DEPARTMENT OF CIVIL ENGINEERING,
AHMADU BELLO UNIVERSITY,
ZARIA, NIGERIA.

A STUDY OF ROUNDABOUT - MODE OF OPERATION AND PERFORMANCE

Thesis submitted in Partial Fulfillment of the requirements for the award of Master of Engineering Degree

BY

AGILI EMMANUEL OFILI

December, 1981

A B S T R A C T

In deciding which type of control to provide at an intersection, there should be some criteria used. A typical criterion used in some situations is to minimize the delay to vehicles.

While there are established methods for estimating dalays at signal control, there does not seem to be any standard or well established method by which the delay to vehicles at roundabout approach can be obtained. This thesis presents a procedure by which this delay can be obtained. The signal control is used here as a typical example of any other control, the delay produced by the two controls (Roundabout and Signal Control), at various demand flows and turning problems are obtained and can then be compared.

Empirically, by modelling the roundabout as a series of "T" junctions, with circulating flow having priority over entering flow, it has been shown in the literatures by previous researchers that the capacity prediction equation for the roundabout entries can be represented by the linear entry/circulating flow relationship. This equation gives the saturated entry flow in terms of circulating flow and some geometric parameters, basically entry width and flare.

Here, with this entry capacity as obtained above, use is made of the low definition rectangular type of demand pattern

ACKNOWLEDGEMENT

I wish to express my sincere gratitude to all my "Friends" for their moral support without which this thesis would never have been completed and ofcourse to Dr. A.S. Alfa for his criticisms and guidance in the course of this work.

I also acknowledge with thanks the assumed permission of all the authors whose works are quoted in this thesis.

TABLE OF CONTENTS

APPENDICES

TABLES

FIGURES

Figs. Cont.

1. INTRODUCTION

As the flow of traffic through an intersection increases, it becomes necessary to introduce some form of controls which must be properly selected, otherwise the flow through the intersection will drop. Two of the most commonly employed controls are the roundabout and the signal control. There has yet not been a clear procedure for comparing the performance of the roundabout to that of the signal controlled intersection. Basically the main objective in this thesis will be to develop a procedure by which the delays to vehicles at an intersection controlled by either of the two controls (roundabout and signal control) could be compared, with the aim of selecting the control that produces the lower delay for a particular geometric and traffic conditions. To achieve this, the mode of operation of the roundabout must be well understood so that the correct assumptions, which when applied, will result in the correct estimation of capacity, can be made. Having obtained the capacity, the delay can then be obtained.

1.1 Roundabout

1.1.1 Mode of operation

initially conventional roundabout were designed with large central islands and parallel-sided weaving sections and entries but newer designs (offside-priority) have smaller islands with wide circulation widths and flared entries.

(See Figs. 1a and 1b). There has been several attempts to establish usable formulae for roundabout capacity prediction, and until recently most of them were based on the "weaving section", the area into which entering circulating traffic merge.

Wordrop (1957) developed a formula which gave the capacity of the weaving section - the maximum expected flow of vehicles along the weaving section - in terms of the geometric parameters defining it's size and shape and the proportion of the traffic which had to "weave". The basic assumption within this approach, that capacity is controlled by "weaving" rather than by the merging/gap-acceptance manoeuver was valid when the formula was developed:- that each entering stream at a round-about have the same priority as traffic already in the weaving section. In the sixties "Give-Way-To-The-Vehicle on the left" rule at roundabouts was introduced, thus the entering traffic now has to give way to circulating traffic.

Consequently the basis for the weaving section formula no longer exists. Instead each approach now operates in a similar manner to a major/minor junction of two one-way road, traffic already in the round-about having priority over that arriving on an approach. Each driver waiting to enter a roundabout must assess the size of the gaps in the priority stream and will move into the roundabout only when a sufficiently large gap presents itself. It then becomes necessary to deal in terms of the capacities of entries,

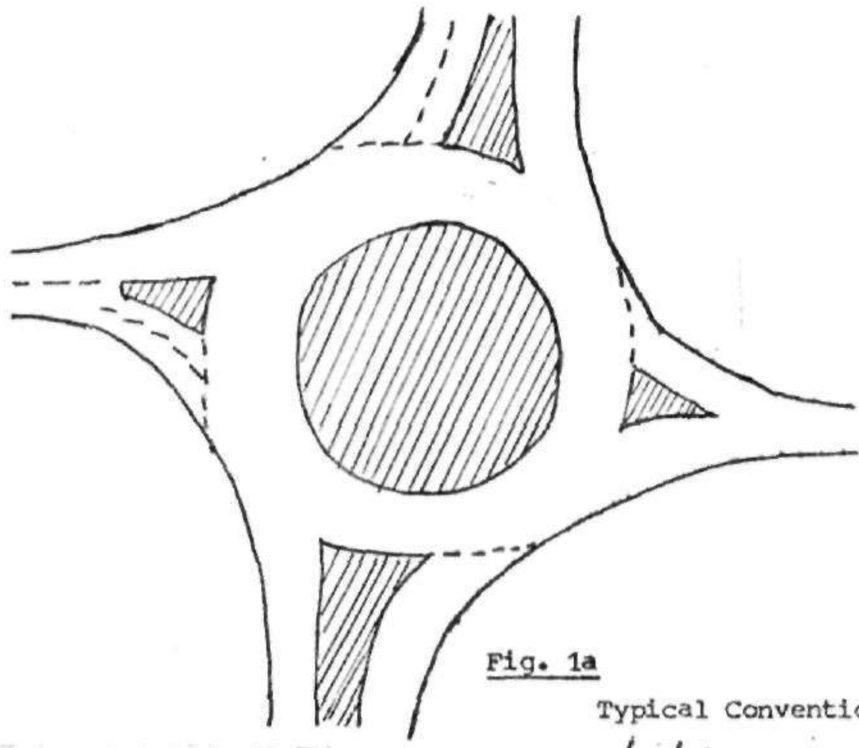


Fig. 1a

Typical Conventional Roundabout

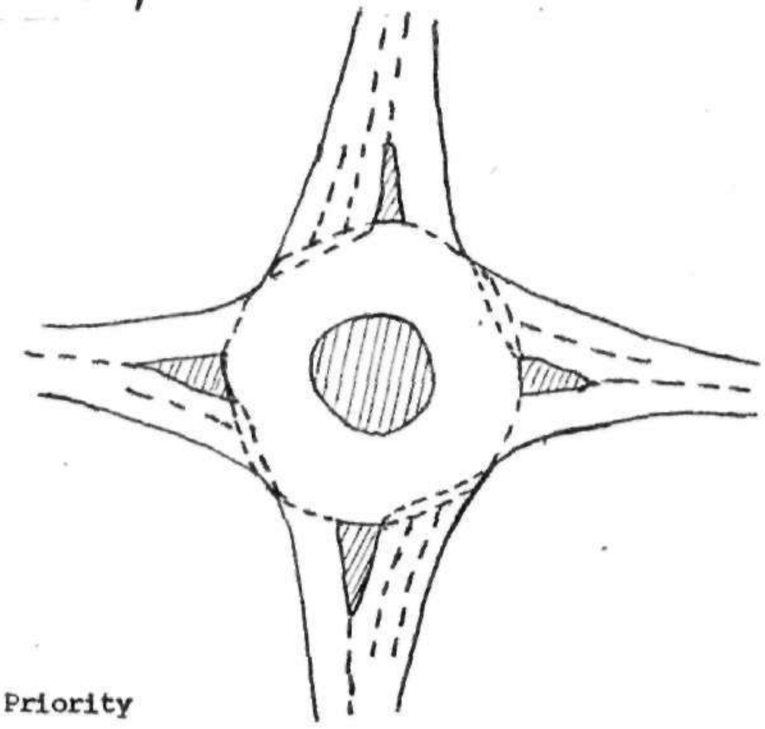


Fig. 1b

Typical Offside Priority Roundabout

TYPES OF ROUNDABOUT

rather than of weaving sections. Thus in recent years, attempts are being made to bring together into common framework in which the capacity is predicted entry by entry. This capacity prediction formula should be applicable to both conventional and off-side priority roundabouts.

1.1.2 The Roundabout - Entry/Circulating Flow Relationships

The entry capacity is defined as the maximum inflow from an entry when the demand flow is such that there is a continuous queueing in the approach. Because of the priority rule, the entry decreases if the circulating flow increases, since there are then fewer opportunities for waiting drivers to enter the circulation. It is therefore necessary to specity the entry capacity at each level of circulating flow. The dependence of entry capacity on circulating flow is known as the entry/circulating flow relationship, and itself depends mostly on the roundabout geometry. The basic task of capacity estimation is to define how this relationship may be predicted from a knowledge of geometry layout. In principle, two strategies are possible, namely:-

a. The first is to establish a theoretical model of the vehicle - vehicle interaction which are taking place at the roundabout entry and to calculate the entry/circulating flow relationship from this model and then to calibrate the parameters of this relationship in terms of roundabout geometry.

b. The second approach is to determine the dependence of the entry/circulating flow relationship on the geometry parameters directly, without recourse to models of vehicle-vehicle interactions. This approach is known otherwise as the empirical method.

1.1.2.1 Vehicle-vehicle Interaction
(The gap acceptance Method)

With this approach, the entry/circulating flow relationship describes the average effect of the vehicle-vehicle interactions that take place in the region of the entry. In the literature, the only vehicle-vehicle mechanism that have received much attention is the "gap-acceptance" and a considerable amount of fundamental work has been done by Tanner (1962) and Ashworth (1969), relating mainly to major/minor priority junctions although the principles are similar for roundabouts. The gap acceptance model is described as follows:-

The circulating flow consists of vehicles which may be subject to certain minimum headway constraints, but are otherwise randomly spaced. Tanner (1962) assumed a negative exponenbal distribution. Gaps occur between groups of one or more circulating vehicles. Vehicles arrive at the entry at random and vehicles waiting to enter move only into gaps exceeding a certain minimum value. The minimum gap size is often assumed to be fixed. Most theories of gap-acceptance are intrinsically passive in the sense that circulating traffic is assumed not to react to the pressence of entering traffic. In addition the gap-acceptance parameters are assumed to be independent of the magnitude of the circulating flow.

However, at roundabouts, other mechanisms are involved and the entry process is in reality somewhat more interactive than the gap-acceptance assumptions flow, as shown below:-

i. "Merging" behaviour often takes place especially at high circulating flows;

ii. Individual vehicles often cause circulating vehicles to slow down and alter their headways;

iii. There are sometimes short periods of priority reversals in which entry vehicles "Force" their way into the junction and circulating traffic has to wait temporarily until the normal priority is regained.

The entry capacity is therefore determined by a variety of mechanisms and although gap-acceptance mechanism as incorporated in theoretical models is very important element in vehicle-vehicle interactions, it is unlikely to be complete and sufficient determinant of the capacity. A comprehensive "vehicle-by-behicle Model" of the entry/circulating flow relationship should include all the various mechanisms, separately identified. But it is not feasible in practice to construct such a model, because of the complexity of:-

i. Separating the mechanisms observationally;

ii. Determining their relative importance from site to site.

iii. Relating a parametric description of each to geometric details of layout.

1.1.2.2 Empirical Methods

Empirically, the roundabout is modeled as a series of "T" Junctions, and the form of the entry/circulating flow relationship is infered directly from capacity observations at the entries. Halcrow Fox and Associates (1978, a & b), Glen, Summer and Kimber (1978) have all shown that since the relationship is inverse (as the circulating flow increases, so the entry flow decreases), the simplest empirical form is a first order model of the form:-

$$Qe = F - fcQc \qquad (1.1)$$

where Qe = entry capacity

Qc = the circulating flow accross the entry

F & fc are positive ~~constraints~~ constants that depend on the geometry of the entry. (See Figs. 2a and 2b).

This is the approach that is adopted in this work and will be dealt with in more detail later.

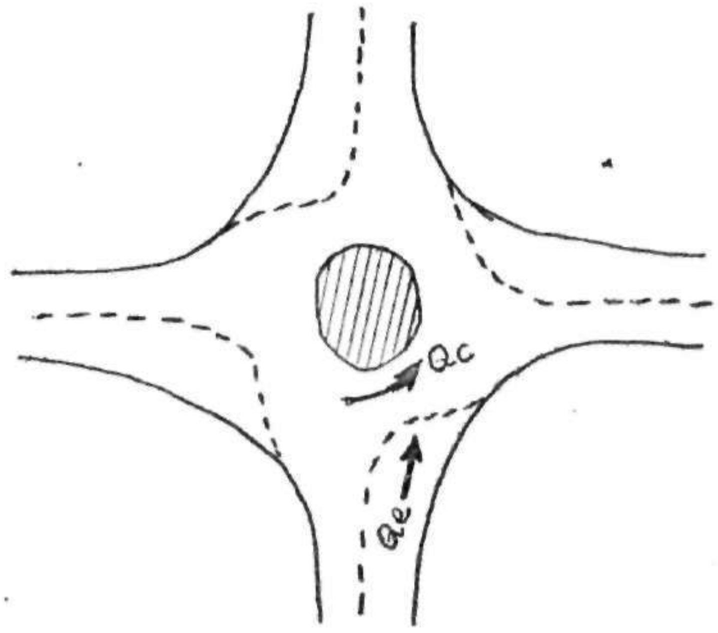


Fig. 2a Traffic Flows At One Entry

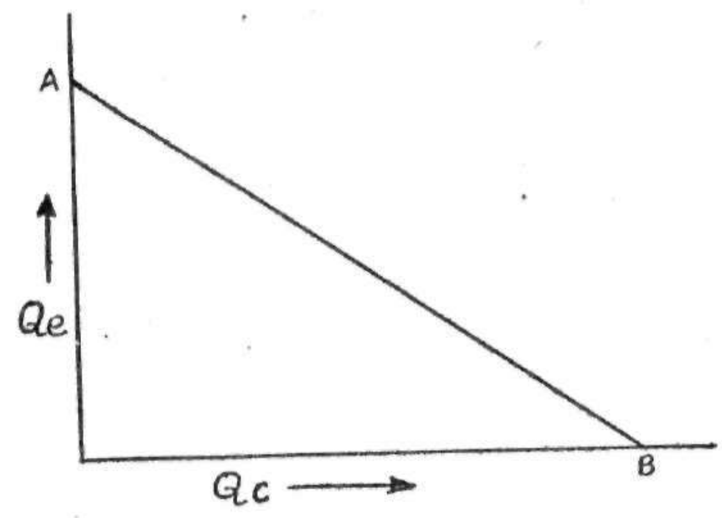


Fig. 2b Entry/Circulating Flow Roundabout

1.2 Traffic Control Signals (Stop and Go)

The use of time-sharing or signal controls eliminates the delays to side street vehicles from waiting for acceptable gaps, but then queueing delays are increased. Signals are useful in providing relief to congested situations where no other control device is adequate. The alternating assignment of right-of-way to intersection legs can eliminate most or all conflicting movements in the intersection area. However since each traffic movement uses the intersection only for a period of time, delay may be increased and capacity reduced, unless the control is properly designed. Consequently the substitution of signal for periority controls will not always reduce the total intersection delay.

The output (and capacity) of uncontrolled or priority intersections decreases once traffic volume increased beyond a certain **level**, but by contrast with signalized intersections, as volume increases the output continues to increase until capacity is reached and then continues at output level equal to capacity with excess volume being accumulated in queues on the intersection approaches.

Considerable amount of work has been done in the literature as to methods of estimating queue length and delay, for example, Webster (1958). Therefore only some aspects of traffic control signals will be discussed later.

2. LITERATURE REVIEW

2.1 The Roundabout - Capacity Estimation

2.1.1 Entry/Circulating Flow Relationship

Investigations of relationships between entry capacity and various streams or manoeuvers by Halcrow Fox and Associates (1978 a and b), Glen, Summer and Kimber (1978) and Marie, Fairweather and Harison (1980) have indicated that only the opposing circulating flow, Qc, has a significant impact on entry capacity. They have also shown that the entry capacity, Qe, is linearly dependent on the circulating flow, Qc. The relationship being represented by equation 1.1 mentioned in chapter 1.

Various detailed breakdown of traffic streams are possible, but according to Halcrow Fox and Associates (1978, b) and Kimber (1980) the most successful was with total entry flow being related to total circulating flow. This relationship is shown in Fig. 2.b. Having gone this far, the problem becomes that of finding the predictable relationships whereby F and fc can be calculated from knowledge of the entry geometry.

The main geometric characteristics to be considered in the study are shown in figs. 3a and 3b. These are:

1. The entry width, e
2. The approach road half-width, v
3. The circulation width, u, at the point of maximum entry deflection.

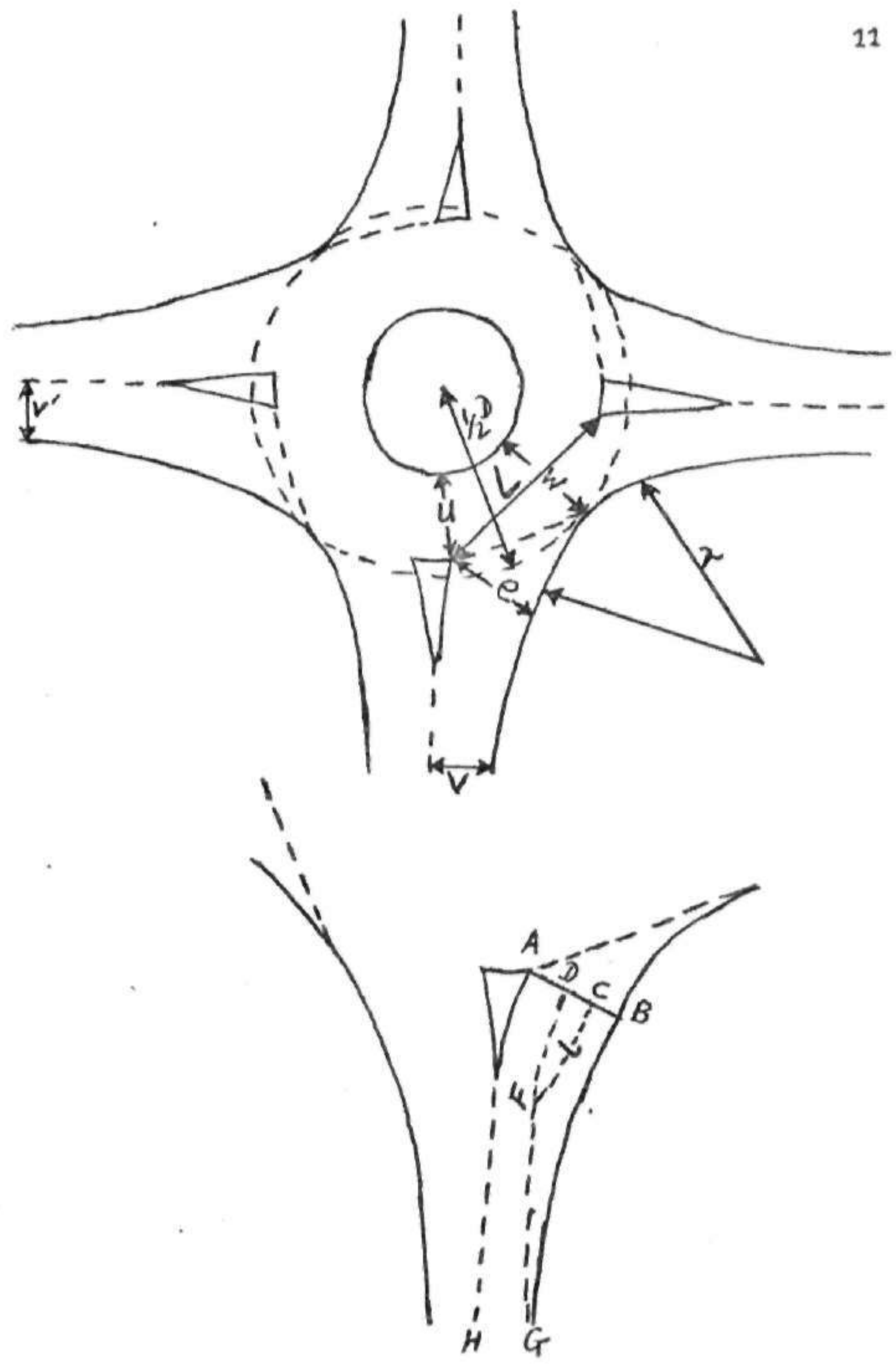


Fig. 3 Definitions of Geometric Parameters (After Kimber, 1980).

