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ANOMALOUS QUALITY OF SERVICE DETERIORATION AT ROUNDABOUTS CAUSED BY RAINFALL

ABSTRACT

The paper investigated the extent to which rainfall influences the quality of service delivery at multilane roundabouts using a novel quality of service approach. Quality of service is defined as how well roundabouts operate based on road users and road providers' perception of service quality. Delay and reserve capacity were used respectively as proxies for road users and road providers' perception of service quality. The entry and circulating traffic data were recorded continuously for eight weeks under dry, light, moderate, and heavy rainfall weather conditions at each surveyed roundabout, then collated, analysed and compared. Linear regression with dummy variable was used to model the roundabout entry capacity and a corrector factor was added to modify the regression function. The corrector factor considered different entry radii and entry angles of surveyed roundabouts. Multi-criteria quality of service table with travel time as proxy for road users and speed as proxy for road providers' perception of service delivery was developed from peak traffic data and used to determine the extent of deterioration. The multi-criteria table introduced in the paper is a clear departure from the speed-based criteria used in many studies. The results show a significant increase in time delay and a decrease in reserve capacity relative to rainfall. The paper has concluded that rainfall has an anomalous negative effect on the quality of service at multilane roundabouts. The findings could be used in a variety of ways in traffic management to predict the travel time at roundabouts under rainy conditions and to prescribe speed limits accordingly.

KEY WORDS

traffic; quality of service; roundabouts; delay; reserve capacity;

1. INTRODUCTION

According to the Florida State Department of Transportation (FDoT) Quality and Level of Service Handbook [7], the quality of service is a traveller-based perception of how well a service or facility is operating. The US Highway Capacity Manual (HCM) uses the term Level of Service (LOS) interchangeably with Quality of Service (QOS) to describe the service quality of a roadway, based on factors such as speed, travel time, manoeuvrability, delay, and safety. The level of service of a facility is designated with a letter, A to F, with A representing the best conditions and F the worst. The FDoT quality of service is based on multi perceptions whereas in HCM a single perception measure is preferred. Akcelik [1] suggested that when assessing the quality of service at roundabouts, delay and degree of saturation are useful parameters. According to Ben-Edigbe [5], the quality of service should be evaluated from the perspectives of service providers and the customers. Kita and Kouchi [12] proposed a discrete choice model for measuring the perceived quality of service of a driver. The method characterizes a driver's perception of the quality of traffic service as based on the microscopic traffic conditions encountered by the driver. In sum, the definitions of the quality of service and the level of service are indeed controversial. However, one thing is clear; they cannot be used interchangeably. The aim of the study is to investigate the quality of service deterioration caused by rainfall at roundabouts. The objectives are to determine and compare multi-perception measures (delay and reserve capacity) with and without rainfall. The delay and reserve capacity are used as proxies for road users and providers' perception of service delivery at roundabouts.

According to Attivor et al. [4] and Johnson [10] the road, vehicle type, traffic, and ambient conditions, as well as the ability to estimate the circulating vehicle speed are among others, the factors that can contribute to time delay at roundabouts. Delays at roundabouts are caused when vehicles slow down and yield to priority vehicles. But how exactly would delay play out during rainy conditions, this can be queried. Rainfall is one of the ambient conditions that affect the quality of service according to Mashros et al. [14]. Given a rainfall scenario at roundabouts, it is necessary to know whether the rainfall could cause a significant increase in delay and reduction in the reserve capacity. It is equally pertinent to know whether the changes caused by rainfall are anomalous or consistent relative to rainfall intensity. Rainfall has been classified in many studies as: light, moderate and heavy, with no clear-cut boundaries between these classes. It is often difficult to ascertain the contribution of rainfall intensity boundary overlapping. It is postulated that rainfall, irrespective of intensity boundary overlapping will have a negative effect on the quality of service at roundabouts. In any case, rainfall is classified into light rain intensity of ≤ 2.5mm/h, moderate rain (2.5 – 10 mm/h) and heavy rain (10 – 50 mm//h) as contained in previous studies.

2. QUALITY OF SERVICE AT ROUNDABOUTS

Roundabout geometric designs include, among others, entry width, circulating entry width, weaving width and weaving length, entry angle and entry radius, inscribed diameter, approach width and entry flare length. Entry width is the width at the point where the entry road meets the circle, usually measured perpendicular from the left edge to the right edge intersection line and the inscribed circle. Entry angle is geometric that represents the entering and circulating traffic stream conflict angle. Entry radius is the minimum radius of curvature of the outside curb of the entry. Approach width is the width of the approaching road along which the traffic stream travels towards the entry to the roundabout. Inscribed circle diameter is the diameter of the outer curb to the outer curb in which the central island diameter, the apron (where applicable) and the circulating roadway are inclusive. Circulatory roadway width is the width of the roadway around the central island along which the circulating flow travels.

