrowski, 2010 & 2014, Tracz & Ostrowski, 2012, Ostrowski & Tracz, 2015, Ostrowski & Chodur, 2015).

3.3 Functional reliability of signalised intersections

Lane operation (exploitation) in saturation states is associated with quality of vehicle service and is renewable over time. In this case, the concept of renewal refers directly to theoretical renewal, when the renewed object reveals the same reliability as directly before overload (Migdalski 1982, Jaźwiński 1989). The correct functioning of traffic lights occurs in periods when demand flow Q is less than capacity C $(Q<C)$. These periods relate to the between-peak hours or traffic peaks at intersections which are not overloaded. Reliability analysis typically covers peak periods, when demand flow exceeds lane capacity and the effects are felt directly by vehicle drivers and indirectly by city residents. The time when overload will occur and its scope are very hard to predict as they depend on a number of factors. Application of reliability theory tools to issues related to overloads helps describe this phenomenon in the language of mathematics, and because of their use of parameters of mathematical statistics, it may become a useful tool in design and traffic management.

Reliability tests are associated with the following concepts:

1) **Critical value of traffic performance measure (plane 1). The value may arise from the following requirements:**
    - traffic management across the road network through the selection of critical values of traffic performance measures arising from the location of the intersection (urban or rural), entry geometry or traffic organization at the entry/group of lanes (e.g. length of an extra lane, distance to the adjacent signals, nuisance to the environment, etc.).
    - maintenance of the required level of service quality, which is important from the point of view of driver needs. Critical values may vary depending on intersection location (isolated or in a network, town or city, urban or rural), intersection's location in the city (zone), and on the entries and the directions on individual lanes.

Critical value of the traffic performance measure may be different from the point of view of road managers and vehicle drivers, but they may also coincide with their needs.

2) **Failure (plane 2).** It is very important to determine it early in the studies, since it closely relates to the description of the situation when a group of functioning lanes reveals reliable or unreliable service within the prescribed period of analysis. At this stage you should decide what measure of traffic performance to use in order to assess the reliability of lane operation. For example, when the measure involves delays and when its values are higher than the assumed marginal values, then such service shall be deemed unreliable ($d \geq d_{gr}$). By contrast, if $d \geq d_{gr}$, service will be deemed to be reliable.

The type of critical value of traffic performance measure (plane 1) should be different from the measure for which failure will be determined (plane 2). For example, the road manager prevents a situation where a queue of vehicles (residual or maximum) exceeds the length of the extra lane for left turn or when it extends to the next intersection. This queue can sometimes be very long and exceed the delay which queued drivers could sustain. Therefore, through the concept of failure and a related but distinct kind of measure (e.g. delay, number of waiting cycles etc.) you can also incorporate drivers' needs into traffic management. Thus, the application of reliability theory tools allows you to manage traffic in two intersecting planes, taking into account the variability of traffic performance measures.

Figure 9 below presents reliability analysis of the critical length of a residual queue of K_{rc} =20 pcu/cycle (plane 1) and delay for which four levels of reliability from N0 to N4 were determined, from which arise marginal values of delay (plane 2). This situation applies to the functioning of a lane which sustains periodic overloads in the subsequent days of the working week. Reliability analyses of the functioning of a signalised intersection at different demand flow levels (including in different weather conditions) are presented in detail in papers (Tracz & Ostrowski, 2012). The selected reliability analysis was performed for a real congested traffic lane carrying through traffic at a signalised intersection at different control parameters (Ostrowski, 2014).

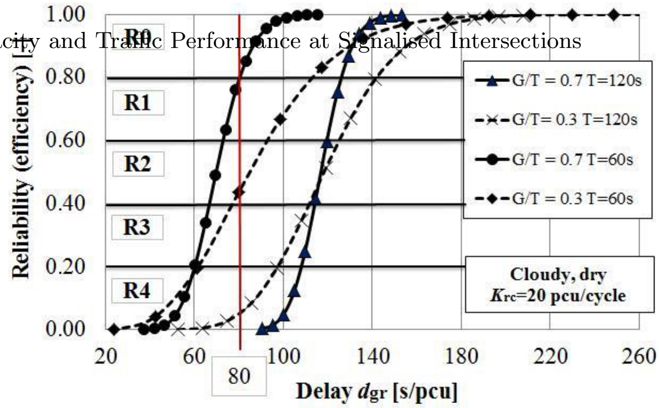

Figure 9: Reliability functions (efficiency) for marginal delays d_{gr} collected for *G/T* = 0.3 and 0.7 at *T* = 60 s and 120 s for the critical value of K_{rc} = 20 pcu/cycle. The reliability function also contains service reliability levels of R0 ÷ R4

The graph shows reliability curves (efficiency) determined for intensities causing overload (max. *Q/C≈1.4*) for selected values of the control parameters for which the curves plot a different course. The individual values of the average delay in cycles correspond to moments in time at which a residual queue (confirming overload) of less than 20 pcu/cycle occurs. It was assumed that drivers do not accept residual queues. In the graph, the curves with a smaller angle correspond to a higher share of the green signal in the cycle and the shift of the curves arises from the length of the signal cycle. The study was performed in favourable weather conditions, without precipitation.