4. The average effective length, l, over which the flare is developed.

5. The sharpness of the flare, S, = (e - v)/l).

6. The entry radius, r.

7. The angle of entry, Ø.

8. The inscribed circle diameter, D.

9. The width of the weaving section, w.

10. The length of the weaving section, L.

The entry width and flare have been found to be by far the most important geometric factors affecting F and fc, with the inscribed circle diameter, D, and the circulating width, u, having relatively little effect(see Halcrow Fox and Associates 1978, a). The influence of entry width and flare arose because they determine:

i. The number of entry queues, n, which predominantly determine both the slope, fc, and the intercept, F, of the capacity relationship. (The number of entry queues is the number of parallel streams at the Stop line).

ii. The width available to each queue, which affects the intercept. See also Kimber, (1980). These two are, ofcourse interdependent, since increase in entry width may increase the number of lanes operating, or the available width per lane or both.

1.2 Number of Ques At Entry, n

The predictive equation for the number of queues at entry, n, is difficult to arrive at since at most roads there is some degree of flare, and entry saturation is often only partial, even when there is continuous queueing in the approach road. In general case, where e is not necessarily equal to v, it has been shown by Halcrow Fox and Associates (1978 a & b), Kimber (1980) and also by Glen, Summer and Kimber (1978) that n can be predicted by means of the equation

$$n = a \left(v + \frac{(e + v)}{1 + CS} \right) \quad \text{- - - - - - - - - - -} \quad (2.1)$$

where

a and C are constants.

2.1.3 Predictive Equation For F and fc

Based on the above relationship for n, Semmens, Fairweather and Harison (1980), Kimber (1980) and also Halcrow Fox and Associates (1978 b) adopted the following equations for F and fc :

$$F = a_0 + a_1 \left(v + \frac{(e - v)}{1 + CS}\right) + a_2 u + a_3 D. \text{ - - -} (2.2.)$$

$$fc = b_0 + b_1 \left(v + \frac{(e - v)}{1 + CS}\right. \quad \text{- - - - - - - -} \quad (2.3).$$

It should be observed that although Qe decends linearly on n (which is it's main determinant), the dependence of n on e, v and S is non-linear for flared entries. Thus except for parallel entries, which are rare, no linear combinations of e, v and S can be used to effectively predict the entry capacity. This was established by a test in which Glen, Summer and Kimber (1980) regressed F and fc against linear combinations of the geometric parameters:

$$F = a_o + \sum_i ai\ gi$$

$$fc = b_o + \sum_i bi\ gi$$

where

gi are the geometric variables, o, u, D, l, v, s, r and Ø. or a subset of them, and ao, ai, bo, bi are coefficient to be determined. It was not possible from these tests to identify the important variables because the coefficients were very sensitive to minor changes in the data base.

To obtain the best value of c in functions of F and fc, Halcrow Fox and Associates, (1978b) calibrated equation 2.1 against observed number of queues. The value of C, that had the highest correlation coefficient, was then obtained as 2. Hence equation 2.1 can now be written as

$$n = a\left(v + \frac{(e - v)}{1 + 2s}\right) = aX_2 \quad \text{------------- } 2.4$$

where $X_2 = \left(v + \frac{(e - v)}{1 + 2s}\right)$.

2.1.4 Current Final Models

Compared with X_2, other geometric variables had a secondary role. Consequently Glen, Summer and Kimber (1978), Halcrow Fox and Associates (1978, b) and also Kimber (1980) adopted the procedure whereby a sensible relationship between entry capacity and circulating flow based on X_2 was established, and subsequently this was correlated for the effects of the additional variables Ø, $\frac{1}{r}$, u and D. As an example, if F and fc are functions of X_2 only, the final relationship will be calibrated in the form:

$$Qe = (a_1 + a_2 \phi + \frac{a_3}{r} + a_4 u + a_5 D)(F - fc\ Qc) \quad \text{— } 2.5$$

The coefficients a_1, a_2, etc would be established by regressing the ratio of observed entry flow to that predicted by the basic equation (F-fcQc) against ϕ, 1/r. u and D.

Two sets of relationship chosen for final consideration are given below. They are basically the same, since the authors employed the techniques already discussed.

Those developed by Halcrow Fox and Associates (1978 B) are :-

i. $fc = 0.27 (1 - \frac{.2}{2})(1 + 0.2X_2)$ ------ 2.6

$= 0.24 (1 + 0.2X)$

$F = -2.82 + 5.45 X_2$ ------------ 2.7

and

$Qe = (0.97 - 0.004 \phi + 0.008 u + 0.0013D)(F - fcQc)$ -- 2.8

Qe being in Pcu/hr.

This produces high correlations for either conventional or offside priority roundabout, about 0.85 (Halcrow Fox and Associates, 1978 B).

ii. Those developed by Kimber (1980) :-

Kimber (1980) showed that the best predictive equation for entry capacity of a roundabout is

$$Q^*_e = K(F - f_c Q_c), \text{ when } f_c Q_c \leq F$$
$$= 0 \qquad f_c Q_c > F \qquad \text{---------- } 2.9$$

where

$K = 1 - 0.00347(\phi - 30) - 0.978((1/r) - 0.05)$

$F = 303 x_2$

$f_c = 0.210 t_D (1 + 0.2 x_2)$

$t_D = 1 + 0.5/(1 + \exp((D-60)/10))$

$x_2 = v - (e-v)/(1 + 2S)$

$S = (e-v)/l'$

The ranges of the geometric parameters in his data base were:-

e :	3.6 – 16.5	(m)
v :	1.9 – 12.5	(m)
l, l' :	1 – ∞	(m)
S :	0 – 2.9	
D :	13.5 – 171.6	m
ϕ :	0 – 77	(o)
r :	3.6 – ∞	(m)

These equations are empirical and are based on results of large numbers of observations of road junctions operating at capacity, and according to Kimber (1980), the equations apply to all roundabout types except those at grade-separated interchanges.

2.2 The Roundabout - Delay Estimation

2.2.1 Application of Queueing Theory

The traffic approaching one arm of the roundabout can be treated as a single stream with a defined demand flow, q, and the capacity available to it, μ. If the system is in equilibrium, the vehicles arrivals at the approach and the departure into the roundabout are randomly distributed, the probability of a queue of n vehicles being Pn, it has been shown that:

$$Pn = (1 - \rho) \quad \text{-------------} \quad 2.10$$

where $\rho = q/\mu$

Pn represents the proportion of the time for which there are n waiting vehicles. In practice the traffic demand and capacity vary in time and time-dependent effect becomes important. Thus the queueing problem becomes that of determining the probabilities as functions of time, given the sequence of q and μ values. Once this has been achieved all qualities of interest can be derived as functions of time Pn(t).

For this model described in the first paragraph of this section, if time-dependent effect is brought in, the average queue length, l, evaluated over a large number of trials, varies with time and can be obtained from

$$L(t) = \sum_{n=1}^{N} nPn(t). \quad \text{---------} \quad 2.11$$

where N is the upper limit of the queue.

Also the total delay surfered between a time t_1 and a time t_2 is simply the area under the curve relating average queue length and time:-

$$D(t_1, t_2) = \int_{t_1}^{t_2} L(t)\,dt \quad \text{---------} \quad 2.12$$

The evaluation of Pn(t) is not an easy task, yet it is necessary to use the queueing theory that considers time-varying effects in calculating queue lengths and vehicle delays because of time varying nature of traffic flow, The more conventional theories, namely, "deterministic" and "stockastic" are unsatisfactory in this situation.

Deterministic queueing theory in which the delay is obtained as a simple integral of demand minus capacity can sometimes be used when demand and capacity vary in time. However, it ignores the statistical nature of traffic arrivals and departures and would predict zero queue length until demand exceeds capacity.

Stochastic theory, on the other hand, which assumes randomness in the vehicle arrivals and service patterns, for steady state, predicts infinite queue lengths whenever demand equals capacity.

It is thus clear that whereas deterministic queueing theory applies for values of traffic intensity, ρ, greater than unity, stockastic theory applies for values of traffic intensity ρ, less than unity.

theory applies for ρ values less than unit. There is therefore the need to use an approach that applies at all levels of ρ . Time-dependent queues approach does this as it provides a much more realistic description of the growth and decay of queues in situations where the demand changes with time. See equations 2.11 and 2.12. and also Morse P.M. The evaluation of equation 2.11 and 2.12 above which involves the calculation of Pn(t) is however costly in computer time and nearly impossible manually. Equations have therefore been developed which give good approximation to the average queue calculated from probabilistic theory. One of such is the co-ordinate transformation approach.

2.2.2 The Co-ordinate Transformation Approach (Principles of The Method)

This technique has been discussed by Catling (1977), Kimber, Marlow & Hollis (1977), Kimber & Hollis (1978), Kimber & Hollis (1979). With this approach, a co-ordinate transformation technique is employed to smooth the steady state stockastic relationship for queue length or vehicle delay into the over-saturated deterministic results obtained by integrating the excess of demand over capacity. It ensures that when the demand considerably exceeds the capacity, queue length and delays approach those calculated deterministically - See Fig 4(b). Kimber and Hollis (1979) have shown that it's use is justified on the basis that it gives predictions close to those of time-dependent approach.

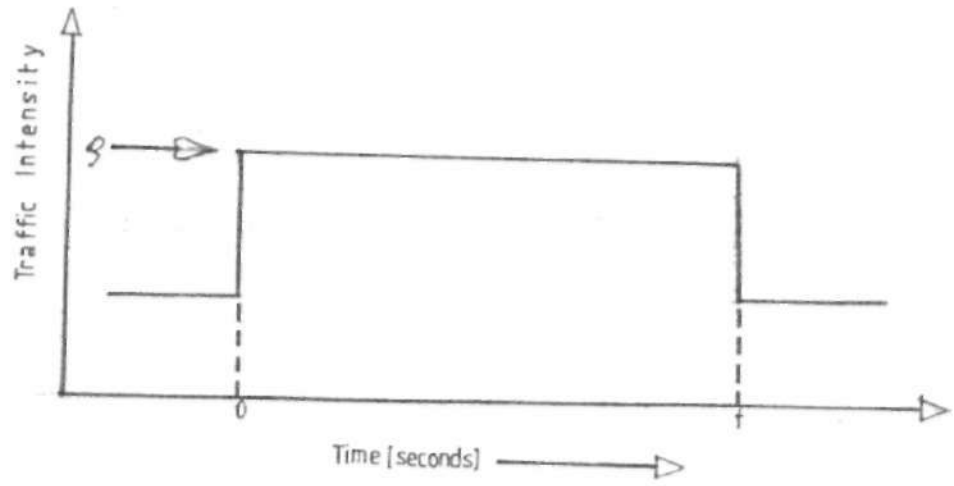


Fig. 4a Prescribed Conditions

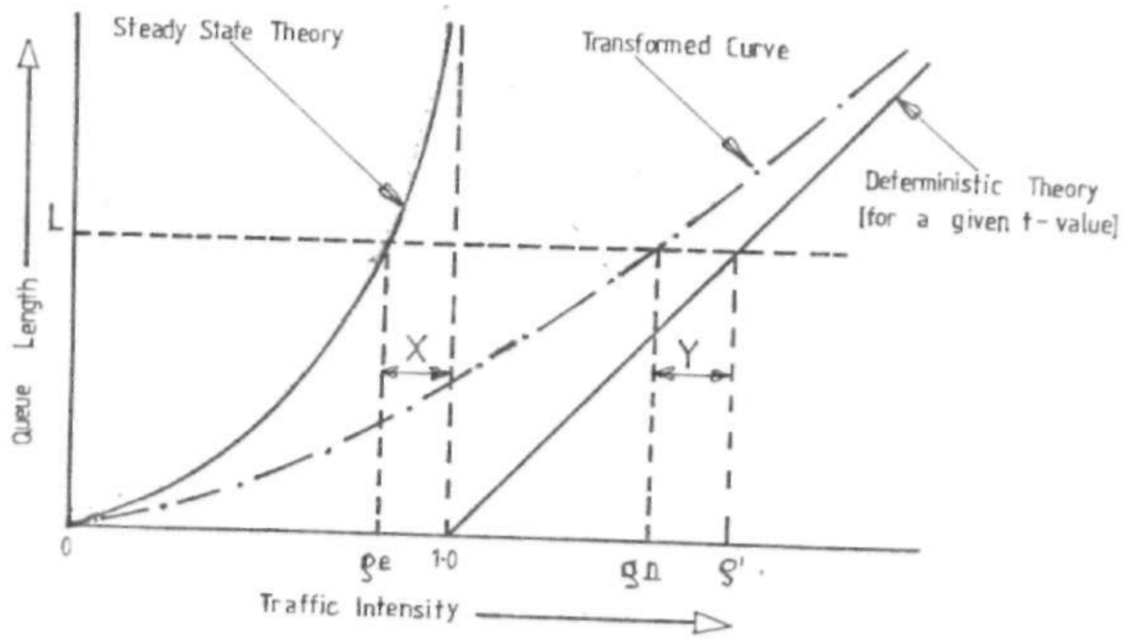


Fig-4b Principle of the Method.

FIG-4 CO-ORDINATE TRANSFORMATION.

Suppose at time $t = 0$, there are Lo waiting vehicles and traffic intensity changes rapidly to ρ from the initial value, see Fig. (4.a) Deterministic theory predicts a queue length, L, given by

$$L = (q - \mu)t + Lo$$
$$= (\rho - 1)\mu t + Lo \quad \text{------------} \quad 2.13$$

Stockastic theory predicts the steady state result for queue length, L, including the vehicle in service

$$L = \rho + C\rho^2/(1-\rho) \quad \text{--------} \quad 2.14$$

If for simplicity, random arrivals and service is assumed,

$C = 1$, then

$$L = \rho/(1-\rho) \quad \text{------------} \quad 2.15$$

The co-ordinate transformation technique transforms the steady state result, equation 2.15, so that instead of L becoming infinite at $\rho = 1$, it approaches the value predicted by the deterministic equation, 2.13, for each value of "t" at high ρ values. Fig. 4(b) illustrates the process graphically for the simple case $Lo = 0$ and an assumed value of t. For a given number of waiting vehicles, L, the steady - state traffic intensity ρ_e is transformed into the new value such that referring to Fig 4(b), X = Y. This makes the deterministic line assymptotic to the transformed curve. Thus:

$$1 - \rho_e = \rho' - \rho_n$$

hence $\rho_e = \rho_n - (\rho' - 1)$

where

ρ' is the intensity corresponding to L in the deterministic case.

From equation 2.14

$$\rho' = (L - Lo)/\mu t + 1$$

and the transformation is equivalent to setting

$\rho = \rho_e = \rho_n - (L - Lo/\mu t)$ in equation 2.16.

The transformed curve is therefore giben by

$$\frac{\rho_e}{1-\rho_e} \rightarrow \frac{\rho_n - (L - Lo)/\mu t}{1 - \rho_n + (L - Lo)/\mu t} = L$$

Kimber and Hollis (1979) obtained a solution for L, in the above equation, as

$$L = \tfrac{1}{2}\left((A^2 + B)^{\frac{1}{2}} - A\right) \quad \text{-------------- 2.16}$$

where

$$A = (1 - \rho)\mu t + 1 - Lo$$
$$B = 4 (Lo + \rho\mu t)$$

Only the positive root of the quadrant in L was needed. Also using the more general equation, (2.14), equation 2.16 was obtained with

$$A = \frac{(1-\rho)(\mu t)^2 + (1 - Lo)\mu t - 2(1-C)(Lo + \rho u t)}{u t + (1 - C)}$$

$$B = \frac{4 (Lo + \rho\mu t)(\mu t - (1 - C)(Lo + \rho\mu t)}{\mu t + (1 - C)}$$

This gives the queue length, L, including the vehicle at the give - way or stop - line.

2.2.3 The Delay Per Arriving Vehicle

During a period of variable demand and therefore of changing queue length, the delay experienced by vehicles arriving at one state of the queue will be different from those arriving at another. The delay per unit time approach produces average values and so can not be used to attribute delay to just those vehicles which arrive within a prescribed period. It includes delays to vehicles already waiting just prior to the interval considered, and excludes those to vehicles left waiting at the end of the period. For the purposes of total delay calculations, over a whole traffic peak, for example, the distinction is unimportant since the values will be the same providing there are as many vehicles in the queue at the end of the whole period considered as at the beginning (or unless these numbers are relatively small anyway which is often the case). However it is sometimes useful to have a measure of the delay to individual vehicles as a function of time or of the average delay per arriving vehicle over an interval. The former can easily be obtained from the queueing curve. $L = L(t)$. For the average delay per arriving vehicle, Dv, over the period 0 - t, the deterministic result is

$$Dv(d) = \frac{(Lo - 1) + \frac{1}{2}(\rho - 1)\mu t}{\mu}$$

and the stochastic result is .

$$Dv(s) = \left(\frac{1}{\mu}\right) \left((1 - C\rho)/(1 - \rho)\right)$$

The transformed time dependent result, developed by Kimber and Hollis (1979) for a particular period T = 0 to T = t, is

$$Dv(t) = \frac{1}{2}\left((J^2 + K)^{1/2} - J\right) \quad \text{------------ 2.17.}$$

where

$$J = \frac{t}{2}(1 - \rho) - \frac{1}{\mu}(Lo - C + 2)$$

$$K = \frac{4}{\mu}\left(\frac{t}{2}(1 - \rho) + \tfrac{1}{2}\rho tC - \left(\frac{Lo + 1}{\mu}\right)(1 - C)\right).$$

This includes the delay at the give-way or stop - line. If this delay is excluded, the result is $D\acute{v}$ and can be obtained from

$$Dv = D\acute{v} + \frac{1}{\mu}$$

These approximation methods allow the growth and decay of queues to be predicted without recourse to probabilistic calculations.