The roundabout is an at-grade intersection that operates on the yield rule where vehicles entering the facilities give priority to the circulating vehicles (see *Figure 1*). Yield rule operates on the availability of gap within the circulating traffic prompting researchers to use the gap theory as an entry capacity estimation tool. Vehicles entering the facility do not have to stop completely at the stop line. Whenever a gap is available, the entry vehicle will look for the safe gap in the circulating traffic before accepting and entering the roundabout. However, sometimes when safe gaps appear in the circulating traffic stream, motorists ignore the gaps. It is not a mandatory requirement that drivers must enter the traffic stream when safe gaps appear. Others may elect to enter the roundabout when it is deemed unsafe thus making the gap theory approach somewhat questionable. Assuming no U-turn, where A denotes ahead, L denotes left-turning vehicles, and R denotes right-turning vehicles, it can be seen from *Figure 1* that at roundabouts the entry flowrate per arm features three turning movements (left (L), ahead (A) and right (R)). According to the Special Report [15], the entry capacity estimate must be obtained before a specific roundabout performance measure can be computed.

$$Q_E = q_L + q_A + q_R \tag{1}$$

where:

Q_E - entry capacity;

q_L, q_A and q_R - demand flows.

The entry capacity of a roundabout is the maximum rate at which vehicles can reasonably be expected to enter the roundabout from an approach during a given time under prevailing traffic and roadway conditions. This can be estimated in many ways that include weaving approach, empirical approach, and the theoretical method. When a merge area is closely followed by diverge area the weaving segments are formed. Weaving segments require intense lane-changing manoeuvres because drivers jockey to access lanes appropriate to

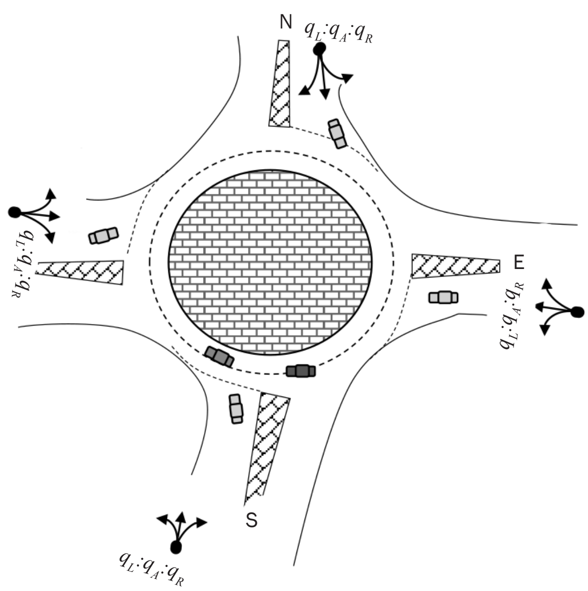


Figure 1 – Typical yield rule movement at roundabouts

their desired exit points. The most critical aspect of weaving segment is lane changing. Weaving approach relies on weaving and entry width parameters to determine the practical capacity. According to HCM [9], the weaving section will operate satisfactorily only if traffic on the approach road is well below the practical capacities of these approaches and the weaving section has one more lane than would normally be required for the combined traffic from both approaches. Hence, practical capacity of multi-lane roundabout can be estimated with *Equation 2*.

$$Q_p = \frac{280w\left(1+\frac{e}{w}\right)\left(1-\frac{p}{3}\right)}{1+\frac{w}{l}} \tag{2}$$

where:

Q_p - practical capacity;
w - weaving width;
p - proportion of weaving vehicles;
l - weaving length;
e - entry width.

The empirical capacity estimation method is based on linear or exponential regression of the entry flow on the circulating flow. Regression estimates are used to explain the relationship between one dependent variable and one or more independent variables. Regression (linear and exponential) methods have proven to be reliable and useful in many empirical studies partly because of geometric sensitivity. Geometrically-sensitive design methods are often preferred by modellers to achieve the required capacity targets while minimizing right-of-way impacts, avoiding high construction costs, and balancing the safety of all users; Lenters and Rudy [13]. Linear regression is the most basic and commonly used predictive analysis. In the United Kingdom [11], the linear regression method is preferred because it has inbuilt substantive geometric sensitivity and a correction factor. The linear function in *Equation 3* can be adjusted with *Equation 4* to take into account different entry angles and entry radii according to Kimber [11]

$$Q_E = F - \gamma q_c \tag{3}$$

$$k = 1 - 0.00347(\emptyset - 30) - 0.978\left(\frac{1}{r} - 0.05\right) \tag{4}$$

So that *Equation 3* becomes, $Q_E = k(F - \gamma q_c)$.

According to HCM 2010, the entry capacity of multilane roundabouts can be estimated with *Equations 5* and *6*;

$$Q_E = F \cdot e^{-\gamma q_c} \tag{5}$$

$$Q_E = 1130 \cdot e^{-0.0007 q_c} \tag{6}$$

where:

Q_E - entry capacity;
F - maximum flowrate;
γ - circulating flowrate coefficient;
q_c - circulating flowrate;
k - correction factor;
e - Napier constant;
r - entry radius;
$\emptyset$ - entry angles.