The interpretation of the graph is as follows: For the road manager, the main thing is not to exceed the critical value of the residual queue of 20 pcu/cycle (plane 1). In turn, drivers do not accept the presence of residual queues and the resulting increased delays (plane 2). Adoption of marginal delay d_{gr} is a compromise between the needs of control and the needs of drivers and in reality translates into the occurrence of certain values of delay and residual queue lengths which will be unacceptable (unreliable) for a number of drivers. The d_{gr} values can be assumed for a specific group of users whose needs and acceptance of traffic performance have been examined empirically, e.g. through surveys. Then, the higher the d_{gr} value is, the lower the drivers' requirements as to traffic performance of a lane group that are reflected. This diversification of requirements may result from the location of the intersection (rural/urban, city centre/outskirts, town/city) or direction of traffic on the lane. Thus, for example the assumption of d_{gr} = 80 s means that with a modelled traffic load causing congestion, 80 % of drivers will deem the conditions to be unreliable when control parameters are *G/T=0.7* and *T=60s*, 45% when they are *G/T*=0.3 and *T*=60s, and 5% when they are *G/T*=0.3 and *T*=120s (Fig. 9). By introducing reliability curves determined in adverse weather conditions, you achieve a lower reliability at the same d_{gr} and at the same control parameters (Ostrowski, 2010, Tracz & Ostrowski, 2012).

The marginal values of selected traffic performance measures (Fig. 9 - delay) in plane 2 should be determined empirically. Empirical research should look at both quantitative traffic performance measures (delay, queues, etc.) and qualitative ones (surveys of drivers' feelings of the quality of a lane group's operation) (Ostrowski, 2014, Ostrowski & Tracz, 2015) and the links between them at different control parameters and intersection locations. Such an approach should be the starting point for further traffic management scenarios, e.g. diversification of d_{gr} values to accommodate the priorities occurring at rural intersections, on national roads (Chodur & Others, 2012). Lower d_{gr} values can then be assumed for the main direction and higher for lower road class entries.

The definition of critical value of the traffic performance measure (service quality classes) allows you to model the duration of congestion t_{cong} and functional availability of lane group AV (Ostrowski & Tracz, 2015). Knowledge of the reliability and functional availability of signalised intersections in a state of overload can be used in traffic management, during the design stage and further to inform drivers about travel time and the predicted traffic conditions, including the difficulties occurring in registered states of unreliability (ITS equipment. variable message signs, etc.).

4. Conclusions

The reliability of saturation flow estimates, and consequently, of capacity and traffic performance at entries to signalised intersections is crucial to the proper design and operation of intersections. Traffic control via signals is used increasingly more often at rural intersections, i.e. on roads with much higher speeds. These intersections have their own specificity in terms of geometry, traffic organisation and road user behaviour. This translates into instantaneous and average values of saturation flow. On the lanes carrying through traffic across rural intersections, the average saturation flows have much lower values (even up to approx. 30%) than at urban intersections. Therefore, at comparable demand flows and intersection design, traffic performance at rural intersections would be worse than at urban intersections (larger delays). Preventive measures can be taken at the design stage by selecting vehicle service parameters appropriate to rural intersections. The study provides a basis for determining the quantitative characteristics of these parameters.

The variability of demand flow intensity at intersections and the variability of service intensity provoke periodic overloads at entries to intersections, even with traffic volumes below capacity ($Q < C$) at longer time periods (e.g. 60 minutes). States of overload and the inherent long queues hinder movement not just at the specific object of road infrastructure e.g. an intersection, but not infrequently over a wider area. Congestion has become a rather common state, especially in urban street networks and is often very difficult or even impossible to prevent. It is therefore important to allow classification of states of congestion and determination of marginal measures of traffic performance to determine traffic performance and to determine their acceptability to road users and road managers. The findings in this field are possible with the use of the reliability theory. The paper shows an example of such an application of the theory.

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