A COMPUTER SIMULATION STUDY OF THE EFFECTS OF FLARING AND TURNING MOVEMENTS ON ROUNDABOUT ENTRY PERFORMANCE

by

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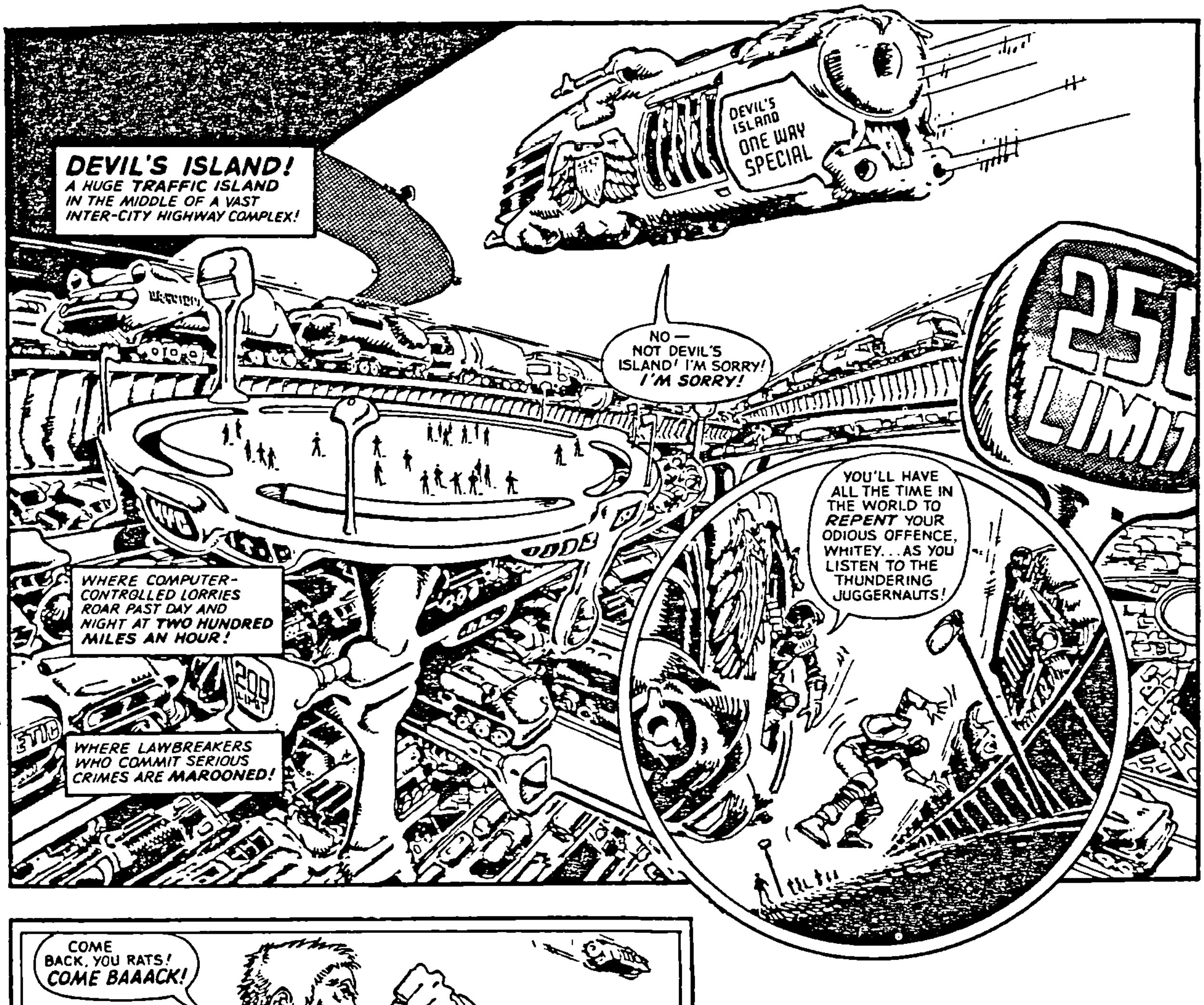
DEVIL'S ISLAND!
A HUGE TRAFFIC ISLAND IN THE MIDDLE OF A VAST INTER-CITY HIGHWAY COMPLEX!
DEVIL'S ISLAND ONE WAY SPECIAL
NO — NOT DEVIL'S ISLAND! I'M SORRY! I'M SORRY!
250 LIMIT
WHERE COMPUTER-CONTROLLED LORRIES ROAR PAST DAY AND NIGHT AT TWO HUNDRED MILES AN HOUR!
200 LIMIT
WHERE LAWBREAKERS WHO COMMIT SERIOUS CRIMES ARE MAROONED!
YOU'LL HAVE ALL THE TIME IN THE WORLD TO REPENT YOUR ODIOUS OFFENCE, WHITEY...AS YOU LISTEN TO THE THUNDERING JUGGERNAUTS!


COME BACK, YOU RATS! COME BAAACK!
AIN'T NO ESCAPE FROM DEVIL'S ISLAND, MAN! THOSE TWO HUNDRED MILE AN HOUR MONSTERS NEVER STOP! YOU'RE HERE FOR LIFE — LIKE THE REST OF US! HEE, HEE!

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SUMMARY

This thesis reports on an investigation carried out to study the effect of flaring and turning movements on the performance of roundabout entries.

A computer simulation program was developed to carry out the investigation. The model simulates an entry with two lanes at the approach section and four at the stop line. It can be modified easily to simulate straight entries by changing the input and one DATA statement.

Data were collected at three public road sites at Sheffield to validate the model. A method of analysing the data was developed to obtain values of the gap-acceptance parameters. The values arrived at were used subsequently as input into the model to allow direct comparison of observed and simulated values. The comparison concluded that the model represents adequately the real conditions.

The results produced showed that average delay for below-capacity operation is reduced by at least 40% when an entry is flared. Capacity improvement, measured as the effective number of lanes of a flared approach, is shown to be influenced by the circulating flow. There is an improvement of 50% for all studied casesfor circulating flow of 2300 veh/hr and more.

Turning proportions do not affect capacity of straight entries but do affect that of flared entries. There is a difference of 25 - 30% between the extreme values depending on the proportion of left-turning vehicles. Turning proportions affect delays of both straight and flared entries. Minimum

delay was obtained for combinations which include 30 - 40% left-turning proportion.

TABLE OF CONTENTS

			Page No
Acknowledgements			i
Summary			ii
Table of Contents			iv
List of Figures			vi
List of Tables			viii
CHAPTER ONE		INTRODUCTION	
	1.1	Roundabout Design	1
	1.2	Roundabout Capacity Theory	2
	1.3	Roundabout Delay Theory	3
	1.4	The Objective of This Study	3
CHAPTER TWO		LITERATURE REVIEW: CAPACITY AND DELAY AT ROUNDABOUTS	
	2.1	Introduction	6
	2.2	Roundabout Operation Before the Priority Rule	6
	2.3	Roundabout Operation Since the Introduction of the Offside Priority Rule	8
	2.4	The Need for New Design Formulae	10
	2.5	Gap Acceptance Models	12
	2.6	Empirical Capacity Models	17
	2.7	Official Design Formulae in Britain	21
	2.8	The Estimation of Delay	24
CHAPTER THREE		COLLECTION OF DATA	
	3.1	Introduction	39
	3.2	The Collection of Gap Acceptance Data	39
	3.3	Turning Movements and Lane Occupancy	43
CHAPTER FOUR		GAP ACCEPTANCE CHARACTERISTICS OF THE ENTERING VEHICLES AND THE MINIMUM HEADWAY OF THE CIRCULATING FLOW	
	4.1	Introduction	62
	4.2	Gap Acceptance Studies	63
	4.2.1	The Critical Gap as the Median of a Distribution	65
	4.2.2	Raff's Critical Lag	66
	4.2.3	Other Methods to Determine the Critical Gap	67
	4.2.4	Gap Acceptance Theoretical Models	68
	4.2.5	The Move-Up Time, β	70
	4.3	The Work of Armitage and McDonald	71
	4.4	Some Linear Models Suggested by Previous Research	77
	4.4.1	Description of the Models	79
	4.4.2	The Comparison of Models by Maher and Dowse	82
	4.5	The Development of a Simple Linear Model	83
	4.5.1	Theoretical Aspects of Linear Regression	83

	4.5.2	Simulated Data	85
	4.5.3	The Use of Rejected Gaps	86
	4.5.4	Extreme Values	88
	4.5.5	The Use of Weights	89
	4.5.6	Dependent and Independent Variables	91
	4.5.7	Effect of Sample Size	92
	4.5.8	The Effect of Assuming the Gap Characteristics not Constant	93
	4.6	Comparison of the Simple Linear Model and Armitage and McDonald's Two-Line Model	93
	4.7	Application of the Simple Linear Model on Observed Data	95
	4.8	The Minimum Headway of the Circulating Flow	96
	4.8.1	Introduction	96
	4.8.2	Distributions of Traffic Headways	97
	4.8.2.1	The Poisson Counting Distribution	98
	4.8.2.2	The Negative Exponential Distribution	99
	4.8.2.3	The Shifted Negative Exponential Distribution	100
	4.8.2.4	Schuhl's Composite Headway Model	100
	4.8.2.5	The Pearson Type III Distribution	101
	4.8.3	Analysis of Collected Data	101
CHAPTER FIVE		THE SIMULATION PROGRAM	
	5.1	Introduction	146
	5.2	Generation of Random Numbers	146
	5.3	Production of the Desired Random Variate	148
	5.4	The Program SIMC	149
	5.5	The Development of SPHT	151
	5.6	Description of SPHT	154
	5.7	Input and Output of SPHT	155
CHAPTER SIX		RESULTS AND COMMENTS	
	6.1	Introduction	160
	6.2	Validation of the Model	160
	6.3	Roundabout Performance	161
	6.3.1	The Effective Number of Lanes	162
	6.3.2	Entry Flow and Turing Proportions	164
	6.3.3	Delay and Turning Proportion	167
	6.3.4	Delay and Flow and Gap Acceptance Parameters	170
CHAPTER SEVEN		CONCLUSIONS	215
REFERENCES			218
APPENDIX 1		THE OBSERVED DATA COLLECTED AT THREE ROUNDABOUTS IN SHEFFIELD	225
APPENDIX 2		THE COMPUTER SIMULATION PROGRAMS	
	2a	Single Entry Simulation Program	229
	2b	Simulation Program SPHT	230
APPENDIX 3		DIFFERENT IN DELAY OBTAINED AT STRAIGHT AND FLARED ENTRIES	243

LIST OF FIGURES

		Page No.
2.1	Wardrop's Formula	28
2.2	Typical Conventional Roundabout Layouts	29
2.3	Small Roundabout Layouts at 3-Way Junctions	30
2.4	Small Roundabout Layouts at 4-Way Junctions	31
2.5	Grade Separated Junction Layout Incorporating Small Roundabouts	32
2.6	Methods of Deflecting Entering Traffic	33
2.7	Mini-Roundabout Layouts	34
2.8	Double Roundabout Layouts	35
2.9	Blackmore's TRRL Experiments	36
2.10	Co-ordinate Transformation	37
3.1	Moore Street Roundabout	48
3.2	Castle Square Roundabout	49
3.3	Park Square Roundabout	50
3.4	Brook Hill Roundabout	51
4.1	Composite Lag and Gap Acceptance Distribution	135
4.2	Raff's Critical Gap	135
4.3	Ramsey-Routledge Method	136
4.4	Multiple Merge Analysis	136
4.5	Armitage and McDonald's Method Geometric Parameters	137
4.6	Armitage and McDonald's Method	138
4.7	Comparison of Input and Predicted Critical Gap Values	139
4.8	Comparison of Standard Deviations of Predicted Critical Gap Values	140
4.9	Comparison of Input and Predicted Move-Up Time Values	141
4.10	Comparison of Standard Deviations of Predicted Move-Up Time Values	142
4.11	Poisson Distribution	143
4.12	Exponential Distribution	143
4.13	Negative Exponential Distribution and Observed Data	144
4.14	Shifted Negative Exponential Distribution	144
4.15	The Erlang Distribution	144
5.1	Generalised Cumulative Probability Distribution	157
5.2	Flow Chart of SPHT	158
6.1	Comparison of Observed and Simulated Capacity	174
6.2	Comparison of Input and Simulated Circulating Flow Values	175
6.3-6.3a	The Effective Number of Lanes and Circulating Flow	176-177
6.4	The Effective Number of Lanes and the Critical Gap	178

6.5	The Effective Number of Lanes and the Move-Up Time	179
6.6-6.9b	Entry Flow and Turning Proportion	180-185
6.10-6.25	Delay and Turning Proportion	186-201
6.26-6.29	Delay and Circulating Flow	202-205
6.30-6.33	Delay and Critical Gap	206-209
6.34-6.35	Delay and Move-Up Time	210-213
A1	Observations at Moore St. Roundabout	225
A2	Observations at Castle Sq.Roundabout	226
A3	Observations at Park Sq. Roundabout	227

LIST OF TABLES

		Page No
3.1	Observed Flows at Three Roundabout Sites	45
3.2	Turning Movements and Lane Usage Observations at Brook Hill Roundabout	46
3.2	Turning Movements and Lane Usage Observations at Moore Street Roundabout	47
4.1	Sites Studied by Armitage and McDonald	106-107
4.2	Average Results of Armitage and McDonald Study	108
4.3	Gap Acceptance Parameters Estimated by Least Squares Analysis	109
4.4-4.17	Comparison of Estimates of Gap Acceptance Parameters by Linear Models	110-122
4.18-4.22	Comparison of Estimates of Gap Acceptance Parameters by Armitage and McDonald's Method	123-125
4.23-4.24	Comparison of Estimates of Gap Acceptance Parameters by All Tested Methods	126-127
4.25	Analysis of Observed Gap Acceptance Data	128
4.26	Frequency of Headways Observed at Castle Sq. Roundabout	129
4.27	Frequency of Arrivals Observed at Castle Sq. Roundabout	129
4.28	Frequency of Arrivals Observed at Moore St. Roundabout	130
4.29	Check of Randomness in the Circulating Flow Data	130
4.30-4.33	χ^2-test on Observed Data of the Circulating Flow Headways and Arrivals	131-134
6.1	Agreement Between Observed and Simulated Capacity of Entry Lanes	172
6.2	Agreement Between Input and Simulated Circulating Flow	173
A3.1-A3.4	Delays Obtained at Flared and Straight Entries	243-246

CHAPTER 1

INTRODUCTION

1.1 Roundabout Design

Intersection control is one of the most important areas of traffic engineering theory and practice, as the performance of any road system and management scheme ultimately depends on the successful design of the intersections.

The cheapest mode of control is the priority junction where one of the intersecting road flows has priority over the traffic on the other roads. This type is suitable for low flows at the minor road. If the combination of the flows reaches certain critical values, the delays incurred by the minor road vehicles exceed acceptable limits. Then, alternative methods of control have to be installed. They include traffic signals, grade-separated layouts and roundabouts.

The first gyratory systems were introduced in Paris in 1907 at the Place de l'Etoile and at the Place de la Nation. They were introduced in Britain in 1925, the Aldwych Island being one of the first in London.

The initial mode of operation of roundabouts did not include a precise definition of the priority of any single stream of traffic at each entry. The two opposing streams were supposed to merge. In practice, however, one or the other of the streams sometimes established priority, forcing the opposing one to wait for suitable gaps in order to continue along its intended path. At high flows in more than one entry, therefore, it became possible for the roundabout to lock. The only solution to that problem, available at the time, was to increase the size of the roundabout, allowing more storage space between entries. The increase of cost associated with larger size layouts, and the decrease of sites that such layouts could be applied usefully, forced researchers to look to alternative ways of improving the performance of roundabouts.

The turning point came in November 1966, when priority to the right was introduced at roundabouts in Britain. This measure prevented any locking, thus stabilizing the flows through the junction and reducing the delays. This allowed the development of design layouts not conforming to the pre-1966 conventions. Size was not significant any longer. It was established that smaller size islands and junctions did give improved performances. New designs suggested include roundabouts with small and mini size islands, layouts incorporating more than one island, and wider entries at the stop line. This design implies that at the stop line there are more lanes for the queueing traffic than further back, on the approach road. However, the above new designs have not replaced completely the conventional large central island roundabouts, which still are used widely, especially at grade-separated intersections. More recently, traffic signals have been introduced in some sites to prevent very long queues and delays suffered in one, or more entries with very heavy flows when the circulating flow is also very heavy.

1.2 Roundabout Capacity Theory

Before the introduction of the priority to the right rule at the roundabouts, their capacity was predicted using formulæ based on the proportion of the traffic weaving within each section. Since 1966, however, and the establishment of a clearly defined priority, weaving does not take place anymore. Up to 1975 the official design formula for conventional roundabouts was based on the weaving proportion; subsequently, however, a modified formula was introduced for these layouts which did not include any weaving parameters. Recently, the T.R.R.L. has published a unified formula to apply both at conventional roundabouts and at new layouts with small islands and flared entries.

The methods and formulæ proposed by various researchers to predict capacities can be divided broadly into two categories, (1) using gap-acceptance theories or (2) relating the capacity to the geometry of the site by empirical observations. (See Chapters 2 and 4 for a detailed presentation of the various suggested methods.)

1.3 Roundabout Delay Theory

Until recently the delay suffered by the entering flows has been estimated either by stochastic or deterministic methods. The former predict adequately delays below capacity, but their predictions tend to infinity as the entering flow approaches capacity; the latter predict zero delay for entering flow below capacity, being better for situations where the capacity is exceeded considerably. The Transport and Road Research Laboratory has proposed time dependent methods of estimating delay which give more realistic results in the region around capacity, being the region of most interest from the point of view of delays (See Chapter 2).

1.4 The Objectives of this Study

During previous work by the present author (Natsinas, 1979), a computer simulation model was developed. That model simulated a single entry to a roundabout with flared lanes, whose approach had two lanes flaring to four at the stop line.

No restrictions were introduced for the lanes used by the entering vehicles that might have been determined by turning movements. The model predicted capacities for the entry as a whole for different combinations of circulating and entering flows and gap acceptance values.

The current project aimed initially to validate the existing simulation model by comparing observed values to the predicted ones. The collection of the data is reported in Chapter 3. For the comparison to be valid, similar conditions as the ones applying to the real situation have to be created by the simulation. The simulation program uses constant values for gap acceptance parameters. During the analysis of the data it became obvious that the abstraction of such parameters was not as straightfoward as envisaged. A lengthy comparison of the available methods became necessary, as well as the development of a new method. Chapter 4 describes the work relating to this aspect.

The computer model was enhanced to include the simulation of turning movements by clearly defining the allowable paths through the entry for each vehicle. The estimation of delay also was improved. Hence, the effect of turning movements on delay and capacity could be studied. The model is described in Chapter 5. Chapter 6 includes the results of the validation and of the improved simulation, which include an estimation of the effective number of lanes of the flared entry. The final conclusions of the study are in Chapter 7.

CHAPTER 2

LITERATURE REVIEW: CAPACITY AND DELAY AT ROUNDABOUTS

2.1 Introduction

Roundabouts as a method of controlling junctions have been employed since the beginning of the century. The first gyratory systems were introduced in Paris in 1907 at the Place de l'Etoile and at the Place de la Nation. In Britain they were introduced in 1925 in London, the Aldwych Island being one of the first (RRL, 1965). Before the Second World War several roundabouts were used at by-pass roads.

Roundabouts in Britain initially operated without a clearly defined priority for any of the two streams of traffic at each entry. Entering traffic had to merge with the circulating. When the entering flow gained priority over the circulating then it could become possible for the whole junction to block. In order to improve this aspect of the operation of roundabouts the offside priority rule was introduced in 1966. This changed radically the operation of roundabouts and gave rise to completely different approaches to their design.

This Chapter concentrates on methods of predicting the capacity and delay at roundabouts since the introduction of the priority rule. Also included are sections on the new layouts of roundabouts and official design procedures.

2.2 Roundabout Operation Before the Priority Rule

Under no clearly defined priority, the operation of roundabouts was based on the weaving of the entry and the circulating traffic streams between successive entry and exit points. Since the 1930's several attempts had been made to estimate the capacity of the weaving sections of roundabouts. However, the most thorough investigation was performed in 1955 and 1956 at the Road Research Laboratory by Wardrop

(Wardrop, 1957). The investigation tested a number of different weaving sections on an artificial test track at Northolt Airport. The study resulted in the following formula for the capacity of each weaving section:

$$Q = \frac{108w(1 + e/w)(1 - p/3)}{1 + w/\ell} \text{ pcu/hr} \qquad \text{(eq. 2.1)}$$

where Q: the capacity of the weaving section (pcu/hr)

w: the weaving width (ft)

e: the average entry width (ft)

p: the proportion of traffic weaving

ℓ: the weaving length (ft)

Figure 2.1 shows the above dimensions.

Subsequent observations at public road sites indicated good agreement in some cases, while in others the calculated capacity overestimated the observed. For this reason the value of the practical capacity, Q_p, was given as 80% of the calculated value

$$Q_p = 0.80 \ Q \text{ (pcu/hr)} \qquad \text{(eq. 2.2)}$$

This relationship was adopted as the official design formula.

Under light or moderate traffic flows the roundabouts functioned satisfactorily, but when the demand approached the capacity locking occurred frequently. This became more pronounced as the late 1950's and early 1960's saw an increase in car ownership and use. Under heavy flows locking was more likely to occur at smaller roundabouts because of the small amount of storage space within the junction. One way, therefore of attempting to avoid locking was to design larger roundabouts. This, however, reduced the possibilities of using roundabouts, especially in urban areas.

At the same time a series of experiments with off-side priority had been conducted. Several local authorities had introduced offside priority since 1956. In 1963, Blackmore drawing from the existing experience up to that time concluded that at roundabouts where the priority-to-the-right rule had been introduced there was an increase in the capacity and reductions in delay and accidents. However, he observed that if the offside rule was followed strictly the capacity would decrease.

2.3 Roundabout Operation Since the Introduction of the Offside Priority Rule

In November 1966 the priority-to-the-right rule was introduced for all roundabouts throughout Britain. Thus, the opposing traffic streams do not weave any more, but, instead, the whole of the roundabout resembles a series of linked T-junctions. The circulatory and entering flows become analogous to the major and minor road flows.

The Road Research Laboratory (RRL, 1969) conducted a series of controlled experiments studying the performance of roundabouts after the new rule was introduced. They reported that the improvement of performance associated with the new mode of operation was not due to an increase in capacity at high demand. Greater capacity at high demand was observed at roundabouts operating under the previous conditions. These high flows, however, were very unstable at saturation and could not be relied as a measure of capacity. The major source of improvement originated from the complete removal of the possibilities of locking.

They looked also at ways of improving further the capacity. Up to then the available ways of improving the capacity of a roundabout was either by increasing its size or by converting it to a multi-level intersection. As both these solutions were very expensive, alternative methods were sought, such that the increase in capacity could be achieved with less expense. One of their observations was that the offside rule improved the performance, and removed locking, even from roundabouts with small central islands. They concluded that the major factor controlling capacity was the shape of the junction.

Blackmore (1970) observed that roundabout capacity was improved if the diameter of the central island was reduced to one third of the diameter of the circle inscribed within the outer kerb line of the roundabout. The capacity was observed to increase more if the entering traffic was deflected to the nearside which would prevent congestion and allow the central island diameter to be reduced further.

The Road Research Laboratory followed the test track experiments by another series conducted on public roads to confirm the above findings.

The first test was at Peterborough (Jervis, 1970) in 1968 where signals controlling a junction were replaced by a series of small roundabouts. The observations showed an increase in capacity of up to 23% as the central island diameter decreased and an overall reduction in delay of up to 50%, though delay at peak hour was not reduced as much. In the early 1970's further tests were carried out at Colchester, Swindon, Sheffield, Halesowen, Hemel Hempstead and Slough. The findings of these tests are summarized by Blackmore and Marlow (1975). At these experiments several designs were tested including mini, multiple and ring layouts. They all showed improvement in capacity and reduction of delay, ranging from 7% to 35%. Blackmore and Marlow compared the small island layouts to the ring junctions. They concluded that the single island ones are more conventional, simpler in design and installation, easier to be understood by drivers, that they give more capacity for 5-arm sites, greater assurance against locking, higher speeds and lower journey times; ring junctions on the other hand are unconventional and, therefore, difficult to understand, but more safe once familiar, they control speeds at lower levels, and are better for pedestrians.

The new layouts have a better safety record where the previous method of control was traffic signals or major/minor priority junctions. However, accidents increase where they replace roundabouts with larger central islands. This was reported by Blackmore and Marlow (1975) and Green (1977).

Figures 2.2 to 2.8 show the layouts of the new designs.

2.4 The Need for New Design Formulae

The priority-to-the-right rule changed radically the way roundabouts operate. As mentioned before, weaving does not occur any more, the junction resembling a series of linked T-junctions. The new types of roundabouts introduced after 1966 had dimensions outside the limits of Wardrop's formula. However, that formula remained as the official design formula for conventional roundabouts until 1975, although the newer small-island layouts were designed in accordance with eq. 2.13.

In 1973, results of two research projects were published which showed that Wardrop's formula was no longer satisfactory for the design of roundabouts. Murgatroyd (1973) showed that the predictions of that formula (eq. 2.1) were overestimating capacity or underestimating it if the 80% practical capacity (eq. 2.2) was used. As the proportion of traffic weaving was no longer relevant he suggested that p should have a value of 1.0 in the formula, i.e. all the traffic should be assumed to weave.

Ashworth and Field (1973) examined the assumed linear relationship of capacity and the weaving proportion in Wardrop's formula with data from two sites in Sheffield. There was no correlation between the two variables, with a slope not significantly different from zero at either site. The observed capacities were considerably different from both the full and 80% practical capacity values.

Ashworth and Laurence (1974) pursued further the examination of the application of Wardrop's formula. Observations from 21 weaving sections were used. The conclusion of the study were that: (1) The capacity of roundabouts is not affected by the proportion of weaving traffic. (2) Observed capacities were approximately 70% of the maximum theoretical capacity as a whole. However, there was considerable scatter for individual entries indicating that Wardrop's formula was no longer reliable. (3) If the weaving proportion was assumed to equal 1.0, the practical capacity predictions were approximately correct overall, but they still produced considerable scatter for individual entries.