Exponential regression is a non-linear model that can be log transformed into a linear model. However, choosing a negative exponential equation based on gap-acceptance theory to define the roundabout entry capacity makes the equation nearly asymptotic to the x-axis. So that when circulating traffic volume is high, it becomes unreliable to model small traffic flowrate. In any case, the operating characteristics of roundabouts are influenced by their geometric elements and have often led to separate entry capacities. Brilon et al. [6] argued that the capacity equations should not be transferred internationally; instead, each country has to find a solution of its own, because of the differences in driver behaviour in different countries.

Within a transportation system, there are two kinds of quality of service delivery: structural and functional. The structural quality of service (SQS) deals with the wellness of transportation fixed facilities, whereas the functional quality of service (FQS) is concerned with traffic flow entity and the control system. The Highway Capacity Manual Special Report [15] presents the level of service (LOS) as a qualitative measure that "characterizes operational conditions within a traffic stream and their perception by motorists and passengers." The draft report suggests that LOS is a qualitative measure of operational conditions and motorists' perception of service delivery. According to the Special Report [15], the degree of saturation, delay and queue length are typically used to estimate the quality of service at a given roundabout. However, the report advises that studies should estimate many parameters in order to obtain the broadest possible quality of service evaluation. Consequently, in this paper, the quality of service at roundabouts is taken as a measure of delay and reserve capacity. Keep in mind that the reserve capacity is an inverse of the degree of saturation.

Reserve capacity is defined in HCM as the unused entry capacity of a movement, or the difference between the actual capacity for a movement and the flowrate for the movement, hence;

$$R_Q = Q_E - q_d \tag{7}$$

where:

R_Q - reserve capacity;
Q_E - entry capacity;
q_d - demand flow.

Reserve capacity is a measure of the overall goodness of the physical design features of the intersection. It provides the best indicator of the roundabout performance and spare reserve availability. It should not be confused with practical reserve capacity that is commonly used to measure the available spare capacity at a traffic signal junction. The concept of a practical reserve capacity has long been used in previous studies as a useful measure of the operational performance of individual signal-controlled junctions. Reserve capacity is one of the parameters used for the measurement of unsignalised intersection performance according to Wong [17]. Reserve capacity is a factor that can be used to determine the total number of vehicles entering the roundabout before the saturation condition is attained. The estimation of the prevailing reserve capacity can be used in traffic management to divert traffic to other routes during construction, accident and road closures. Since reserve capacity is a direct derivation of entry capacity, it follows that the entry capacity shrinkage would result in reserve capacity loss.

HCM 2010 rightly prescribes delay as the primary measure of effectiveness for roundabouts and intersections. Geometric delays are defined as those delays encountered during travel through the intersection. Geometric delays are measured as the time it takes a vehicle to traverse the intersection from entry point to exit point. The geometric delay excludes the queuing time at the roundabout entry; it could be more than the delay in congestion with the exception of when the traffic approaches the capacity [11]. Control delay is the time a driver decelerates to a queue, stays in the queue, while at the front of the queue, waits for an acceptable gap and accelerates out of the queue [16]. Traffic delay or yield delay occurs when entering vehicles are delayed by the presence of vehicles already in the intersection. Geometric delay can be taken as any delay experience when a vehicle is traversing the roundabout in the absence of any other vehicles if the driver could identify that they are traversing the roundabout in isolation [2, 11, 18]. The value is usually high for a stopping vehicle because of the time it takes to accelerate to the design speed of the roundabout (Rodegerdts, 2007). HCM 2010 identified three types of delay at roundabouts, namely: traffic delay (d_1), control delay (d_2), and geometric delay (d_3) as shown in *Equation 8*.

$$d = \frac{3600}{Q_E} + 900T\left\{\left(\frac{q_d}{Q_E} - 1\right) + \sqrt{\left(\frac{q_d}{Q_E} - 1\right)^2 + \frac{\left(\frac{3600}{Q_E}\right)\left(\frac{q_d}{Q_E}\right)}{450T}}\right\} + 5 \tag{8}$$

where:

$$d = d_1 + d_2 + d_3; \quad d_1 = \frac{3600}{Q_E};$$

$$d_2 = 900T\left\{\left(\frac{q_d}{Q_E} - 1\right) + \sqrt{\left(\frac{q_d}{Q_E} - 1\right)^2 + \frac{\left(\frac{3600}{Q_E}\right)\left(\frac{q_d}{Q_E}\right)}{450T}}\right\};$$

$$d_3 = 5\text{ s}.$$

$$Q_{95} = 900T\left\{\left(\frac{q_d}{Q_E} - 1\right) + \sqrt{\left(\frac{q_d}{Q_E} - 1\right)^2 + \frac{\left(\frac{3600}{Q_E}\right)\left(\frac{q_d}{Q_E}\right)}{150T}}\right\}\frac{Q_E}{3600} \tag{9}$$

where:

d - delay;
T - time period (0.25 for a 15-minute period);
Q_E - entry capacity;
q_d - demand flow;
Q_{95} - 95th percentile queue;
d_1 - traffic delay;
d_2 - control delay;
d_3 - geometric delay.