This equation 2.17, is the final result adopted in this thesis for the estimation of delay experienced by vehicles at an approach of the roundabout. The assumed demand model is discussed hereunder.

2.2.4 Flow/Delay Relationships

Traditionally, flow/delay relationships have been derived from stochastic (steady state) theory, and provide good predictions only if ρ never exceeds the critical value ρ_c ($\rho_c < 1$). This restriction is quite demanding since even if the average intensity during peak period is below ρ_c, the maximum intensity may exceed it. Thus although the application of steady state theory is obviously wrong when $\rho = 1$ and queues and delays are apparently infinite, it is in most cases also inappropriate for ρ values somewhat less than unity, say for ρ values greater than 0.95.

The division of traffic intensity into two regimes broadly distinguished by the criteria $\rho \leq \rho_c$ and $\rho > \rho_c$ enables time-dependent, low-definition, flow-delay relationship to be formulated. The basic low-definition problem is to predict the delay associated with a period of above average demand, such periods usually correspond to morning or evening peak hours. During the peak, ρ_c may be exceeded and the traditional stochastic (steady state) theory no longer applies.

The simplest "model" of peak period traffic demand is illustrated in Fig.5. At time t = 0, the traffic intensity changes rapidly from 0 to ρ, **then** remains constant until at t=T, when it falls rapidly to zero again, the capacity, μ remains constant throughout the interval t = 0 – t = T.

Since the formulae for the queueing problem and that of the co-ordinate transformation technique relate to the time dependence of various queueing quantities during a time of segment of demand pattern in which ρ and μ are effectively constant, and it is some times necessary to employ flow/delay relationships to give the average delay associated with average flows over a longer period, without recourse to

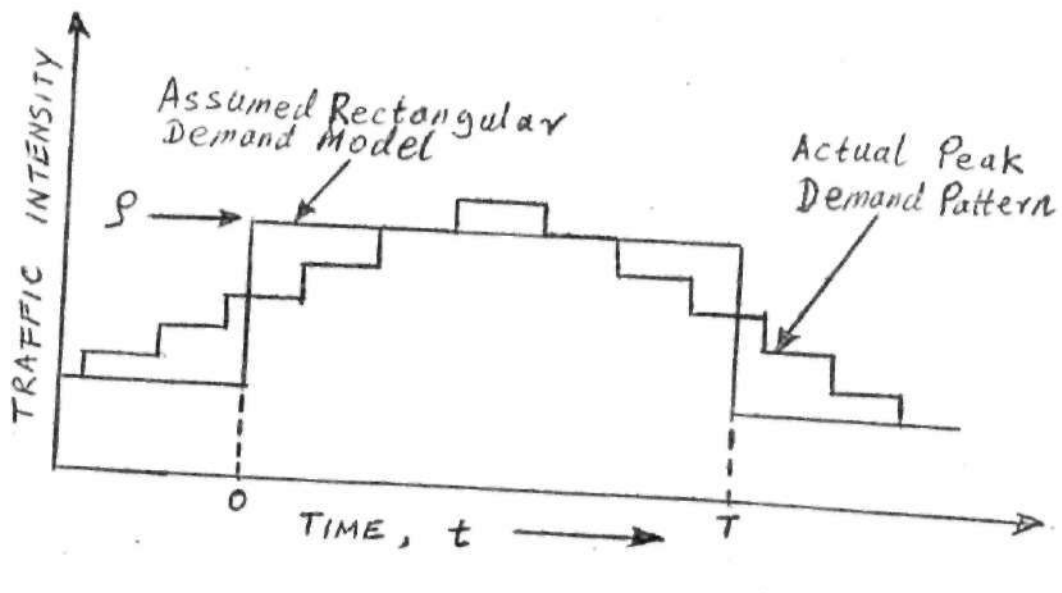


Fig. 5 Peak Period Rectangular Demand Model

detailed queueing calculations, assumptions must be made about the shape of traffic demand profile with time. Such assumptions have to strike a balance between the realism of the shape and the complexity of the resulting flow/delay relationships. In low definition problems, only very broad descriptors are available for traffic demand - for example the average intensity, average capacity and approximate duration of peak period (0 - T) in Fig. 5.

The flow/delay relationships corresponding to this model are described by Kimber and Hollis (1979) and also by Hollis, Semmens and Dennis (1980). Fig. 5 is obtained when Lo = 0 and C = 1. For delay estimates, the model is less artificial than it appears at first sight, as Catling (1977) has shown. Although the traffic intensity before the peak is in reality non-zero, it is non the less below the critical value, and steady state conditions are reached rapidly. Thus whether the intensity is represented initially as zero or non-zero is not very important. The fact that the intensity is really greater than zero after the peak is rather more important, since the decay of the queue depends largely on the difference between demand and capacity.

However, provided that the "block time", 'T' is suitably defined, the delay estimates are sound and the model contains all the essential features necessary to account for time - dependent effects, and in particular, Catling (1977) and Kimber and Hollis (1978), have shown that delays at and near $\rho = 1$ are realistic.

2.3 Signal Controlled Intersection – Delay Estimation

A lot of work has been done on signal controlled intersections and extensive literature on the subject abounds. The relevant results that will be used in this thesis shall be quoted but no attempts will be made to cover the literature here.

There are two major types of traffic signal controls, namely, the fixed-time signal and the vehicle-actuated signal. The former is commonly used here in Nigeria. Whilst over-saturation occurs at some signalized intersections, it is a once in a while situation during the day.

2.3.1 Fixed-Time Signal – Unsaturated case

In this work only the case of under-saturation is considered for the comparism with roundabout, because it is more straight forward to apply. Besides, the mathematics involved in considering over-saturated case is fairly complicated and if considered, might cause distraction from the main objective. Nevertheless, if the actual situation is different from this, the delay can be estimated by the methods described in sections 2.3.2 and 2.3.3 below.

Webster (1958) obtained, by a combination of queueing theory and digital computer simulation, the average delay, d, per vehicle on a particular intersection approach as

$$d = \frac{C(1-\lambda)^2}{2(1-\lambda\chi)} + \frac{\chi^2}{2q(1-\chi)} - 0.65\left(\frac{C}{q^2}\right)^{\frac{1}{3}} \chi^{2+5\lambda} \quad -- \; 2.18$$

where

d = average delay per cycle (secs)

λ = g/c, (Proportion of cycle time the signal is green)

χ = $q/\lambda s$, degree of saturation

y = q/s

q = The flow (arrival rate) veh/sec.

s = Saturation flow.

2.3.2 Fixed-Time Signal - Oversaturated case

In general, experience indicates that for oversaturated intersections, long cycles result in less total delay. On the other hand, if the cycle length is extremely long, the approach becomes unsaturated, and green time may be wasted and delay increases. This will occur if during the peak period, the output curve intersects the imput curve on the approach. Furthermore cycle length affects maximum queue lengths, and in order to avoid blocking the adjacent intersection, it may be advantageous to use shorter cycle length resulting in higher total delays. If it is thus established that no green time is wasted, then flow per cycle (output flow) could be obtained as effective green time, multiplied by the saturation flow for that approach. Also the total delay could be obtained, as the area between the cumulative input and output curves, if the cumulative output curve do not intercept the cumulative input curve, i.e the queue does not become negative.

2.3.3 Vehicle - Actuated Signals

With vehicle - actuated signals, the green time is extendable in accordance with traffic demand, but under heavy demand, at peak periods, successive phases frequently run to the maximum value (as set on the controller) and the operation of the signals approximates to fixed-time working. In this condition then, equation 2.18 is applicable. Under lighter traffic, when the green periods are varying according to the flow, the delay can be estimated by first assuming a mean demand rate, then working out the optimum fixed-time signal settings - cycle time and green times. These values are then substituted in the fixed-time formula of equation 2.18, to estimate the delay.

3. ESTIMATING DELAYS ON ROUNDABOUTS AND SIGNALIZED INTERSECTIONS

3.1 Roundabout - Estimating Entry Capacity

The entry capacity of the roundabout is to be determined from knowledge of geometric and traffic conditions, thence the delay could be obtained.

It is not intended here that the rudiments of roundabout design should be gone through. Steps to be adopted in the process of designing the "best" possible roundabout has been fully set out by Kimber (1980). The geometrics chosen, take into account, the traffic demand intensity, traffic composition, turning proportions, available land and other constraints which may be considered important. In practice therefore, it will not be possible to design a roundabout without considering the above items.

By considering the geometric characteristics, the capacity of roundabout entry, Qe, was obtained and represented by the "linear entry capacity/circulating flow relationships", of the form

$$Qe = F - fcQc \quad \text{(See equation 1.1).}$$

Here the slope fc and the intercept, F are obtained using "the best predictive equation" - equation 2.9 - which was developed by Kimber (1980). This equation could be written as

$$Qe = K(F' - f'cQc) \text{ for } fcQc \leq F$$
$$= 0 \quad \text{otherwise}$$

where

$K = 1 - 0.00347 (\phi - 30) - 0.978 \left(\left(\frac{1}{r}\right) - 0.05 \right)$

$F' = 303X_2$

$f_c' = 0.210 t_D (1 + 0.2X_2)$

$t_D = 1 + 0.5/(1 + \exp((D - 60)/10))$

$X_2 = v - (e - v)/(1 + 2S)$

$S = (e - v)/l$

The entry capacity obtained as above should then be used together with traffic conditions (obtained by counting or estimating to get a final entry capacity under the given traffic conditions. Since it is the maximum possible flow through the entry that is of interest here, the roundabout approaches are assumed saturated. When all the arms are thus saturated (i.e. all the demand flows substantially exceed their respâctive arm capacity) the capacities satisfy the complete set of similtaneous equations. For example in the case of a four-arm roundabout with no U-turners.

$$\mu_A = F_A - f_{cA} (\mu_C P_{CB} + \mu_D (P_{DB} + P_{DC})) \quad --- 3.1a$$

$$\mu_B = F_B - f_{cB} (\mu_A P_{AC} + \mu_B (P_{BD} + P_{DA})) \quad --- 3.1b$$

$$\mu_C = F_C = f_{cC} (\mu_C P_{AD} + \mu_B (P_{BD} + P_{BA})) \quad --- 3.1c$$

$$\mu_D = F_D - f_{cD} (\mu_B P_{BA} + \mu_C (P_{CA} + P_{CB})) \quad --- 3.1d.$$

Where $(\mu_C P_{CB} + \mu_D (P_{DB} + P_{DC}))$ is the circulating flow past approach "A"; - - - - - - - etc.

μ_A is the capacity of entry A

P_{ij} is the proportion of flow from arm i to j

$(i, j = A, B, C, D)$.

Since it is assumed that the arms are all saturated, each arm is operating at it's capacity. These set of equations are solved to obtain the entry capacities – μ_A, μ_B, μ_C & μ_D.

To show how this process is carried out, typical values for e, v, l, D, r and Ø are used in the analysis and values of F and fc calculated.

e	was	varied	from	2.5m	to	11.0	m
v	"	"	"	2.0m	to	11.0	m
l	"	"	"	5m	to	30	m
D	"	"	"	10m	to	120m	
r	"	"	"	4m	to	70m	
Ø	"	"	"	30°	to	60°	

The method used was to allow only one parameter to vary at a time while others are constant during that analysis. For each of the above cases, values of F and fc are calculated and then shown in appendix 'A'. The results obtained appear reasonable and the variation of F or fc with the various geometric parameters of the roundabout can be observed. These results conform to the linear entry/circulating flow relationship and is confirmed by the data collected at three roundabouts in Kaduna. (See section 3.3 and figures 8, 9 and 10). Since the above calculations could be very tedious, the "HP.21" programable pocket calculator was used.

To illustrate how the entry capacities could be obtained knowing the linear entry/circulating flow relationship – F & fc – in addition to traffic conditions in the roundabout, two major assumptions are made:-

i. The roundabout approaches are saturated.

ii. The roundabout is geometrically symetric – (for simplicity).

Thus

$$F_A = F_B = F_C = F_D$$
$$f_{cA} = f_{cB} = f_{cC} \quad f_{cD}$$

Two sets of values for the geometric parameters are used with the resulting values of F and fc.

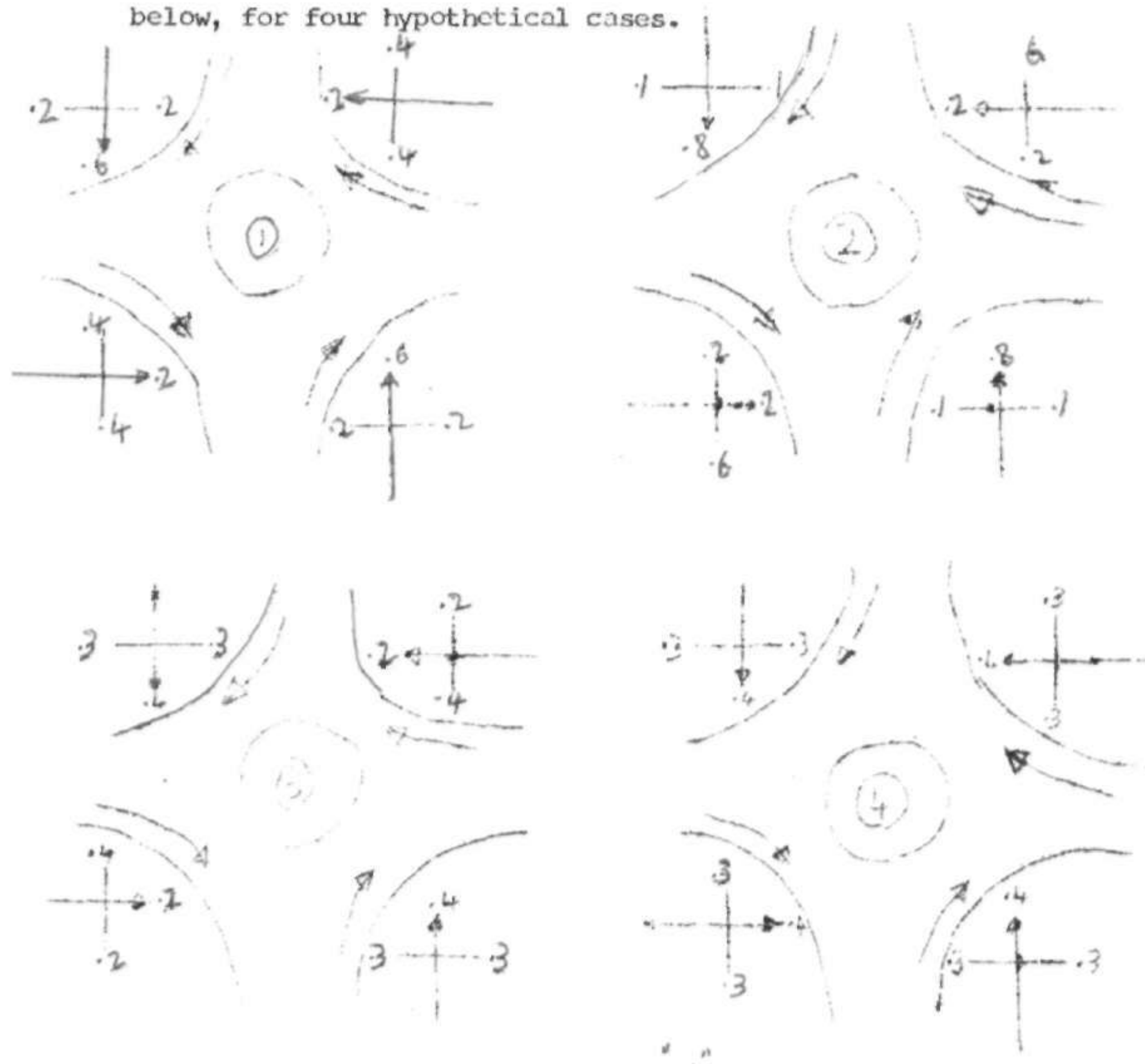
e	5m,	9m
v	3m,	6m
l	10m,	20m
D	10w,	30m
r	4m,	15m
ø	30°,	40°

The resulting values of F and fc are.

F = 1342 and 2517

fc = 0.593 and 0.8251

As already stated, the only information needed for computation of entry capacities are turning proportions for each approach. The roundabout is considered while operating at each of the different conditions of turning movements shown below, for four hypothetical cases.



With all the arms saturated and no "U"-turners the four sets of simultaneous equations for each of the above, from equations 3.1 (a, b, c, d) are :-

Case 1.

$$\mu_A = F_A - f_{cA}(\cdot 2\mu_C + \cdot 6\mu_D)$$
$$\mu_B = F_B - f_{cB}(\cdot 4\mu_D + \cdot 8\mu_A)$$
$$\mu_C = F_C - f_{cC}(\cdot 2\mu_A + \cdot 6\mu_B)$$
$$\mu_D = F_D - f_{cD}(\cdot 4\mu_B + \cdot 8\mu_C)$$

~~Case 2.~~

~~$\mu_A = F_A - f_{cA}(.2\mu_C + .6\mu_D)$~~

~~$\mu_B = F_B - f_{cB}(.4\mu_D +$~~

Case 2.

$$\mu_A = F_A - f_{cA}(.1\mu_C + .4\mu_D)$$
$$\mu_B = F_B - f_{cB}(.2\mu_D + .9\mu_A)$$
$$\mu_C = F_C - f_{cC}(.1\mu_A + .4\mu_B)$$
$$\mu_D = F_D - f_{cD}(.2\mu_B + .9\mu_C)$$

Case 3

$$\mu_A = F_A - f_{cA}(.3\mu_C + .8\mu_D)$$
$$\mu_B = F_B - f_{cB}(.6\mu_D + .7\mu_A)$$
$$\mu_C = F_C - f_{cC}(.3\mu_A + .8\mu_B)$$
$$\mu_D = F_D - f_{cD}(.6\mu_B + .7\mu_C)$$

Case 4

$$\mu_A = F_A - f_{cA}(.3\mu_C + .7\mu_D)$$
$$\mu_B = F_B - f_{cB}(.3\mu_D + .7\mu_A)$$
$$\mu_C = F_C - f_{cC}(.3\mu_A + .7\mu_B)$$
$$\mu_D = F_D - f_{cD}(.3\mu_B + .7\mu_C)$$

These sets of equations were solved by method of interaction, using initial values as $\mu_A = F_A$, $\mu_B = F_B$, $\mu_C = F_C$ and $\mu_D = F_D$ After four or five interactions, the entry flows from each arm converse to their final values. Because of the tedious nature of the above calculation, it was performed by the "HP.21" programable calculator. The two sets of values for F and fc were used - F = 1342 & 2517

fc = 0.593 & 0.8251

with the four sets of equations obtained from the turning movements shown, and the following results obtained:

with F = 1342, fc = 0.593

Case 1

		ITERATIONS				
		1st	2nd	3rd	4th	5th
μ_A	=	708	969	976	975	975
μ_B	=	689	707	214	714	714
μ_C	=	938	1003	1005	1005	1005
μ_D	=	734	698	696	696	696

Case 2

	ITERATIONS				
	1st	2nd	3rd	4th	5th
μ_A =	944	1117	1119	1120	1120
μ_B =	679	666	667	667	667
μ_C =	1101	1144	1145	1145	1145
μ_D =	674	652	652	652	652

Case 3

	ITERATIONS				
	1st	2nd	3rd	4th	5th
μ_A =	447	834	834	846	846
μ_B =	671	719	723	734	734
μ_C =	785	862	865	865	865
μ_D =	777	728	722	722	722

Case 4

	ITERATIONS				
	1st	2nd	3rd	4th	
μ_A =	546	846	842	842	
μ_B =	876	835	842	842	
μ_C =	739	839	842	842	
μ_D =	879	845	842	842	

Similarly with F = 2517 and fc = 0.8251, the following results were obtained:

Case 1 – After five iterations, μ_A = 1760, μ_B = 1053
μ_C = 1838, μ_D = 956

Case 2 – After five iterations, μ_A = 2090, μ_B = 702
μ_C = 2180, μ_D = 707

Case 3 – After six iterations, μ_A = 1431, μ_B = 1152
μ_C = 1648, μ_D = 1088

Case 4 – After five iterations, μ_A = 1379, μ_B = 1379
μ_C = 1379, μ_D = 1379.