Since then a number of alternative formulaehave been produced to predict the capacity. They are described in the subsequent sections.

2.5 Gap Acceptance Models

Before 1966, Tanner (1962) had developed a model of operation of T-junctions based on the gap acceptance behaviour of drivers. Once the operation of roundabouts became similar to that of T-junctions, his model and the gap acceptance parameters formed the basis of a large portion of the research to develop new formulae relating to roundabout performance.

Tanner (1962, 1967) derived the following capacity formula for priority junctions:

$$q_2 = \frac{q_1(1 - \beta_1 q_1)}{e^{q_1(\alpha-\beta_1)}(1 - e^{-\beta_2 q_1})} \qquad \text{(eq. 2.3)}$$

with the following assumptions

(1) The major stream flow consists of a single traffic stream equal to q_1(veh/s); there is a minimum headway, β_1(sec), between successive vehicles in the major stream.

(2) The entering vehicles arrive at the intersection at a rate greater than q_2(veh/s), where q_2 is the entry capacity.

(3) β_2(sec) is the minimum headway of successive entering vehicles.

(4) The critical gap, α(sec) is assumed constant for all drivers.

The above formula formed the basis of a significant portion of the subsequent research on the capacity of roundabouts.

Wohl and Martin (1967) considered roundabouts operating under no clearly defined priority. They assumed that weaving did occur, but they introduced the concept of the critical gap in their formula:

$$C_{max} = \frac{R+1}{R} \frac{T}{t} \log_e(R+1) \qquad \text{(eq. 2.4)}$$

where C_{max}: the capacity of a weaving section, (in vehicles)

R: the weaving ratio = q_1/q_2 where q_1 and q_2 are the weaving flows through the section,

T: the duration of flow in seconds

t: the critical gap in seconds.

The above formula can be derived from Tanner's (Eq. 2.3).

In 1971, Bennett suggested that Tanner's formula could be used for predicting capacities of roundabout entries as follows:

$$q_L = \frac{q_s(1-\beta q_s)}{e^{q_s(\alpha-\beta)}(1-e^{-\gamma q_s})} \qquad \text{(eq. 2.5)}$$

where q_L: the entry flow (veh/s),

q_s: the circulating flow (veh/s),

α: the minimum gap accepted in the circulating flow (sec)

β: the minimum headway in the circulating flow (sec),

γ: the move-up time in the entry flow (sec)

He observed that another factor affecting the capacity is the number of entry lanes. He used 90% of q_L as the practical capacity.

Ashworth and Field (1973) derived an alternative model for capacity prediction. They based it on Wohl and Martin's approach, with the difference that parameter R was defined as the ratio of circulating (Q_1) to entering (Q_2) flows. Then by plotting $\log_e(2R+1)$ vs Q_1 they obtained the following equation

$$Q_2 = \frac{2Q_1}{e^{Q_1/1100} - 1} \qquad \text{(eq. 2.6)}$$

valid for 2-lane entries to roundabouts of the type studied. (Q_1 and Q_2 are both in veh/hr).

In 1974, Horman and Turnbull proposed a simplification of Tanner's formula. They assumed the minimum circulating headway β_1, to be equal to zero, equivalent to a two-lane circulating flow. This reduced Tanner's formula to

$$q_2 = \frac{q_1 \, e^{-\alpha q_1}}{(1 - e^{-\beta_2 q_1})} \qquad \text{(eq. 2.7)}$$

For a 2-lane entry the capacity $Q_2 = 2q_2$ (veh/s). They found the predictions successful if suitable α and β_2 values were used. They also proposed that 80% of the above value was a practical though conservative estimate.

Armitage and McDonald (1974) modified Tanner's formula to take into account the effect of flared entries.

They assumed that:

(1) when vehicles are entering, they move forward in the ranks in which they are waiting,

(2) when they are queuing all available spaces would be filled by the vehicles.

The formula they derived is

$$Q_2 = \frac{Q_1(1 - \beta Q_1)}{e^{Q_1(\alpha-\beta)}(1 - e^{-\gamma Q_1})} \left[N - (e^{-\gamma Q_1})^{C_1} - (e^{-\gamma Q_2})^{C_2} \cdots - (e^{-\gamma Q_n})^{C_n} \right] \qquad \text{(eq. 2.8)}$$

where α: the critical gap (sec),

β: the minimum circulating headway (sec),

γ: the minimum entering headway (sec),

N: the number of lanes at the stop lane,

C_1: the number of carlengths back to the first loss of lane,

C_n: the number of carlengths back to the nth loss of lane,

Q_1: the circulating flow (veh/s),

Q_2: the entry capacity (veh/s).

This formula was found to provide accurate estimates of capacity at 15 roundabouts studied.

Following the earlier work leading to eq. 2.6, Ashworth and Laurence (1975, 1977, 1978) examined a series of models to predict capacity. Based on the analysis of results from 42 roundabout sections in different parts of Great Britain, they proposed the following equation as the most satisfactory:

$$Q_2 = \frac{N\,Q_1}{e^{Q_1/A} - 1} \qquad \text{(eq. 2.9)}$$

where Q_1: the circulating flow (veh/hr),

Q_2: the entry capacity (veh/hr),

N: the number of standard width entry lanes (standard entry width = 3.65m),

$A = 3600/t$,

$t = \alpha = \beta_2$,

α: the critical gap (sec),

β_2: the move-up time (sec).

For the purposes of developing the above model α and

β_2 were assumed to be equal. A value of $A = 1120$ gave the best-fit to the observed data, i.e. $t = 3.21$ sec.

They also found that the linear equation

$$Q_2 = N(868 - 0.2Q_1) \qquad \text{(eq. 2.10)}$$

was satisfactory for the range of data examined but appeared likely to be inaccurate for low circulating flows.

Armitage and McDonald (1977,1978) also extended their previous work by developing an approach using the concepts of lost time and saturation flow. They assumed that each circulating vehicle is associated with a certain length of lost time, L seconds, during which it is not possible for vehicles to enter. At all other times vehicles enter at the saturation flow rate, q_s (veh/sec).

The capacity formula they propose is

$$q_2 = q_s(1 - \beta_1 q_1)\, e^{-q_1(L-\beta_1)} \qquad \text{(eq. 2.11)}$$

where q_1: the circulating flow (veh/s)

β_1: the minimum headway for circulating vehicles that have been held up (sec)

The parameters L and q_s were related to geometric characteristics of the roundabouts. For a further discussion of this aspect see section 4.3 in Chapter 4.

Roundabouts are not widely used in continental Europe. However some work has been done on gap-acceptance models to predict capacities at priority junctions. A model developed in Germany by Harders is described in the OECD (1975) publication "Capacity of at-grade junctions". The formula is

$$q_{max} = \frac{Q\, e^{-\beta}}{e^{\alpha} - 1} \qquad \text{(eq. 2.12)}$$

where q_{max}: the maximum minor flow (veh/hr)

Q: the major priority flow (veh/hr)

$\alpha = \frac{Qt_2}{3600}$

$\beta = \frac{Q}{3600}(t_1 - t_2)$

t_1: the minimum gap acceptable by drivers (sec)

t_2: the minimum time interval required for one vehicle to follow another from the minor stream-termed the "following-gap" (sec).

2.6 Empirical Capacity Models

Most of the work under this heading tries to relate the capacity of roundabouts to geometric characteristics of the junctions. The majority of this work has been developed at the Transport and Road Research Laboratory.

The first attempt to describe the performance of the new layouts was carried out at the TRRL and reported by Blackmore (1970). The formula suggested deals with the whole of the junction and gives a single value of capacity.

$$Q = K(\sum w + \sqrt{a}) \qquad \text{(eq.2.13)}$$

where Q: the capacity (pcu/hr)

K: an efficiency coefficient

$\sum w$: the sum of the basic road widths in metres used by traffic in both directions to and from the junction

a: the area of widening, i.e. the area within the intersection including islands, if any, lying outside the area of the basic crossroads (m^2) (see Fig. 2.9).

Blackmore reported that the highest capacities obtained by different junction types were approximately equal for the same values of parameter a.

In 1969 Grant investigated some roundabouts in Aberdeen with dimensions outside the limits of the Wardrop equation. He treated each approach separately as a priority junction and developed a graphical relationship between the capacity of each entry and the dimensions of the entry. He observed that smaller gaps than usual were accepted at the small roundabouts, resulting in high capacities.

Murgatroyd (1973), while examining the validity of Wardrop's formula, proposed an alternative one. It is similar to Wardrop's with p = 1.00 and with a subtractive constant:

$$Q = \frac{90w(1 + e/w)}{1 + w/\ell} - 1100 \text{ (pcu/hr)} \qquad \text{(eq. 2.14)}$$

The above symbols have the same significance as for Wardrop's equation (eq. 2.1), and again all dimensions are in feet.

In 1974 Maycock proposed a model from which the capacity is determined by the conflict of entering traffic with traffic already using the circulation. He proposed a linear model approximating Tanner's relationship:

$$q = q_m(1 - c/c_m) \qquad \text{(eq. 2.15)}$$

where q: the maximum entry flow (pcu/hr),

c: the corresponding circulating flow,

q_m and c_m: constants specific to the roundabout.

q_m would be equal to the entering flow when there is no circulating flow, while c_m is the circulating flow at which no entering flow would be possible.

The research carried out at TRRL since then has been concentrated on estimating predictive equations for the constants in equation 2.15. These constants were related to geometric parameters of each roundabout. Initially two sets of equations were published, one for conventional the other for offside priority roundabouts. Eventually one unified formula was developed. Here the formulae relating to conventional roundabouts will be described, as well as the unified set, because they are the basis of the design methods of TE Design Note No. 1, (see section 2.7).

The equations for conventional roundabouts were presented by Philbrick (1977). The linear model was presented in the following form

$$Q_E = F - f_c Q_c \qquad \text{(eq. 2.16)}$$

where Q_E: the entry flow (pcu/hr),

Q_c: the circulating flow (pcu/hr),

f_c and F constants for each site.

The relationship of f_c and F to traffic and geometric parameters was examined. It was concluded that no traffic parameter significantly explained the results, while from the geometric ones the following were significant:

e_1: the entry width (m) which was the most significant factor,

r_1: the radius of entry (m),

w: the section width (m).

The two best relationships for the parameters were

$$f_c = 0.0449\ (2e_1 - w) + 0.282 \qquad \text{(eq.2.17)}$$

$$F = 233\ e_1 (1.5 - 1/\sqrt{r_1}) - 255 \qquad \text{(eq.2.18)}$$

Philbrick concluded that the new formulae were much more successful than Wardrop's formula at predicting the within-sections variation of Q_E and Q_c, but that the equations chosen were unlikely to represent the final solution for design purposes.

The unified formulae were presented by Kimber (1980). The general form is:

$$\begin{aligned} Q_e &= k(F - f_c Q_c) && \text{when } f_c Q_c \leqslant F \\ &= 0 && \text{when } f_c Q_c > F \end{aligned} \qquad \text{(eq.2.19)}$$

where $k = 1 - 0.00347(\phi - 30) - 0.978((\frac{1}{r}) - 0.05)$,

$F = 303 \ x_2$,

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

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

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

$S = (e-v)/\ell \ (= 1.6(e-v)/\ell')$

where the geometric parameters used are (with their respective ranges):

e: the entry width, 3.6 - 16.5 (m),

v: the approach road half-width, 1.9 - 12.5 (m)

ℓ: the average effective length over which the flare is developed, 1 - ∞ (m),

ℓ': approximately $\ell' = 1.6\ell$,

S: the sharpness of flare, $S = (e-v)/\ell$, 0 - 2.9,

D: the inscribed circle diameter, 13.5 - 171.6 (m),

ϕ: the angle of entry, 0 - 77 (degrees),

r: the entry radius, 3.4 - ∞ (m)

The primary elements of design are e and ℓ (or ℓ'). A method has been described allowing the equations to be corrected to take account of local operating conditions at overloaded existing sites. Also, the following form of the equation has been proposed:

$$\Delta Q_e = \left\{ \left(\frac{e_2 - v}{1 + 2S_2} \right) - \left(\frac{e_1 - v}{1 + 2S_1} \right) \right\} (303 - 0.042\ t_D\ Q_C) \qquad \text{(eq.2.20)}$$

This equation allows the prediction of the effect on capacity of a change in the geometric parameters from S_1, e_1 and ℓ_1 to S_2, e_2 and ℓ_2. S_1 and S_2 are the initial and final values of the sharpness of flare.

In 1982, Semmens extended the unified formula to cover grade-separated roundabouts. The modified formula suggested was

$$Q_e = 1.11F - f_c\ Q_c \qquad \text{(eq.2.21)}$$

where all parameters have the same significance as for the unified formula.

2.7 Official Design Formulae in Britain

Wardrop's formula was the official design formula for conventional layouts until 1975. The formula, as given in "Layout of Roads in Rural Areas" (Ministry of Transport, 1968), is the following:

$$Q_p = \frac{282w(1 + e/w)(1 - p/3)}{1 + w/\ell} \qquad \text{(eq.2.22)}$$

which is the practical capacity, Q_p = 80% Q_m, where Q_m is the maximum theoretical capacity; e, w and ℓ are in metres. The above value of Q_p was corrected depending on various layout characteristics, eg gradient and angles of entry or exit.

The above formula was not amended until 1975. After the priority rule was introduced various researchers, mentioned in section 2.4, demonstrated that Wardrop's formula was no longer applicable. This led to the publication of an interim design formula for conventional roundabouts until a new comprehensive one was developed.

Technical Memorandum H2/75, (Department of the Environment, 1975), included both this interim formula and one introduced previously for use with the new layouts with small islands and flared entries.

H2/75 defined the following types of roundabouts:

(a) Conventional: a roundabout having an one-way carriageway, which may be composed of weaving sections, around a circular or asymmetrical central island and normally without flared entries.

(b) Small: a roundabout having an one-way circulatory carriageway around a central island 4 metres or more in diameter, and with flared approaches.

(c) Mini: a roundabout having an one-way circulatory carriageway around a flush or slightly raised circular marking less than 4 metres in diameter, with or without flared entries.

(d) Double: an individual junction with two small or mini roundabouts either contiguous or connected by a short link road.

(e) Multiple: an individual junction with three or more small or mini roundabouts either contiguous or interconnected by short link roads.

(f) Ring Junctions: a junction having a two-way circulatory carriageway around a central island linking mini-roundabouts at the mouth of each entry to the junction.

These types are illustrated in Figs. 2.2 to 2.8.

For small, mini and double roundabouts Blackmore's formula (eq. 2.13) was suggested.

$$Q_p = K(\sum w + \sqrt{a}) \text{ veh/hr}$$

where K has a value between 40 and 70, depending on the type of roundabout and the number of approach arms. 85% Q_p is used for design purposes.

For conventional roundabouts, the practical capacity of each "weaving section" was proposed to be estimated by the following formula:

$$Q_p = \frac{160w(1 + e/w)}{1 + w/\ell} \quad \text{(veh/hr)} \qquad \text{(eq. 2.23)}$$

which is the same as eq.2.22 with the $(1 - p/3)$ term removed and the constant being 160 rather than 282. Again, a value of 85% Q_p is suggested.

T.E. Design Note No. 1, (Department of Transport, 1978), considers Philbrick's formula (eq. 2.16, 2.17, 2.18) for conventional roundabouts. Because of the interim nature of that formula, H2/75 was not modified. However, designers were advised to examine the effect of applying Philbrick's formula to the design of conventional roundabouts, particularly for those situations in which its use would overcome difficulties with land-take, earthworks or the environment. It was then proposed that in order to adopt a layout based on the new formula specific approval should be obtained, being a departure from standards.

2.8 The Estimation of Delay

The development of a unified formula for roundabout capacity at TRRL, (see section 2.7), was part of a wider examination of traffic behaviour at road junctions. This included an investigation of the methods available for predicting delays at priority junction. They concluded that the existing methods were not satisfactory, and therefore, they produced a new set of formulae relating to delay. This section describes all these methods briefly.

The methods previous to the ones suggested by TRRL can be divided into two groups. The first is based on steady state queueing theory, the second on deterministic queueing theory. Kimber and Hollis (1978, 1979) and Catling (1977) describe the disadvantages of both groups. Models belonging to the first group (e.g. Tanner, 1962) are suitable for situations where the demand flow and the capacity of entries are constant over the period of interest. However, at cases of varying flow and when the capacity is exceeded by the demand flow steady state theories predict infinite queues and delays. This is contradicted by the actual behaviour of traffic flows, which when demand is close to capacity, or even exceeds it for short periods, the development of the queue and the increase in delay lags behind the predictions of steady state theory. Models based on deterministic queueing theory (e.g. May and Keller, 1967) assume that queues grow at a rate determined by the excess of demand over capacity and decay when the demand is less than the capacity at a rate equal to the difference. This ignores the statistical nature of traffic arrivals and departures and seriously underestimates the delay unless the capacity is exceeded by a considerable margin. In fact zero delays are predicted until demand reaches capacity, contrary to experience.

Therefore, both sets of models perform worse at the region when capacity and demand are equal or of similar value which, Kimber and Hollis observe, in practical terms is the most important region of operation. They proceeded to develop an alternative model based on time-dependent demand-capacity interaction.

They define μ as the capacity and q the demand flow. They assume that these values vary in time, and that they represent average values at each fraction of the period of interest. Each section of this period represents a possible set of arrivals to the queue and departures from it. The proportion of occurrences of a queue of n vehicles at time t is $p_n(t)$. Both the average queue length and average vehicular delay can be derived as functions of time from $p_n(t)$.

Hollis, Semmens and Denniss (1980) report on a computer program to model capacities, queues and delays at roundabouts which is based on an approximate method of the above principle. This employs a co-ordinate transformation technique to smooth the steady state stochastic relationship for queue length or vehicle delay into the over-capacity deterministic results obtained by integrating the excess of demand over capacity. An example of a graph is given in Fig. 2.10.

The queue lengths and delays are calculated according to the following rules:

Over a short time interval, t, with capacity μ and demand q assumed constant, traffic intensity is defined as $p = q/\mu$. Several cases exist depending on p, the queue at the start of the time interval L_o, and the equilibrium queue length

$\ell = p/(1 - p)$.

If F_n is a queueing function defined for x (a time variable) by:

$$F_n(x) = 0.5 \{((\mu x(1-p) + 1)^2 + 4p\mu x)^{\frac{1}{2}} - (\mu x(1-p) + 1)\} \quad \text{(eq.2.24}$$

then the average queue length, L, after a time, t, is given by the following expressions:

(i) for $p \geq 1$: $L(t) = F_n(t+t_o)$ where $t_o = L_o(L_o+1)/\mu(p(L_o+1) - L_o)$

(ii) for $p < 1$:

(a) $0 \leq L_o < \ell$: $L(t) = F_n(t+t_o)$ where $t_o=L_o(L_o+1)/\mu(p(L_o+1)-L_o)$

(b) $L_o = \ell$: $L(t) =$

(c) $\ell < L_o \leq 2\ell$: $L(t) = 2\ell - F_n(t+t_o)$ where $t_o=(2\ell-L_o)(2\ell-L_o+1)/\mu(p(2\ell-L_o+1)-(2\ell-L_o)$

(d) $L_o > 2\ell$

$$:L(t) = \begin{cases} L_o + (p - L_o/(L_o + 1))\mu t & 0 \leq t \leq t_c \\ 2\ell - F_n(t-t_c) & t > t_c \end{cases}$$

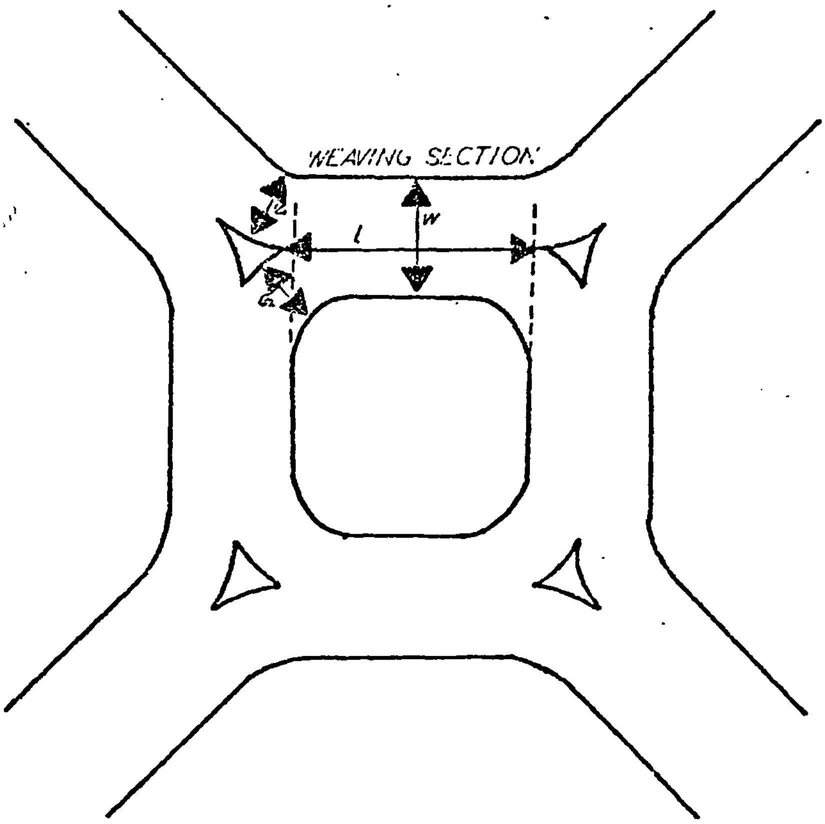
where $t_c = (2\ell-L_o)/\mu(p - L_o/(L_o + 1)$

These equations represent the growth or decay in queue length within the time interval t. The total average delay during this time is obtained by integrating the appropriate queue length equation over the time interval.

Thus, given the demand flow, q, and capacity, μ for a short time interval and the queue length at the beginning of the interval, the equations above allow the queue length at the end of the interval to be calculated. Therefore if any period is divided into a sequence of short time intervals the queues at the end and beginning of each period can be estimated. The program allows variation of both q and μ at every interval.

The program can be used to assess the efficiency both of existing roundabout layouts and of new designs. They announce that there are plans to enhance the model to include geometric delays and to allow optimisation of geometric dimensions.

$$q = \frac{108\,w\left(1 + \frac{e}{w}\right)\left(1 - \frac{p}{3}\right)}{\left(1 + \frac{w}{L}\right)}$$

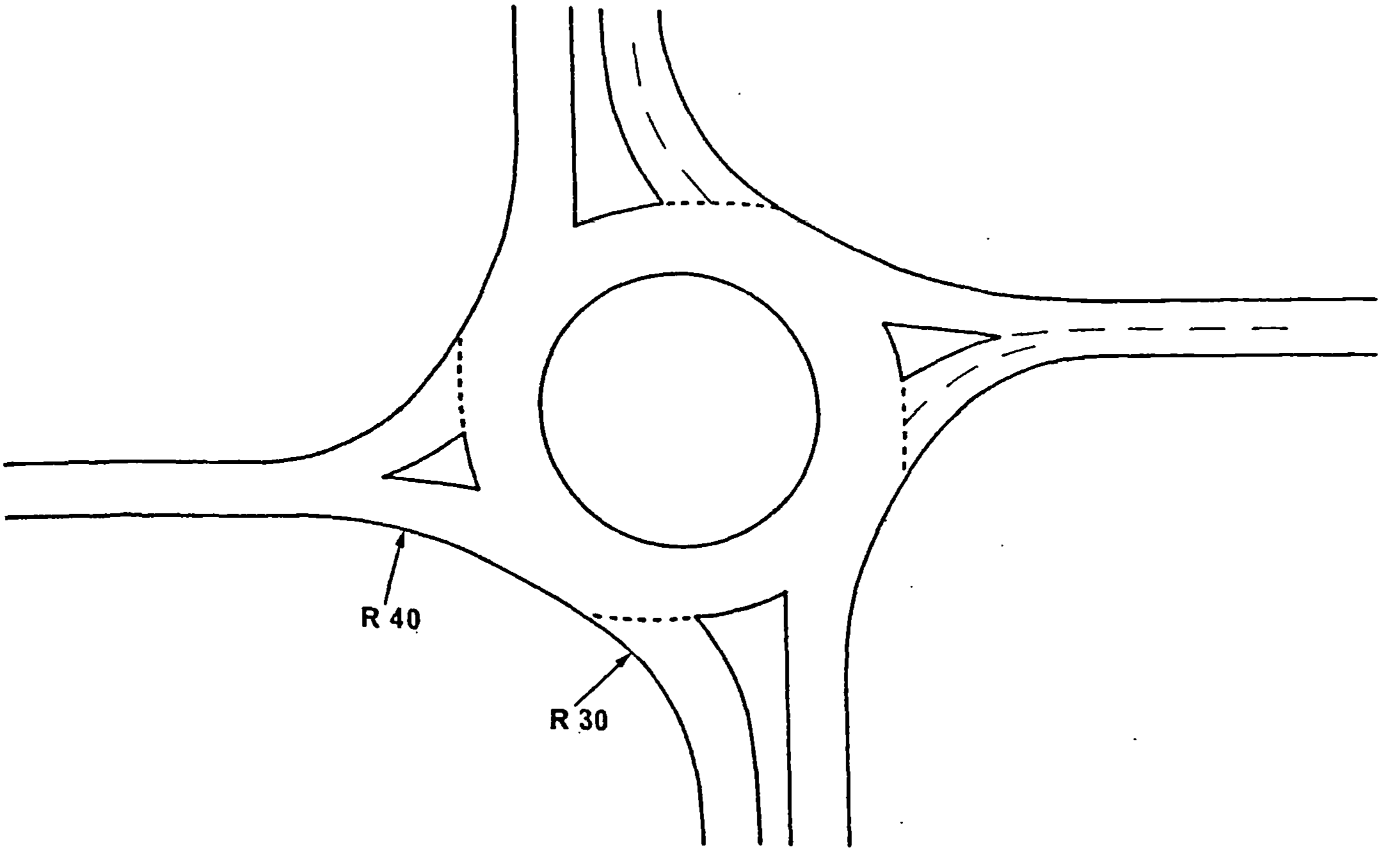


w Weaving Width
l Weaving Length
e Average Entry Width (= 0.5[e_1+e_2])
q Total Capacity of Weaving Section (p.cu/hr.)
p Proportion of Weaving Traffic

Figure 2.1 Wardrop's Formula

TYPICAL CONVENTIONAL ROUNDABOUT LAYOUTS.