Note that model *Equation 9* has no fixed time allowance for geometric delay, whereas, in HCM 2010, 5 s allowance is made for the geometric delay in *Equation 8*. Akcelik, [1] model equation with allowance for variation over time the equation is similar to HCM 2010 model equation:
Average delay,

$$d = d_m + 900T\left(\left[\frac{v}{c} - 1\right] + \sqrt{\left(\frac{v}{c} - 1\right)^2 + \frac{[8k]\frac{v}{c}}{c \cdot T}}\right) \tag{10}$$

Overflow parameter, $k = \frac{c \cdot d_m}{3600}$; Yield line delay,

$$d_m = \frac{e^{\lambda(\alpha - \Delta)}}{\varphi \cdot v_c} - \alpha - \frac{1}{\lambda} + \frac{\lambda\Delta^2 - 2\Delta + 2\Delta\varphi}{2(\lambda\Delta + \varphi)}$$

$$\varphi = 0.75(1 - \Delta \cdot v_c); \quad = \frac{\varphi \cdot v_c}{1 - \Delta \cdot v_c}; \quad \Delta = 2s$$

$$c = \frac{3600(1 - \varphi)v_c e^{-\lambda(t_c - \zeta)}}{1 - (e^{-\lambda t_f})}$$

$$\lambda = \frac{(1 - \phi)v_c}{1 - \zeta v_c}$$

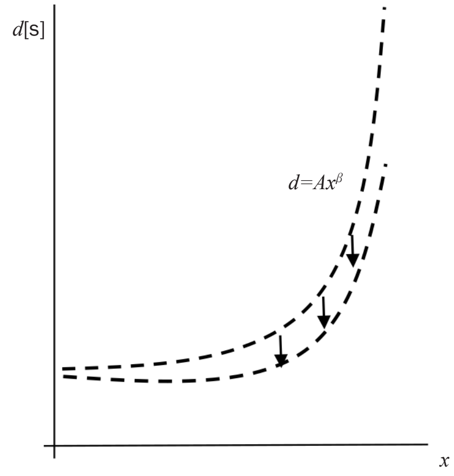
For $d = d1 + d2 + d3$; $d_1 = d_m$;

$$d_2 = 900T\left(\left[\frac{v}{c} - 1\right] + \sqrt{\left(\frac{v}{c} - 1\right)^2 + \frac{[8k]\frac{v}{c}}{c \cdot T}}\right); \quad d_3 = 0$$

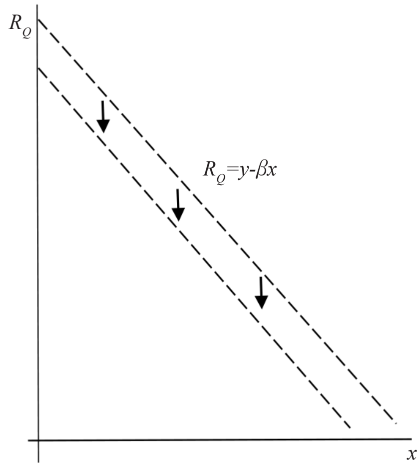
where:

d - delay;
T - time period (0.25 for a 15-minute period);
v - traffic volume;
q_d - demand flow;
d_1 - traffic delay;
d_2 - control delay;
d_3 - geometric delay;
e - Napier constant;
c - entry capacity;
v_c - circulating flow;
t_c - critical gap;
t_f - follow-up headway;
φ - proportion of bunched vehicle in circulating stream;
τ - minimum circulating stream headway.

Interestingly, road providers and users consider delay as a key parameter when road providers should be interested in the reserve capacity for the purpose of management planning. This can be argued. The degree of saturation is a ratio of demand to capacity and a reciprocal of reserve capacity. The relationships between the degree of saturation, delay, and reserve capacity are illustrated in *Figure 2*. *Figure 2a* shows the hypothetic relationship between the delay and the



a) Degree of saturation vs Delay



b) Degree of saturation vs Reserve capacity

Figure 2 – Delay and reserve capacity vs degree of saturation

degree of saturation whilst *Figure 2b* shows the hypothetic relationship of the reserve capacity (R_Q) and the degree of saturation (x).

Generally, the roundabout quality of service delivery deals with the assessment of the functional and structural quality of service provided by road providers for the users. Functional quality of service is premised on the idea that the drivers and road providers' perceptions of quality are key assessment parameters. In this regard, delay, reserve capacity and degree of saturation are an important quality of service parameters. Delay is an important parameter that is used in the performance evaluation of intersections. It is a key parameter used to measure the performance of an intersection. Delay in a roundabout can be defined as the time spent on traversing a roundabout in excess of traffic-free flow at the roundabout and it was the primary service delivery for a roundabout [16]. Reserve capacity is a measure of sufficiency. Reserve capacity values alert road providers to areas where traffic mitigation measures should be considered for deployment because once capacity is reached, congestion sets into the traffic stream. The degree of saturation is the volume to capacity ratio. Delay is always present at the roundabout because of the geometric design. The roundabout geometric design is one of the major factors that influence delay [3]. The value is usually small for a small roundabout, but large for a large diameter roundabout and it could be significant. In sum, delay and reserve capacity are parameters that provide a unique perspective on the functional quality of service at roundabouts under prevailing conditions. Therefore, it is useful to estimate these parameters in order to obtain the broadest possible assessment of the roundabout qualitative performance. In all cases, the roundabout entry capacity estimate must be obtained.