These values are the entry capacities of the individual approaches, and appear quite reasonably close in magnitude, being comparable with the values got from measurements. Flow measurements obtained vary between 12 vehicle/min and 25 veh/min. This is (720 – 1500) veh/hr.

3.2 Roundabout – Estimating Delay

3.2.1 Delays Per Arriving Period During Peak Period

As discussed earlier, low definition approach, with only very broad descriptors are used. Only average intensity, ρ, and approximate duration of peak, t, are needed to calculate the above delay, given only the entry capacity, μ, of the roundabout approach. The pertinent equation is given here as equation 2.17, and in order to keep the calculation simple, it is assumed that the arrival and service patterns are completely random, thus C is assumed to have a value of 1.

The simple model of peak period traffic demand is shown in Fig. 5. At time $t = 0$, the traffic intensity changes rapidly from 0 to ρ, then remains constant until $t = T$, when it falls rapidly to zero again. It is assumed that the capacity remains constant at a value μ, which corresponds to the entry capacity of the roundabout approach. Also Lo is assumed to be approximately zero.

i.e. $Lo \simeq 0$.

To illustrate how this method works, four values for the duration of peak period were used (15 min, 30 min, 45, 60 min.) while the value of traffic intensity, ρ, was varied from 0.7 to 1.3. Three values for entry capacity, were used (μ = 975, 1200, 1740 veh/hr.). The delay per arriving vehicle was then computed and the results shown below:

T = 15 mins.

ρ	0.7	0.8	0.9	0.95	1.0	1.1	1.2	1.3
μ = 975, D_v =	11.6	16.2	25.0	32.4	42.6	71.8	108.9	149.8
μ = 1200, D_v =	9.5	13.4	21	28.3	38.2	68	105	147
μ = 1740, D_v =	6.7	9.6	15.8	22.0	31.6	62.1	101.3	143.6

Dv = Delay per arriving vehicles (secs)

T = 30 min

	0.7	0.8	0.9	0.95	1.0	1.1	1.2	1.3
μ = 975, Dv =	11.9	17.2	28.9	40.6	59.6	121.1	200.3	285
μ = 1200, Dv =	9.7	14.1	24.3	35	53	111.6	197	282
μ = 1740, Dv =	6.8	10.0	17.7	26.8	44.3	109.2	191.8	278

T = 45 Min

	0.7	0.8	0.9	0.95	1.0	1.1	1.2	1.3
μ = 975, Dv =	12.1	17.6	30.8	45.6	725	168.3	200.1	420.5
μ = 1200 Dv =	9.8	14.4	25.7	39.1	65.2	103	287	[illegible]18
μ = 1760 Dv =	6.8	10.0	18.4	29.4	53.9	155.1	282.1	[illegible]13.8

T = 60 min

	0.7	0.8	0.9	0.95	1.0	1.1	1.2	1.3
μ = 975, Dv =	12.1	17.8	31.9	49.1	83.4	214.6	381.1	555.7
μ = 1200, " =	9.8	14.5	26.5	42	75	208	377	553
μ = 1740, " =	6.8	10.1	18.9	31.2	62.1	200.6	372.1	548.8

This results were obtained using the ordinary Hp.21 programable pocket calculator.

3.3 Checking The Reliability of the Method

The acceptability of this approach was tested by comparing the results predicted by it with those obtained from actual traffic data obtained at three roundabouts in Kaduna. These roundabouts are:

Singer Roundabout – termed site 1.
Balcni Motors Roundabout – termed site 2.
Chelatems Roundabout – termed site 3.

3.3.1 Data Collection

The data sheet which was prepared and used is shown in Appendix B and C. The sheet is rulled in columns, representing minutes and each column is further divided into sixty units, each unit representing a second. The sheet takes 25 minutes traffic data·Digital watches were used and an observer was positioned at each of the four positions shown in figure 6. Before the commencement of the exercise, all the watches were synchronized and a period after which the count should start decided (e.g. four minutes later) and also the duration of the count. As much as possible the count was conducted only during periods when it was certain that there would be queues at the roundabout approach.

The passage of a vehicle accros any of the

Positions (1 - 4), during the exercise was indicated by a "dash" marked by the operator on the sheet at the precise time that the vehicle passes him) - one dash representing one vehicle and indicating the time in second, at which it passed. One sheet is used for each of the four positions. During analysis the four sheets are treated together to determine the variation of flow with time accross each of the positions. A lot more informations could be obtained from the sheets according to need. In the absence of more sophisticated traffic counting gadgets this improvised method gave quite reasonable results.

3.3.1 Treatment of data

The data obtained are shown in Appendix B. ~~and~~ . for sites 1 and 2. and also Appendix C. for site 3. The flow per minute accross each of the positions is got by adding the number of "dashes" in that column. Thus the flow per minute for the three positions (which are pertinent here) is obtained for the three sites for the duration of the counts. These are shown in tables 1, 2 and 3.

The total delay (vehicle minute) is obtained by the sumation of the queue length over the whole period, and dividing this by the total vehicle arrival during the whole period gives the average dalay per arriving vehicle, Dv.

Table 1 - Site 1

Singer Round-About - Kaduna Approach

	μ_1	μ_2	μ_3	1-2	Queue Length	Delay Vehicle-Minute
1.	12	15	3	-3	-3	-
2.	13	9	9	4	4	4
3.	18	14	4	4	8	12
4.	11	10	10	1	9	21
5.	15	7	14	8	17	38
6.	21	12	8	9	26	64
7.	21	10	12	11	37	101
8.	17	10	6	7	44	145
9.	18	13	9	5	49	194
10.	18	12	6	6	55	249
Σ	164	112	81			

Notes

$\bar{\mu}_3$ = 8.1 veh/min.

Thus from Fig. 7 the capacity of the entry μ_e is got to be

11.7 veh/min = (702 veh/hr)

Also demand rate, λ = $\bar{\mu}_1$ = 16.4 veh/min (984 veh/hr)

Thus traffic intensity = ρ = 1.4

Fig. 7

Singer Roundabout - Kaduna Approach

Qc =	3	4	6	8	9	10	12	14
Qe =	15	14	12	12	13	10	7	

$$Qe = -0.62\ Qc + 1004 \text{ veh/hr.}$$

$$r^2 = 0.86$$

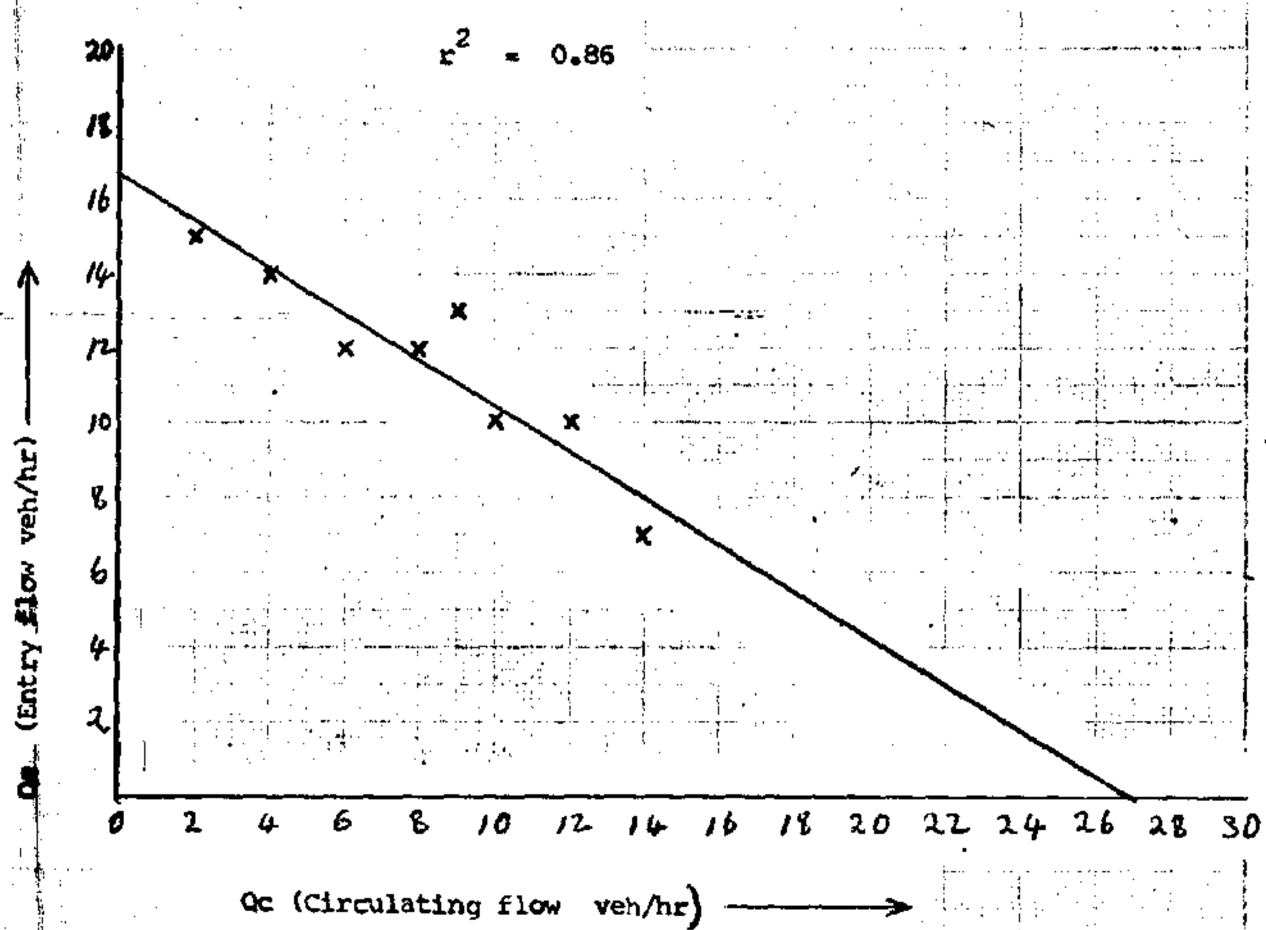


Table 2 - Site 2

Baloni Motors Roundabout - Zaria Approach

	μ_1	μ_2	μ_3	1-2	Queue Length	Delay Vehicle-Minute
1.	23	17	1	6	6	6
2.	22	16	2	6	12	18
3.	20	16	4	4	16	34
4.	21	14	3	7	23	57
5.	21	15	3	6	29	86
6.	25	14	5	11	40	126
7.	23	16	2	7	47	173
8.	19	13	4	6	53	226
9.	17	12	1	5	58	283
10.	22	12	3	10	68	352
11.	26	18	1	8	76	428
12	15	18	3	-3	73	501
13	13	16	4	-3	70	571
14	12	12	3	0	70	641
15	19	11	2	8	78	719
Σ	299	220	41			

Notes

$\bar{\mu}_3$, = 2.73 veh/min

From Fig. 8 the capacity of the entry μ_e is got to be 16.1 veh/min (966 veh/hr)

Also demand rate, $\lambda = \bar{\mu}_1$ = 19.9

= 19.9 veh/min (1196 veh/hr)

Thus traffic intensity, ρ = 1.25.

Fig. 8

Singer Roundabout - Kaduna Approach

Qc =	1	2	3	4	5
Qe	18	16	15	16	14

$Qe = -0.8\ Qc + 1098$ veh/hr.

$r^2 = 0.73$

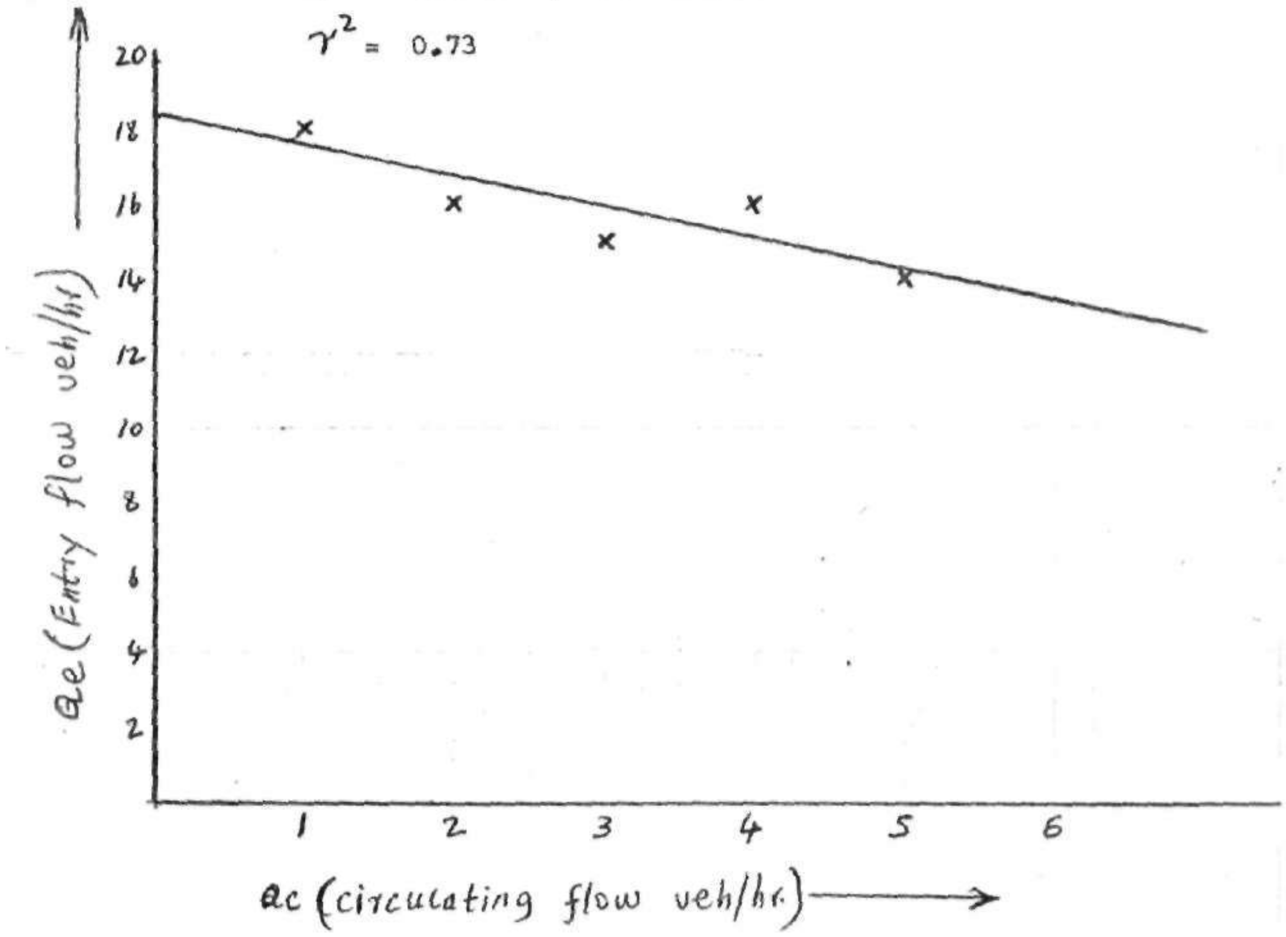


Table 3 - Site 3

Chelaram's Roundabout - Zaria Approach

	1	2	3	1–2	Queue-Length	Delay Veh/Minute
1.	23	26	15	1.3	0	
2.	24	24	15	0	0	
3.	24	19	10	5	5	5
4.	27	25	18	2	7	12
5.	24	26	12	−2	5	17
6.	23	27	9	−4	1	18
7.	18	20	9	−2	0	18
8.	15	19	17	−4	0	18
9.	29	23	9	6	6	24
10.	27	24	19	3	9	33
11.	26	19	11	7	16	49
12.	16	21	11	−5	11	60
13.	16	21	9	−5	6	66
14.	29	19	9	10	16	82
15.	24	26	10	−2	14	96
16.	17	24	15	−7	7	103
17.	21	16	11	6	13	116
18.	21	25	17	−4	9	125
19.	21	17	11	4	13	138
20.	25	25	13	0	13	151
21	21	15	11	6	19	170
22.	20	27	11	−7	12	182
Σ	491	488	272			

Notes

$\bar{\mu}_3 =$ 12.36 veh/min

From Fig. 9 the entry capacity, μ_e = 25.9 veh/min (1554 veh/hr).