A. At Grade Junction.



B. Grade Separated Junction.

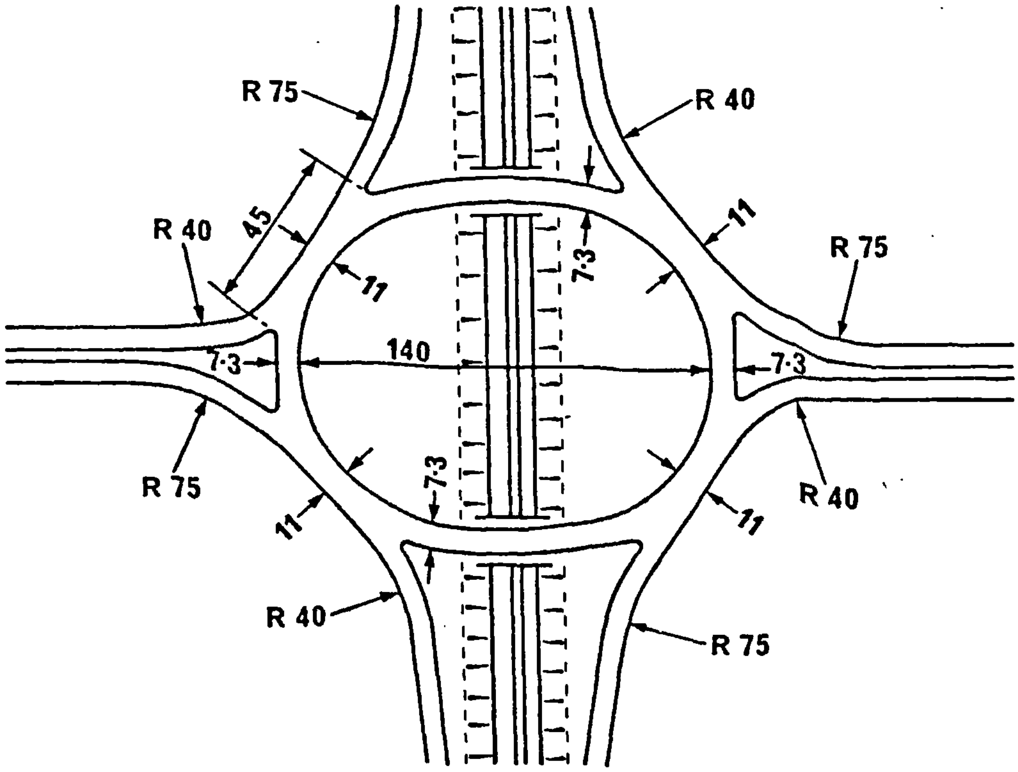
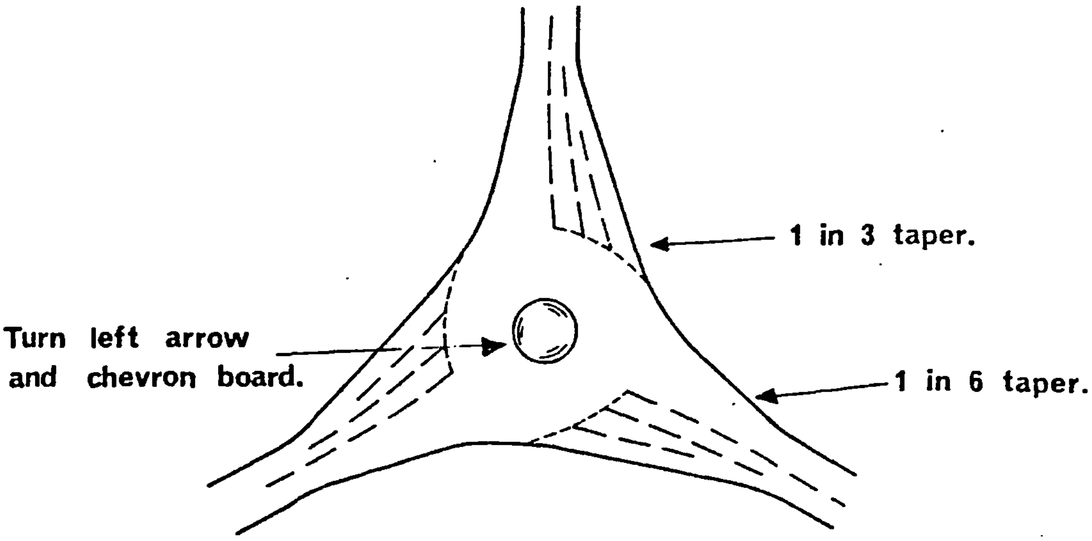


Figure 2.2

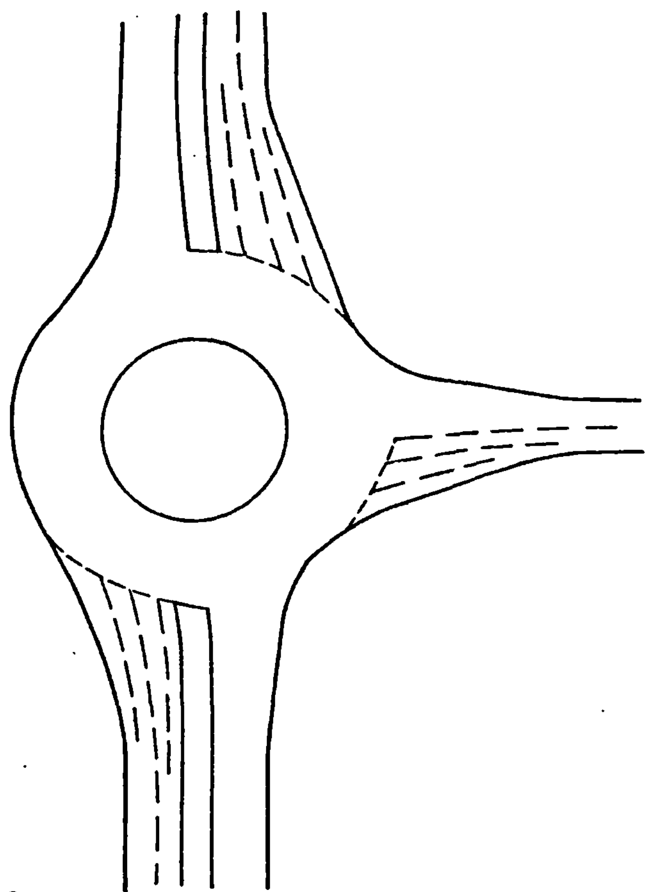
Dimensions in Metres.

EXAMPLES OF SMALL ROUNDABOUT LAYOUTS AT NEW 3 WAY JUNCTIONS.

A. For total design flow 3200 veh/hr.
Approach roads all 7·3m wide.



B. For total design flow 5000 veh/hr.
Approach roads dual 7·3m and single 7·3m wide.

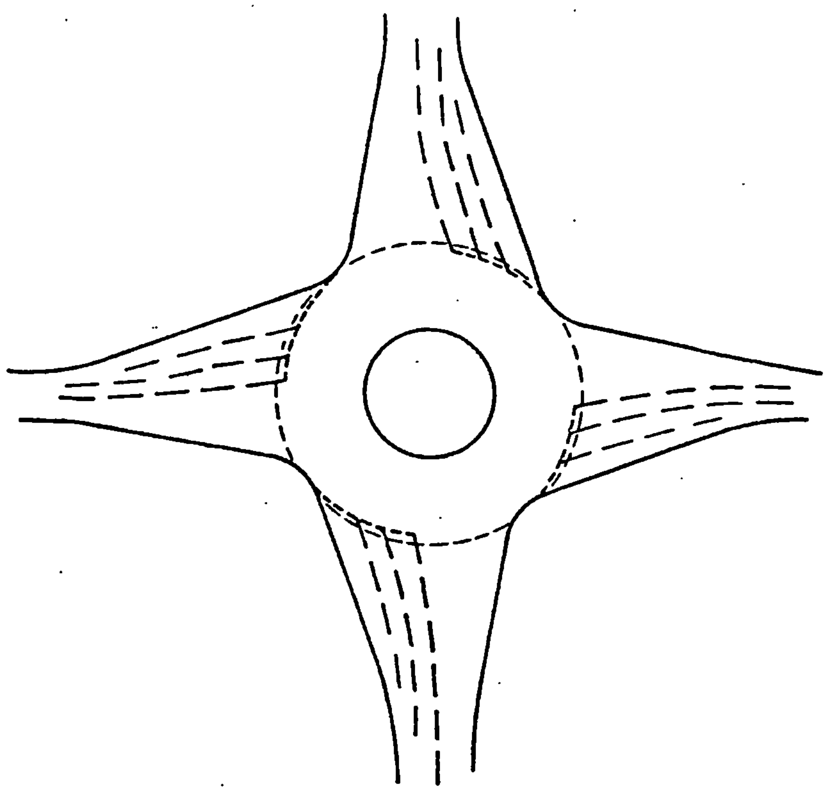


Scale 1:1000

Figure 2.3

EXAMPLES OF SMALL ROUNDABOUT LAYOUTS AT NEW 4 WAY JUNCTIONS

A. For total design flow 2750 veh/hr with 20% HGV
Approach roads single 10m and 7·3m wide



B. For total design flow 3500 veh hr
Approach roads dual 7·3m and single 7·3m wide

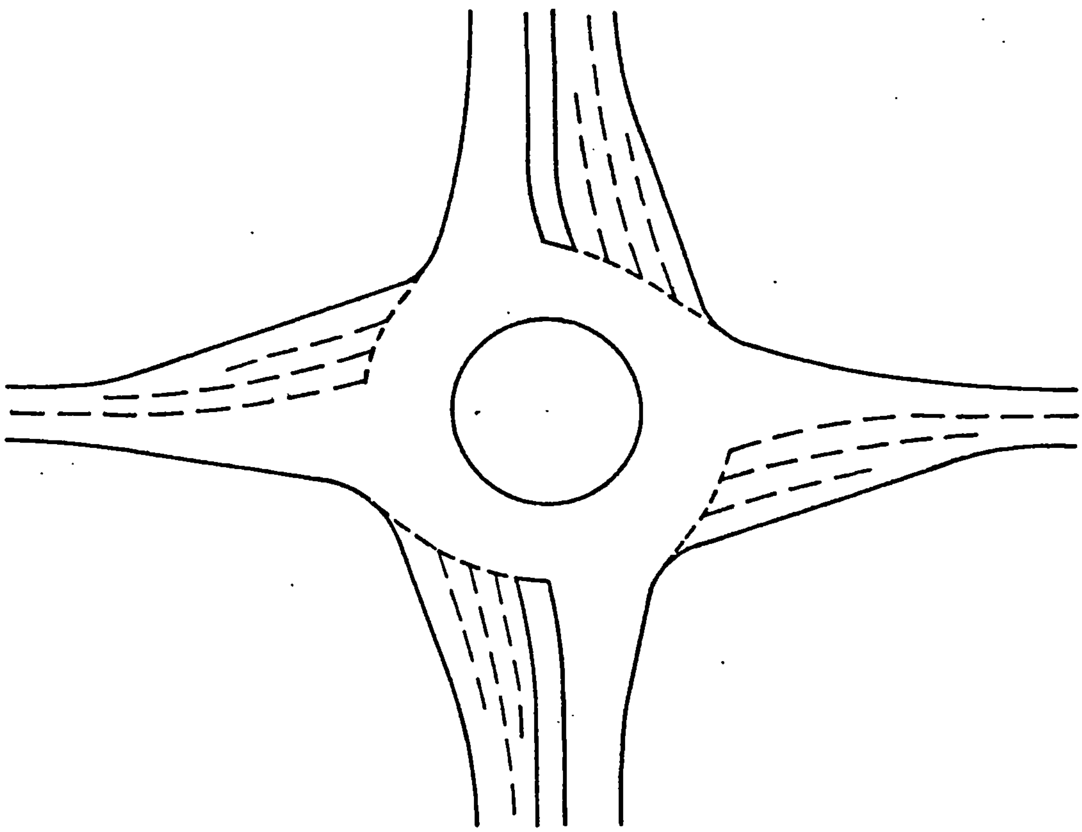


Figure 2.4

Scale 1:1000

EXAMPLE OF GRADE SEPARATED JUNCTION LAYOUT INCORPORATING SMALL ROUNDABOUTS.

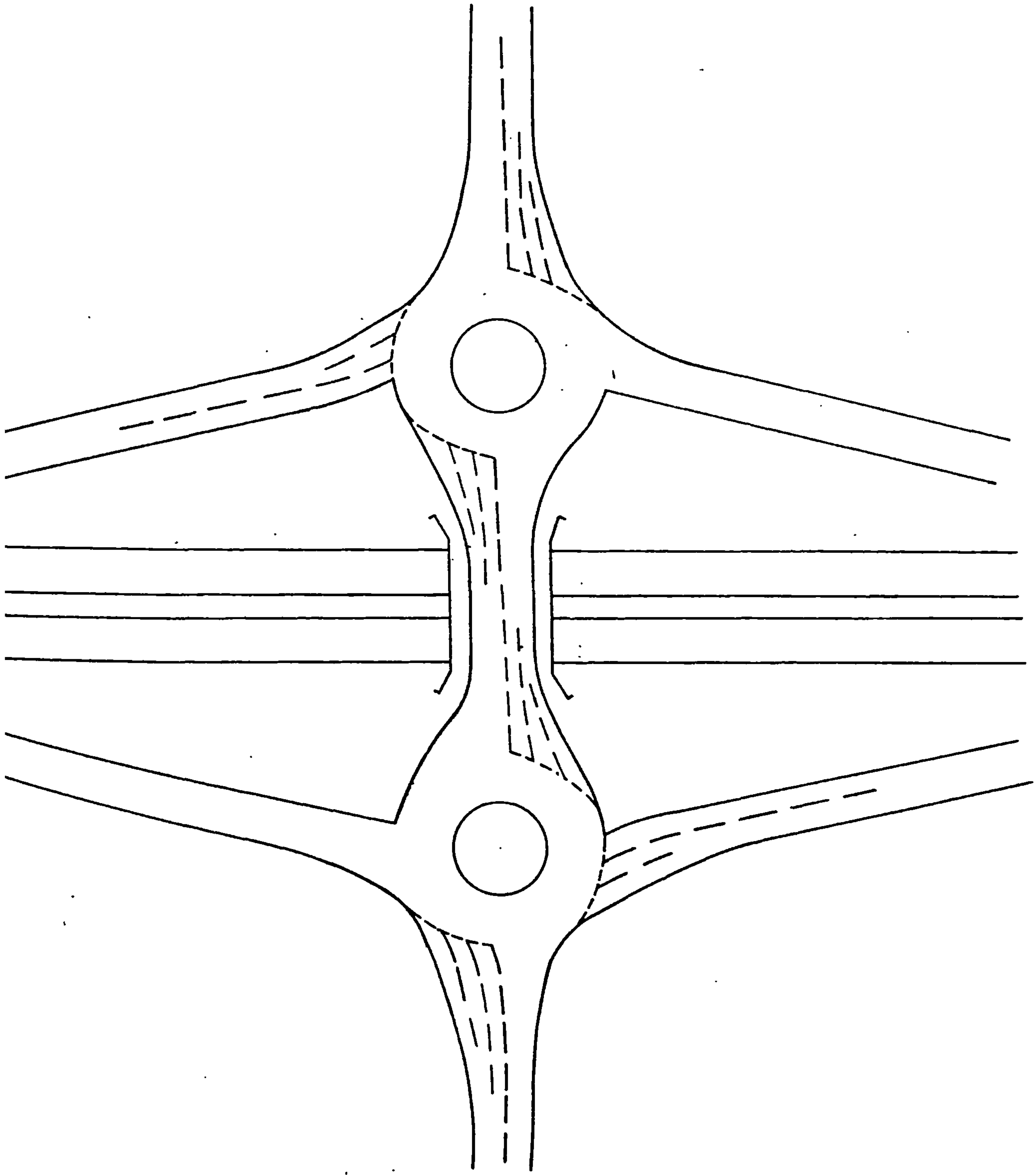


Figure 2.5

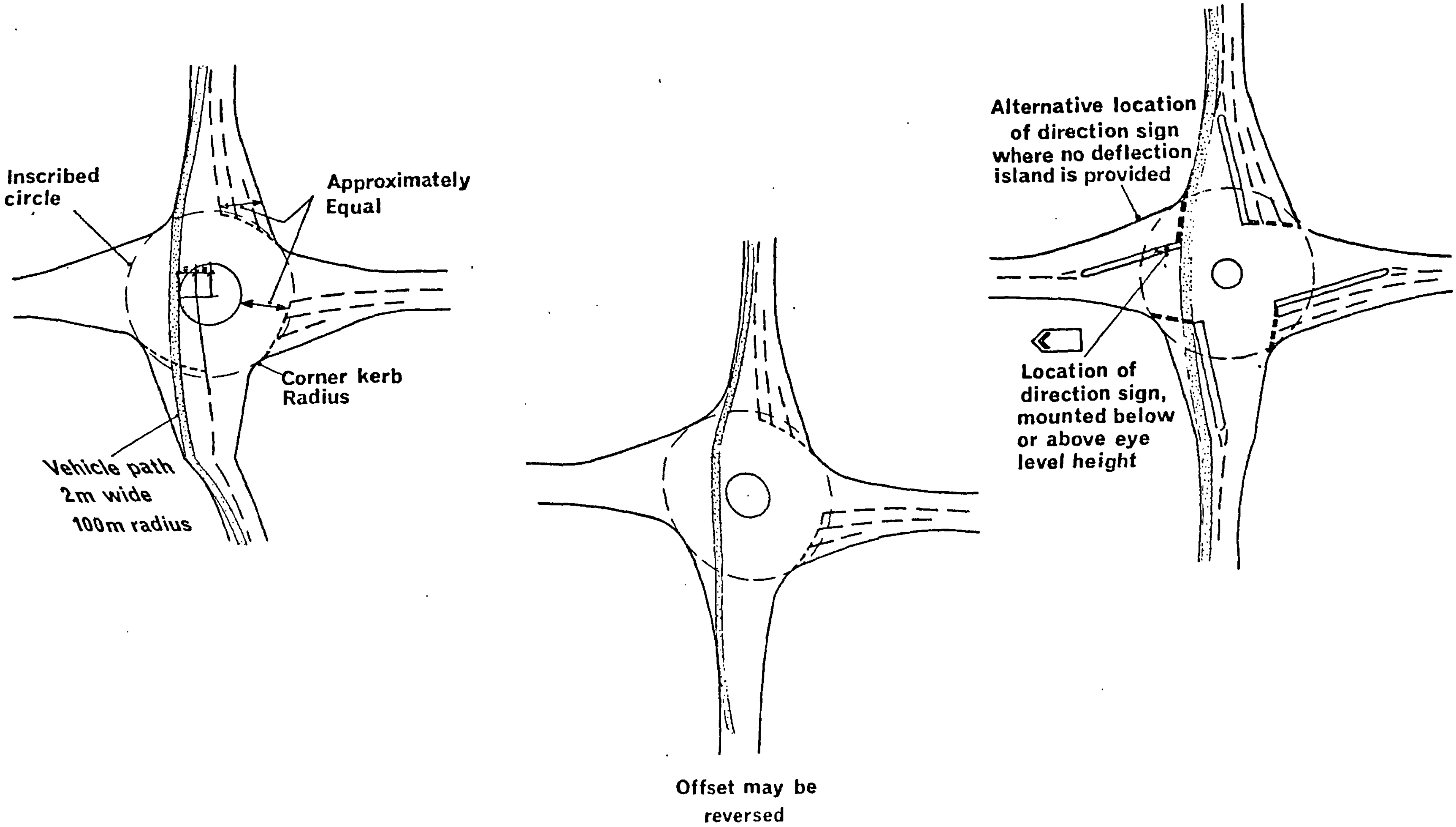
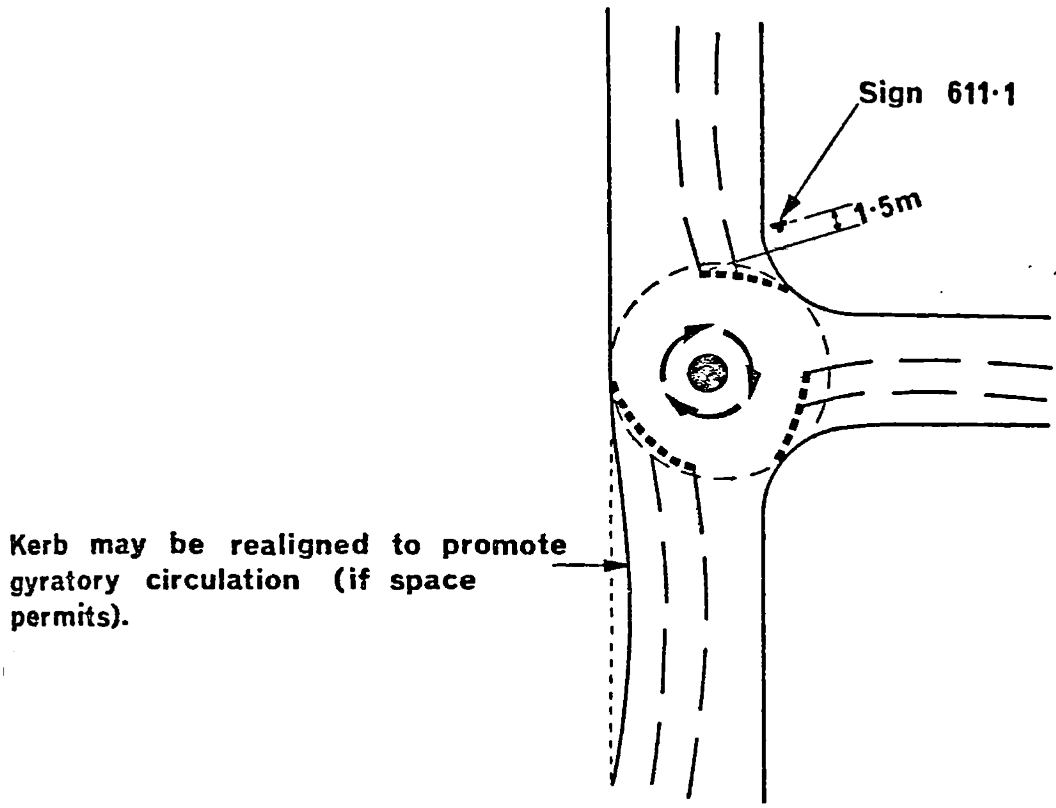


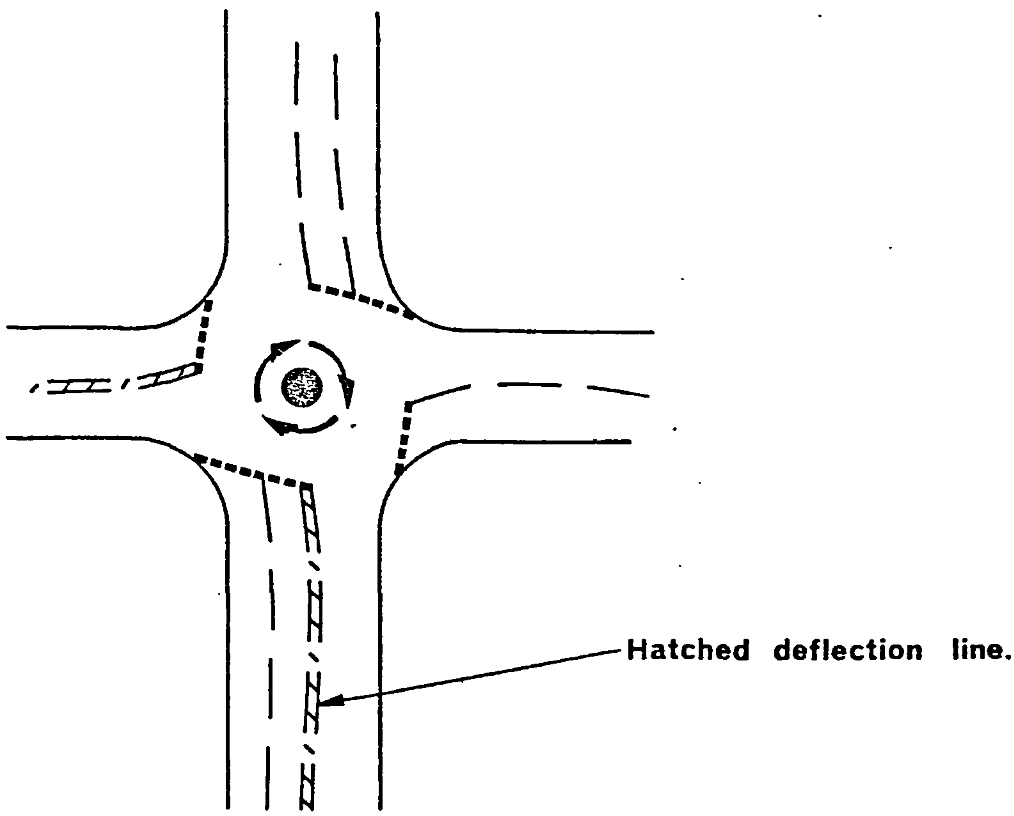
Figure 2.6 Method of Deflecting Entering Traffic

EXAMPLES OF MINI-ROUNDABOUT LAYOUTS AT EXISTING JUNCTIONS.