3. DATA COLLECTION

The study data were collected at four selected roundabouts in Durban, South Africa. The intensity of rainfall varied from province to province in South Africa. The amount of precipitation in South Africa varies tremendously, which makes it difficult to predict the variation in the amount of rainfall accurately. Rainfall usually occurs during the months of November to March. Three classes of rainfall intensity were recorded; light rain-LR (intensity < 2.5 mm/h), moderate rain-MR (2.5 ≤ intensity < 10 mm/h) and heavy rain-HR (10 ≤ intensity < 50 mm/h). The study used automatic traffic counter to collect the entry and circulating traffic volume, headway, and vehicle-type data continuously for eight weeks at each site. Typical site layout is shown in *Figure 3*. Note that ATC denotes automatic traffic counter, RGS denotes rain gauge station. Surveyed roundabouts have bituminous surfaces, functional and effective drainage, about one kilometre from the rain gauge station. Although not part of the studies, poor driver visibility, changing driver behaviour, reduced speed and general discomfort and anxiety were observed in passing at survey sites

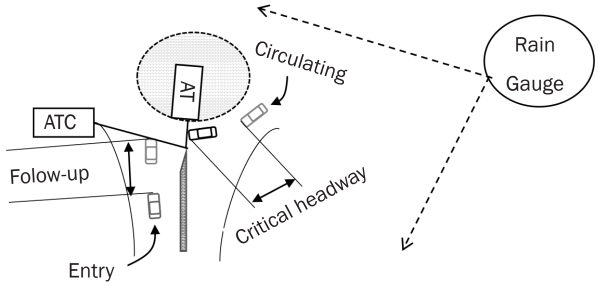


Figure 3 – Typical site setup

Table 1 – Summary of roundabouts observed parameters

Site		01	02	03	04
Approach half-width [m]		7	8	7	7
Entry width [m]		10	10	12	10
Entry radius [m]		30	45	30	35
Inscribed diameter [m]		50	55	50	55
Entry angle [°]		50	45	55	45
Correction factor		95	98	93	97
Passenger cars	Entering vehicles	94	91	89	95
	Circulating vehicles	92	97	94	93
Light vans	Entering vehicles	4	8	9	4
	Circulating vehicles	7	1	5	6
Heavy goods vehicles, trucks and buses	Entering vehicles	2	1	2	1
	Circulating vehicles	1	2	1	1

during rainy conditions, suggesting that differential traffic flowrate would result from rainfall. See *Table 1* for a summary of roundabout parameters.

4. RESULTS AND DISCUSSION

Typical entry and circulating flows depicting dry, light, moderate and heavy rainfall are shown in *Table 2*. Collate peak and off-peak entry and circulating traffic flows under dry, light, moderate and heavy rainy weather conditions. Peak data are used to develop criteria table and off-peak data are used for assessing prevailing conditions. Use appropriate passenger car equivalent (PCE) values to convert vehicles per hour to PCE per hour. Modify PCE values if at all necessary and note the effect of such modifications on study outcomes. The criteria table is a management tool developed to address road users and providers' perception of service delivery. It enables changes resulting from rainfall to be evaluated and appropriately classified. The proposed criteria table in this paper has six classes (A to F) just like HCM 2010 and SIDRA level of service. Class A is the best and F is the worst. Once entry capacity is reached, the degree of saturation, x = 1 and the appropriate class is F. When x is less than one but not greater than 0.85 the class is E (assuming 0.85 is the threshold), the remainder classes are; Class D ($x \leq 0.85$); Class C ($0.60 \geq x \leq 0.70$); Class B ($0.50 \geq x \leq 0.60$) and Class A ($x \leq 0.50$). Unlike HCM and SIDRA, the columns in the criteria table will include delay and reserve capacity. Using *Equation 6* and the appropriate degree of saturation, delays can be estimated for each class in the criteria table. This remainder of this section is presented using a stepwise approach for ease of explanation and clarity. Note

Table 2 – Typical entry and circulating flows

Entry flow [PCE/h]					Circulating flow [PCE/h]				
Period	Dry	Rainfall			Period	Dry	Rainfall		
		L	M	H			L	M	H
1	1,286	1,093	1,076	902	1	828	521	712	629
2	1,385	1,004	988	969	2	607	780	712	703
3	1,382	1,092	960	938	3	852	821	492	657
4	1,123	1,006	960	954	4	1,053	667	501	734
5	1,181	1,100	984	874	5	1,070	676	897	593
6	1,490	1,228	1,200	1,120	6	787	732	619	729
7	1,464	1,288	1,101	1,027	7	796	664	679	813
8	1,336	1,262	1,097	919	8	864	734	463	1,021
9	1,063	1,001	1,076	906	9	1,048	842	741	744
10	1,075	1,099	912	956	10	1,202	955	969	693
11	1,665	1,099	936	1,004	11	607	928	888	624
12	1,123	876	888	946	12	979	1,079	979	864
Average	1,298	1,096	1,015	960	Average	891	783	721	734