Also demand rate, $\lambda = \bar{\mu}_1$ = 22.3 Thus traffic intensity, ρ = 0.861

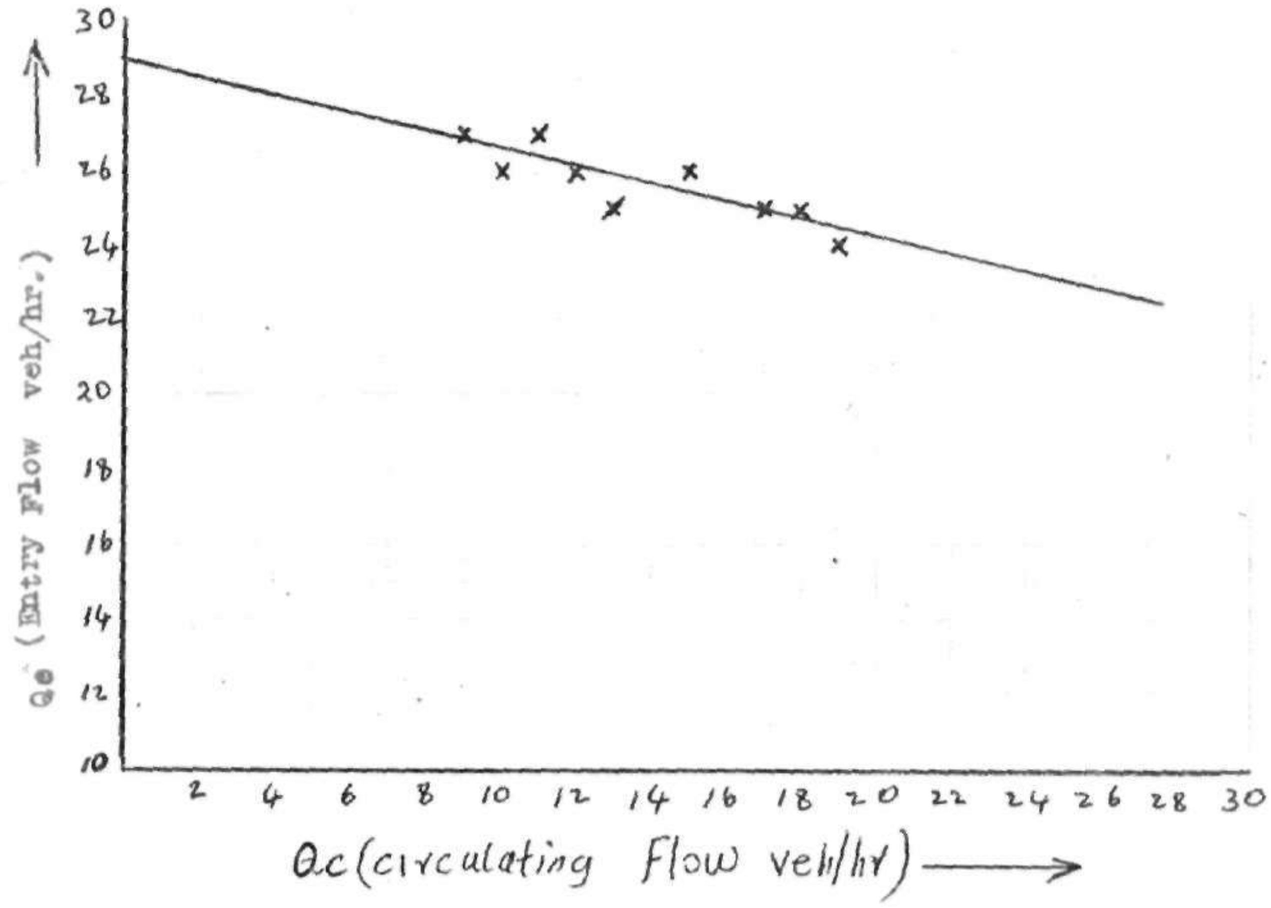
Fig. 9

Singer Roundabout - Kaduna Approach

Qc =	9	10	11	12	13	15	17	18	19
Qe =	27	26	27	26	25	26	25	25	24

$Qe = -0.23\ Qc + 1733$ veh/hr.

$r^2 = 0.72$



Also graph of entry flow against circulation flow is plotted. The graph for site 1 is shown in Fig. 7, that for site 2 in Fig. 8 and that for site 3 in Fig. 9. The values of entry flow used for this is the highest value corresponding to each level of circulating flow. (This ensures that there was queue, as it was not possible to keep very accurate records of periods of queue occurence). Therefore only the underlined figures are plotted. By this then the graph will represent the maximum expected entry flow - (entry capacity) against circulating flow for the roundabout approach.

The entry capacity, μ_e, of an approach is then given by the entry flow that corresponds to the mean circulating flow for the peak period under consideration, and dividing the mean arrival rate, $\bar{\mu}$, by this gives the traffic intensity, ρ .

3.3.3 Results

Graphs of entry flow against circulating flow plotted for the three sites (Figs. 7, 8 and 9) conform to the linear entry/circulating flow relationship. Infact the data gave following equations and coefficients of correlation:

Site 1, $Qe = -0.62\,Qc + 16.7$ veh/min, $\gamma^2 = 0.86$

2, $Qe = -0.8\,Qc + 18.3$ veh/min, $\gamma^2 = 0.73$

3, $Qe = -0.23Qc + 28.9$ veh/min, $\gamma^2 = 0.72$

where

Qe = entry flow.

Qc = circulating flow.

Bearing in mind the crude nature of the data, the values of delay per arriving vehicle, Dv obtained from the data appear reasonably close in magnitude to those predicted by the theoretical method. The results obtained for the sites at the traffic intensity, ρ, entry capacity, μ_e and peak duration, t, are shown below:

Site 1. μ_e = 707 v/hr , ρ = 1.4, t = 10 min

Dv	Experimental value	=	91 secs
	Theoretical value	=	136 secs

Site 2. μ_e = 966 veh/hr , = 1.25, t = 15 min.

Dv	Experimental value	=	144 secs
	Theoretical value	=	129 secs

Site 3. μ_e = 1554 veh/hr , = 0.86, t = 22 min

Dv	Experimental value	=	22.2 secs
	Theoretical value	=	14.7 secs

The experimental value was estimated by considering the area between the cumulative input and cumulative output curves which represents the total delay. This value was then divided by total input during the interval.

3.4 SIGNALIZED INTERSECTION

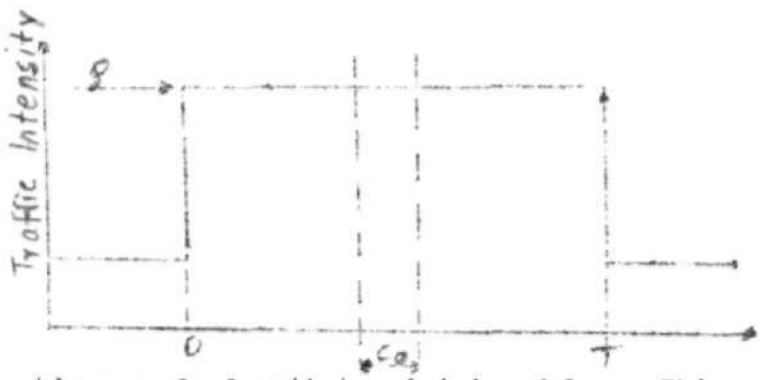
– Delay to traffic during Peak Period

The computation of the above will be based on the following assumptions:-

i Arrival is random

ii. Mean arrival rate λ)
) are constant
Mean service μ)

The low definition rectangular model is assumed and it is also assumed that the period of study falls within the interval 0 to T. as shown below:



Co = optimum cycle length to minimize delay. This was given by Webster as

$$Co = \frac{1.5L + 5}{1 - \sum_{i=1}^{n} y} \qquad \text{------------ } 3.2$$

The actual intersection layout that can be superimposed on the available land in place of the roundabout is adopted. The traffic signal is then designed since **the following** are known:

i. Demand flow for each approach .

ii. Turning movements .

iii. The geometrics of the intersection.

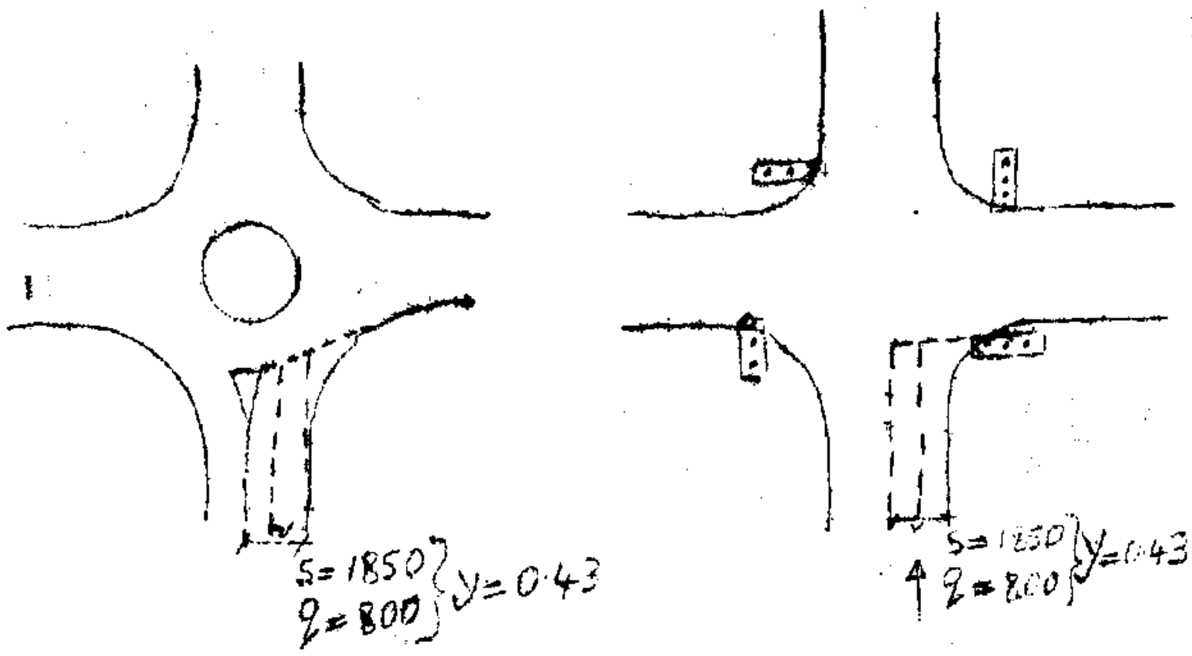
The delay formula and it's minimization are both approximate and the results apply only when there is no oversaturation. (If in a particular case the conditions differ from the above then methods discussed in sections 2.3.2 and 2.3.3 could be applied). This limitation can not be regarded as very serious, in simple terms, it means that the delays should not exceed the cycle length. - which is enough delay already to worry about.

The delay formula to be used is that developed by Webster, which gave the average delay per vehicle on particular intersection arm, this is here given as equation 2.18.

This delay formula has been tested under actual road conditions at several fixed - time and vehicle - actuated intersections and the variations between observed and calculated values was no greater than would be expected owing to random fluctuations.

To be able to compare the delays experienced by vehicles in a roundabout to those experienced in a signalized intersection it would be necessary to superimpose the most suitable signalized intersection on the plan of the roundabout. Then for the same traffic demand flow and turning movement, the delay could be estimated. (This is done for the peak period). The most important geometric factors for the signalized intersection is the approach width at the stop line and the turning radii. From these, the saturation flow, S, could be determined. Also from turning movements and desired flow, optimum cycle time, Co, is determined.

The process could look like this:-



For simplicity, only two phase situation is treated, (more complicated traffic movement can similarly be treated). For a symetrical traffic demand and turning propertions;

$y = 0.43$ and $Y = 0.86$

If a six seconds intergreen period is adopted then

$l = (6-3) + 2 = 5$ Secs

$L = 2 \times 5 = 10$ Secs.

Then from equation 3.2, Co is obtained as 143 Secs.

Total effective green is then obtained as 143 - 10, Hence g = 66.5 Secs.

The delay, d is then obtained from equation 2.18 as 57.05 Secs.

This delay is not affected by the length of the peak period because it has been assumed that there was no oversaturation as all arrivals within a cycle are cleared within that cycle, thus keeping the maximum possible delay less than the cycle length.

For the purpose of comparing the performance of the two traffic controls, it will not be possible to represent the results in a general form where it could be graphed. This is because of the fact that it is not easy to relate the geometrics of the roundabout to those of the signalized intersection. It is suggested that any particular intersection is handled on its merit, taking into account the available land traffic demand intensity, turning movements and traffic composition. Thus a specific intersection is considered for a roundabout and a signalized intersection, the control that gives the lower delay is then prefered for that intersection.

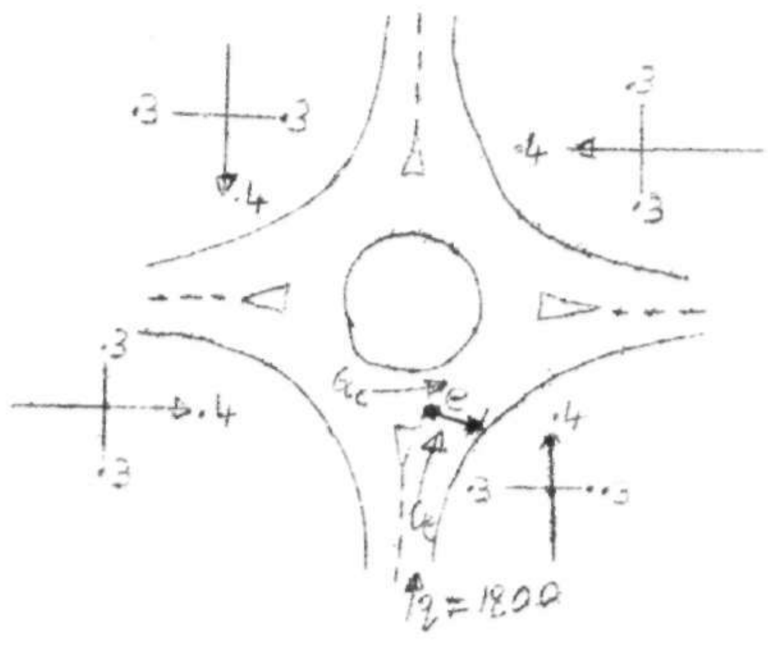
4. ILLUSTRATING THE PROCEDURE

4.1 The Roundabout

4.1.1 Design of the Roundabout

A geometrically symetric roundabout, with four approaches is designed to handle an expected flow of 1800 pcu/hr, per approach. A ballanced flow with turning proportions - 40% straight ahead, 30% right turning and 30% left turning is assumed. The roundabout is designed according to the method set out by Kimber (1980). The process of selecting the appropriate values for the parameters that determine the capacity is itera-tive because the parameters are subjective to constraints which has to be adjusted as the design proceeds. This process is shown below:-

The expected demand flow assumed is 1800 pcu/hr. An entry capacity, Qe, of about 110% of the expected demand flow is required. Thus Qe = 1980 pcu/hr.



.3
.3
.4
.4
.3
.3
.3
.4
.3
.4
.3
q = 1800

The circulating flow, Qc = 1800 x .3 + 1800 (.3 + .4)

= 1800.

Values are then chosen for the geometric parameters (taking the various constraints into account) that will lead to the required traffic capacity, Qe of 1980 pcu/hr. For this hypothetical case, the "central" values, (Kimber, 1980) of D, Ø and r are sellected.

Thus D = 60m

Ø = 30^{o}

= 20m

X_2 is obtained from, $X_2 = \dfrac{Qe + 0.26\ Qc}{303 - 0.053\ Qc}$

$X_2 = 11.8$

If V = 7.3m is fixed by the half width of the approach and is estimated at maximum acceptable to be 10m.

e is obtained from, $e = \dfrac{V - (X_2 - v)/1}{1 - 3.2\ (X_2 - v)}$

e = 14.3.

Assuming that these geometric parameters have been tested on site and are found to be consistent with site conditions, thence

X_2 is further re-calculated from

$X_2 = \dfrac{(Qe/k) + 0.21 t_D Qc}{303 - 0.042\ t_D Qc}$

where k and t_D are as defined in eq(2.9).

The value of X_2 got from this is 11.76 $\simeq$ 11.8, there is then no need for further adjustment.

The final Design Geometric Parameters are

$D = 60$
$\phi = 30^{\circ}$
$\gamma = 20m$
$v = 7.3m$
$l = 40m$

with value of $X_2 = 11.76$

Substituting these values in the unified capacity formula for roundabout, eq (2.9).

These results are obtained

$K = 1$
$F = 3563$
$fc = 0.88$

The linear entry/circulating flow relationship for the approaches is thus given by an intercept of 3563 pcu/hr on the entry axis with a slope of 0.88.

i.e. $Qe = 3563 - 0.88\ Qc$ pcu/hr.

With a circulating flow of 1800, the entry capacity in this case is 1979 $\simeq$ 1980 which is the design capacity. The roundabout has thus been designed with entry capacity of 1980 pcu/hr.

This entry capacity would have been obtained for a general case by solving these four simultaneous equations. See equations 3.1 (a, b, c & d).

$$\text{i. } Q_{eA} = F_A - f_{cA}(0.3Q_{eC} + 0.7Q_{eD})$$
$$\text{ii. } Q_{eB} = F_B - f_{cB}(0.3Q_{eD} + 0.7Q_{eD})$$
$$\text{iii. } Q_{eC} = F_C - f_{cC}(0.3Q_{eA} + 0.7Q_{eB})$$
$$\text{iv. } Q_{eD} = F_D - f_{cD}(0.3_{eB} + 0.7Q_{eC})$$

These are solved by iteration using the initial values $Q_{eA} = F_A, Q_{eB} = F_B, Q_{eC} = F_C, Q_{eD} = F_D$ and also since the roundabout is geometrically symetric by taking $F_A = F_B = F_C = F_D$

After 6 iteration the values converge at

Q_{eA} = 1980

Q_{eB} = 1980

Q_{eC} = 1980

Q_{eD} = 1980

The turning proportion are not further varied because the roundabout entry capacity is not affected by turning proportions when the flow is balanced - the flow and turning proportions are all identical for all the approaches - because the circulating flow accross the approaches are all equal to the entry flow. When the flow is not balanced varying turning proportions makes the circulating flows to assume different values and the entry capacities of the approaches with higher circulating flows are reduced.

4.1.2 Delay Estimation - The roundabout

Since the roundabout has an estimated demand flow of 1,800 pcu/hr and entry capacity of 1980 pcu/hr., the traffic intensity, ρ is therefore equal to 0.91 Average delay, Dv(t) to vehicles arriving during a peak period, t, of any duration can therefore be estimated using equation (2.17).

The results obtained when = 0.91 and entry flow = 1000 for peak durations of 10 mins to 60 mins are shown below

Duration of peak, t (min)	10	15	30	45	60
Dv (Secs)	15.0	16.4	18.5	19.4	20

4.2 Design of Signal-Controlled Intersection with same Traffic factors Also Delay Estimation

A four-leg intersection in which both roads have priority is assumed. The four approach roads have half-width of 7.3m each lane widths are all 3.65m. A balanced traffic conditions is assumed in which the proportion of right turners are equal to that of left turners. The flow into all the approaches are each equal to 1800 pcu. Therefore the flow out of each approach is also equal to 1800 pcu.

Unlike in the case of the roundabout where for a balanced flow, the turning proportions do not affect the entry capacity, the signal controlled intersection is always affected by turning proportions. Therefore the turning proportions are varied and three cases are investigated here. These cases are:-

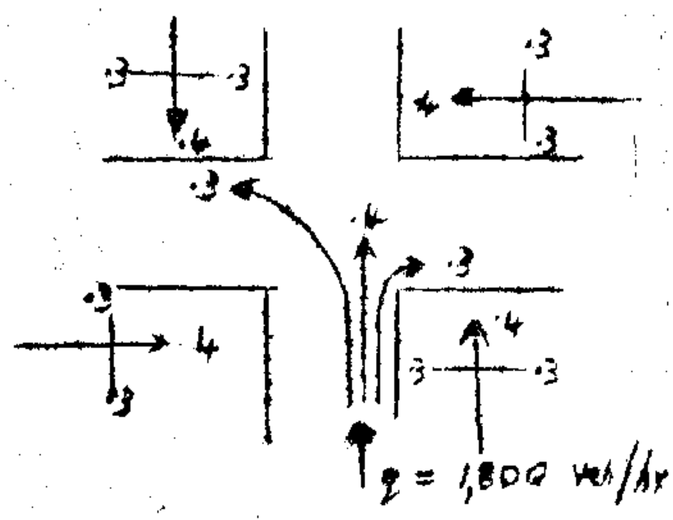
i. The proportion of straight-ahead is 0.4 and that for the right and left-turnings 0.3 each.

ii. The proportion of straight-ahead vehicles is 0.6 and that for right and left turners, 0.2 each.