A. 3 way 'T' Junction.



B. 4 way Junction.



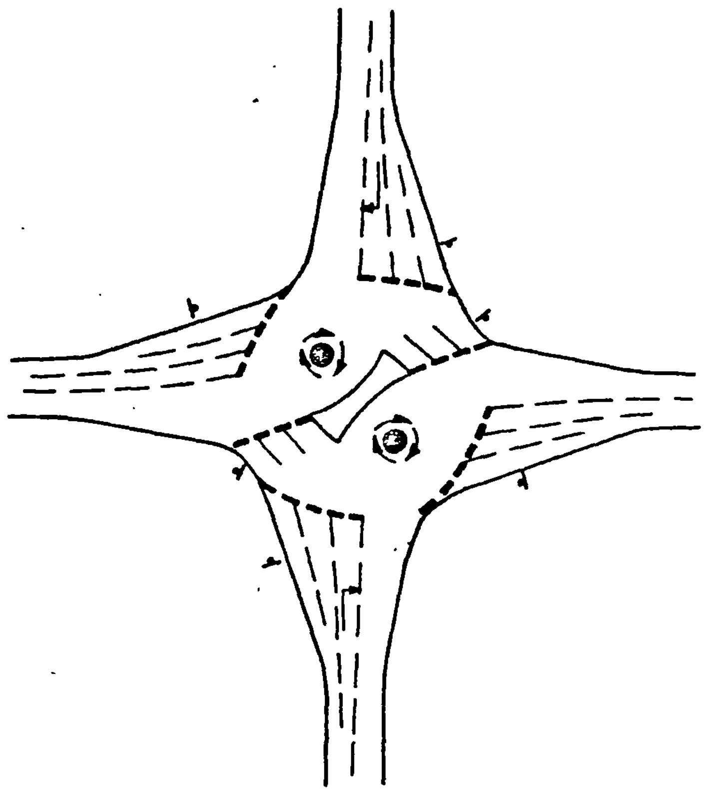
Scale 1:500

Figure 2.7

EXAMPLES OF DOUBLE ROUNDABOUT LAYOUTS AT EXISTING JUNCTIONS.

See paragraphs 71 & 72.

A. 4 way Junction with large right turning flows.



B. 4 way Scissor Junction.

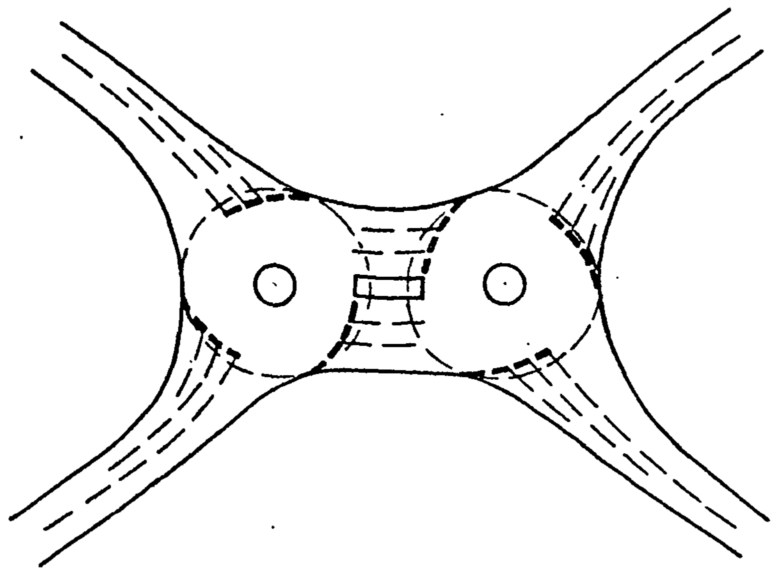


Figure 2.8

Scale 1:1000

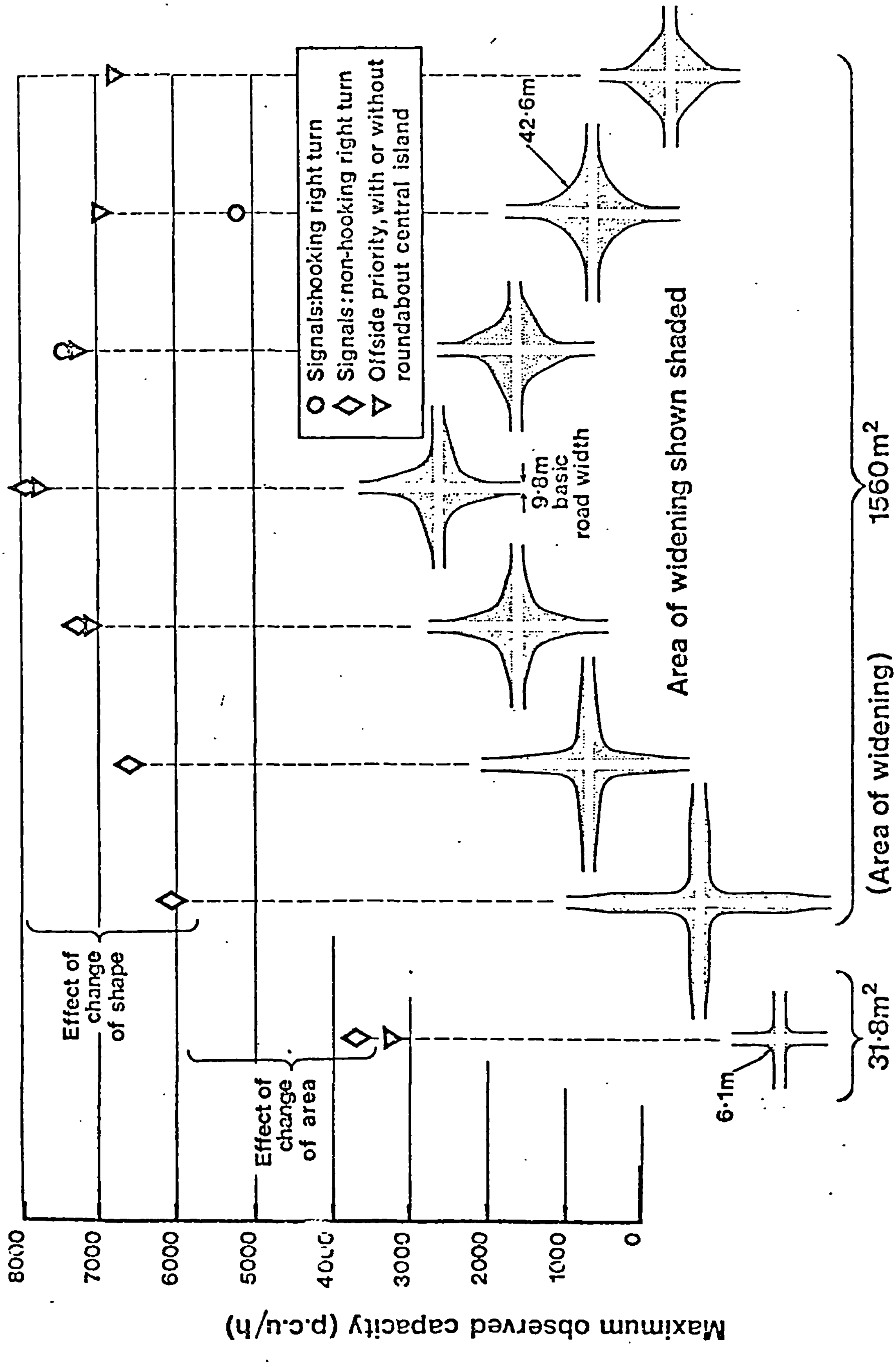
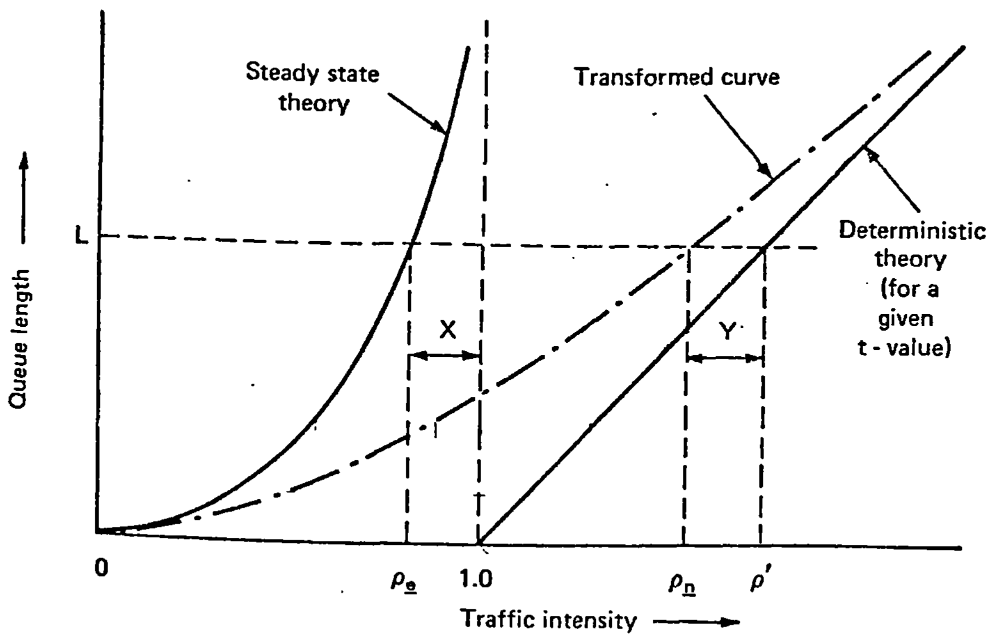


Figure 2.9 Blackmore's TRRL Experiments



PRINCIPLE OF THE METHOD

CO-ORDINATE TRANSFORMATION
Figure 2.10

CHAPTER 3

COLLECTION OF DATA

3.1 Introduction

The development of a simulation model does not require the collection of large number of data. In the present study, data were required on two occasions. The first one was the validation of model SIMC developed previously, (see Chapter 5); the second was to provide an indication of the way entering vehicles position themselves at the available lanes, taking into account their turning movement.

Both sets of data observations were not extensive and were conducted over a brief period of time, (generally 30 minutes).

3.2 The Collection of Gap-Acceptance Data

Data were required to validate the simulation program developed previously, which formed the basis of the present work. Program SIMC uses constant values for the critical gap and the move-up time to produce an estimate of the capacity associated with each circulating flow value. The observations, at this stage, were required to provide values for the circulating and entering flows, and for the gap-acceptance parameters. Therefore, the sites had to fulfil certain criteria: at least one of the entries had to operate at capacity for a considerable length of time; a suitable vantage point had to be available for positioning the video camera and recorder used for data recording; no pedestrian crossings or other forms of traffic control should be affecting the approach to the entry; and the entry should have more than one lane.

Preliminary investigations showed that very few sites, fulfilling all the above criteria, were available in

Sheffield. It was observed that, in general, each lane of the entries behaved in a different way. This was particularly true for the nearside lane at the entry position. It was decided to treat each lane of the entries separately, rather than assume each entry as a uniform entity. The peculiarities of each site are discussed below, in greater detail.

Three sites near the centre of Sheffield were considered acceptable:

(1) Moore Street Roundabout,

(2) Castle Square Roundabout, and

(3) Park Square Roundabout.

Moore Street Roundabout (Figure 3.1) is at the junction of the Inner Ring Road, Moore Street and Ecclesall Road. During the morning peak period heavy delays and long queues occur at Ecclesall Road entry. This entry carries traffic approaching the city centre, while large volume of traffic uses the Inner Ring Road, resulting in the heavy delays and long queues along Ecclesall Road. The entry has four lanes at the stop line; the nearside one is used by a large number of buses, while a bus stop is positioned near the stop line. Thus, the nearside lane is not continuously saturated. Therefore, it was decided not to take into account the data from that lane. Similarly lane 2 was not saturated for long enough periods for the data related to it to be suitable for capacity calculations.

Castle Square Roundabout (Fig. 3.2) is very near the city centre, it forms the junction of Arundel Gate, High Street, Angel Street and Commercial Street. During the morning peak period heavy flows are observed along Arundel Gate and from Commercial Street towards Angel Street. The flow entering from Commercial Street forms the majority of the circulating flow at the Arundel Gate entry. There are long queues and delays at Arundel Gate as a result of this. It is of interest to note that High Street is used mainly by buses as it is a no-through road for all other classes of vehicles. Two immediate results are that (1) only a small proportion of the entering flow at Arundel Gate turns left; and (2) the circulating flow has a high percentage of buses. This arrangement of flows allows vehicles from the nearside lane of Arundel Gate to merge with the circulating flow, rather than accept offered gaps. Data from the other two lanes only were taken into account.

Park Square Roundabout (Fig. 3.3) is a large size roundabout, having six entries and seven exits. It is near the city centre and provides the entry to the main link road with the M1 Motorway. The entry from where data was collected is the Corn Exchange which is the immediately previous entry to the Parkway (the M1 link road). During the evening peak period heavy flows from Sheaf Street and Commercial Street, directed towards the Parkway, cause long queues and delays to traffic entering from Corn Exchange. The entry has four lanes at the stop line, however the flow from the nearside one is not seriously impeded by the circulating flow, allowing entering vehicle to filter into the junction and exit at the Parkway. It should be noted that traffic entering from the Parkway during the morning peak period was subjected to extremely long delays; to alleviate this condition traffic signals have been installed to the junction since the observations collected for this project, changing radically the operation of this roundabout.

At all sites the entry under study was recorded using a SONY AV3420CE portable monochrome video tape recorder, belonging to the Civil and Structural Engineering Department of the University of Sheffield. The duration of each observation was thirty minutes as all of the entries were not saturated for longer periods. The resulting tape was subsequently transferred in the laboratory onto another tape on which a time base was superimposed using a National NV.8030 recorder and an Aston VNG IT video number generator to generate the time base. This tape subsequently was analysed by being played back on a monitor using the slow and stop motion facilities of the National recorder. The time base was accurate to an 1/50th of a second. Therefore the available gaps of the circulating flow as offered to the entering traffic could be easily abstracted. The headway measurements were concerned with the time interval between successive vehicles moving along the circulating carriageway, though not necessarily in the same traffic lane. These headways are referred to also as 'gaps'; however this does not imply that the quantity measured was the inter-vehicle time gap. The abstraction of the headway data although simple was long and tedious, however the most important advantage of using video tapes, over other automated methods of recording, is that a permanent record of the whole operation of the junction becomes available. Thus if any supplementary details are required they can be abstracted using the same videotape.

The quantities abstracted included the size of all the accepted gaps of the circulating flow, the number of vehicles entering each gap from each lane of the entry, the total circulating flow and the composition of the flows. Table 3.1 includes the results of the analysis of the traffic volumes for all three sites. It should be noted that two-wheeled vehicles, classified under motorcycles, were included only for the circulatory flow. It was assumed that they did not have any effect on the entering flow as, usually, they did not follow the lane markings and entered additionally to other vehicles.

3.3 Turning Movements and Lane Occupancy

The simulation program developed during this study assigns each entering vehicle to specific positions at the entry, according to its turning movement (see Chapter 5). It was decided to carry out a limited series of observations to obtain an indication of how vehicles use the entry. The observations were carried out at the Brook Hill Roundabout near the University (Fig. 3.4). The roundabout has five arms; the one studied was Upper Hanover Street which forms part of the Inner Ring Road. During the evening peak period there are heavy flows along Brook Hill towards Western Bank and Bolsover Street, and along Netherthorpe Road towards Upper Hanover Street; (Netherthorpe Road forms part of the Inner Ring Road also). The observations were carried out over four days. Each period lasted 40 minutes which was divided into four 10 minute sections per lane. The results are included in Table 3.2. It was possible also to analyse the lane usage of one of the entries recorded at Moore Street Roundabout. The results of this analysis are included in Table 3.3. The observations at Brook Hill Roundabout were carried out manually using hand tallies. The Moore Street Roundabout figures were abstracted from the video tape used to obtain the gap-acceptance data and circulation flows. This same tape could have provided a similar analysis of turning movements of the entries from

Ecclesall Road. However, this distribution was affected by the presence of buses at the nearside lane due to the bus stop near the stop line. Therefore, the choice of lane would have been affected by other factors apart from the intended exit.

As can be seen from the tables, the nearside lane at the 4-lane Upper Hanover Street entry is used almost exclusively by left-turning vehicles, the next two lanes, again almost exclusively, are used by straight through traffic, while the offside lane is mainly used by right-turning vehicles. The Clarence Street entry of the Moore Street Roundabout has only three lanes. Here, the nearside lane was used heavily by straight through traffic, it must be noted, however, that the left-turning volume is very low. The offside lane flow included a small number of straight through vehicles but comprised mainly right-turning vehicles. The assumptions made about the use of the lanes of the entry in the simulation model are included in Chapter 5.

TABLE 3.1

Site	Position of flow	Total flow (veh/hr)	Passenger cars (veh/hr)	Heavy goods vehicles (veh/hr)	Motor cycles (veh/hr)
Moore Street 28/02/80	Circulatory	2596	2420	104	71
	Lane 1	413	398	16	
	Lane 3	380	360	20	
Castle Square 18/06/80	Circulatory	1059	788	236	34.5
	Lane 1	518	495	23	
	Lane 2	590	580	10	
Park Square 03/09/80	Circulatory	1980	1845	81	54
	Lane 1	453	432	21	
	Lane 2	435	399	36	
	Lane 3	495	459	36	

TABLE 3.1 Observed Flows at 3 Roundabout Sites

It should be noted that the convention adopted for numbering the lanes is the offside one to have number 1, while the nearside to have the highest number

TABLE 3.2

Day	Entry Lane	Total flow (veh/10min)	Flow per exit (veh/10 min)				
			1	2	3	4	5
Tuesday 04/05/82	1	12	0	0	4	8	0
	2	37	0	2	35	0	0
	3	55	1	44	10	0	0
	4	23	22	1	0	0	0
Wednesday 05/05/82	1	10	0	0	1	8	1
	2	42	0	4	38	0	0
	3	60	1	38	21	0	0
	4	22	20	2	0	0	0
Thursday 06/05/82	1	6	0	0	0	6	0
	2	45	0	2	43	0	0
	3	76	3	57	16	0	0
	4	15	15	0	0	0	0
Friday 07/05/82	1	12	0	0	3	9	0
	2	39	0	3	36	0	0
	3	68	3	51	14	0	0
	4	19	18	1	0	0	0
Total	1	40	0	0	8	31	1
	2	163	0	11	152	0	0
	3	259	8	190	61	0	0
	4	79	75	4	0	0	0

TABLE 3.2 Turning Movements and Lane Usage Observations at Brook Hill Roundabout, Upper Hanover Street entry.
Lane numbers 1 offside, 4 nearside
Exit numbers 1 Western Bank
2 Bolsover Street
3 Netherthorpe Road
4 Brook Hill
5 Upper Hanover Street

TABLE 3.3

Entry Lane	Total flow (veh/30 min)	Flow per exit (veh/30min)			
		1	2	3	4
1	82	0	13	58	1
2	110	0	106	4	0
3	116	20	95	1	0

TABLE 3.3 Turning Movements and Lane Usage Observations at Moore Street Roundabout, Clarence Street entry (28/02/80)
Lane numbers 1 offside, 3 nearside
Exit numbers 1 Moore Street
2 St. Mary's Gate
3 Ecclesall Road
4 Clarence Street