Table 3 – Typical peak entry and circulating flow veh/km for dry weather

Q_E	1,286	1,385	1,382	1,123	1,181	1,490	1,464	1,336	1,063	1,075	1,665	1,123
Qc	828	607	852	1,053	1,070	787	796	864	1,048	1,202	607	979

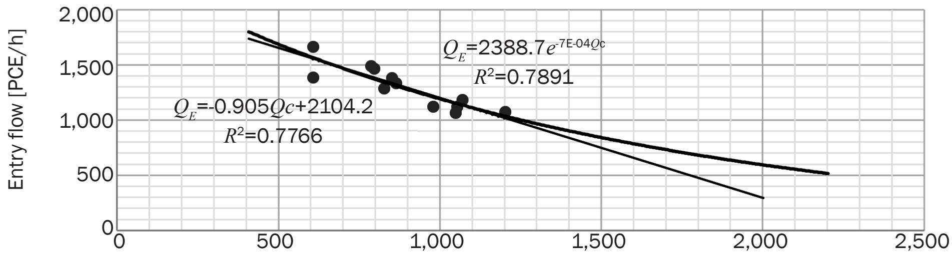


Figure 4 – Linear vs Exponential Regression Functions

that peak and off-peak traffic data were analysed. Dry weather peak traffic data were used to develop the criteria table whilst off-peak data were used to determine the traffic performance under dry weather, light rainfall, moderate rainfall, and heavy rainfall conditions. Note also that linear and exponential regression methods were tested for suitability.

Step 1: Use the peak traffic data in *Table 3*. Dry weather data were used to determine linear and exponential functions as illustrated in *Figure 4*.

Step 2: Test the model equations for statistical fitness. The coefficients of determinant (R^2) are above 0.5, which indicates that the model equations are reliable. T-test results are greater than 2.2 at 95% level of confidence which shows that the parameters used are significant, and the F tests are greater than 4 which indicates that the model equations did not occur by chance. Therefore, the statistical results are satisfactory and the model equations are accepted.

Step 3: Apply correction factor (k) to *Equation 11*, then compare the computed entry capacity outcomes from the linear and exponential functions

$$Q_E = 2104 - 0.905Qc \quad R^2 = 0.78 \tag{11}$$

where entry angle is 40° and entry radius is 40 m, using *Equation 3*, the correction factor, k=0.98; apply it to the linear model *Equation 11*, so that it becomes; Q_E=2061–0.89Qc.

Linear entry capacity equation per lane,

$$Q_E = 1030 \text{ PCE/h/lane} \tag{12}$$

$$Q_E = 2388e^{-0.0007Qc} \quad R^2 = 0.79 \tag{13}$$

Exponential entry capacity equation per lane,

$$Q_E = 1194e^{-0.0007Qc} \tag{14}$$

Chi-square test suggests that there is a significant difference between the linear and exponential entry capacity because of the calculated χ^2(22.5)>tabulated χ^2(3.84). It is suggested that exponential regression methods overestimate the entry capacity due to geometric sensitivity weakness and the absence of y-intercept. Of course, if the interest is finding the slope and intercept then the exponential curve has to be converted to linear function using logarithms. For example when; Q_E=1194 e$^{-0.0007Qc}$→logQ_E= log 1194 –0.0007logQc→y-intercept=log 1194.

Step 4: Comparative assessment of empirical exponential and HCM 2010 exponential equations where: HCM 2010 entry capacity equation, Q_E= 1130e$^{-0.0007Qc}$; empirical exponential equation, Q_E=1194e$^{-0.0007Qc}$.

Chi-square test suggests that there is no significant difference between the empirical exponential and HCM 2010 exponential equation because of the calculated χ^2 (3.62) < tabulated χ^2(3.84).

Step 5: For the purpose of developing a criteria table, divide the degree of saturation into six classes A=0.50, B=0.60, C=0.70, D=0.85, E=1.0. Note that HCM 2010 and International Capacity Utilization (ICU) have 0.50 as the starting class A. Thereafter, ICU uses a constant 0.10 interval from class B(0.60) to F(1.0), whereas HCM 2010 uses irregular intervals from class B(0.7) to E(1.0). Note that the threshold value of 0.85 is not a sacrosanct value, it can be duly adjusted. ICU threshold value of 0.90 is classified as E, whereas HCM 2010 threshold value of 0.92 is classified as D and SIDRA's threshold value of 0.95 is classified as class D.