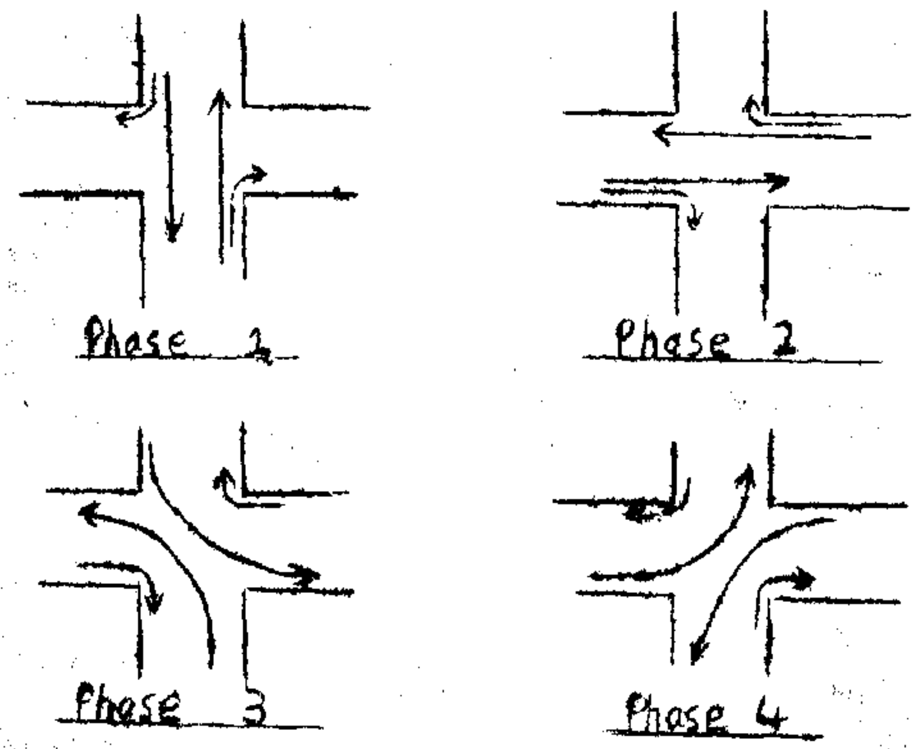
iii. The proportion of straight-ahead vehicles is 0.8 and that for right and left turners, 0.1 each.

For these three hypothetical cases it is, as in the case of the roundabout, assumed that all the vehicles are passenger cars. There is no conversion then to compensate for the effect of traffic composition.

The design of the first case is as follows :-



Only traffic on one approach is shown above, (i.e. South Approach). Due to high proportion of left-turners and considering safety and ease of operation, a four-phase signal control is thought to be most suitable. The phases are as shown below:



All right turners have green, twice in the cycle as shown (e.g. for South approach, the right turners have green in phases 1 and 4). Because of capacity restrictions, and to avoid having Y value greater than unity, the approaches are widened to

5-lanes of 3.65m each, and distributed thus : left-turning flows have 2 lanes on each approach, straight ahead flows also have 2 lanes on each approach and right turning flows have one lane on each approach.

The following assumptions are needed. (See Salter, 1974).

Saturation flow, S, for approach width of 3.65m, S = 1900 pcu/hr

for approach width of 7.3m S = 4015 "

for left right turnings

Single-file flows S = 1600 "

double-file flows " = 2700 "

		flow pcu/hr	Saturated flow	y
1.	North approach, straight ahead	720	4015	0.18
2.	North approach, right turning	270	1600	0.17
3.	North approach, left turning	540	2700	0.2
4.	South approach, straight ahead	720	4015	0.18
5.	South approach, right turning	270	1600	0.17
6.	South approach, left turning	540	2700	0.2
7.	West approach, straight ahead	720	4015	0.18
8.	West approach, right turning	270	1600	0.17
9.	West approach, left turning	540	2700	0.2
10.	East approach, straight ahead	720	4015	0.18
11.	East approach, right turning	270	1600	0.17
12.	East approach, left turning	540	2700	0.2

Phase 1 lanes 1, 4, 2 and 5 will flow, $y_{MAX} = 0.18$

Phase 2 lanes 3, 6, 8 and 11 will flow, $y_{MAX} = 0.2$

Phase 3 lanes 7, 10, 8 and 11 will flow, $y_{MAX} = 0.18$

Phase 4 lanes 9, 12, 2 and 5 will flow, $y_{MAX} = 0.2$

$\therefore Y = .18 + .2 + .18 + .2 = 0.76$

Due to the size of the intersection, the intergreen period is chosen as 6 secs to allow the vehicles time to clear. The starting delay is also taken as 2 secs and Amber time 3 secs.

Total lost time per cycle, $L = (4 + 2)4 = 24$ secs.

The optimum cycle time is then given by Equation 3.2 as

$$Co = 170.8 \simeq 171 \text{ secs.}$$

Total effective green time is obtained as

$$171 - 24 = 147 \text{ secs}$$

Effective green time for phase , $g_{phase\,1} = \frac{147}{0.76} \times .18 = 34.8$ secs

Green time for phase 2, $g_{phase\,2} = \frac{147}{0.76} \times .2 = 38.7$ secs

Green time for phase 4, $g_{phase\,3} = \frac{147}{.76} \times .18 = 34.8$ secs

Green time for phase 4, $g_{phase\,4} = \frac{147}{.76} \times .2 = 38.7$ secs.

With lost time due to starting delays as 2 secs;

the actual green times are:

Phase 1	=	34.8 + 2 − 3	=	33.8 secs.
Phase 2	=	38.7 + 2 − 3	=	37.7 secs.
Phase 3	=	34.8 + 2 − 3	=	33.8 secs.
Phase 4	=	38.7 + 2 − 3	=	37.7 secs.

Delay estimation

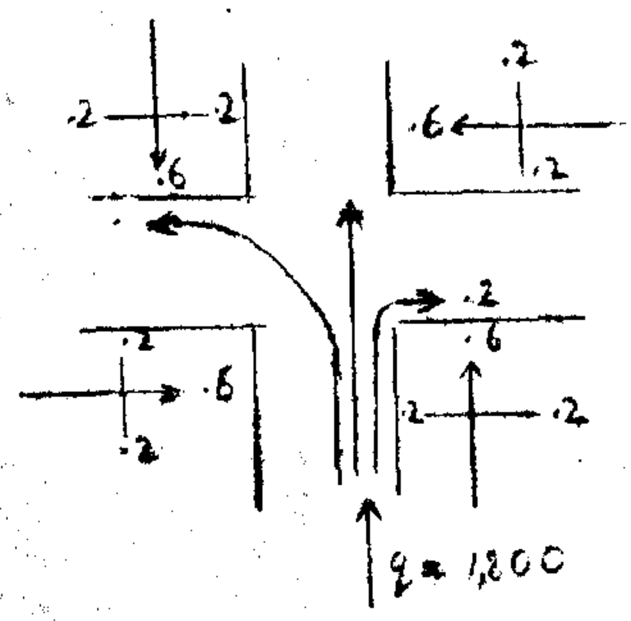
The delay encountered by vehicles at the signal controlled intersection is estimated by equation 2.18 according to Webster (1958). The calculations were done with the aid of Hp.21 prorumable calculator. The results are shown below, for the three flows at an approach. The mean is then taken as the average delay to vehicles at the approach. Since a balanced flow was assumed, this delay is also the average delay to vehicles at other approaches.

	g	C	S	q	1st term	2nd term	3rd term	$\bar{d}$	Total Delay
4	34.6	171	4015	720	66.1	16.3	7.2	75.2	54144
5	34.6	85.5	1600	540	25.5	60	3.5	28.0	15139
6	38.7	171	2700	540	64	22.4	8.4	77.7	41958
									111241 secs.

$\therefore$ mean Delay $= \frac{111241}{1800}$

$= 61.8$ secs (Average per vehicle)

2nd Case   The design of the second case is as follows:

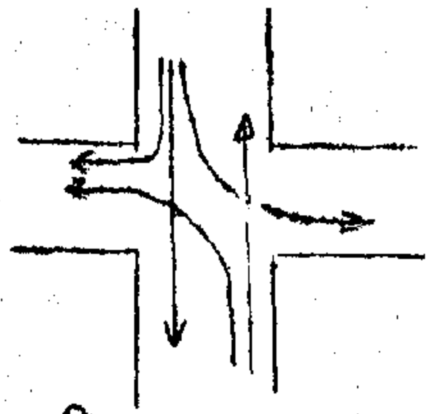


Again 5-lanes are used as less number would result to Y value greater than 1.0. Thus with 5-lanes of 3.65m on each approach, the lanes are distributed as below: left turning flows have 2 lanes on each approach, straight ahead flows also have 2 lanes on each approach and right turning flows have one lane on each approach. Again balanced flow is assumed.

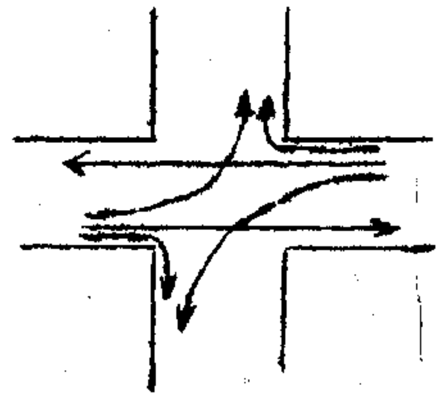
		Flow	S	y	y_{max}
1.	North Approach, straight ahead	1080	4015	0.27	.27
2.	North Approach, right turning	360	1600	0.23	or
3.	North Approach, left turning	360	2700	0.13	.13 + .27
4.	South Approach, straight ahead	1080	4015	0.27	
5.	South Approach, right turning	360	1600	0.23	
6.	South Approach, left turning	360	2700	0.13	
7.	West Approach, straight ahead	1080	4015	0.27	.27
8.	West Approach, right turning	360	1600	0.23	or
9.	West Approach, left turning	3600	2700	0.13	.13 + .27
10.	East Approach, straight ahead	1080	4015	0.27	
11.	East Approach, right turning	360	1600	0.23	
12.	East Approach, left turning	360	2700	0.13	

$$Y = 0.8$$

A two phase, with early cut-off signal control is adopted.



Phase 1



Phase 2

Using intergreen period of 7 sec. since the intersection is large.

Total lost time per cycle = $(4 + 2) \times 2$ = 12 secs.

The optimum cycle time, Co, is obtained from

equation 3.2 as, 115 secs.

Total effective green time = 115-12 = 103 secs.

green time for phase 1,

g_{phase_1} = 51.5 secs

green time for phase 2

g_{phase_2} = 51.5 secs

The green time is divided into 2 stages for both phases in the ratio 0.27 : 0.13

i.e. 34.8 secs and 16.7 secs

With this early cut-off technique the straight ahead have right of way for the first 34.8 secs of the green time, after this they lose the right of way and the left turners then have right of way for the remaining 16.7 seconds. This is the most economical of all the alternatives tried.

Delay estimation

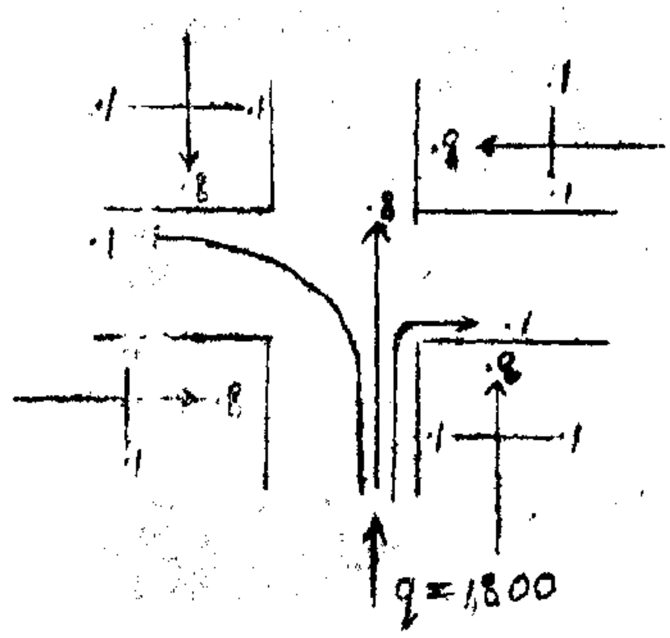
The delay encountered by vehicles at this signal controlled intersection is estimated by equation 2.18 according to Webster (1958). The calculations were done with the aid of H.P. 21 programable calculator. The results are shown below, for the 3 flows at south approach. The mean is taken as the average delay to vehicles at the approach. Since a balanced flow was assumed, this delay is also the average delay to vehicles at other approaches.

	g	c	s	q	1st term	2nd term	3rd term	$\bar{d}$	Total Delay
4	34.8	115	4015	1080	38.3	11.9	4.7	45.6	49248
5	51.5	115	1600	360	22.6	2.5	0.8	24.3	8748
6	16.7	115	2700	360	48.5	51.5	11.5	88.5	31860
									89856 secs

Mean Delay $= \frac{89356}{1800} = 49.9$ secs

$= 49.9$ secs (Average per vehicle)

3rd Case

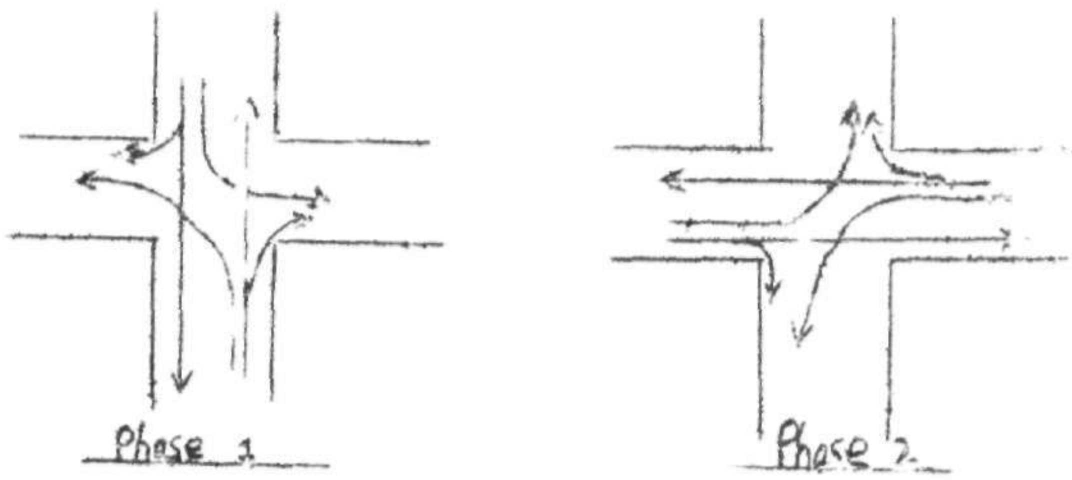


3 lanes can not be used without Y being more than 1.0, 4 lanes are therefore assumed for each approach and distributed as follows :- Left turning flows occupy one lane, on each approach straight ahead flows and right turning flows share 3 lanes (i.e they are mixed). This is the most economical (in land needed of all alternatives considered.

	Flow	S	y	y_{max}
North approach, straight ahead	1440	6025	.29	.20 or .29 + .11
North approach, right turning	315			
North approach, left turning	180	1600	0.11	
South approach straight ahead	1440	6025	0.29	
South approach, right turning	315			
South approach, left turning	180	1600	0.11	
West approach, straight ahead	1440	6025	0.29	.29 or .29 + .11
West approach, right turning	315			
West approach, left turning	180	1600	0.11	
East approach, straight ahead	1400	6025	0.29	
East approach, right ahead	315			
East approach, left turning	180	1600	0.11	

Y = .8

A two phase with early cut-off signal control is adopted.



Using intergreen period of 6 secs (because of the size of the intersection) starting delay of 2 secs. and Amber time of 3 secs

Total lost time per cycle = (3 + 2) X_2 = 10 secs.

Optimum cycle time, Co, is obtained from equation 3.2 as

100 secs.

Total effective green time = 100 − 10 = 90 secs.

Green time for phase 1 = 45 secs

Also green time for phase 2 = 45 secs

This time is divided into 2 stages for both phases in the ratio 0.29 : 0.11, to give

32.6 secs and 12.6 secs.

The actual green times are 31.6 secs and 11.6secs. Thus with the early cut-off technique, the straight ahead flow have right of way for the first 31.6 secs., of the green time, after this they lose the right of way and the left turning flows then have right of way for the remaining 11.6 secs.

Delay Estimation

The delay encountered by vehicles at this signal controlled intersection is estimated by equation 2.18 according to Webster, 1958. The calculations were done with the aid of HP.21 programable calculator. The results are shown below, for the 3 flows at the south approach. The mean is taken as the average delay to vehicles at the approach.

g	c	s	q	1st Term	2nd Term	3rd Term	$\bar{d}$	Total Delay
32.6	100	6	1755	32	7.7	3.2	36.5	64057.5
12.4	100	1600	180	44.2	88.8	17.2	114.8	20664
								84721.5

$$\text{Average Delay} = \frac{84721.5}{1800}$$

$$= 67.1 \text{ secs. (Average per vehicle)}$$

The results obtained, in this section are summarized below.

The roundabout

Geometrically symetric and designed to handle a balanced flow of 1,800 vehicles per hour at traffic intensity, of 0.91 For balanced flow the entry capacity is not affected by turning proportions, e = 14.3m, v = 7.3m and D = 60m amongst other results.

The delay for various values, duration of peak period, (block time) are:

Duration of peak Period, t, min	10	15	30	45	60
Dv (secs)	15	16.4	18.5	19.4	20

The Signal Control

Three hypothetical cases of signal control studied gave the following results:

i. When the proportion of straight-ahead vehicles is 0.4 and that for right and left-turnings are 0.3 each.

Number of lanes (half-roundwidth) = 5 lanes
Delay estimate (average per vehicle) = 61.8 secs.

ii. The proportion of straight-ahead vehicles is 0.6 and that for right and left-turnings are 0.2

Number of lanes (half-roundwidth) = 5 lanes
Delay estimate (Average per vehicle) = 49.9 secs.

iii. The proportion of straight-ahead is 0.8 and that for right and left-turnings 0.1 each.

Number of lanes (half-roundwidth = 4 lanes
Delay estimate (Average per vehicle) = 47.1 secs.

From the above results, the performance of the two controls at different values of turning proportion can be observed and decision could then be taken as to any particular intersection.