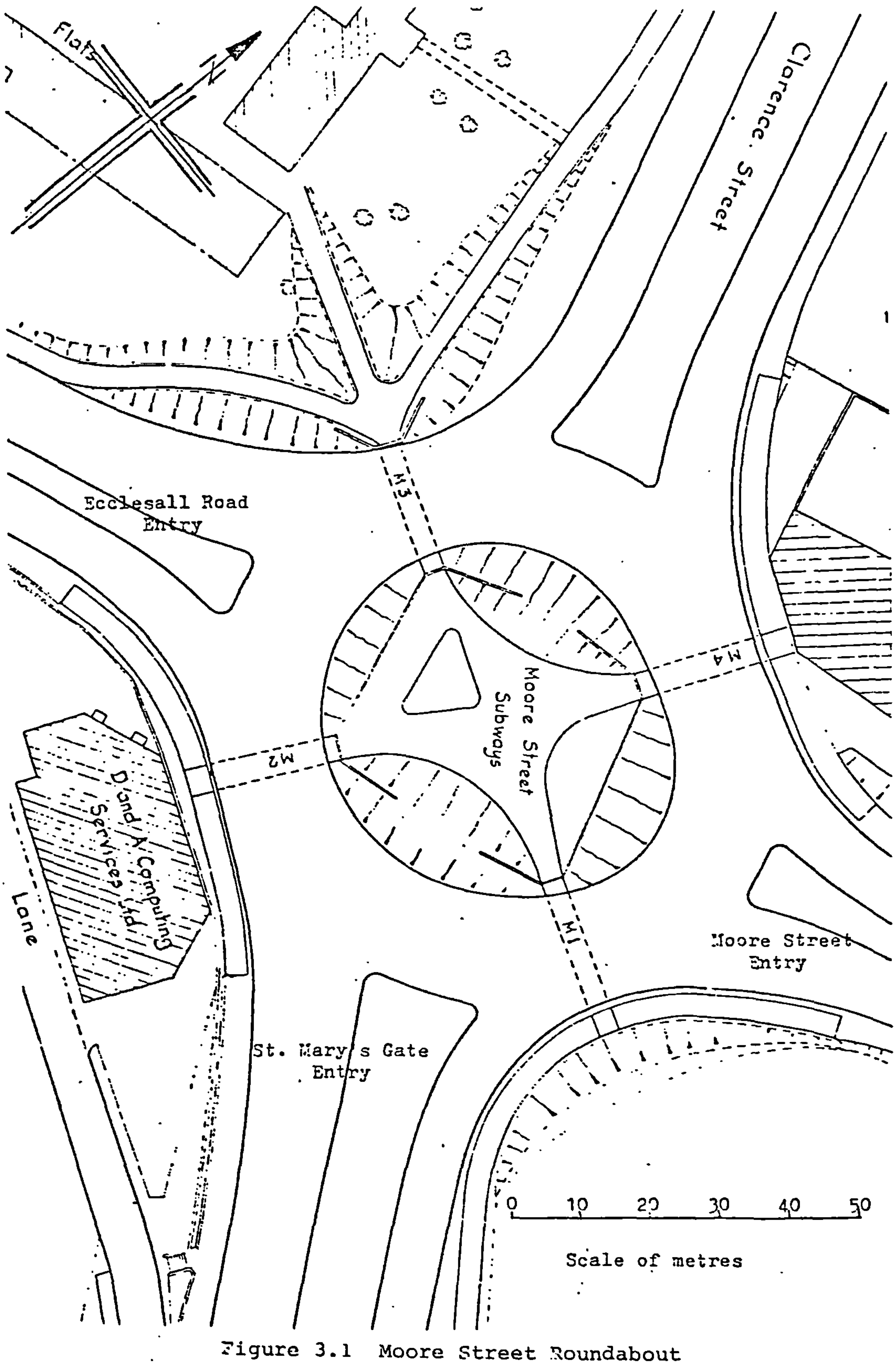


Figure 3.1 Moore Street Roundabout

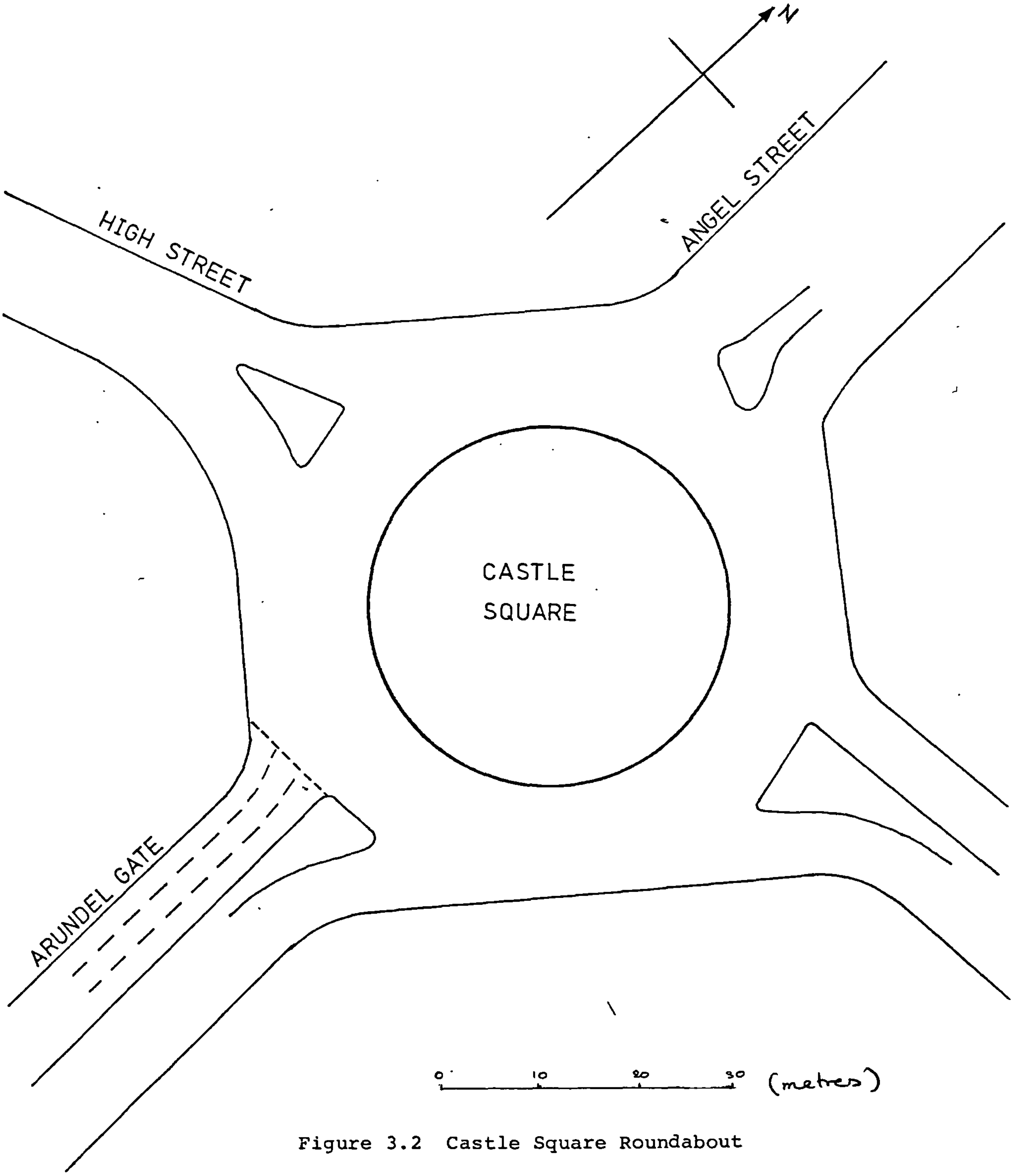


Figure 3.2 Castle Square Roundabout

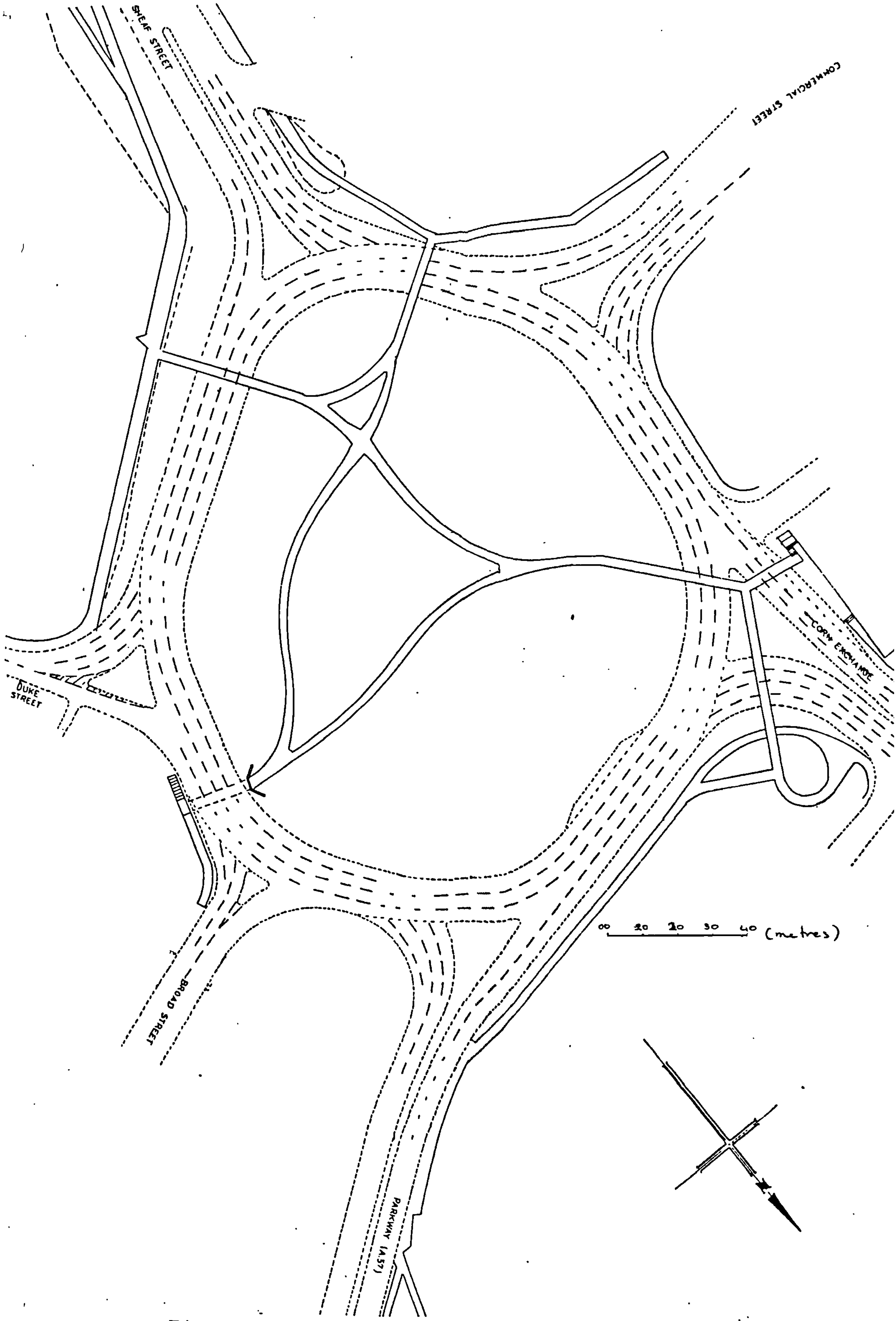


Figure 3.3 Park Sq. Roundabout

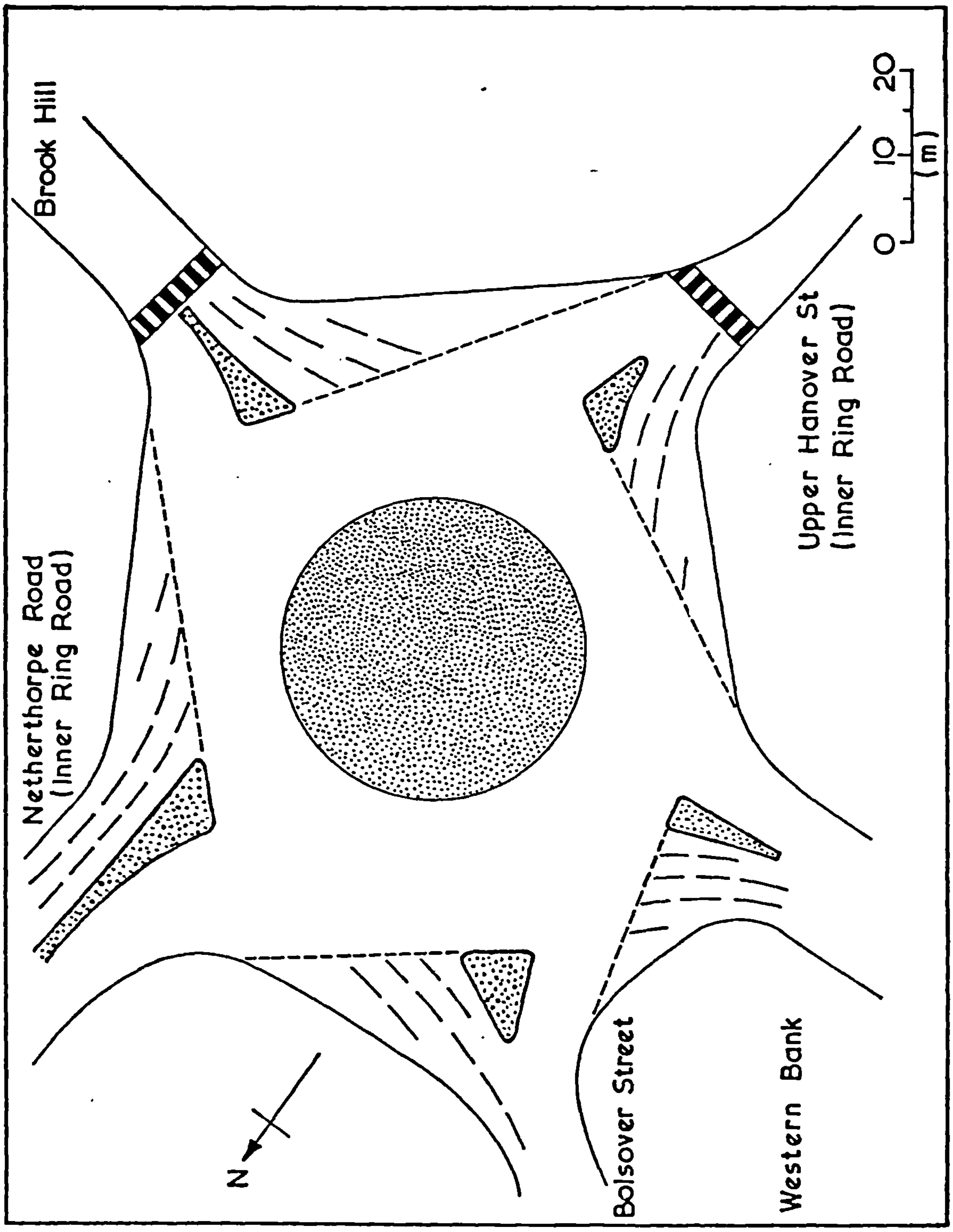


Figure 3.4 Brook Hill Roundabout

CHAPTER 4

GAP ACCEPTANCE CHARACTERISTICS OF THE ENTERING VEHICLES AND THE MINIMUM HEADWAY OF THE CIRCULATING FLOW

4.1 Introduction

The computer program, developed to simulate an entry into a roundabout, is designed to generate individual entry vehicles which progress through the queue of traffic until they reach the stop line, where they reject or accept the gaps in the circulating flow as they are presented to them; (for further description of the model see Chapter 5). This entails the assignment to the entering vehicles of a set of parameters related to their gap-acceptance behaviour. The parameters involved are the critical gap, α, and the move-up time, β. The critical gap is a measure of the minimum length in time, between circulating vehicles, for the first vehicle in the queue to join the circulating flow; the move-up time is a measure of the additional length required for any subsequent queueing vehicles to accept the same gap. Another parameter involved is the minimum headway, τ, of the circulating flow.

The values of these parameters are significant as they describe the performance of the queueing vehicles in the simulation and the size of gaps offered to them. For the model to give realistic predictions these parameters must have values that correspond to observed data.

This Chapter describes various methods to obtain these values from observations proposed by previous research. It suggests some modifications to these methods, and finally describes the analysis of the collected data to obtain the values used to validate the simulation.

Notation: There has been no uniform notation which has been universally adopted by previous researchers in this field. The conventions used here are:

α the critical gap (sec)

β the move-up time (sec)

τ the minimum circulating headway (sec).

These notations will be applied throughout this Chapter. When previous research, which has used different notation, is described the present notation will be used instead. However, it will be made clear that a change has taken place from what was the original notation.

4.2 Gap Acceptance Studies

The study of parameters associated with the acceptance of gaps was initially related to priority junctions and pedestrians crossing roads at non-signalized positions. Gap acceptance became relevant to studies of roundabouts only after the introduction of priority-to-the-right rule in 1966. The operation of a roundabout was likened to that of a series of T-junctions, and Tanner's formula of capacity prediction for priority junctions was applied to roundabouts (Tanner 1962, Tanner 1967). Tanner's formula uses two parameters relevant to the minor stream, the critical gap, α, and the minimum headway, β_2. The latter is defined as the time between successive vehicles accepting the same gap, therefore β_2 is analogous to the move-up time, β, used in the present study. Before the introduction of the priority rule at roundabouts, a lot of research was carried out relating to the estimation of the critical gap parameter for T-junctions. After 1966 this research has become relevant to roundabouts. Simultaneously, other models of theoretical gap-acceptance behaviour

Cooper et al (1977), Wennell and Cooper (1981)). Other distributions used were the shifted negative exponential (Herman and Weiss (1961), McNeil and Morgan (1962), Blumenfeld and Weiss (1979)), the Erlang distribution (Blunden et al (1962)), and the Pearson Type III (Gamma) distribution (Drew (1967)). McNeil and Morgan (1968) have developed a method of building up a distribution from the available data rather than fitting a theoretical model on the data. One problem associated with the inclusion of all offered gaps in the acceptance probability distribution has been the bias introduced by the inclusion of comparatively more rejections by drivers with large critical gaps. This inclusion results in critical gap values larger than the true values. To avoid this bias, Greenshield et al (1947) included in their analysis, only the lags whereas Blunden et al (1962) used an equal number of accepted and rejected gaps by first assuming that all gaps larger than the one accepted by a driver would also be accepted and that all gaps shorter than the ones he rejected would also be rejected, and then factoring the latter to equalise the two totals. Drew (1967) used only the accepted gaps and the largest rejection of each driver. Ashworth (1968, 1970) quantified the bias, assuming a fixed critical gap for each driver, and proposed as the corrected median critical gap, α_c, the following

$$\alpha_c = m - s^2 q \qquad \text{(eq.4.1)}$$

where m: the median value of the observed gap acceptance distribution (sec)

s^2: the variance of the observed gap acceptance distribution (sec^2)

have been developed that use more complex descriptions than Tanner's formulae. These models usually assume that the critical gap follows some distribution between drivers rather than assume a single value for α. The analysis of the data provides a measure of the mean or median and of the variance of this distribution. In the following sections some of these methods of analysis will be presented. It should be noted that all the methods included in sections 4.2.1 - 4.2.4 provide a measure of the critical gap only. The present study was interested in methods estimating both the critical gap and move-up time parameters. These methods are described in sections 4.3 and 4.4 in more detail.

4.2.1 The Critical Gap as the Median of a Distribution

Most methods suggested are variations of the one introducted by Greenshields et al (1947). Here only the lags offered were considered and the percentage acceptance of each size group was determined. A lag is defined as the time interval between the arrival of the side road vehicle at the stop line and the passage of the next major road vehicle. Their method defined the critical lag as the one with 50% probability of being accepted. Since then other researchers have used all available data in the acceptance distribution, i.e. both offered lags and gaps. A number of different theoretical distributions have been fitted to the data to obtain the median value. The most common distributions applied were the normal distribution (Worrall et al (1967), Ashworth (1968, 1969, 1970), Ashworth and Bottom (1977), Powell and Glen (1978)) and the log-normal distribution (Solberg and Oppenlander (1966), Wagner (1966), Ashton (1971), q: the major road (circulating) flow (veh/sec). Figure 4.1 shows a typical example of a cumulative gap-acceptance distribution. Miller (1971) compared the last three methods using simulated data, and determined that the methods proposed by Blunden et al (1962) and Drew (1967) gave very biased results, while the correction given by Ashworth (1968, 1970) did remove the bias and gave satisfactory results.

Ashworth and Bottom (1977) carried out repeated observations on a number of drivers entering into major roads from a T-junction. That enabled them to build acceptance probability distributions for each driver. To obtain each driver's mean critical gap they fitted cumulative normal distributions on each driver's data.

Blumenfeld and Weiss (1978, 1979), analysing the same data, used a shifted negative exponential distribution to describe each driver's behaviour. The mean value and the variance of the distribution can be expressed in terms of the two parameters which define each driver's distribution.

4.2.2 Raff's Critical Lag

One of the first definitions of a gap-acceptance parameter was by Raff and Hart (1950). They only considered lags presented to the minor road flow. They defined as critical lag, L, the size lag for which the number of accepted lags shorter than L is the same as the number of rejected lags longer than L. The value of L was determined graphically as shown in Fig. 4.2. They noted that if lags and gaps are considered together, the percentage of intervals accepted for a particular size is not a true measure of the proportion of drivers who accept such gaps, since several rejected intervals, but only one acceptable, may be counted for each driver.

Similar definitions were used by other researchers, specifically Drew (1967), Armitage and McDonald (1974), and Bendtsen (1972).

Ashworth (1970) compared Raff's critical lag to the mean α of the critical gap distribution. When this distribution has variance s^2, and the circulating (major road) flow is q veh/sec, the relationship is $L = \alpha - s^2q/2$. Thus it is incorrect to equate the two parameters apart from the case of a constant critical gap associated with a step function.

Miller (1971) arrived at the same relationship. He compared this method with other estimators of critical gaps to conclude that it is biased.

4.2.3 Other Methods to Determine the Critical Gap

When the distribution of the acceptance probability is known Maximum Likelihood Estimates (MLE) equations can be derived to give the maximum likelihood values of the gap acceptance parameters. Moran (1966) and Miller (1971) derived MLE equations assuming normal distributions. Miller compared his method to eight other estimators to conclude that the maximum likelihood method and Ashworth's method both gave satisfactory results, the MLE method being slightly more precise but, also, more laborious. Since then Maher and Dowse (1982) have used MLE methods (see section 4.4.2).

Ramsey and Routledge (1973) evaluated the critical gap using a histogram of all offered gaps and a histogram of the accepted gaps. They assume that all drivers are consistent and their method estimates the proportion of drivers in each gap range having critical gap less than or equal to the middle value of the range. Troutbeck (1975) compared this method with the ones included in Miller's (1971) comparison and determined that it is not better than the MLE or Ashworth's method. This method has the disadvantage that in certain conditions it can result in negative values. Troutbeck showed that the mean critical gap in the Ramsey-Routledge method is equal to the mean accepted gap minus the reciprocal of the flow (or the average offered gap). Figure 4.3 shows the histograms used in this method.

The critical gap has been related to the speed of the approaching vehicles (Cooper et al (1976), Cooper et al (1977)). In these studies the accepted and rejected gaps were classified according to the speeds of the approaching vehicles and a log-normal gap acceptance function was fitted to the data in each 5 mile/hour speed-band. Median accepted gaps for each speed, V, were expressed in terms of both time, T, and distance, D (=VT). The median accepted gap is expressed in terms of a constant time and a constant distance. They note that in their method it was not possible to remove the flow bias and derive 'absolute' gap acceptance functions.

4.2.4 Gap Acceptance Theoretical Models

The value of the critical gap has been associated with a number of theoretical models of the acceptance behaviour of minor road vehicles. Plank (1982) has grouped all these models into four categories, as follows:

Model (1) The gap-acceptance distribution is a step function. All drivers have the same critical gap, and consistently accept all gaps greater than or equal to the critical gap, and reject all gaps less than the critical gap. This is the model used by Tanner (1962).

Model (2) Individual drivers follow a step function gap-acceptance distribution, but the critical value is a variable distributed over the population of drivers, i.e. the drivers are consistent but not homogeneous. This is the model used by Ashworth (1968, 1969, 1970) and by Miller (1971).

Model (3) The minimum acceptable headway is described by a probability distribution but is the same for all drivers, i.e. they are homogeneous but not consistent. This is the model used by Herman and Weiss (1961) and by Blumenfeld and Weiss (1978, 1979).

Model (4) Each driver has a gap acceptance distribution given by F(t;w) where parameter F(t) is the probability of accepting a gap of size t, while parameter w has a distribution over the driver population, i.e. the drivers are neither homogeneous, nor consistent.

Model (4) is the most sophisticated and will most accurately describe the true situation. However, Plank suggests that any of the other models will still yield reasonable results with less practical and mathematical difficulty.

Ashworth and Bottom (1977) showed that Model (3) is a more appropriate simplification than Model (2), since the major source of variability in gap acceptance is within drivers rather than between them.

Blumenfeld and Weiss (1978, 1979) support this conclusion. They also compare the statistics for Models (2) and (4) as well as for Models (1) and (3). They conclude that the simplified models (1) and (2), compared with models (3) and (4) respectively, lead to an accurate estimate of the true average delay, a slightly overestimated probability of no delay (highway transparency) and capacity of the minor road, and seriously overestimated variance of delay.

4.2.5 The Move-Up Time, β

Tanner (1962, 1967) uses, in his capacity and delay formulae, the parameter β which is defined as the time between successive vehicles accepting the same gap. According to this theory a gap, T, would be accepted by one vehicle if it is equal or greater than the critical gap, α, i.e. if $T \geq \alpha$, by two vehicles if $T \geq \alpha + \beta$, and by n if $T \geq \alpha + (n-1)\beta$. The estimation of the move-up time has not received the same attention as the critical gap. In general the value of β has been assumed to be constant in the theoretical models of gap-acceptance, although a few researchers have proposed a specific move-up time for each position in the queue of entering vehicles. In most cases the value of β has been abstracted as the mean of the observations of the extra time that vehicles in the queue after the leading one need to accept the same gap (Bendtsen (1972), Uber (1978), Powell and Glen (1978)).Cooper and Wennell (1978) used the median of the distribution. Armitage and McDonald (1974) chose the value of β such that when the critical gap is calculated, by a modified Raff method, the two together have the effect that the total observed entries are equal to the total number of entries predicted from the same gap data. Pearson and Ferreri (1961) and Worrall et al (1967) built cumulative acceptance distributions for the extra time used by subsequent vehicles in a merging platoon. Although they do not report any values as the move-up time or by any other definition, Worrall et al conclude that there is no significant difference among the acceptance distributions for the second, third, and fourth vehicles in line in a multiple merge. See Fig. 4.4 for an example of the acceptance probability curves.

Bendtsen (1972) gives different β values for the second, third, fourth and any subsequent vehicle which are progressively smaller, 4.2, 3.9, 3.8, 3.7 sec respectively.

Uber (1978) gives the following values for the same vehicles 3.54, 3.53, 3.74 and 4.10 sec.

Cooper and Wennell (1978) report values for the second, third and any subsequent vehicles, which were 2.9, 3.2, 2.9 sec respectively.

The last three studies were conducted at priority T-junctions. Powell and Glen (1978) studied gap acceptance at roundabouts. They arrived at one value for all vehicles in a multiple acceptance. However, they suggested different values for the various types of roundabouts they studied. The values they suggest ranged from 2.0 to 3.3 sec.

It is of interest to note that some roundabout capacity models proposed by Wohl and Martin (1967), Ashworth and Field (1973) and Ashworth and Laurence (1975) assume the move-up time to be equal to the critical gap.

The methods discussed in the following sections provide values for both gap-acceptance parameters simultaneously.

4.3 The Work of Armitage and McDonald

Armitage and McDonald conducted a series of studies on roundabout performance during the 1970's. The investigation included the prediction of gap acceptance parameters from roundabout geometry and they proposed a method for obtaining these parameters from observed data using a least squares best-fit curve.

Armitage and McDonald (1977, 1978) assumed that roundabouts operate as a series of linked T-junctions. They were interested in developing a formula that would predict the capacity and not the delay of the entering vehicles. This allowed them to use assumptions that gave simpler formulae. Thus they developed two concepts incorporated in their capacity formula. They were the concepts of lost time and saturation flow. Lost time is assumed to be a period associated with the passage of each vehicle of the circulating flow. During this time no entry vehicle can join the circulating flow, while at all other times they join at a constant rate which is the saturation flow.

The formula they proposed as the most useful is the following:

$$q_2 = q_s(1 - \tau q)\ e^{-q_1(L-\tau\ )} \qquad \text{(eq. 4.2)}$$

where q_2: entering flow (veh/s)

q_1: circulating flow (veh/s)

q_s: saturation flow (veh/s)

L : lost time (s)

τ : minimum headway of circulating flow (s)

For further description of their capacity formula see Chapter 2 section 5. Originally they used the notation β_1 for the minimum headway.

They related the gap-acceptance parameters, q_s, L and τ, to the geometric characteristics of the layout. In order to achieve this they collected data on all of the above parameters at a large number of public road sites and also in a series of test track experiments conducted by the Transport and Road Research Laboratory. Each of the sites was described by the geometric factors shown in Fig. 4.5. They tested all three gap-acceptance parameters against all these characteristics. The formulae they proposed are the following:

$$q_s = 0.12\ EO + 0.04(E1 + EO) \text{ for non-flared entries} \qquad \text{(eq.4.3)}$$

$$q_s = 0.12(EO + F1(E1 + EO)/(F1 + 69)\ ) \text{ for flared entries} \qquad \text{(eq. 4.4.)}$$

$$L = 2.3 + 0.006K1 - 0.04\ W2 \qquad \text{(eq.4.5)}$$

$$\tau_{(i)} = 1/(0.12\ EO_{(j)} + 0.04(E1_{(j)} - EO_{(j)})\ ) \qquad \text{(eq.4.6)}$$

where all the geometric notations are as defined in Fig. 4.5. The subscripts (i) and (j) in eq. 4.6 signify the following:

(i): parameters relating to the study entry

(j): parameters relating to the immediately previous entry.

Five different methods were used to estimate the minimum circulating headway. Briefly, these methods were:

(i) the theoretical headway distribution was fitted to the observed headway data by the method of moments;

(ii) the theoretical headway distribtuion was fitted to the observed headway data by minimizing χ^2;

(iii) the minimum circulating headway was related to the mean rejected headway;

(iv) after estimating q_s and L, τ was varied to give a least squares fit to the flow data;

(v) τ was assumed to be the reciprocal of the saturation flow of the arm from which the main circulating flow emerges.

Method (iv) was described as the most consistent, with the disadvantage that for certain flow conditions it did not give satisfactory results; however Armitage and McDonald preferred to use method (v) as can be seen from equation 4.6 where the denominator of the right-hand side is the expression for the saturation flow.

The other two parameters, q_s and L, were estimated together by the method of least squares. Taking the simplest case of a single lane of traffic entering a roundabout, two straight lines were fitted to a plot of the number of entries (y) during each gap against the length, (x), of the gap. The line for $x \le L$ was $y = 0$, while for $x \ge L$, it was $y = q_s*(x-L)$. This is illustrated by Fig. 4.6 for a 2-lane entry where the model is fitted to some sample data and compared with the conventional gap-acceptance step function model which uses parameters α and β. The model uses both accepted and rejected gaps. However it should be noted that all rejected gaps less than L have a zero contribution to the least squares value. Also all accepted gaps less than L have a constant contribution since the line for $x \le L$ cannot change slope being defined as $y = 0$. Therefore those points have no influence on the slope of the line for $x > L$ which determines q_s. As L decreases more rejected gaps are contributing to the sum of the squares of differences, but it is not possible to know in advance which rejected gaps should or should not be abstracted from the data. This results in a considerable number of rejected gaps which although abstracted from the data, are not utilized finally. It should be noted that the rejected gaps will be numerous and proportionally the majority of all the gaps, especially at high circulating flows. Therefore this method is very inefficient in the use of data which have to be manually abstracted.

It should be noted that Fig. 4.6 refers to a 2-lane entry of a roundabout. The slope indicated by q_s on the figure is in fact half the value of the actual slope. This is necessary in order to estimate the saturation flow per lane. Also, it should be noted that no rejected gaps less than L were included on the diagram.

It is of interest to examine the relationship between the parameters q_s and L, used by Armitage and McDonald, and the parameters critical gap, α, and move-up time, β, as used in the present study. As can be seen from Fig. 4.6, the move-up time, β, is the reciprocal of the saturation flow, q_s, and the critical gap, α, is related to L and q_s as is shown in eq. 4.7

$$\alpha = L + \frac{\beta}{2} = L + \frac{1}{2*q_s} \qquad \text{(eq. 4.7)}$$

$$\beta = \frac{1}{q_s} \qquad \text{(eq. 4.8)}$$

These two relationships allow the reinterpretation of the data given in Armitage and McDonald (1977) into the conventional parameters. These are included in Table 4.1.

This is useful in providing a direct comparison with the results obtained by the analysis of the present study for both observed and simulated data.