Step 6: Then, estimate the delay and queue; for example where entry time headway of 3.5 s and degree of saturation =0.5, then, $d = \frac{3600}{1030} +$

$$900 \cdot 0.25\left\{(0.5-1) + \sqrt{(0.5-1)^2 + \frac{\left(\frac{3600}{1030}\right)(0.5)}{450 \cdot 0.25}}\right\} + 5 = 11.9\,\text{s}$$

$$Q_{95} = 900 \cdot 0.25\left\{(0.5-1) + \sqrt{(0.5-1)^2 + \frac{\left(\frac{3600}{1030}\right)(0.5)}{150 \cdot 0.25}}\right\}\frac{1030}{3600} =$$

4 vehicles

Table 4 – Computed criteria table 1st Iteration

Class	Degree of saturation (x)	Delay [s] ±20%	R_Q = 1 - x
A	0.50	d ≤ 12	0.50
B	0.60	12 ≥ d ≤ 16	0.40
C	0.70	16 ≥ d ≤ 21	0.30
D	0.85	21 ≥ d ≤ 30	0.15
E	1.00	30 ≥ d ≤ 48	0.00
F	N/A	d > 48	N/A

Compute delay and queue length for each class based on the degree of saturation and construct a criteria table as shown in *Table 4*. It is clear from the table that class A, B, and C have variance overlapping, hence the class distribution must be adjusted to prevent variance overlapping.

Step 7: Criteria table adjustment. Repeat step 6 using class A=0.50, B=0.70, C=0.80, D=0.90, E=1.0 and note that the threshold value has been adjusted to 0.90 along ICU line. *Table 5* shows criteria table, the second iteration based on adjusted values. It is clear from *Table 5* that class A, B, and C variance overlapping has now been removed, hence the adjusted class distribution is a better criteria table fit.

Step 8: Compare the computed criteria table values with HCM 2010 LOS, SIDRA and ICU LOS as shown in *Table 6*. As shown in the table, all the criteria tables (FQS, HCM, ICU, and SIDRA) have six classes (A to F) with different degree of saturation intervals. In all cases, the delay values increase relative to decrease in the degree of saturation values. SIDRA has the highest threshold value of 0.95 compared to HCM value of 0.92, ICU and FQS have the same value of 0.90. ICU has no delay values. Chi-square test suggests that there is no significant difference between the empirical delay values and HCM 2010 delay values because of the calculated χ^2(1.12) < tabulated χ^2(3.84). Note that in *Table 6*, FQS denotes the functional quality of service from empirical data.

Step 9: Repeat steps 1, 2 and 3 using off-peak traffic data to estimate the capacity values for dry weather, light, moderate and heavy rainfall. Introduce dummy variable (zero for dry and 1 for rainfall) into the linear regression as shown in *Table 7*. Thereafter, estimate the capacity values for dry, light, moderate and heavy rainfall conditions. For example, at site 01: where the model equation is Q_E=2280-1.17 Qc-146 R_L, modify with k (0.95) so that Entry capacity equation becomes;

$$Q_E = 2166 - 1.11Qc - 139R_L \quad (15)$$

Entry capacity per lane for dry weather, Q_E=921 PCE/h/lane

Entry capacity per lane for light rainfall, Q_{LR}=861 PCE/h/lane

Step 10: Estimate off-peak time headway

Entry time headway per lane for dry weather, Q_D=3600/921=3.91s

Entry time headway per lane for light rainfall, Q_{LR}=3600/861=4.18s

Step 11: Repeat step 6. Estimate off-peak delay and queue length caused by rainfall where the degree of saturation for dry weather is about 0.71 and 0.84 for rainfall is. For example, at site 01;

Delay, d_D≈17 s; Queue, d_D=6 veh: Delay, d_{LR}≈27 s; Queue, d_{LR}=10 veh

Table 5 – Computed criteria table 2nd Iteration

Class	Degree of saturation (x)	Delay [s] ±20%	Queue [veh]	R_Q = 1 - x
A	0.50	d ≤ 12	3	0.50
B	0.70	12 ≥ d ≤ 16	6	0.30
C	0.80	16 ≥ d ≤ 21	9	0.20
D	0.90	21 ≥ d ≤ 30	13	0.10
E	1.00	30 ≥ d ≤ 48	20	0.00
F	N/A	d > 48	N/A	N/A

Table 6 – Functional quality of service criteria table comparisons

Class	Average Delay [s]			Reserve Capacity (R_Q)	Degree of Saturation (x)			
	FQS	HCM 2010	SIDRA	FQS	FQS	HCM 2010	ICU Method	SIDRA
A	$d \leq 12$	$d \leq 10$	$d \leq 10$	0.50	0.50	0.50	0.50	$x \leq 0.85$
B	$12 \geq d \leq 16$	$10 \geq d \leq 15$	$10 \geq d \leq 20$	0.30	0.70	0.70	0.60	
C	$16 \geq d \leq 21$	$15 \geq d \leq 25$	$20 \geq d \leq 35$	0.20	0.80	0.85	0.70	
D	$21 \geq d \leq 30$	$25 \geq d \leq 35$	$35 \geq d \leq 50$	0.10	0.90	0.92	0.80	$x \leq 0.95$
E	$30 \geq d \leq 48$	$35 \geq d \leq 50$	$50 \geq d \leq 70$	0.00	1.00	1.00	0.90	$x \leq 1.0$
F	$d > 48$	$d > 50$	$d > 70$	N/A	N/A	N/A	1.00	$x > 1.0$