5. DISCUSSIONS AND CONCLUSIONS

The roundabout has been modelled as a series of T Junctions linked by the common circulating carriage-way. The results obtained from three roundabouts in Kaduna support the linear entry/circulating flow relationship. For the purpose of estimating delays at roundabout entry, the demand flow can be assumed to be single stream and delays computed by applying the co-ordinate transformation approach. The results obtained are close in magnitude to those obtained from measurements at three roundabouts in Kaduna, bearing in mind the crude nature of the data.

Selecting a particular type of control depends on a number of factors namely, traffic-handling capacity, expected delay to vehicles, land needed for the control, cost of construction and maintenance amongst others. The factor considered here is primarily delay to vehicles, although in practice, land needed for the control and safety of users cannot be ignored.

Equation 2.17, has been adopted for the prediction of delays (average per vehicle) at roundabout entries. This equation needs only the expected demand flow, μ, the traffic intensity, ρ , duration of the peak period t, to estimate the average delay per arriving vehicle during the period.

The behaviour of equation 2.17, with changes in demand flow was illustrated with graphs of delays predicted by it at two levels of traffic intensity (0.9 and 1.0) and "block times" of 15 mins. 30 mins. 45 mins. and 60 mins., as examples. These are presented in Fig. 10. Delays at roundabout entries were computed, also based on equation 2.17, for roundabouts designed for entry capacity of 800 veh/hr. and 1,800 veh/hr. These values are selected because they are typical values obtained during peak periods at intersection. The results are shown in Fig. 12.

When the traffic intensity, ρ , is kept constant by redesigning the roundabout as the demand flow increases, equation 2.17, predicts lower delay to vehicles, with increase in demand flow. This results in abnormally very high delay at very low demand flow, as seen in Fig 12. This increase is as a result of having the demand flow μ as a denominator in equation 2.17, thus making the equation predict infinite delay at zero demand flow. This effect starts after a minimum value of delay has been attained at traffic intensity, ρ , of 0.5 (Fig. 12). Since in this thesis, the intersections are being

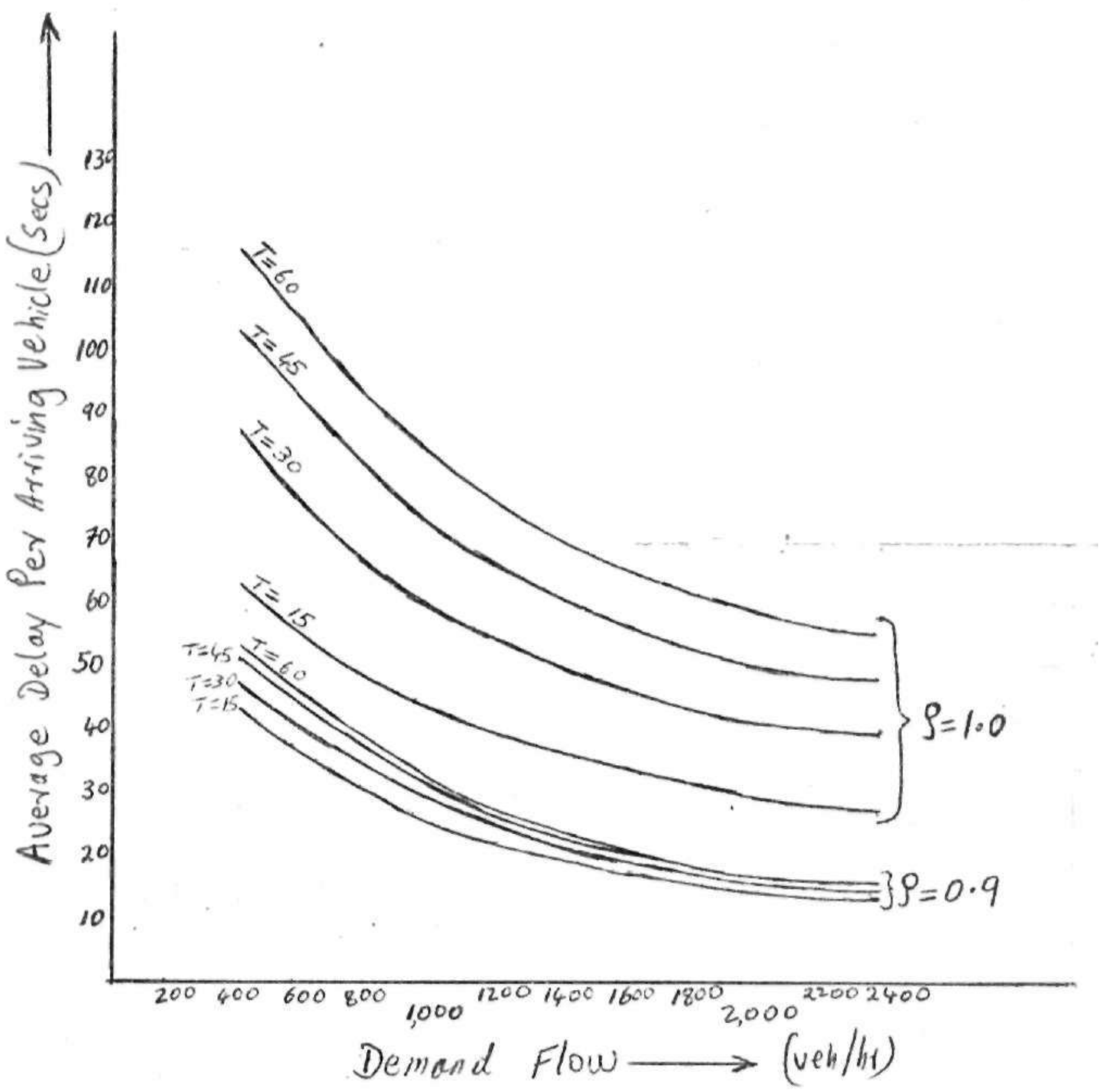


FIG.10 · Roundabout delays at two levels of traffic intensity

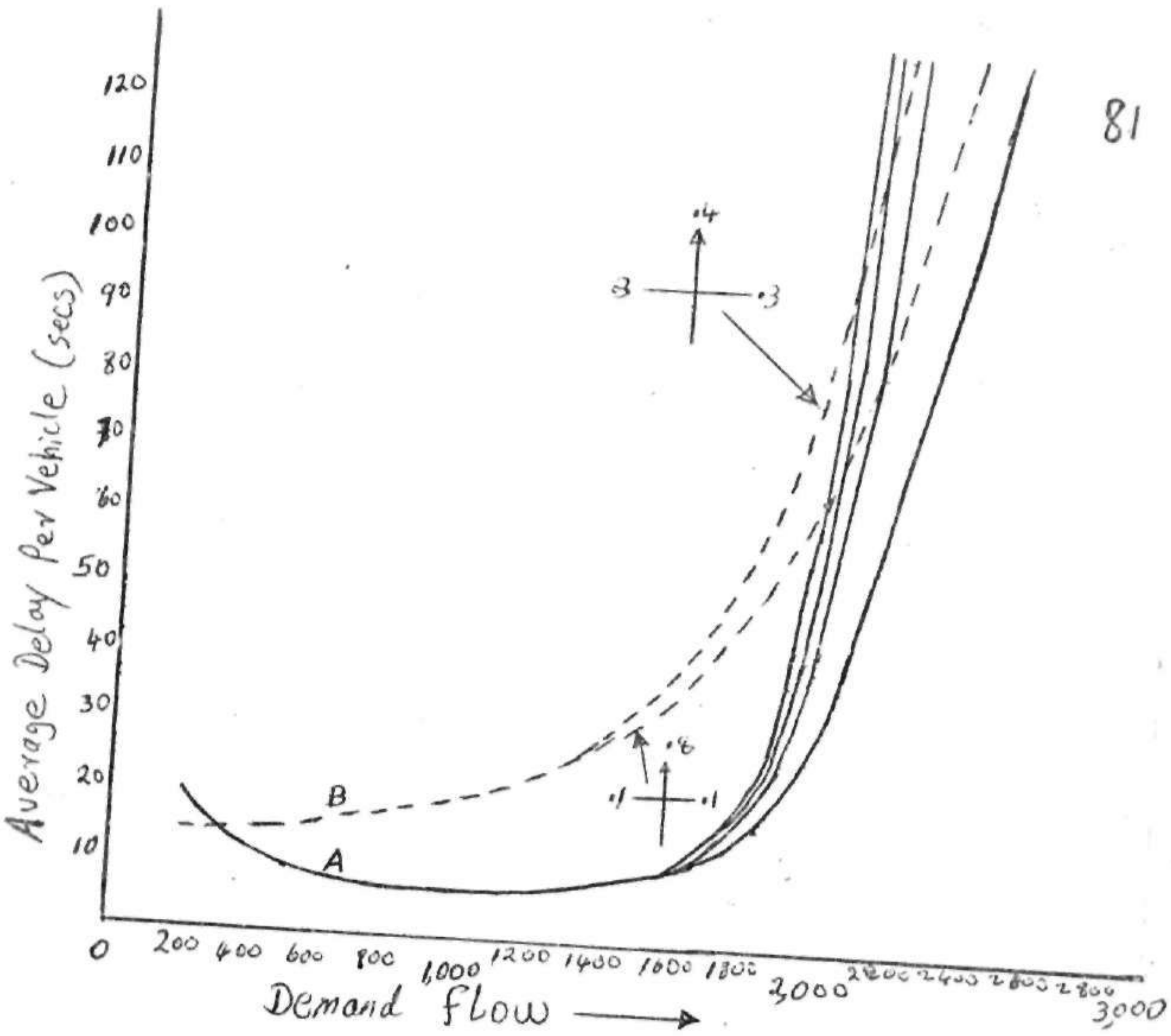
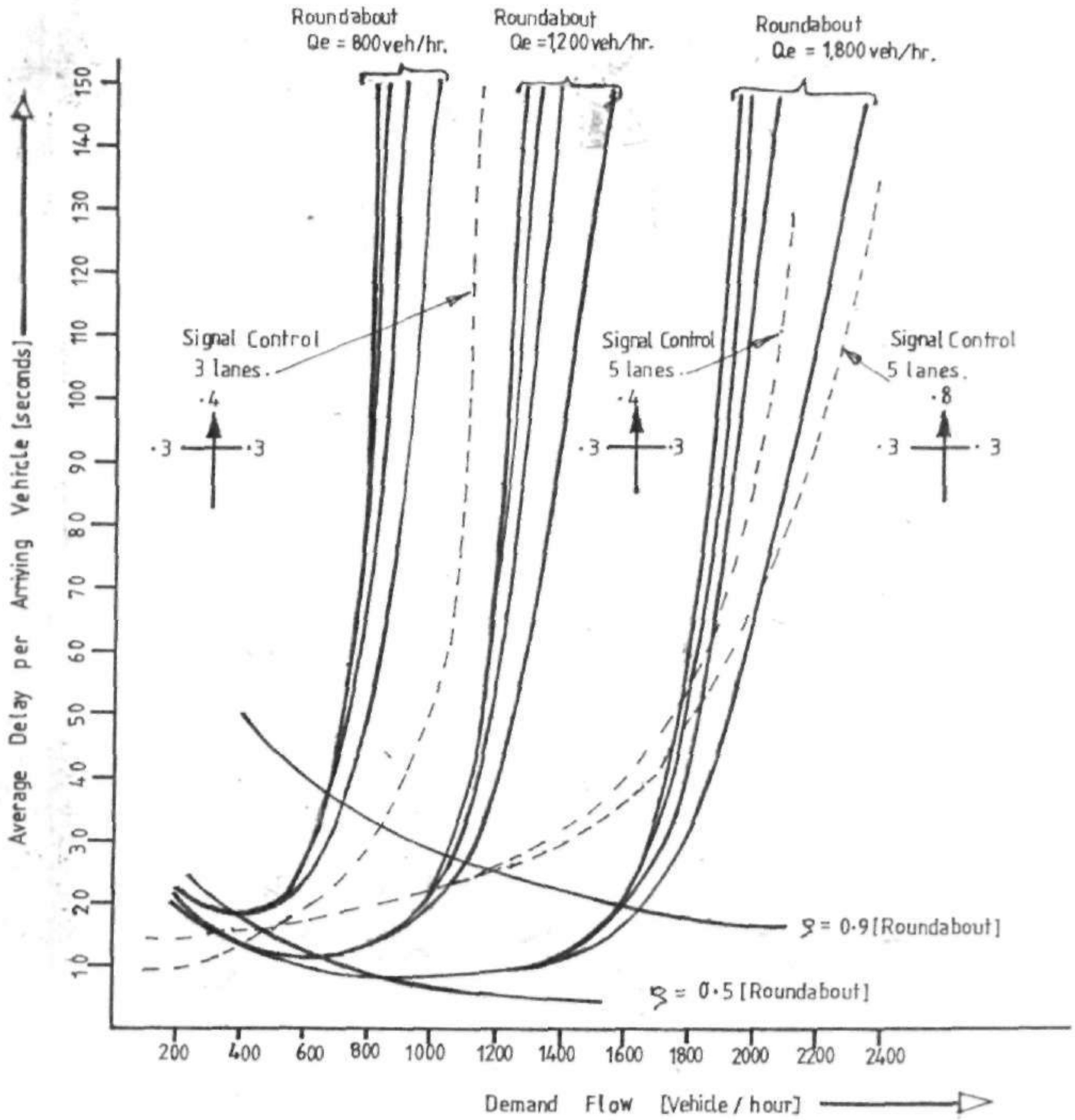


Fig. 11 A Delays at Roundabout entry, designed for entry capacity of 1980 veh/hr. to handle flow of 1,800 veh/hr. at traffic intensity, ρ, of 0.91. The entry width is 14.3m.

B Delays at Signal Control approach, with 5 - lanes of 3.65m.

Demand flow (veh/hr)



Fig·12 Delays at Roundabout And signal Control.

considered during peak periods, the case where the traffic intensity, ρ , becomes less than 0.5, will rarely be encountered. Also, from Fig. 12, when traffic intensity exceeds 1.0, the delays become excessive and increase rapidly as the curves become very steep. On these bases therefore, it is suggested that equation 2.17 be used only when the value of traffic intensity lies between 0.5 and 1.0, because, the equation behaves well between these limits.

Using equation 2.18, delays are estimated for two typical sizes of signal control (approach half-roadwidth of three and five lanes) for varying demand flows and turning proportions. The method adopted in the design is to keep the geometric parameters constant while only changing the timings in accordance with design standards, (see for example salter R.J. (1974).) From these results shown in Fig. 12, it is seen that the approach half-road width of three lanes cannot handle flows of more than 1,000 vehicles per hour without the delay becoming too high with the delay curve becoming very steep. The approach half-roadwidth of five lanes on the other hand can handle flows up to 1,800 vehicle per hour width low delay to vehicles and even after this level the delay curve is less steep.

By considering the results obtained for the approach half-roadwidth of five lanes, it is seen that vehicles encounter more delay when the turning proportions are 40% straight-ahead, 30% right-turning and 30% left-turning, than when the turning proportions are 80% straight-ahead, 10% right-turning and 10% left-turning.

This method of comparing the performance of the two controls can be seen from Fig. 11. Here the signal control was designed on a five-lane approach half-roadwidth (i.e. 18.25m at 3.6m per lane). The designed roundabout, the details are given in section 4.1.1, has entry width e, of 14.3m amongst other parameters given in section 4.1.1. This roundabout was designed foi entry capacity of 1980 vehicles per hour, such that at the expected level of operation - 1,800 vehicles per hour the traffic intensity, ρ , is 0.91. From Fig.11, for this hypotetical case, it could be said that the roundabout gave lower delay than the signal control for values of demand flow up to 2,000 vehicles per hour. If due to other constraints, the signal control has to be used, then the number of lanes available will be increased from five to six or seven. The resulting half-roadwidth of 21.9m or 25.55m, may pose other problems. One may therefore settle for a roundabout at this level of demand flow. In practice, other combinations should be tried, before making any choice.

Similarly from Fig.12, the signal control may be preferred for flows less than 1,000 vehicle per hour. This is because a signal control on approach half-roadwidth of three lanes can easily be designed at most intersections to handle this flow (no consideration is given to monetary cost of constructing any of the two controls, delay is the basis for comparism, land needed by the control and safety to users are also given some considerations), with resulting low delay to vehicles. This delay becomes high when demand flow approaches 1,000 vehicles per hour but increases rapidly after this level as the delay curve becomes very steep. To use the signal control for demand flows above 1,000 vehicles per hour, the intersection has to be redesigned, but the redesigned intersection to handle higher flow will of course need more land which may not always be readily available. If on the other hand, roundabout must be used for demand flows less than 1,000 vehicles per hour, the redesigned one may not compare favourably to the signal control in terms of delay, land needed for the control, safety of users amongs other criteria.

To keep the delays to vehicles at roundabout entries low, it is suggested that the design of the roundabout should be such that at the expected level of operation, the traffic intensity should lie around 0.9. From Fig. 10, it is observed also that at this level of operation, the effect of "block time" on delay is small, and the steepness of the delay curve is low.

For a geometrically symetric roundabout, with balanced traffic condition, the entry capacities of the approaches are not affected by varying the turning proportions.

If the delay to vehicles at an intersection is to be kept at minimum, it is clear from the hypothetical cases studied that turning proportions of vehicles at the approach and the demand flow are important factors in determining the control to be used.

The delay with signal control increases with increase in left-turning proportion.

The delay to vehicles at roundabout entries can be estimated by applying equation 2.17. For a particular intersection, operating at a given traffic condition, this delay can be compared with that produced by other controls, such as the signal control.

If the adopted criterian in selecting the type of control to provide at the intersection is minimizing the delay to vehicles, then the control that achieves this can be chosen.

REFERENCES

1. Ashworth, R.
The Capacity of Priority Intersections with a non-Uniform Distribution of critical acceptance Gaps.
Transpn. Res. 3, 1969.

2. Blackmore, F.C.
Priority at Roundabouts
Traffic Engineering and Control,
Feb. 1963 - pp.104-106.

3. Catling, I.
A time-dependent approach to junction delays.
Traffic Engineering and Control - Nov. 1977.

4. Erica M. Hollis, Marie C. Semmens & Sharon L. Dannies
ARCADY: A computer program to model capacities queues and delays at roundabouts
TRRL laboratory report 940, 1980.

5. Glen M.G., Summer S.L. and Kimber R.M.
The capacity of Offside Priority Roundabout Entries
TRRL Supplementary Report, 436, Crowthorn 1978.

6. Halcrow Fox and Associates (1978a)
The capacity of Entries to Offside Priority Roundabouts.
Feb. 1978.

7. Halcrow Fox and Associates (1978 b)
The capacity of Entries To Offside and Conventional Roundabouts.
May, 1978.

8. Hollis, Erica M.
A Program To Model Flows, Queues And Delays at a Roundabout.
Traffic Engineering and Control. pp.163-165, April, 1979.

9. Kimber, R.M., Erica M. Hollis
"Traffic Queues And Delays At Road Junctions
TRRL Laboratory Report 909, Crowthorne 1979.