It should be noted, however, that the values of q_s as supplied in Armitage and McDonald (1977, Appendix 1) might be a source of errors. In that study it is not mentioned whether the values given have been divided by the number of lanes for each site. If they have consistently followed the practice of dividing the slope by the number of lanes, as indicated in Fig. 4.6, then the values provided can be used to calculate β by equation 4.8. Otherwise serious errors can be introduced. Studying the results of Table 4.1 the values of β calculated as above often appear very low, sometimes they are less than 1 second. This suggests that the values of q_s given are for the whole entry and are not per lane. However, among the data provided for each site, the number of lanes is not included, therefore it is difficult to justify any other use of the q_s value.

The gap-acceptance parameters estimated in the above way have been grouped according to area and whether the site was a public road or a test track at TRRL. For each group the average values of α and β were calculated. They are included in Table 4.2. The mean values over all the sites are the following

$$\alpha = 2.86 \text{ sec}$$

$$\beta = 1.43 \text{ sec.}$$

The α value compares favourably with values proposed by other researchers. However, the β value is lower than any proposed by Bennett (1971), Horman and Turnbull (1974) and Armitage and McDonald (1974). The lowest suggested value by any of the above is 2.00 seconds. This discrepancy must arise because the q_s values have not been divided by the number of lanes of each entry.

Furthermore, there are no data relating to the actual use of the entries, some of the plans included in the 1977 report do not indicate the number of lanes each entry was designed to have, and finally the entry width, E1, as defined (see Fig. 4.5) does not represent a satisfactory alternative to the number of lanes.

From the above, it follows that if β is underestimated so will be the value for α, the critical gap, as the two are related. This can be seen in equation 4.7. Therefore, both averages given above should not be considered accurate, as both underestimate the true values.

4.4 Some Linear Models Suggested by Previous Research

The analysis of data to abstract values for gap-acceptance characteristics is similar for both T-junctions and roundabouts. In both cases the entry/minor road vehicles give way to circulating/major road vehicles while they wait for a suitably long gap to enter or cross the priority flow. Therefore the concepts of "critical gap" and "move-up time" are relevant to both situations. Some previous research into gap acceptance at T-junctions has proposed models for estimating these parameters which are directly relevant to the current project. They are described in more detail in the following section.

Some aspects of T-junction operation are significantly different from roundabout operation. They have to be taken into account when the models for T-junctions are compared with models for roundabouts. The major points of difference are:

(1) The major road flow can be in two directions while the circulating flow is always one directional;

(2) The minor road vehicles can either merge with the stream coming from the right or cross that stream and merge with the stream from the left;

(3) There might be right-turning major road vehicles whose queue can inhibit the right-turning minor road vehicles;

(4) The major road vehicles usually have higher speeds than the circulating ones at roundabouts since they do not have to slow down as they approach the junction;

(5) The design of a T-junction minor road and an entry road to a roundabout differ in such ways as to be easier for vehicles to enter from a roundabout entry than from a minor road at a T-junction, for example flaring is almost exclusively used at roundabouts, there is better visibility at roundabouts especially for vehicles not at the give way line etc.

From the above it is clear that methods developed for T-junctions are not directly relevant for roundabout operation. However, the analysis of the acceptance behaviour by minor road vehicles can distinguish left- and right-turning streams. In such cases the relationships for the left-turning minor road stream have similarities with roundabout operation. Even in such cases, however, only the form of the relationship is relevant and not the reported values for the gap acceptance parameters which tend to be larger than the respective ones for roundabouts.

4.4.1 Description of the Models

The four models described here are linear relationships between the number of vehicles entering, N, and the gap-length in seconds, T.

Pearson and Ferreri (1961) examined queue acceptance in terms of the percentage of gaps of a given size accepted by streams of vehicles entering a freeway. From their gap acceptance distributions, they derived a linear relationship between N and T:

$$N = 0.28\,T - 1.07 \qquad \text{(eq. 4.9)}$$

They claim a high correlation coefficient for this relationship but the method of derivation is not clear.

In 1974, Watson proposed a capacity model for roundabouts which related the gap-acceptance parameters to the geometry of the site. He reported that N and T have a linear relationship. The two gap-acceptance parameters used were m and c, where m was the slope of the straight line and c the intercept with the y-axis. In the regression no rejected gaps are included, data from the whole entry are included, and N is assumed to be the independent variable. In his analysis Watson does not relate the gap-acceptance parameters of his method to the critical gap and the move-up time, but the relationships are as follows

$$\beta = \frac{1}{m}$$

$$\alpha = \frac{1}{m}\left(c + \frac{1}{2}\right)$$

However, the values he reports are not strictly comparable to the ones in this study as they refer to the entrance as a whole.

He does suggest, though, that the two parameters are related by

$$m = 0.45c + 0.16$$

which he rounds up to

$$c = 2m$$

According to this

$$\alpha = \frac{1}{m}\,(2m + \frac{1}{2})$$

$$= 2 + \frac{1}{2m}$$

$$= \frac{\beta}{2} + 2$$

Uber (1978) considered the behaviour of queues of turning vehicles moving into large gaps at a T-junction controlled by a STOP sign. The relationship he derived between N and T is based on the median start-up times of the first and subsequent vehicles making a left turn and the median remainder rejected lag:

$$N = 0.29T - 0.74 \qquad \text{(eq. 4.10)}$$

Cooper and Wennell (1978) proposed two models which they call "the direct linear relationship" and "the explanatory model" respectively. Both models are developed to describe a merging and a crossing manoeuvre. Only the merging relationships are mentioned here.

The direct linear relationship is

$$T = 2.8N + 4.9 \qquad \text{(eq. 4.11)}$$

The explanatory model takes the form

$$T = S + N.M + R \qquad \text{(eq. 4.12)}$$

where S: median start-up time (sec)

M: median move-up time (sec)

R: median residual gap (sec)

This relationship becomes

$$T = 3.0N + 3.0 \qquad \text{(eq. 4.13)}$$

for the merging manoeuvre they were studying. They consider the explanatory model more useful as it enables the effect of changes in the individual components of queue acceptance on the overall relationship to be evaluated.

Considering equations 4.9, 4.10, and 4.11 it is of interest to note that Cooper and Wennell interchange the dependent and independent variables. Instead of treating T as the independent variable they assume it is the dependent variable. They regard T as inappropriate to be the independent variable for the data they were using, since they are sampled from continuous distributions of gap sizes for fixed, integer, values of N.

They also comment on the applicability of the term "regression" for such models. They note that the distribution of the lengths of gaps accepted by a given number of vehicles is markedly skew, i.e. there is always a larger number of gaps at the lower values of the range. This is contrary to the normality assumption of any linear regression model. Therefore they do not use the term "regression" for the direct linear relationship. They conclude that the explanatory model is better than the direct linear relationship for the analysis of queue acceptance.

4.4.2 The Comparison of Models by Maher and Dowse

Maher and Dowse (1982) compared six models of predicting gap-acceptance parameters. They included four simple linear models, the Armitage and McDonald method, and a method using Maximum Likelihood Estimates (MLE) which they developed. The four simple linear models were regressions of N on T, and T on N, firstly using all the data, and secondly, excluding the rejected gaps.

As to the applicability of the term regression they comment that the model assumptions, of either (i) independent errors with zero mean and constant variance, or (ii) normally distributed errors, do not hold in these cases. They conclude that any special status which least squares regression might hold as a method is inappropriate, but the validity of any method of estimating α and β depends on the assumptions made about the underlying mechanism of gap-acceptance.

They tested the six methods for unbiassedness and efficiency. A method is unbiassed if the estimator $\tilde{\theta}$ has a mean (or expected) value of θ, i.e. $E(\tilde{\theta}) = \theta$. A method is asymptotically unbiassed if $E(\tilde{\theta}) \rightarrow \theta$ as, the sample size, $n \rightarrow \infty$. The most efficient one is that which has minimum mean squared error, i.e. $E(\tilde{\theta}-\theta)^2$ is minimum. The relative efficiency of two unbiassed estimators is the ratio of their mean squared errors or variances, $Var(\tilde{\theta}_1)/Var(\tilde{\theta}_2)$. The efficiency of an estimator depends on the statistical assumptions made. If one can be confident of the model assumed then the best estimator can be used, if not a robust or insensitive to model assumption estimator should be used. The disadvantage of any MLE method is that specific probabilistic model assumptions need to be made, the form of the estimates being specific to that model. Furthermore the estimates need to be calculated by means of some numerical iterative scheme for maximising the likelihood function.

In their comparison for bias they conclude that three of the six methods are not seriously biassed; the MLE method, Armitage and McDonald's, and the linear model assuming T as the dependent, N as the independent variable while excluding all rejected gaps.

Comparing the relative efficiencies, they conclude that the MLE method is the most efficient, while Armitage and McDonald's method was more efficient than the simple linear model.

4.5 <u>The Development of a Simple Linear Model</u>

As has been suggested by previous research of Pearson and Ferreri (1961), Uber (1978) and Cooper and Wennell (1978) there can be a direct linear relationship between the size of the gap and the number of vehicles entering during the gap. Their findings are described in more detail in section 4.4. Here, the development of such a linear model is described.

4.5.1 <u>Theoretical Aspects of Linear Regression</u>

Mood and Graybill (1963) define a simple linear model as following:

"Let y, y_2, ..., y_n be uncorrelated, observable random variables such that $y_i = \alpha + \beta x_i + e_i$, where α and β are unknown parameters, x_i are observable mathematical (non-random) variables, and e_i are uncorrelated, unobservable random variables with mean 0 and variance σ^2, where σ^2 is not a function of α, β, or x_i".

Two points of interest arise concerning the use of such a linear model with the type of data involved in the current study. The first is the definition of dependent and independent variables, the second is the distribution of the variables.

From a purely explanatory point of view it would seem obvious to define as dependent variable the number of vehicles entering while the size of the gap is defined as the independent variable.

However, from the point of view of errors due to observational mistakes, it is very unlikely that any should be present in the counting of the number of entries associated with each gap. On the other hand such errors are much more likely in the estimation of the size of the gaps. Furthermore, the number of entering vehicles is a step function while the distribution of the gap lengths accepted by a given number of vehicles is markedly skew i.e. there are more smaller such gaps than longer ones. Thus the normality assumption of linear regression models is violated. This does not allow the full benefits of the linear regression method to be exploited. However, it does not invalidate the use of a linear model. It points to the possibility of introducing modifications to produce reasonable predictions, accepting the fact that the least squares regression assumptions will not be met. Thus, the term "least squares regression" will not be used, instead the model will be referred to as "simple linear". This is in line with the arguments of Cooper and Wennell (1978). It is termed "simple" to distinguish it from the two-line model proposed by Armitage and McDonald.

The justification of such a model will be dependent on its ability to successfully analyze data and provide values for the gap-acceptance parameters which are as near to their true values as it is possible to determine. In order to arrive at the best model, data with known gap-acceptance parameters have been produced using computer simulation. These data are analyzed using the available linear models. This way the model producing the best results can be chosen.

4.5.2 Simulated Data

The data for the checking were produced using a computer program simulating a continually saturated single-lane entry to a roundabout. The program is given in Appendix 2. It assumes a shifted negative exponential distribution for the circulating flow. It allows changes in the values of the critical gap, α, the move-up time, β, the circulating flow and the minimum headway, τ. Values of the gap-acceptance parameters α and β were constant throughout the simulation for the initial runs, although later work allowed variation in these parameters (see section 4.5.8). The period of the simulation can be extended indefinitely; however, the pseudo-random generating subroutine has a cycle of 16384; therefore the pattern of the circulating flow repeats itself after 16384 gaps. The actual simulated time thus depends on the circulating flow.

The accepted gaps were divided in groups of limited number (see section 4.5.7 on sample size effects). Each group was then analyzed and the gap-acceptance parameters calculated: Below the following aspects of the analysis are discussed

1. The use of rejected gaps, (section 4.5.3).
2. The effect of extreme values, (section 4.5.4).
3. The use of weights in the model, (section 4.5.5).
4. The definition of dependent and independent variables, (section 4.5.6).
5. The effect of sample size, (section 4.5.7).
6. The use of variable gap characteristics as input to the simulation, (section 4.5.8).

Finally section 4.6 compares the overall performance of the models tested. Throughout the section the notation used is N, for the number of vehicles accepting a gap, and T, the length of the gap.

4.5.3 The Use of Rejected Gaps

As mentioned in section 4.3, Armitage and McDonald included only the rejected gaps greater than L, the lost time. The data on which the analysis is performed have such distributions that the number of gaps will always be disproportionally larger at the value N = 0, i.e. for no acceptances, than at all other values of the dependent variable. This influences the slope and the intercept of the linear model.

The effect of excluding the rejected gaps was tested by analyzing sets of simulated data with and without the rejected gaps. The results of the analysis are shown in the table below. The table shows the results from two sets of input values for the gap-acceptance parameters

input values for simulation	predicted values with rejected gaps	predicted values without rejected gaps
(i)		
β: 2.25	3.33	2.40
α: 4.27	3.51	3.84
(ii)		
β: 2.25	3.03	2.49
α: 3.99	3.36	3.45

It can be seen that for both sets of data the predictions were improved when the rejected gaps were not included in the analysis. As expected, the most marked improvement was for the value of β, which is the reciprocal of the slope of the straight line. The above results also point to a feature that was observed consistently throughout the study of the linear model, i.e. the predictions for β were always in much better agreement with the input values than the predictions for α. The explanation can be that the value of α is obtained by extrapolating outside the range of the used data to find the intercept, while β is related directly to the slope of the linear model.

Finally, the inclusion of rejected gaps would, obviously, use a larger part of the collected data since accepted gaps tend to be in a minority position in relation to the total number of gaps available. However, abstracting the data (e.g. from video tapes) involves considerable labour which is disproportionally increased if the rejected gaps are required. It is interesting to note in this respect, that

Armitage and McDonald's two-line least squares method involves the abstraction of all or most of, the rejected gaps since the value of L, the intercept, is not known in advance, while the majority of these rejected gaps, the ones less than L, will not in fact be considered in the analysis.

4.5.4 Extreme Values

The distribution of the circulating flow used in the simulation was shifted negative exponential (see section 4.8. for a more detailed description). This distribution allows the occasional large gap to be present even though in reality such gaps are often more common than suggested by the theoretical distribution. When the simulated data were divided in groups, the frequency of large gaps per group was very small; often no such gap was present. Furthermore, it was difficult to define a consistent way of determining the lower limit of these extreme values. For example, the simulated gap distribution based on a circulating flow of 0.44 veh/s had only a few gaps allowing 4 vehicles to enter, less allowing 5 vehicles, and none allowing more than 5 vehicles. The 20 groups into which these gaps were divided were analyzed with and without the gaps allowing 4 or 5 vehicles to enter. The results are given in Table 4.3 which also includes the results of analyzing the same groups but reversing the definition of dependent and independent variables (see section 4.6). As can be seen from the table, some groups did not have gaps of length that were large enough to be excluded, and therefore, no gap-acceptance values are shown under the heading "highest values excluded". The exclusion of top values decreased the values of α, and increased the value of β at all groups. The effect on the mean value over all 20 groups is shown on Table 4.4. In the case of assuming the number of entries as the dependent variable, exclusion decreased the accuracy of the prediction of the mean but also reduced the standard deviation. In the case of the gap size as dependent variable, the prediction was improved, giving the best results of the four sets. However, the criterion for excluding extreme values was not satisfactory, as it could not be explicitly defined. The effect of exclusion of large gaps for data collected in the field would be very uncertain as the total number of gaps would be very small compared to the simulated data. It was decided therefore to develop other procedures for ensuring reasonable predictions without resorting to exclusion of the extreme values.

4.5.5 The Use of Weights

In general, weights are introduced into a least squares analysis to counterbalance distributions of data which overrepresent certain parts of the range, since the latter may introduce inaccuracies in the parameters of the analysis. The distribution of the gaps is of a type that more smaller gaps are present than larger. This would occur with either a negative exponential or a shifted negative exponential distribution assumed for the circulating flow. This over-representation of the smaller gaps would be observed not only over the whole range but also each value of the step function describing variable N would exhibit a similar distribution, for example there should be more smaller gaps accepted by two vehicles than larger ones. Two weights were suggested, one for each distribution:

$$W = e^{qt} \qquad \text{(eq. 4.13)}$$

for the negative exponential distribution

$$f(t) = qe^{-qt} \qquad \text{(eq. 4.14)}$$

and

$$W = e^{q\left(\frac{t-\tau}{1-q\tau}\right)} \qquad \text{(eq. 4.15)}$$

for the shifted negative exponential distribution

$$f(t) = \frac{1}{\bar{t}-\tau}\, e^{-(t-\tau)/\bar{t}-\tau)} \qquad \text{(eq. 4.16)}$$

where W: the weight

q: the flow (veh/sec)

t: the size of the gap (sec)

f(t): the probability density function

τ: the minimum headway of the circulating flow (sec)

$\bar{t}$: 1/q (sec/veh)

As the circulating flow in the simulation program was assumed to have a shifted negative exponential distribution the weight applied was eq. 4.15. Tables 4.5 and 4.6 contain the results of weighted analysis of simulation data based on two sets of initial values. They contain results from analysing the data using two definitions of dependent/independent variables. The means and standard deviations of the predictions over all the groups are included in Table 4.7. All four predictions are satisfactory, while the definition of number of entries as dependent variable gave better predictions in set (i), and the definition of gap size as dependent variable resulted in better predictions for set (ii) of the initial values. The predictions of the follow-up time, β, are in much better agreement with the input values than the predictions of the critical gap, α. Similarly the standard deviations associated with the mean of β are less than half of the standard deviations associated with α. This indicates that the confidence associated with the prediction of individual groups is less for the critical gap than the move-up time. The importance of this fact is that observed data collected at a half period during the peak period are likely to be less than 500 accepted gaps which is the number included in each of the groups analysed here (see section 4.5.7).

4.5.6 Dependent and Independent Variables

Previous research on gap-acceptance, which has proposed linear models, has not determined the optimum definition of dependent and independent variables. Pearson and Ferreri, Uber, Armitage and McDonald assume the number of vehicles accepting a gap, N, as the dependent variable while Cooper and Wennell, Maher and Dowse prefer the gap size, T, as the dependent variable (see sections 4.3 and 4.4). Some justifications for using T as the dependent variable are included at section 4.5.1. Therefore it was decided to test both definitions.

Tables 4.8, 4.9. 4.10. 4.11 and 4.12 include the results of using both definitions, both weighted and unweighted analysis. Table 4.7 contains the mean and standard deviations using weighted analysis only. As mentioned in the previous section, this did not allow a conclusive decision. Table 4.13 includes the results for the same sets of initial values but for unweighted analysis. The predictions overall are worse than the ones of weighted analysis. However, standard deviations are much lower. Comparing the results of the two definitions, the results of analysis using the gap size (T) as the dependent variable are much better. The conclusion of these two sections is that best mean values are provided by weighted analysis while the standard deviation associated with the mean values is much smaller for unweighted analysis which assumes T as the dependent variable.

4.5.7 Effect of Sample Size

Another point investigated was the effect of reducing the sample size on the predictions and especially on the standard deviation of the mean over all the groups. The groups up to now consisted of 500 accepted gaps. However during a half-hour observation period the accepted gaps are usually much less. The groups of one set were divided up to form groups of 200 vehicles. Table 4.14 shows the results. Table 4.15 is a collection of the respective results for a sample size of 500 vehicles. Comparison of the results shows that the reduction in sample size affected the prediction only by 0.01 sec while it increased the standard deviation by only a very small amount. Therefore the predictions based on a sample size of 200 are not significantly worse than those based on a sample size of 500.

4.5.8 The Effect of Assuming the Gap Characteristics not Constant

Up to now the simulation assumed that α and β are the same for all drivers. This simplifying assumption was replaced by defining distributions of values for both the parameters. The distribution used was normal in each case. The simulation was carried out only for one set of initial values. Each entry vehicle was assigned its own critical gap and move-up time with the restriction that no value should be outside the following range $m + 2s \geq x \geq m - 2s$ where m is the input mean and s the input standard deviation. The accepted gaps were divided into 20 groups of 500 acceptances each. The analysis was performed as described previosuly. Tables 4.16 and 4.17 show the results of this set of simulation data.

The predictions compare with the input values equally well as the predictions for constant α and β. The standard deviation of the mean, however, is slightly larger than in the previous cases. (Compare Table 4.17 with Tables 4.14 and 4.15.) The increase is very small and does not invalidate the method when α and β vary between drivers.

4.6 Comparison of the Simple Linear Model and Armitage and McDonald's Two-line Model

As has been described in section 4.3, Armitage and McDonald have proposed a model which fits two straight lines to the data. This model is only slightly more complicated to apply once the data have been abstracted but requires the abstraction of rejected gaps. The simple linear model requires the abstraction of the accepted gaps only. If the two models give equally satisfactory predictions, the simple linear model would be easier to apply since it involves less labour in abstracting the data.

The models tested with the Armitage and McDonald method are the following: (1) A weighted linear model assuming the gap size, T, as the dependent variable, and the number of entries, N, as the independent variable. (2) An unweighted linear model, SRTN assuming the same dependent and independent variables as in (1). (3) An unweighted linear model, SRNT, with the dependency inverted. The reason that another weighted model was not used was that, as section 4.5.5 demonstrated, the performance of the two weighted models was similar.

The test was carried out on computer simulation data generated using 5 sets of initial values. Tables 4.18, 4.19, 4.20, 4.21, 4.22 include the detailed results of the analysis of each group of the simulated data using the Armitage and McDonald method. The comparison is summarised in Tables 4.23 and 4.24. Table 4.23 includes the mean values and standard deviations of the predictions over all the simulated data groups of each input values set, for all 4 methods of analysis.

Comparison of the mean value of the predictions demonstrates that the method provided by far the best results is the weighted linear model. This can be seen in Fig. 4.7 for the predictions of the critical gap and in Fig. 4.9 for the move-up time. For the critical gap, the weighted linear model gives the best prediction in all five cases; while for the move-up time, it gives the best prediction in two cases. The main problem of this method is the very high standard deviations associated with the predictions. This is demonstrated on Fig. 4.8 and Fig. 4.10. Therefore, the predictions derived from any one group of data is very likely to vary from the true value. This becomes very important when the method is applied on observed data; in such cases the number of data points available is restricted compared with simulated data. The other three methods had much lower standard deviations, of a similar order.

These three methods are compared in Table 4.24. Since the standard deviations were of a similar order only the means are compared. The Armitage and McDonald method consistently overestimates α, the range of percentage over-estimations is 3.3% - 6%. The linear models consistently underestimate α, SRTN by -4.0% to -7% and SRNT by -8.7% to -15.7%. The predictions of β are much better, the Armitage and McDonald method underestimates β by -1.6% to -5.0%, SRTN has a range of -1.0% to +1.1%, while SRNT overestimates β by 3.2% to 10.4%, SRNT provides the worst predictions in both cases. The other two methods predict values much closer to the input values. According to the criteria set out in the beginning of the section the linear model SRTN was adopted for the analysis of the data collected for this study.

4.7 Application of the Simple Linear Model on Observed Data

The simple linear model described in the previous sections was applied to the data collected from the three roundabouts in Sheffield as described in Chapter 3. As the data abstracted were separated into gap acceptances for each lane, the model was applied separately on each lane providing parameters in each case. The values arrived at are included in Table 4.25. As can be seen, the results show some difference between the predictions for each lane. Consistently the value of the critical gap of the offside lane is higher than for any of the other lanes. This is more pronounced in the cases of the Castle Square and the Park Square Roundabouts. The explanation could be that vehicles using the offside lane tend to be right-turning. Their manoeuvre usually involves circulating near the island, therefore they have to take into account the flow pattern of all streams circulating. The manoeuvre can be described as involving merging and weaving.

Also, often the angle they approach the give-way line is sharper than at the other lanes, especially the flared ones. Their manoeuvre has some similarities to "crossing" at a priority T-junction. Vehicles turning left or going straight ahead have only to merge with the nearside circulating flow stream.

The difference in the predicted values of the move-up time does not demonstrate any consistent pattern. The higher values observed at Castle Square Roundabout may be associated with the poor visibility of vehicles in the queue at Arundel Gate.

4.8 The Minimum Headway of the Circulating Flow

4.8.1 Introduction

The simulation model developed in the previous research (Natsinas, 1979) assumed that the headway distribution of the circulating flow follows a shifted negative exponential distribution. The input to the program included a variable, TAU, that described the minimum allowable headway of the distribution. During that research the value of TAU was assumed to be constant and equal to 1 sec.

When the model was validated by comparing its predictions to observed data, it became important to examine critically the behaviour of TAU in the simulation. One of the first tests was to examine the sensitivity of the capacity predictions to variations of the minimum headway. It soon became obvious that the predictions were very sensitive to the TAU value. For example, one of the simulation runs assuming 2596 veh/hr circulating flow, predicted a capacity μ = 774 veh/hr for τ = 0.50 sec and μ = 129 veh/hr for τ = 1.00 sec

It was considered necessary to examine in more detail the suitable models for the headway distribution, as well as, the suitable value of TAU if a shifted negative exponential distribution was used. This section looks at some theroetical models proposed for the headway distribution, presents the results of the analysis of the observed and simulated data, and concludes by proposing the use of shifted negative exponential distribution with the minimum headway equal to 0.20 sec.

Note on notation: throughout the section the notation followed is the following

τ: the minimum headway of the circulating flow.

4.8.2 Distributions of Traffic Headways

The description of the traffic distribution along a road has attracted considerable attention from traffic engineers and statisticians. Statistical distributions are useful in describing a wide variety of phenomena where there is a high element of randomness. Such distributions can be divided into counting and interval distributions. Counting distributions describe the occurrence of events that can be counted, while interval ones describe the distribution of the time intervals between events. In this section one counting distribution has been included, the Poisson distribution - and the following interval distributions: the negative exponential, the shifted negative exponential, the Pearson Type III and Schuhl's composite headway model.

4.8.2.1 The Poisson Counting Distribution

The use of this distribution in trafffic studies was introduced by Kinzer (1934), Adams (1936) and Greenshields et al (1947). This distribution gives the probability of any number of vehicles to arrive during a period of given length. If this probability is P(x), its mathematical formulation is

$$P(x) = \frac{e^{-m}\, m^{x}}{x!} \qquad \text{(eq. 4.17)}$$

where

m: the mean number of arrivals expected in the given time

e: the base of natural logarithms = 2.71828

Figure 4.11 shows the distribution for m = 5.