Table 7 – Summary of delay, queue length reserve capacity, and FQS outcomes

Site	Model equations	Entry flowrate [PCE/h/lane]		Entry time headway [s]		Delay [s] / Queue [veh]		Reserve capacity R_Q (FQS class)	
		Dry Q_E	Rain Q_E	Dry Q_E	Rain Q_E	Dry	Rain	Dry	Rain
01 k=0.95	Q_E = 2,280-1.17Q_c-146R_L	921	861	3.9	4.2	17 / 6	27 / 10	20% (C)	10% (D)
	Q_E = 2,211-1.09Q_c-302R_M	893	770	4.0	4.7	18 / 6	29 / 10	20% (C)	10% (D)
	Q_E = 2,159-1.04Q_c-345R_H	872	732	4.1	4.9	18 / 6	30 / 10	20% (C)	10% (D)
02 k=0.98	Q_E = 1,967-1.23Q_c-71R_L	819	790	4.4	4.6	19 / 6	29 / 9	20% (C)	10% (D)
	Q_E = 1,908-1.17Q_c-191R_M	795	715	4.5	5.0	19 / 6	31 / 9	20% (C)	<10% (E)
	Q_E = 1,805-1.06Q_c-264R_H	752	642	4.8	5.6	20 / 6	33 / 10	20% (C)	<10% (E)
03 k=0.93	Q_E = 1,985-0.99Q_c-110R_L	785	741	4.6	4.9	20 / 6	30 / 10	20% (C)	10% (D)
	Q_E = 2,038-1.04Q_c-270R_M	806	699	4.5	5.1	19 / 6	31 / 9	20% (C)	<10% (E)
	Q_E = 1,900-0.92Q_c-379R_H	751	601	4.8	6.0	20 / 6	35 / 9	20% (C)	<10% (E)
04 k=0.97	Q_E = 1,902-1.19Q_c-61R_L	784	758	4.6	4.8	20 / 6	29 / 10	20% (C)	10% (D)
	Q_E = 1,716-0.88Q_c-149R_M	707	646	5.1	5.6	21 / 6	33 / 9	20% (C)	<10% (E)
	Q_E = 1,556-0.61Q_c-183R_H	642	565	5.6	6.4	23 / 6	36 / 9	10% (D)	<10% (E)

Delay, d_D≈18 s; Queue, d_D=6 veh: Delay, d_{MR}≈29 s; Queue, d_{MR}=10 veh

Delay, d_D≈18 s; Queue, d_D=6 veh: Delay, d_{HR}≈30 s; Queue, d_{HR}=10 veh

Step 12: Determine the reserve capacity and quality of service as shown in *Table 7*. Results show that the average increase in delay due to light rainfall is about 10 s, moderate rainfall about 12 s and heavy rainfall about 13 s. The average reserve capacity is reduced by about 10% with one class reduction. It appears that heavy rainfall would cause bunching and traffic congestion that is characterized by slower speeds, increased delay time, and increased traffic queue. Note that in *Table 7*, LR - light rain, MR - moderate rain, HR - heavy rain, FQS - functional quality of service; R_Q reserve capacity. The findings could be used in a variety of ways in traffic management to predict travel time at roundabouts under rainy conditions and prescribe speed limits accordingly.

5. CONCLUSION

The aim of the study has been to investigate the extent of anomalous quality of service deterioration caused by rainfall. The study objectives were to determine the delay, reserve capacity; hence the quality of service for dry weather, light, moderate and heavy rainfall conditions. In the paper the empirical criteria table was developed and used to assess traffic performance under prevailing conditions. At the onset, the paper presented the quality of service as different from the level of service and argued that the quality of service is a multi-parameter measure. The analyses and comparisons made in this investigation confirmed a postulation that rainfall may cause the anomalous quality of service deterioration. The comparison of dry weather and rainy conditions indicate that the empirical method is the most suitable capacity estimation method. Results show that there are differences between linear and exponential regression entry capacity estimation methods. Further analyses of the effect of rainfall on the quality of service reveal that increasing rainfall intensity does not significantly affect the quality of service when traffic flowrate is nearing or at capacity. However, it is simply logical that heavy rainy conditions are required for significant consistency in quality of service deterioration and higher volume of capacity ratios for significant delay differentials. In terms of reserve capacity, increasing volume/capacity, ratio has a reciprocal influence on it. Based on the empirical findings, it is correct to conclude that the effect of rainfall on quality of service deterioration at roundabouts is anomalous. It is also correct to suggest that rainfall will cause an increase in travel delay and a decrease in reserve capacity.

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