10. Kimber, R.M.
The Traffic Capacity of Roundabouts
TRRL Laboratory Report, 942, Crowthorne 1980.

11. Kimber, R.M. & Erica M. Hollis
Peak - Period Traffic Delays At Road Junctions
And other Bottlenecks
Traffic Engineering And Control Oct. 1978

12. Kimber, R.M., M. Marlow and Erica M. Hollis
Flow/Delay Relationships For Major/Minor Priority
Junctions
Traffic Engineering and Control November, 1977

13. Marie C. Semmens, P.J. Fairweather & I.B. Harison
Roundabout capacity; Public Road Experiment At
Winchheap, Canterbury.
TRRL Supplimentary Report 554, Crowthorne 1980.

14. Mc Donald M. Armitage D.J.
The Capacity of Roundabouts
Traffic Engineering and Control
pp.447 - 450, October 1978.

15. Michalopoulos P.I. and Stephanopoulos G.
Oversaturated Signal Systems with Queue length constraints -1

16. Morse P.M.
Queues, Inventories and Maintenance
John Willey & Sons.

17. Salter R.J.
Highway Traffic Analysis and Design
Addion-Wesley Publishing Company Inc. (1974)

18. Tanner, J.C.
A Theoretical Analysis of Delays At an
Uncontrolled Intersection.
Biometrika, 49, pp.163-170, 1962.

19. Wardrop, J.C.
The Traffic capacity of Weaving Sections of Roundabouts
Proc. 1st Conference on Ops. Res. London
pp.266-80, 1957.

20. Webster, F. U.
Traffic Signal Settings
Road Res. Tech. Pap. No.39 HMSO
London, 1958.

21. Webster, F.V.
Greenford Roundabout Experiment.
Traffic Engineering & Control,
pp. 266-271, May, 1962.

APPENDIX A

Dependence of F And f_c On Various Geometric Parameters.

Roundabout Entry Capacity, $Q_e = F - f_c Q_c$.

$e_{(m)}$	$V_{(m)}$	$l_{(m)}$	$D_{(m)}$	$r_{(m)}$	ϕ^o	S	X_2	t_D	K	$F=(KF')$	$f_c=(Kf_c')$
5	3	10	10	4	30	0.2	4.43	1.5	0.804	1079.7	0.478
"	"	"	"	"	35	"	"	"	0.787	1056.4	0.468
"	"	"	"	"	40	"	"	"	0.770	1033.0	0.457
"	"	"	"	"	45	"	"	"	0.752	1009.8	0.447
"	"	"	"	"	50	"	"	"	0.735	986.6	0.437
"	"	"	"	"	55	"	"	"	0.718	963.3	0.426
"	"	"	"	"	60	"	"	"	0.700	940.0	0.416
5	3	10	10	4	30	"	"	"	0.804	1079.7	0.476
"	"	"	"	10	"	"	"	"	0.951	1276.6	0.565
"	"	"	"	15	"	"	"	"	0.984	1320.4	0.584
"	"	"	"	20	"	"	"	"	1.00	1342.3	0.594
"	"	"	"	25	"	"	"	"	1.01	1355.4	0.600
"	"	"	"	30	"	"	"	"	1.016	1361.2	0.604
"	"	"	"	35	"	"	"	"	1.021	1370.0	0.607
"	"	"	"	40	"	"	"	"	1.024	1375.1	0.609
"	"	"	"	50	"	"	"	"	1.029	1381.7	0.612
"	"	"	"	70	"	"	"	"	1.035	1389.2	0.615
5	3	10	10	4	30	"	"	1.5	0.804	1079.7	0.478
"	"	"	20	"	"	"		1.49	"	"	0.475
"	"	"	30	"	"	"	"	1.48	"	"	0.472
"	"	"	50	"	"	"	"	1.37	"	"	0.436
"	"	"	70	"	"	"		1.13	"	"	0.360
"	"	"	90	"	"	"	"	1.02	"	"	0.325
"	"	"	120	"	"	"	"	1.001	"	"	0.318
5	3	5	10	4	30	0.4	[illegible]	1.5	"	1001.7	0.462
"	"	7	"	"	"	0.29	4.27	"	"	1040.7	0.470
"	"	12	"	"	"	0.17	4.5	"	"	1096.8	0.481
"	"	17	"	"	"	0.12	4.62	"	"	1126.0	0.488
"	"	22	"	"	"	0.09	4.69	"	"	1143.1	0.491
"	"	30	"	"	"	0.07	4.76	"	"	1160.1	0.495

APPENDIX A. CONTD.

e(m)	V(m)	L(m)	D(m)	γ(m)	φ°	S	X_2	t_g	K	$F=(KF')$	$f_c=(Kf'_c)$
5	2.0	5	10	4	30	0.6	3.36	1.5	0.804	818.9	0.424
"	2.5	"	"	"	"	0.5	3.75	"	"	914.0	0.443
"	3.0	"	"	"	"	0.4	4.11	"	"	1001.7	0.462
"	3.5	"	"	"	"	0.3	4.44	"	"	1082.2	0.478
"	4.0	"	"	"	"	0.2	4.71	"	"	1148.0	0.492
"	5.0	"	"	"	"	0.0	4.50	"	"	1218.6	0.507
"	6.0	"	"	"	"	-0.2	4.33	"	"	1055.4	0.470
2.5	3	5	10	4	30	-0.1	2.38	1.5	0.804	5801.1	0.374
3.0	"	"	"	"	"	0.0	3.0	"	"	731.2	0.405
4.0	"	"	"	"	"	0.2	3.71	"	"	904.2	0.441
5.0	"	"	"	"	"	0.4	4.11	"	"	1001.7	0.462
6.0	"	"	"	"	"	0.6	4.36	"	"	1062.7	0.474
7.0	"	"	"	"	"	0.8	4.54	"	"	1106.5	0.483
8.0	"	"	"	"	"	1.0	4.67	"	"	1138.2	0.490
9.0	"	"	"	"	"	1.2	4.74	"	"	1155.3	0.494
10.0	"	"	"	"	"	1.4	4.84	"	"	1179.0	0.499
11.0	"	"	"	"	"	1.6	4.9	"	"	1194.3	0.502

BALONI MOTORS AND SINGER ROUNDABOUT - POSITION 1

NAME: ARATAN LANRE

SHEET 1

(1) BALONI MOTORS ROUNDABOUT ZARIA APPROACH

(2) SINGER ROUNDABOUT KADUNA APPROACH

TIME: 26/8/

POSITION : 1

	00	01	02	03	04		05	06	07	08	09		10	11	12	13	14		15	16	17	18	19		20	21	22	23	24	
00						00						00						00						00						00
01						01						01						01						01						01
02						02						02						02						02						02
03						03						03						03						03						03
04						04						04						04						04						04
05						05						05						05						05						05
06						06						06						06						06						06
07						07						07						07						07						07
08						08						08						08						08						08
09						09						09						09						09						09
10						10						10						10						10						10
11						11						11						11						11						11
12						12						12						12						12						12
13						13						13						13						13						13
14						14						14						14						14						14
15						15						15						15						15						15
16						16						16						16						16						16
17						17						17						17						17						17
18						18						18						18						18						18
19						19						19						19						19						19
20						20						20						20						20						20
21						21						21						21						21						21
22						22						22						22						22						22
23						23						23						23						23						23
24						24						24						24						24						24
25						25						25						25						25						25
26						26						26						26						26						26
27						27						27						27						27						27
28						28						28						28						28						28
29						29						29						29						29						29
30						30						30						30						30						30
31						31						31						31						31						31
32						32						32						32						32						32
33						33						33						33						33						33
34						34						34						34						34						34
35						35						35						35						35						35
36						36						36						36						36						36
37						37						37						37						37						37
38						38						38						38						38						38
39						39						39						39						39						39
40						40						40						40						40						40
41						41						41						41						41						41
42						42						42						42						42						42
43						43						43						43						43						43
44						44						44						44						44						44
45						45						45						45						45						45
46						46						46						46						46						46
47						47						47						47						47						47
48						48						48						48						48						48
49						49						49						49						49						49
50						50						50						50						50						50
51						51						51						51						51						51
52						52						52						52						52						52
53						53						53						53						53						53
54						54						54						54						54						54
55						55						55						55						55						55
56						56						56						56						56						56
57						57						57						57						57						57
58						58						58						58						58						58
59						59						59						59						59						59

APPENDIX B.2 (Sites 1 and 2)

BALONI MOTORS AND SINGER ROUNDABOUT - POSITION 2

SHEET 1 LOCATION: KADUNA BALONI MOTORS R/ABOUT (ZARIA APPROACH) (1) 15 MINUTES 26/8/80 TIME 9.34 - 9.48

DATE: 26/8/80

POSITION: (2)

(2) SINGER R/ABOUT (KADUNA APPROACH) 15 MINUTES

Ex: Clement Solomon

	00	01	02	03	04		05	06	07	08	09		10	11	12	13	14		15	16	17	18	19		20	21	22	23	24	
00						00						00						00						00						00
01						01						01						01						01						01
02						02						02						02						02						02
03						03						03						03						03						03
04						04						04						04						04						04
05						05						05						05						05						05
06						06						06						06						06						06
07						07						07						07						07						07
08						08						08						08						08						08
09						09						09						09						09						09
10						10						10						10						10						10
11						11						11						11						11						11
12						12						12						12						12						12
13						13						13						13						13						13
14						14						14						14						14						14
15						15						15						15						15						15
16						16						16						16						16						16
17						17						17						17						17						17
18						18						18						18						18						18
19						19						19						19						19						19
20						20						20						20						20						20
21						21						21						21						21						21
22						22						22						22						22						22
23						23						23						23						23						23
24						24						24						24						24						24
25						25						25						25						25						25
26						26						26						26						26						26
27						27						27						27						27						27
28						28						28						28						28						28
29						29						29						29						29						29
30						30						30						30						30						30
31						31						31						31						31						31
32						32						32						32						32						32
33						33						33						33						33						33
34						34						34						34						34						34
35						35						35						35						35						35
36						36						36						36						36						36
37						37						37						37						37						37
38						38						38						38						38						38
39						39						39						39						39						39
40						40						40						40						40						40
41						41						41						41						41						41
42						42						42						42						42						42
43						43						43						43						43						43
44						44						44						44						44						44
45						45						45						45						45						45
46						46						46						46						46						46
47						47						47						47						47						47
48						48						48						48						48						48
49						49						49						49						49						49
50						50						50						50						50						50
51						51						51						51						51						51
52						52						52						52						52						52
53						53						53						53						53						53
54						54						54						54						54						54
55						55						55						55						55						55
56						56						56						56						56						56
57						57						57						57						57						57
58						58						58						58						58						58
59						59						59						59						59						59

APPENDIX B.3 (Sites 1 and 2)

BALONI MOTORS AND SINGER ROUNDABOUT - POSITION 3

NAME ZAJI M. BUNU SHEET 1 TIME: 9:37 – 9:4? a.m.

LOCATION BALONI MOTORS ROUND ABOUT 1st 15 minutes DATE 26/8/1.. ISHOUNN APPROACH 11:13 – 11:20

POSTION (3) THREE ZARIA APPROACH SINGER ROUND ABOUT – NEXT 10 MINS

	00	01	02	03	04	05	06	07	08	09	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
00																			—						
01						—																		—	
02		—							—									—						—	
03																									
04																								—	
05														—											
06															—										
07																—				—					
08								—																	
09																									—
10														—						—	—			—	
11																							—		
12																								—	
13																	—								
14																							—		
15										—							—								
16			—																						
17			—														—								
18						—													—						
19																			—						
20		—			—																				
21																	—					—			
22												—	—								—	—			
23																									
24																	—								
25																			—			—			
26																							—		
27						—					—											—			
28													—											—	
29																									
30																						—			
31																									
32				—																		—			
33																						—			
34																		—	—	—					
35			—												—		—				—				
36													—												
37																	—	—			—	—			
38				—	—					—										—					
39																									
40																			—	—		—			
41				—										—						—					
42																			—						
43						—																	—		
44								—					—						—	—				—	
45										—										—					
46							—												—					—	
47																				—					
48																—									—
49																				—					
50	—																			—					
51												—												—	—
52																						—			
53			—		—																—				
54		—						—									—	—		—	—	—			—
55						—										—	—		—	—					
56							—					—									—				—
57								—															—		
58																			—	—	—				—
59																									

APPENDIX B.4 (Sites 1 and 2)

BALONI MOTORS AND SINGER ROUNDABOUT - POSITION 4

Position 4

KADUNA ① R.T. BRISCO R/About. 26/8/80

ZARIA Approach → 1st 15 mins

② Singer R/Abt Kaduna

	00	01	02	03	04		05	06	07	08	09		10	11	12	13	14		15	16	17	18	19		20	21	22	23	24	
00						00						00						00						00						00
01						01						01						01						01						01
02						02						02						02						02						02
03						03						03						03						03						03
04						04						04						04						04						04
05						05						05						05						05						05
06						06						06						06						06						06
07						07						07						07						07						07
08						08						08						08						08						08
09						09						09						09						09						09
10						10	5		2		4	10	6	1	6	4	[illegible]	10						10						10
11						11	4		7			11						11						11						11
12						12						12						12						12						12
13						13						13						13						13						13
14						14						14						14						14						14
15						15						15						15						15						15
16						16						16						16						16						16
17						17						17						17						17						17
18						18						18						18						18						18
19						19						19						19						19						19
20						20					5	20	3	4	5	5	4	20						20						20
21						21		3				21						21						21						21
22						22						22						22						22						22
23						23						23						23						23						23
24						24						24						24						24						24
25						25						25						25						25						25
26						26						26						26						26						26
27						27						27						27						27						27
28						28						28						28						28						28
29						29					5	29				2		29						29						29
30						30						30		6	3	7	3	30						30						30
31						31						31						31						31						31
32						32						32						32						32						32
33						33						33						33						33						33
34						34						34						34						34						34
35						35						35						35						35						35
36						36						36						36						36						36
37						37						37						37						37						37
38						38						38						38						38						38
39						39						39						39						39						39
40						40					4	40	2	3	2	2		40						40						40
41						41						41						41						41						41
42						42						42						42						42						42
43						43						43						43						43						43
44						44						44						44						44						44
45						45						45						45						45						45
46						46						46						46						46						46
47						47						47						47						47						47
48						48						48						48						48						48
49						49						49						49						49						49
50						50						50	5	3	4	2		50						50						50
51						51						51						51						51						51
52						52						52						52						52						52
53						53						53						53						53						53
54						54						54						54						54						54
55						55						55						55						55						55
56						56						56						56						56						56
57						57						57						57						57						57
58						58						58						58						58						58
59						59						59	3	2		-		59	22	14	12	14	6	59	13	5	18	4	13	59

APPENDIX C.1 (Site 3)

CHELERAM'S ROUNDABOUT - POSITION 1

DATE :- 14/8/80 SHEET 2 TIME : 3 [illegible]

POSITION 1 / 1

LOCATION : CHELLARAM'S ROUND ABOUT (Bank Approach) ABATAN LODGE

	00	01	02	03	04	05	06	07	08	09	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
04	7	4	7	7	5	7	10	8	5	4	2	9	7	6	5	3	7	5	4	4	5	4	7	6	3
18	9	5	6	4	5	6	5	8	7	3	1	6	6	5	5	5	7	8	5	6	5	8	6	8	6
35	5	8	7	5	8	10	-	-	2	6	2	8	7	6	2	6	8	7	3	5	6	[illegible]	[illegible]	-	6
50	3	6	7	7	6	1	6	4	7	5	7	6	7	6	4	2	7	7	5	6	5	[illegible]	[illegible]	[illegible]	-
59	24	23	27	23	26	24	22	[illegible]	[illegible]	18	15	23	22	[illegible]	16	11	[illegible]	24	17	21	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

APPENDIX C.2 (Site 3)

CHELERAM'S ROUNDABOUT - POSITION 1

POSITION 2

LOCATION: ZARIA APPROACH (CHELLARAMS APPROACH. roundabout)

Eq. Schikun

	00	01	02	03	04	05	06	07	08	09	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
05–06	8	4		6	4	4	7	5	5	4	5	4	7	3	6	6	5	7	7	5	8	7	4	7	6	3
09–10	3																									
21–22	4		-	[illegible]	7	7	3	7	7	5	4	8	6	4	7	6	3	6	3	3	5	1	7	3	10	6
24	5																									
34–35	7		3	4	5	5	8	8	7	5	7	[illegible]	7	8	5	5	4	6	8	6	8	5	7	4	8	3
54–55	6	3		8	6	3	7	6	8	6	3	5	10	4	3	4	7	7	6	2	4	4	5	1	3	[illegible]
59	33			26	24	19	25	26	27	20	19	23	24	19	21	21	19	[illegible]	24	[illegible]	25	[illegible]	25	[illegible]	[illegible]	

APPENDIX C.3 (Site 3)

CHELERAM'S ROUNDABOUT - POSITION 3

CHELLARAMS ROUNDABOUT 14/8/80 TIME 11:30 a.m

ZARIA APPROACH

POSITION THREE (3) Zagi M. Bunu

Sec	00	01	02	03	04	05	06	07	08	09	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
00																									
01																									
02																									
03																									
04	3	3	5	3	3	2	4	2	2	2	5	1	4	1	3	3	2	2	4	1	1	5	3	3	3
05																									
06																									
07																									
08																									
09																									
10																									
11																									
12																									
13																									
14																									
15																									
16																									
17																									
18																									
19																									
20																									
21	3	5	1	2	3	4	4	4	1	1	3	2	5	3	3	1	0	3	2	4	8	2	5	4	1
22																									
23																									
24																									
25																									
26																									
27																									
28																									
29																									
30																									
31																									
32																									
33																									
34																									
35																									
36	3	3	6	5	5	3	4	3	1	2	4	4	3	2	2	4	3	2	2	3	4	2	4	3	4
37																									
38																									
39																									
40																									
41																									
42																									
43																									
44																									
45																									
46																									
47																									
48																									
49	4	5	3	5	4	2	6	3	3	4	5	2	7	5	3	1	4	3	7	3	4	2	1	1	3
50																									
51																									
52																									
53																									
54																									
55																									
56																									
57																									
58																									
59	13	[illegible]	15	15	15	10	15	12	9	9	17	9	19	11	11	[illegible]	9	[illegible]	15	11	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

APPENDIX C.4 (Site 3)

CHELERAM'S ROUNDABOUT - POSITION 4

Sec.	00	01	02	03	04	05	06	07	08	09	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
14/15	6	6	8	7	5	5	4	7	5	3	6	7				7	7	5		9	7			5	5
29/30	7	7	4	5	6	9	7	4	1	5	7	5	4			7		7	8		5	7	6	9	4
44	3	5	6	7	7	4	6	5	6		5	4	8	7		8	9		5		8	[illegible]	[illegible]		4
45	7	5		7	0	6	8	5	7		9	7	10	10		7	7			9	7	3	7	3	5
59		19		27		32	24	27	26	23	26	26	25			26					2				