The Poisson distribution is appropriate for describing discrete random events. Gerlough and Huber (1975) note that it will provide satisfactory results when the traffic flow is light and it is not affected by any disturbing control systems. However, at high flow or when there is some cyclic disturbance the Poisson distribution does not describe the conditions adequately.

The Poisson distribution has equal mean and variance. Therefore, if the observed data have markedly different mean and variance the Poisson distribution is not suitable.

4.8.2.2 The Negative Exponential Distribution

The Poisson distribution is discrete. However, another traffic characteristic of interest is the interval between the occurrence of events, for example the gap size between successive vehicles along a road.

Adams showed that P(0), i.e. the probability of zero arrivals using the Poisson counting distribution, is also the probability for a headway equal or greater than t, the time interval used in the Poisson distribution. If h is the headway then

$$P(h \geq t) = e^{-qt} \qquad \text{(eq. 4.18)}$$

where

q: the average flow (veh/sec).

The probability of a headway being less than t is

$$P(h < t) = 1 - e^{-qt} \qquad \text{(eq.4.19)}$$

The distributions of equations 4.18 and 4.19 are shown in Fig. 4.12.

Furthermore

$$P(t_1 < h < t_2) = e^{-qt_1} - e^{-qt_2} \qquad \text{(eq. 4.20)}$$

The negative exponential distribution predicts the greatest number of headways in the smallest time interval between t = 0 sec and t = t_1 sec, where t_1 is the time interval considered. This coincides with observations only when traffic flows are light and there are several lanes available to the traffic.

The agreement becomes poor as soon as the traffic increases in intensity when interaction between vehicles increases. At high flow situations vehicles move in platoons with a minimum headway between successive vehicles. This is more pronounced if headways along one lane only are observed. In such conditions, the number of very small headways reduces, the largest number of observed headways being around the value of the minimum gap. Gerlough and Huber demonstrated the disagreement between theoretical and observed headways using the probability density curve, Fig. 4.13.

Because of this disagreement a number of different distributions have been used which predict fewer small headways.

4.8.2.3 <u>The Shifted Negative Exponential Distribution</u>

This distribution introduces a minimum allowable headway, τ. Equation 4.18 becomes

$$P(h \geq t) = e^{-(t-\tau)/(\bar{t}-\tau)} \qquad \text{(eq. 4.19)}$$

where

$\bar{t} = \frac{1}{q}$ the mean headway (sec)

The shifted negative exponential distribution cumulative curve is shown in Fig. 4.14.

4.8.2.4 <u>Schuhl's Composite Headway Model</u>

Schuhl (1955) proposed a model which assumes some vehicles in a flow to be in bunches having a minimum headway, while the rest to flow in a random manner. The probability of a headway h being less than t is

$$P(h < t) = (1 - \theta)[1 - e^{-t/\bar{t}_1}] + \theta[1 - e^{-(t-\tau)/\bar{t}_2 - t)}] \qquad \text{(eq. 4.22)}$$

where

θ: the proportion of restrained vehicles

$\bar{t}_1$: the mean headway of the free-flowing vehicles (sec)

$\bar{t}_2$: the mean headway of the restrained vehicles (sec)

τ: the minimum headway of the restrained vehicles (sec)

Cowan (1975) has proposed that the minimum headway should be considered as a random variable within the bunches of restrained vehicles.

4.8.2.5 <u>The Pearson Type III Distribution</u>

The generalised equation for the Pearson Type III (or Gamma) distribution is

$$f(t) = \frac{t^{k-1}}{\Gamma(k)} (qk)^k e^{-qkt} \qquad \text{(eq. 4.23)}$$

where

k: a constant

q: the traffic flow (veh/sec)

$$\Gamma(k) = \int_{z=0}^{\infty} z^{k-1} e^{-z} dz \text{ : the gamma function}$$

When k is a positive integer $\Gamma(k) = (k-1)!$ and

$$f(t) = \frac{t^{k-1}}{(k-1)!} (qk)^k e^{-qkt} \qquad \text{(eq. 4.24)}$$

which is the Erlang distribution.

The major advantage of the Erlang distribution is that it can describe headway distributions ranging from complete random flow (k=1) to completely regular flow (k=∞). Fig. 4.15 shows four of the Erlang family of curves.

4.8.3 <u>Analysis of Collected Data</u>

The circulating flow data collected at Castle Square

Roundabout were analysed to provide both a counting and a headway distribution. The results are shown in Tables 4.26 and 4.27. The data from Moore Street Roundabout were analysed to provide just a counting distribution, see Table 4.28.

The observed distributions were examined to establish if the simpler theoretical models, i.e. the Poisson counting, the negative exponential and the shifted negative exponential distributions could be used.

Using the observed headway distribution for the Castle Square data the value of k in the Erlang distribution was derived. In that distribution k is given by

$$k = \frac{(\text{mean})^2}{\text{variance}}$$

The mean of the distribution was 3.42 sec, the standard deviation 3.15 sec, and the variance 9.92 sec^2.

$$k = \frac{3.42^2}{9.92} = \frac{11.696}{9.92} = 1.179$$

When this value is rounded to the nearest integer k became 1 and the Erlang distribution was reduced to the negative exponential. It should also be noted that the ratio (mean/standard deviation) equalled 1.086, which is very close to unity.

The observed counting distributions were examined to establish if the data, when grouped into fifteen second intervals, are random. Each data point representing the number of arrivals per 15 sec interval was regressed on the corresponding value for the immediate previous 15 sec interval. For the data to be random, with 95% confidence, the correlation coefficient of the regression should lie within the range $\pm\frac{1.96}{\sqrt{n}}$, where n is the number of pairs of data points. The results of the regressions are given in Table 4.29. As can be seen the hypothesis for either roundabout cannot be rejected.

The above analysis suggests that the use of the Poisson counting distribution and either the simple or the shifted negative exponential interval distribution is suitable for the traffic conditions observed.

The data were therefore compared with theoretical distributions using the χ^2-test. The two observed counting distributions were first compared to the theoretical Poisson counting distribution. In neither case was the observed distribution rejected at the 5% level, (see Tables 4.30 and 4.31). However, it was found that the headway distribution at Castle Square Roundabout could not be accepted as similar to either a theoretical negative exponential distribution or a shifted negative exponential distribution with $\tau = 0.27$ sec (see Table 4.32). (The figure of 0.27 sec was arrived at as the difference of the mean and the standard deviation of the observed distribution and gives a theoretical distribution having the same mean and variance as the observed distribution of headways).

Since the analysis so far had proved inconclusive in suggesting a suitable value for τ, it was decided to use the simulation program to produce headways based on a shifted negative exponential distribution. The value of the shift, τ, was input to the simulation together with the other flow characteristics. The value of τ was varied from 0.0 to 1.0 sec and the circulating flow used was the value obtained at the study sites. The headways produced by the simulation were then analysed to provide counting distributions for each of the separate values of τ. These distributions were then used as theoretical distributions to compare them with the observed ones. This method would allow the determination of whether the best model to describe the real flows was the simple negative exponential or a shifted negative exponential; furthermore, it should be possible to arrive at a suitable value for the shift, τ.

For Castle Square, three values of τ were used in the simulation, 0.00, 0.50 and 1.00 sec. When the observed counting distribution was compared, the hypothesis that it was the same as the simulated could not be rejected for the cases of τ = 0.00 and τ = 0.50 sec, while it was rejected for τ = 1.00 sec. For Moore Street, the hypothesis was not rejected for τ = 0.00 sec, only; both other cases were rejected.

The simulated flows were then analysed to produce headway distributions using values of τ varying from 0.0 to 1.0 sec in 0.1 sec increments. These were compared with the headway distribution obtained at Castle Square but the hypothesis that they were similar was rejected for all values of τ. However the ones that came nearest to the value that would have permitted no rejection were in the region of τ = 0.60 - 0.75 sec, contrary to expectations. Inspecting the results it could be seen that the main contributors to the difference were the intervals 0 - 1.0 sec and 1.0 - 2.0 sec, the first two, which contain the smallest offered gaps. This can be seen in Table 4.32 which shows the χ^2-test for two theoretical exponential distributions and the observed. The agreement in all other intervals was reasonable.

From the point of view of which gaps influence the behaviour of the entry flow, it is obvious that the most important ones are the ones which would allow vehicles to join the circulating flow, i.e. the ones larger than the critical gap. Therefore, the precise distribution of the gaps less than 2 seconds in the two categories would not influence at all the capacity and delay of the entering vehicles. What is of more importance is that the total number of gaps in these two categories combined is similar in both simulated and observed distributions. Therefore, it was decided to repeat the χ^2-test by combining the first two intervals of the distributions. The results were radically different from the previous analysis. The values of τ for which the hypothesis was not rejected were in the range of 0.00 to 0.40 sec with the value of $\tau = 0.20$ sec giving the lowest χ^2 value. As this value was also the mid-point of the range, it was decided to adopt $\tau = 0.20$ sec as the value to be used in the roundabout simulation model. (See Table 4.33 for the χ^2 values obtained at this stage of the analysis.)

TABLE 4.1

No.	Location of Sites	q_s	β	L	α
1	Redbridge Roundabout, Southampton	0.9984	1.00	3.1631	3.66
2	Millbrook Roundabout, Southampton	0.7868	1.27	2.0787	2.71
3	Sports Centre Roundabout, Southampton	0.4579	2.18	1.4232	2.52
4	Winchester Road Roundabout, Southampton	0.7771	1.29	2.2733	2.92
5	Housdown Roundabout, Southampton	0.6944	1.44	2.2335	2.95
6	Casham Roundabout, Portsmouth	1.0847	0.92	1.9960	2.45
7	" " "	0.4735	2.11	0.7166	1.77
8	Hilsea Roundabout, Portsmouth	1.0095	0.99	2.7865	3.28
9	" " "	1.2575	0.80	2.0652	2.46
10	Fareham Roundabout	0.7183	1.39	2.0949	2.79
11	Roundabout Hotel, Fareham	0.8422	1.19	3.0088	3.60
12	Whitton, London area	0.7350	1.36	1.7984	2.48
13	" " "	0.7252	1.38	2.0478	2.44
14	" " "	0.8211	1.22	2.2243	2.83
15	" " "	0.9170	1.09	2.1710	2.72
16	A310/316 Junction Roundabout, London area	0.8602	1.16	1.9632	2.54
17	" " " "	0.7656	1.31	2.3876	3.04
18	" " " "	0.7650	1.31	2.0720	2.73
19	" " " "	0.6468	1.55	1.8935	2.67
20	Turk's Head Roundabout London area	0.7538	1.33	1.9621	2.63
21	" " " "	0.9713	1.03	2.9132	3.43
22	" " " "	0.6489	1.54	1.9166	2.69
23	" " " "	0.6353	1.57	1.7402	2.53
24	Stoke Roundabout, Guildford	0.8059	1.24	2.0173	2.64
25	Stoke Roundabout, Guildford	0.6899	1.45	3.1383	3.86
26	Tite Hill Roundabout, Surrey	0.3699	2.70	2.3442	2.70
27	Runnymede Roundabout, Surrey	0.9174	1.09	2.0177	2.56
28	Crooked Billet Roundabout, London area	1.3457	0.74	2.0154	2.39
29	" " " "	1.2693	0.79	2.3913	2.79
30	" " " "	1.0287	0.97	1.4641	1.95
31	" " " "	0.8394	1.19	2.2579	3.15
32	Bedfont Roundabout, London area	0.8726	1.15	1.9767	2.55
33	" " "	0.8191	1.22	1.8316	2.44
34	Thames Ditton Roundabout, London area	0.4236	2.36	2.2808	3.46
35	Nottingham Roundabout	1.0244	2.41	0.9762	2.90
36	Durham Roundabout	0.4939	2.04	2.0247	3.05
37	Blue House Roundabout, Newcastle	0.7890	2.61	1.2674	3.24
38	TRRL Test Track Experiment	0.2893	2.68	3.4566	4.40
39	" " " "	0.3575	3.42	2.7972	4.22
40	" " " "	0.5177	2.83	1.9316	3.79
41	" "	0.6180	1.62	2.2360	3.05
42	"	0.3217	3.11	2.2190	3.77
43	"	0.3795	2.64	2.5417	3.86
44	"	0.6190	1.62	2.4613	3.27
45	"	0.9025	1.11	1.7072	2.26
46	"	0.5149	1.94	1.5258	2.50
47	"	0.4616	2.17	1.8218	2.91
48	"	0.9707	1.03	2.6552	3.17
49	"	0.9289	1.08	2.2902	2.83
50	"	0.7027	1.42	1.9442	2.66
51	"	0.8300	1.20	1.9547	2.56

TABLE 4.1 (continued)

No.	Location of Sites	q_s	β	L	α
52	T.R.R.L. Test Track	1.0670	0.94	1.6174	2.09
53	"	0.9028	1.11	1.5891	2.14
54	"	0.8777	1.14	1.5116	2.08
55	"	0.8300	1.20	1.5731	2.18
56	"	0.9592	1.04	2.2080	2.75
57	"	0.9694	1.03	2.3797	2.90
58	"	1.0568	0.95	2.187	2.66
59	"	1.2013	0.83	2.0239	2.44
60	"	1.0153	0.98	1.93377	2.43
61	"	0.9701	1.03	1.7645	2.28
62	"	data of last site not available			

Table 4.1 The sites studied by Armitage and McDonald and the results of the regression on the collected data

TABLE 4.2

Nos. of sites	Location	α:mean	α:st.deviation	β:mean	β:st.deviation
1-5	Southampton	2.95	0.43	1.43	0.45
6-9	Portsmouth	2.49	0.62	1.20	0.61
10-11	Fareham	3.20	0.58	1.29	0.14
12-15	Whitton, London	2.69	0.15	1.26	0.14
16-19	A310/316 roundabout, London	2.74	0.21	1.33	0.16
20-23	Turk's Head " "	2.82	0.41	1.37	0.25
24-25	Guildford	3.25	0.87	1.35	0.15
26-27	Surrey	3.13	0.80	1.13	1.83
28-31	Crooked Billet, London	2.57	0.52	0.92	0.20
32-33	Bedfont, "	2.50	0.08	1.18	0.05
34	Thames Ditton "				
35	Nottingham				
36	Durham				
37	Newcastle				
38-61	TRRL track experiment	2.91	0.75	1.56	0.76
12-23/28-34	London sites	2.72	0.36	1.28	0.35
1-11/24-27/ 35-37	Non-London sites	2.95	0.54	1.41	0.52
1-37	All public road sites	2.83	0.46	1.34	0.44
1-61	All sites	2.86	0.59	1.43	0.59

TABLE 4.2 The sites studied by Armitage and McDonald grouped according to certain characteristics; average results over each group

TABLE 4.3

group	number of entries on gap size				gap size on number of entries			
	all values included		highest values excluded		all values included		highest values excluded	
	α	β	α	β	α	β	α	β
1	4.35	2.41	3.65	2.96	4.48	2.37	4.16	2.62
2	3.18	3.35			3.73	3.04		
3	4.08	2.64	3.40	3.34	4.30	2.52	4.17	2.66
4	4.36	2.41	3.63	2.96	4.50	2.37	4.06	2.69
5	3.19	3.39			3.72	3.09		
6	4.00	2.64	3.62	3.01	4.24	2.51	4.34	2.41
7	3.62	2.98			4.01	2.74		
8	3.18	3.44			3.71	3.11		
9	4.01	2.63	3.66	2.98	4.24	2.51	4.35	2.40
10	3.61	3.02			4.01	2.76		
11	3.77	2.90	3.15	3.43	4.26	2.67	3.69	3.11
12	4.27	2.42	3.72	2.89	4.39	2.38	4.33	2.44
13	3.26	3.35			3.76	3.06		
14	3.91	2.73	3.46	3.14	4.19	2.59	3.94	2.82
15	4.28	2.42	3.77	2.84	4.39	2.38	4.35	2.42
16	3.26	3.31			3.75	3.03		
17	3.99	2.72	3.46	3.21	4.30	2.56	4.00	2.82
18	4.27	2.42	3.69	2.92	4.38	2.39	4.31	2.45
19	3.26	3.29			3.80	2.97		
20	4.04	2.73	3.38	3.37	4.34	2.57	3.88	3.00

Table 4.3 Gap acceptance parameters estimated by least square analysis.
This set had the following initial values: α = 4.01 sec, β = 2.77 sec, q = 0.44 veh/sec

TABLE 4.4

		α		β	
		mean	st.dev.	mean	st. dev.
(i)	all values	3.793	0.437	2.858	0.378
	excluded top	3.457	0.212	3.158	0.207
(ii)	all values	4.125	0.283	2.681	0.273
	excluded top	4.003	0.243	2.781	0.261

TABLE 4.4 Comparison between predictions based on the inclusion and exclusion of top values and on two definitions of the dependent variable;
set (i) assumes as dependent variable the number of entries
set(ii) assumes as dependent variable the size of the gaps

initial values α = 4.01 sec β = 2.77 sec q = 0.44 veh/sec

Table 4.5

	NT		TN	
	β	α	β	α
1	2.95	3.17	2.89	3.34
2	2.94	3.17	2.86	3.39
3	2.68	3.71	2.65	3.83
4	2.95	3.20	2.92	3.33
5	2.88	3.23	2.79	3.47
6	3.04	2.89	3.02	3.02
7	2.89	3.30	2.84	3.48
8	2.87	3.25	2.86	3.35
9	2.86	3.54	2.82	3.70
10	2.87	3.41	2.72	3.72

TABLE 4.5 Gap- acceptance parameters estimated by linear models initial value $\alpha = 3.50$ sec, $\beta = 2.80$ sec, $q = 0.26$ veh/sec

NT: N, dependent; T, independent variables
TN: T, dependent; N, independent variables
N: the number of entering vehicles per gap
T: the length of the gap (sec)
β: move-up time (sec)
α: critical gap (sec)
q: circulating flow (veh/sec)

both methods of analysis were weighted linear models

TABLE 4.6

	NT		TN	
	β	α	β	α
1	2.07	2.27	2.04	2.36
2	1.91	2.56	1.87	2.68
3	1.88	2.61	1.81	2.78
4	2.04	2.17	2.04	2.22
5	2.03	2.46	1.93	2.69
6	1.85	2.79	1.82	2.93
7	1.84	2.74	1.80	2.86
8	1.99	2.47	1.94	2.65
9	2.19	2.09	2.17	2.19
10	2.09	2.24	2.05	2.36

TABLE 4.6 Gap-acceptance parameters estimated by linear models. initial value α = 2.50 sec, β = 2.00 sec, q = 0.35 veh/sec
method of analysis (both weighted linear models)
NT: N, dependent, T, independent variables
TN: T, dependent, N, independent variables
N: the number of entering vehicles per gap
T: the length of the gap (sec)
α: move-up time (sec)
β: critical gap (sec)
q: circulating flow (veh/sec)

TABLE 4.7

		α		β	
		mean	st.dev.	mean	st.dev.
(i)	NT	2.44	0.240	1.99	0.116
	TN	2.57	0.267	1.95	0.125
(ii)	NT	3.29	0.225	2.89	0.093
	TN	3.46	0.237	2.84	0.104

TABLE 4.7 Comparison between predictions
Mean and standard deviation of predictions using weighted least squares analysis
Two sets of initial values
(i) α = 2.50 sec, β = 2.00 sec, q = 0.35 veh/sec
(ii) α = 3.50 sec, β = 2.80 sec, q = 0.26 veh/sec
Two definitions of dependent/independent variables
NT: number of entries (N), dependent; gap size (T) independent
TN: gap size (T), dependent, number of entries (N), independent

TABLE 4.8

	WRNT		SRTN		SRNT	
	β	α	β	α	β	α
1	2.54	2.84	2.73	2.52	2.45	2.80
2	2.33	3.26	2.62	2.66	2.39	2.90
3	2.25	3.35	2.69	2.60	2.39	2.91
4	2.56	2.58	2.61	2.69	2.46	2.86
5	2.42	3.16	2.78	2.51	2.52	2.77
6	2.74	2.75	2.89	2.48	2.62	2.74
7	2.23	3.47	2.81	2.60	2.43	2.95
8	2.42	3.16	2.71	2.55	2.50	2.76
9	2.71	2.63	2.85	2.42	2.61	2.66
10	2.54	2.92	2.83	2.48	2.50	2.79
11	2.51	2.91	2.82	2.41	2.53	2.69
12	2.53	2.92	2.83	2.45	2.49	2.75
13	2.63	2.82	2.71	2.58	2.52	2.79
14	2.51	3.03	2.78	2.51	2.55	2.75
15	2.40	3.07	2.65	2.58	2.44	2.81
16	2.75	2.68	2.98	2.32	2.56	2.70
17	2.50	2.95	2.72	2.59	2.49	2.81
18	2.47	3.07	2.72	2.51	2.46	2.79
19	2.54	2.93	2.73	2.53	2.50	2.79
20	2.40	3.46	2.77	2.53	2.57	2.73

TABLE 4.8 Gap-acceptance parameter estimation
Input balues α = 3.00 sec, β = 2.50 sec, q = 0.30 veh/sec
Methods of analysis (simple linear models)
WRNT: weighted, N: dependent, T: independent variables
SRTN: unweighted, T: dependent, N: independent variables
SRNT: unweighted, N: dependent, T: independent variables
N: the number of entering vehicles per gap
T: the length of the gap (sec)
β: move-up time (sec)
α critical gap (sec)
q: circulating flow (veh/sec)

TABLE 4.9

	WRNT		SRTN		SRNT	
	β	α	β	α	β	α
1	2.42	3.33	2.51	2.89	2.60	2.74
2	2.44	3.13	2.49	2.84	2.57	2.71
3	2.50	2.91	2.46	2.91	2.58	2.76
4	2.63	1.85	2.50	2.87	2.56	2.76
5	2.52	3.01	2.52	2.90	2.60	2.77
6	2.60	2.63	2.48	2.92	2.57	2.77
7	2.42	3.07	2.47	2.79	2.58	2.63
8	2.52	2.97	2.52	2.81	2.60	2.67
9	2.48	3.26	2.52	2.90	2.59	2.78
10	2.44	3.16	2.46	2.93	2.57	2.79

TABLE 4.9 Gap-acceptance parameter estimation
Input values α = 3.00 sec, β = 2.50 sec, q = 0.20 veh/s
Method of analysis (all simple linear models)
WRNT: weighted; N: dependent, T: independent variables
SRTN: unweighted; T: dependent, N: independent variables
SRNT: unweighted; N: dependent, T: independent variables
N: the number of entering vehicles per gap
T: the length of the gap (sec)
β: move-up time (sec)
α: critical gap (sec)
q: circulating flow (veh/sec)

TABLE 4.10

	WRNT		SRTN		SRNT	
	β	α	β	α	β	α
1	2.07	2.27	1.96	2.34	2.19	2.11
2	1.91	2.56	1.91	2.41	2.11	2.22
3	1.88	2.61	1.94	2.39	2.20	2.13
4	2.04	2.17	1.98	2.39	2.10	2.25
5	2.03	2.46	2.03	2.36	2.25	2.14
6	1.85	2.79	1.97	2.40	2.17	2.19
7	1.84	2.74	1.97	2.41	2.29	2.12
8	1.99	2.47	2.00	2.33	2.17	2.16
9	2.19	2.09	2.08	2.24	2.26	2.05
10	2.09	2.24	1.97	2.32	2.23	2.08

TABLE 4.10 Gap-acceptance parameter estimation
Input values α = 2.50 sec, β = 2.00 sec, q = 0.35 veh/s
Methods of analysis (all simple linear models)
WRNT: weighted; N: dependent, T: independent variables
SRTN: unweighted; T: dependent, N:independent variables
SRNT: unweighted, N: dependent, T: independent variables
N: the number of entering vehicles per gap
T: the length of the gap (sec)
β: move-up time (sec
α: critical gap (sec)
q: circulating flow (veh/sec)

TABLE 4.11

	WRNT		SRTN		SRNT	
	β	α	β	α	β	α
1	2.61	3.99	2.64	3.74	2.85	3.51
2	2.93	3.46	2.77	3.56	3.05	3.26
3	2.68	3.98	2.75	3.58	2.98	3.31
4	2.62	4.27	2.77	3.58	2.94	3.39
5	2.86	3.67	2.75	3.73	2.95	3.50
6	2.64	4.01	2.71	3.65	3.00	3.35
7	2.97	3.39	2.79	3.71	2.96	3.49
8	2.82	3.63	2.75	3.60	2.97	3.36
9	2.82	3.62	2.70	3.73	2.92	3.47
10	2.76	4.08	2.88	3.57	3.11	3.30

TABLE 4.11 Gap- acceptance parameter estimation
Input values α = 3.85s, β = 2.75s, q = 0.26 veh/s
Method of analysis (all simple linear models)
WRNT: weighted, N: dependent, T: independent variables
SRTN: unweighted, T: dependent, N: independent variables
SRNT: unweighted, N: dependent, T: independent variables
N: the number of entering vehicles per gap
T: the length of the gap (sec)
β: move-up time (sec)
α: critical gap (sec)
q: circulating flow (veh/sec)

TABLE 4.12

	WRNT		SRTN		SRNT	
	β	α	β	α	β	α
1	2.95	3.17	2.82	3.29	3.08	3.02
2	2.94	3.17	2.87	3.12	3.12	2.84
3	2.68	3.71	2.77	3.32	2.94	3.12
4	2.95	3.20	2.78	3.40	3.01	3.14
5	2.88	3.23	2.81	3.21	3.08	2.92
6	3.04	2.89	2.85	3.26	3.06	3.01
7	2.89	3.30	2.82	3.36	3.05	3.11
8	2.87	3.25	2.79	3.33	3.03	3.06
9	2.86	3.54	2.90	3.23	3.14	2.95
10	2.87	3.41	2.87	3.25	3.24	2.88

TABLE 4.12 Gap-acceptance parameter estimation
Input values α =3.50s, β = 2.80, q = 0.26 veh/s
Methods of analysis (all simple linear models)
WRNT: weighted; N: dependent; T: independent variables
SRTN: unweighted; T: dependent; N: independent variable
SRNT: unweighted; N: dependent; T: independent variable
N: the number of entering vehicles per gap
T: the length of the gap (sec)
β: move-up time (sec)
α: critical gap (sec)
q: circulating flow (veh/sec)

TABLE 4.13

		α		β	
		mean	st.dev.	mean	st.dev
(i)	NT	2.15	0.062	2.20	0.062
	TN	2.36	0.053	1.98	0.047
(ii)	NT	3.01	0.104	3.08	0.081
	TN	3.28	0.081	2.83	0.043

TABLE 4.13 Mean and standard deviation of predictions using unweighted least squares analysis
Two sets of initial values
(i) $\alpha = 2.50$ sec, $\beta = 2.00$ sec, $q = 0.35$ veh/sec
(ii) $\alpha = 3.50$ sec, $\beta = 2.80$ sec, $q = 0.26$ veh/sec
Two definitions of dependent/independent variables
NT: number of entries (N), dependent; gap size (T), independent
TN: gap size (T), dependent; number of entries (N), independent

TABLE 4.14

	T on N	N on T
β mean	2.450	2.385
st.dev.	0.102	0.097
α mean	2.832	3.024
st.dev.	0.238	0.225

TABLE 4.14 Mean predictions using reduced sample size of 200 gaps per group
Input value $\alpha = 2.95$s, $\beta = 2.42$ sec, $q = 0.20$ veh/sec

TABLE 4.15

	T on N	N on T
β mean	2.430	2.389
st.dev.	0.093	0.094
α mean	2.882	3.024
st.dev.	0.229	0.213

TABLE 4.15 Mean prediction using sample size 500 per group
Same input values as in Table 4.14

TABLE 4.16

	no. of entries on gap size		gap size on no. of entries	
group	α	β	α	β
1	2.91	2.71	3.34	2.60
2	3.37	2.63	3.48	2.59
3	3.40	2.49	3.63	2.40
4	3.38	2.50	3.56	2.45
5	3.56	2.42	3.68	2.40
6	3.07	2.65	3.41	2.57
7	3.30	2.52	3.67	2.39
8	3.85	2.27	3.95	2.25
9	3.66	2.34	3.98	2.25
10	3.49	2.47	3.80	2.37
11	3.29	2.48	3.37	2.47
12	3.17	2.65	3.44	2.56
13	3.14	2.59	3.38	2.54
14	3.50	2.44	3.74	2.38
15	3.47	2.53	3.73	2.45
16	3.72	2.41	3.87	2.37
17	2.93	2.65	3.31	2.56
18	2.78	2.69	2.96	2.67
19	3.40	2.46	3.74	2.33
20	2.98	2.65	3.34	2.54

TABLE 4.15 Results of applying proposed method on simulation data that involve normally distributed values of the gap acceptance parameters
Input values
α: mean = 3.50 sec, standard deviation 0.50 sec
β: mean = 2.50 sec, standard deviation 0.25 sec

TABLE 4.17

	T on N	N on T
β mean	2.53	2.46
st.dev.	0.122	0.118
α mean	3.32	3.57
st.dev.	0.287	0.255

Input values

	β	α
mean	2.50	3.50
st. dev	0.25	0.50

TABLE 4.17 Overall averages of the results of applying proposed method on simulation data assuming gap acceptance parameters to be normally distributed.

TABLE 4.18

	L	q_s	β	α
1	1.98	0.427	2.34	3.15
2	2.02	0.429	2.33	3.19
3	2.05	0.431	2.32	3.21
4	1.98	0.419	2.39	3.17
5	2.00	0.421	2.38	3.19
6	1.83	0.396	2.53	3.09
7	2.09	0.426	2.35	3.26
8	1.97	0.422	2.37	3.16
9	1.89	0.405	2.46	3.12
10	2.00	0.420	2.38	3.19
11	1.90	0.418	2.39	3.10
12	2.07	0.434	2.31	3.22
13	1.99	0.418	2.39	3.19
14	2.05	0.422	2.37	3.24
15	1.92	0.422	2.37	3.11
16	2.00	0.420	2.38	3.19
17	1.98	0.418	2.39	3.19
18	1.90	0.415	2.41	3.10
19	1.99	0.422	2.37	3.18
20	2.09	0.421	2.37	3.28

TABLE 4.18 Gap-acceptance parameter estimation
Input values $\alpha = 3.00$ sec, $\beta = 2.50$ sec, $q = 0.30$ veh/sec
Method of analysis Armitage and McDonald

TABLE 4.19

	L	q_s	β	α
1	1.99	0.410	2.43	3.21
2	1.80	0.405	2.47	3.03
3	1.89	0.411	2.43	3.11
4	1.79	0.402	2.49	3.03
5	1.90	0.404	2.47	3.13
6	1.89	0.406	2.46	3.12
7	1.80	0.411	2.44	3.02
8	1.79	0.401	2.49	3.04
9	1.90	0.404	2.48	3.13
10	1.98	0.414	2.41	3.19

TABLE 4.19 Gap-acceptance parameter estimation
Input values $\alpha = 3.00$ sec, $\beta = 2.50$ sec, $q = 0.20$ veh/s
Method of analysis Armitage and McDonald

TABLE 4.20

	L	q_s	β	α
1	1.70	0.54	1.87	2.63
2	1.69	0.53	1.88	2.63
3	1.70	0.53	1.89	2.64
4	1.69	0.52	1.92	2.65
5	1.70	0.52	1.93	2.66
6	1.69	0.52	1.91	2.64
7	1.69	0.52	1.92	2.65
8	1.68	0.52	1.91	2.63
9	1.69	0.52	1.93	2.66
10	1.70	0.53	1.88	2.64

TABLE 4.20 Gap-acceptance parameter estimation
Input values $\alpha = 2.50$ sec, $\beta = 2.00$ sec, $q_s = 0.35$ veh/s
Method of Analysis Armitage and McDonald

TABLE 4.21

	L	q_s	β	α
1	2.74	0.387	2.58	4.03
2	2.64	0.377	2.65	3.87
3	2.64	0.378	2.65	3.96
4	2.63	0.376	2.66	3.96
5	2.64	0.370	2.70	3.99
6	2.75	0.383	2.61	4.06
7	2.73	0.371	2.69	4.08
8	2.75	0.385	2.60	4.05
9	2.69	0.377	2.65	4.02
10	2.64	0.365	2.74	4.01

TABLE 4.21 Gap-acceptance parameter estimation
Input values $\alpha = 3.85s$, $\beta = 2.75s$, $q = 0.26$ veh/s
Method of Analysis Armitage and McDonald

TABLE 4.22

	L	q_s	β	α
1	2.30	0.367	2.72	3.66
2	2.19	0.366	2.73	3.56
3	2.39	0.377	2.65	3.72
4	2.30	0.364	2.74	3.67
5	2.30	0.373	2.68	3.64
6	2.29	0.363	2.75	3.67
7	2.39	0.369	2.71	3.74
8	2.30	0.367	2.72	3.66
9	2.29	0.361	2.77	3.68
10	2.28	0.363	2.75	3.66

TABLE 4.22 Gap-acceptance parameter estimation
Input values $\alpha = 3.50s$, $\beta = 2.80s$, $q = 0.26$ veh/s
Method of Analysis Armitage and McDonald

TABLE 4.23

	input values	AMM mean	AMM st.dev.	WRNT mean	WRNT st.dev	SRTN mean	SRTN st.dev.	SRNT mean	SRNT st.dev.
q	0.30								
α	3.00	3.18	0.053	3.00	0.256	2.79	0.074	2.53	0.087
β	2.50	2.38	0.048	2.50	0.143	2.50	0.064	2.76	0.092
q	0.20								
α	3.00	3.10	0.069	2.93	0.428	2.88	0.048	2.74	0.052
β	2.50	2.46	0.028	2.50	0.073	2.49	0.025	2.58	0.013
q	0.35								
α	2.50	2.64	0.012	2.57	0.267	2.36	0.053	2.15	0.062
β	2.00	1.90	0.022	1.95	0.125	1.98	0.047	2.20	0.062
q	0.26								
α	3.85	4.01	0.043	3.81	0.293	3.65	0.075	3.39	0.092
β	2.75	2.65	0.049	2.77	0.130	2.75	0.063	2.97	0.071
q	0.26								
α	3.50	3.67	0.048	3.46	0.237	3.28	0.081	3.01	0.105
β	2.80	2.72	0.036	2.84	0.104	2.83	0.043	3.08	0.081

TABLE 4.23 Comparison of predictions by various methods
AMM: Armitage and McDonald
WRNT: weighted linear model, number of entries (N) dependent variable
SRTN: unweighted linear model, gap size (T) dependent variable
SRNT: unweighted linear model, number of entries (N) dependent variable
q: circulating flow (veh/sec)
α: critical gap (sec)
β: move-up time (sec)

TABLE 4.24

	initial value	AMM mean prediction	AMM absolute difference from input value	AMM % difference	SRTN mean prediction	SRTN difference from input: absolute	SRTN difference from input: %	SRNT mean prediction	SRNT different from input: absolute	SRNT different from input: %
q	0.30									
α	3.00	3.18	+0.18	6.0%	2.79	-0.21	-7.0%	2.53	-0.47	-15.7%
β	2.50	2.38	-0.12	-4.8%	2.50	0.00	0.0%	2.76	+0.26	10.4%
q	0.20									
α	3.00	3.10	+0.10	3.3%	2.88	-0.12	-4.0%	2.74	-0.26	- 8.7%
β	2.50	2.46	-0.04	-1.6%	2.49	-0.01	0.4%	2.58	+0.08	3.2%
q	0.35									
α	2.50	2.64	+0.14	5.6%	2.36	-0.14	-5.6%	2.15	-0.35	-14.0%
β	2.00	1.90	-0.10	-5.0%	1.98	-0.02	-1.0%	2.20	+0.20	10.0%
q	0.26									
α	3.85	4.01	+0.16	4.2%	3.65	-0.20	-5.2%	3.39	-0.46	-11.9%
β	2.75	2.65	-0.10	-3.6%	2.75	0.00	0.0%	2.97	+0.22	8.0%
q	0.26									
α	3.50	3.67	+0.17	4.9%	3.28	-0.22	-6.3%	3.01	-0.49	-14.0%
β	2.80	2.72	-0.08	-2.9%	2.83	+0.03	1.1%	3.08	+0.28	10.3%

TABLE 4.24 Comparison of predictions by various methods
AMM: Armitage and McDonald   SRTN: unweighted linear mode; dependent variable: gap size (T)
SRNT: unweighted linear model; dependent variable: number of entries, N
q: circulating flow   α: critical gap (sec)   β: move-up time (sec)

TABLE 4.25

Site	Lane No.	β(sec)	α(sec)
Moore St. Roundabout Ecclesall Rd entry	1	1.77	2.82
	2	1.59	2.72
	3	1.68	2.80
Castle Sq Roundabout Arundel Gate entry	1	2.60	3.75
	2	2.59	3.22
Park Square Roundabout Corn Exchange entry	1	1.89	3.50
	2	2.18	3.42
	3	2.19	3.10

TABLE 4.25 Results of analysis of gap acceptance data from observations at public sites in Sheffield

The lane numbers start at the offside and increase as they approach the nearside lane
All three sites had one more lane, the nearside one, which did not provide adequate data to be analysed

TABLE 4.26

Time interval (sec)	Observed Frequency
0.00-1.00	52
1.01-2.00	162
2.01-3.00	84
3.01-4.00	43
4.01-5.00	32
5.01-6.00	21
6.01-7.00	26
7.01-8.00	15
8.01-9.00	10
9.01-10.00	11
≥10.01	19

Table 4.26 Castle Square Roundabout Observed frequency of headways in circulating flow

TABLE 4.27

Arrivals per 15 sec interval	Observed Frequency
0	1
1	7
2	18
3	16
4	25
5	23
6	12
7	11
≥8	7

TABLE 4.27 Castle Square Roundabout Observed frequency of arrivals per 15 sec interval in circulating flow

TABLE 4.28

Arrivals per 15 sec interval	Observed Frequency
0	0
1	0
2	0
3	Q
4	1
5	1
6	12
7	6
8	8
9	15
10	9
11	8
12	10
13	14
14	10
≥15	14

TABLE 4.28 Moore Street Roundabout Observed frequency of arrivals per 15 sec interval in circulating flow

TABLE 4.29

Site	No. of groups	$\frac{\pm 1.96}{\sqrt{n}}$	regressesion correlation coefficient
Castle Square Roundabout	60	±0.25	-0.05
Moore Street Roundabout	54	±0.27	0.10

TABLE 4.29 Checking for Randomness in the Circulating Flow Data

TABLE 4.30

No. of passing vehicles	Poisson Probab.	Expected Frequency	Observed Frequency	$0^2/E$
0.	0.0123	> 8.0	1	> 8
1	0.0540		7	
2	0.1188	14.3	18	22.7
3	0.1743	20.9	16	12.2
4	0.1917	23.0	25	27.2
5	0.1687	20.2	23	26.2
6	0.1237	14.8	12	9.7
7	0.0778	9.3	11	13.0
≥ 8	0.0787	9.4	7	5.2
	1.0000	119.9	120	123.8

df = 7 $\quad \chi^2 = 3.8 < 12.59$ @ 0.05%

TABLE 4.30 χ^2 test, Poisson counting distribution with Castle Square Roundabout data

TABLE 4.31

No. of passing vehicles	Poisson Probab.	Expected Frequency	Observed Frequency	$0^2/E$
0	0.0000	↑	0	↑
1	0.0002		0	
2	0.0012		0	
3	0.0043	9.4	0	20.9
4	0.0116		1	
5	0.0250	↓	1	↓
6	0.0450		12	
7	0.0694	7.5	6	4.8
8	0.0936	10.1	8	6.3
9	0.1124	12.1	15	18.6
10	0.1214	13.1	9	6.2
11	0.1192	12.9	8	5.0
12	0.1072	11.6	10	8.6
13	0.0851	9.6	14	20.4
14	0.0687	7.4	10	13.5
≥15	0.1317	14.2	14	13.8
	1.0000	108	108	118.1

df = 10 $\chi^2 = 10.1 < 15.51$ @ 0.05%

TABL/ 4.31 χ^2-test Poisson counting distribution with Moore Street Roundabout data

TABLE 4.32

Time Interval	Observed Frequency	P(h<t) Simple Exponential	Expected Frequency	P(h<t) Shifted Exponential	Expected Frequency	$0^2/E$ Simple	$0^2/E$ Shifted
0-1	52	0.254	120	0.207	98	22	28
1-2	162	0.189	90	0.216	103	292	256
2-3	84	0.141	67	0.157	75	105	94
3-4	43	0.105	50	0.114	54	37	34
4-5	32	0.079	37	0.083	40	27	26
5-6	21	0.059	28	0.061	29	16	15
6-7	26	0.044	21	0.044	21	32	32
7-8	15	0.033	16	0.032	15	14	15
8-9	10	0.024	12	0.023	11	9	9
9-10	11	0.018	9	0.017	8	14	15
10-14	11	0.037	18	0.033	16	7	8
≥14	8	0.017	8	0.013	6	8	11
	475					584	542

Simple negative exponental df = 10 $\chi^2 = 109 >> 20.98$ @ 0.05%

Shifted negative exponential df = 9 $\chi^2 = 67 >> 19.02$ @ 0.05%

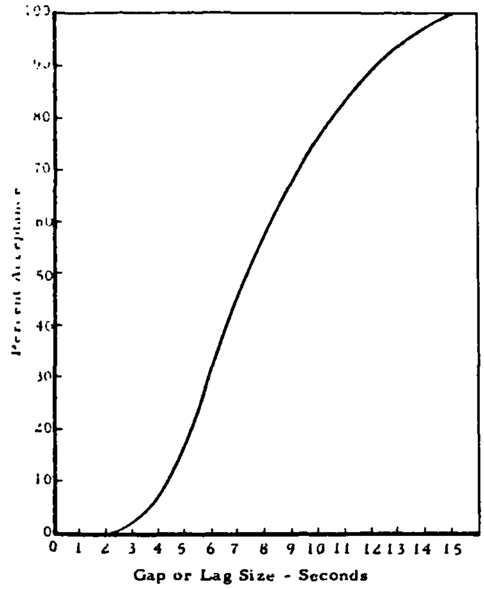
Table 4.32 χ^2-test, simple and shifted negative exponential distribution with Castle Square Roundabout data

TABLE 4.33

τ(sec)	χ^2	df
0.00	11.36	8
0.10	10.47	7
0.20	10.20	7
0.30	10.67	7
0.40	12.00	7
0.50	14.37	7
0.60	17.19	6
0.70	22.50	6
0.80	29.71	6
0.90	39.28	6
1.00	51.87	6

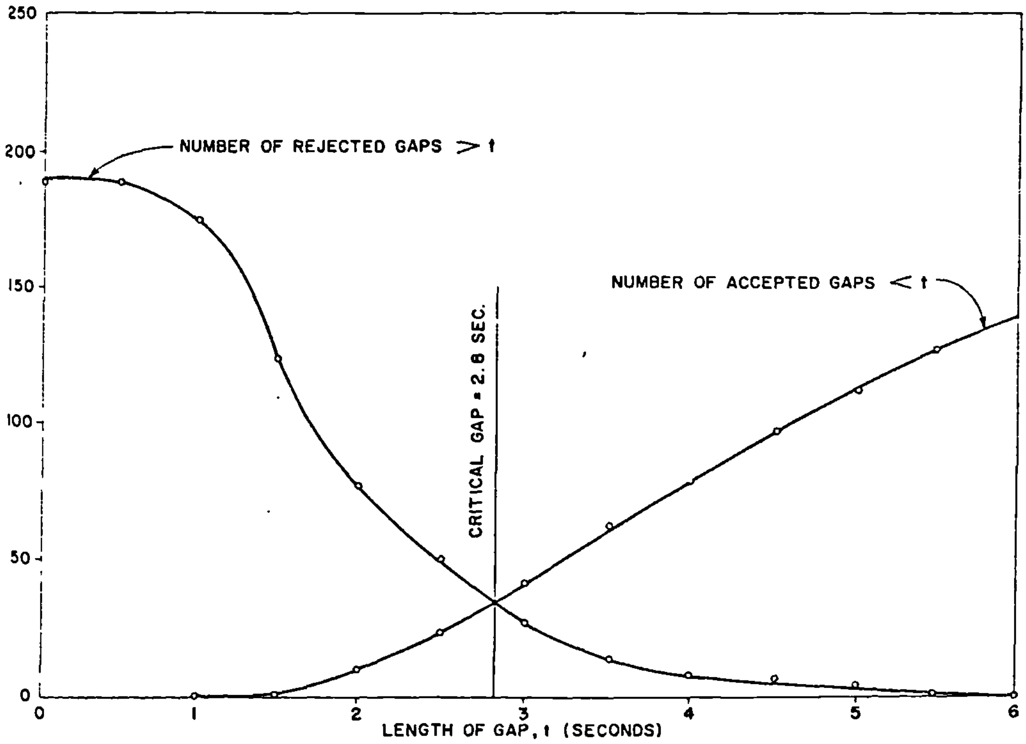
Acceptance @ 5% $\chi^2_{0.05}$ = 14.07 for 7 degrees of freedom

TABLE 4.33 χ^2-tests, Simulated headway distributions and Castle Square Roundabout data



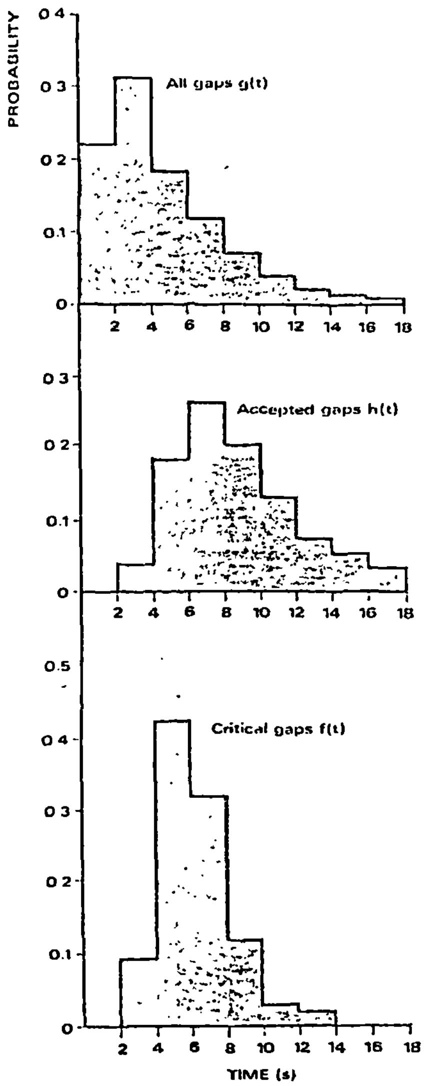
Composite lag and gap acceptance distribution.

Figure 4.1



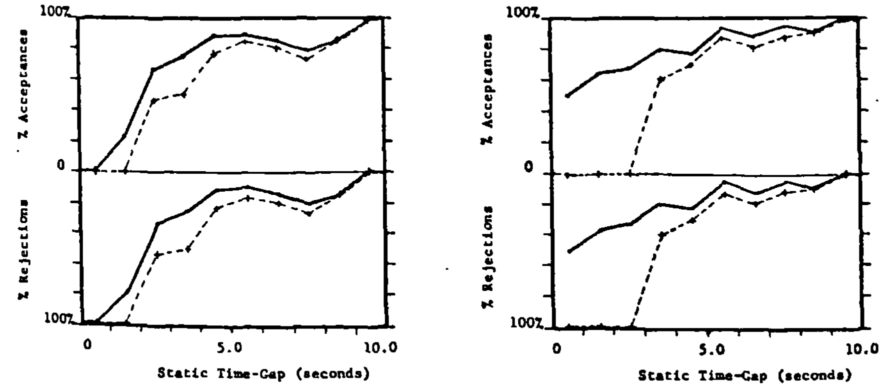
Accepted and rejected gaps for all vehicles

Figure 4.2 Raff's Critical Gap



Density of all, accepted and critical gaps

Figure 4.3 Ramsey Routledge Method

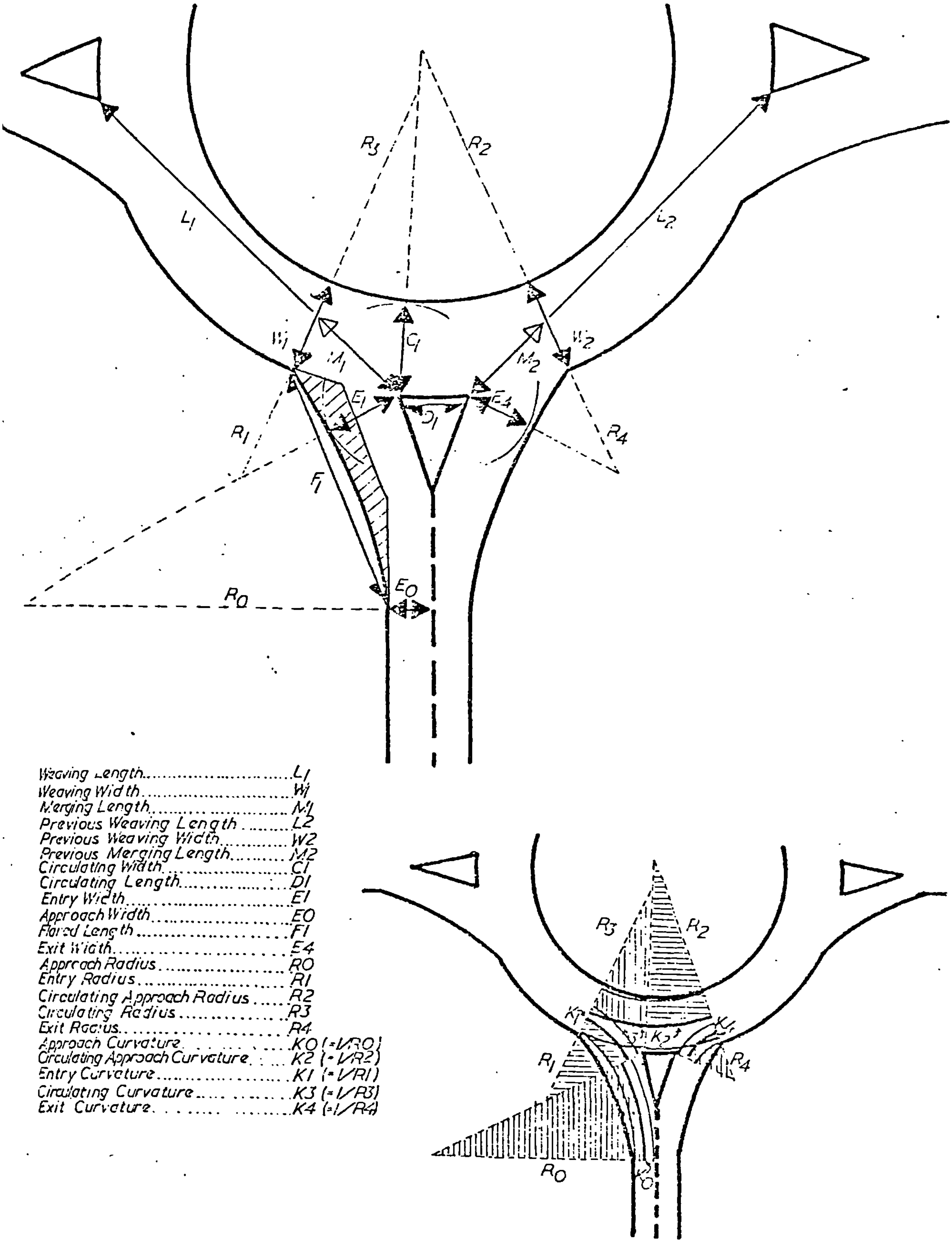


Notation:

——————— Single Vehicle Merges, i.e. one ramp vehicle only entering the mainstream
+ - - - - + - - - - + Two Vehicle Merges, i.e. two ramp vehicles accepting a single mainstream gap

Multiple merge analysis: Acceptance and rejection of static time-gaps by multiple vehicles.

Figure 4.4



. Geometric factors considered in this study.

Figure 4.5 Armitage and McDonald's Method Geometric Parameters

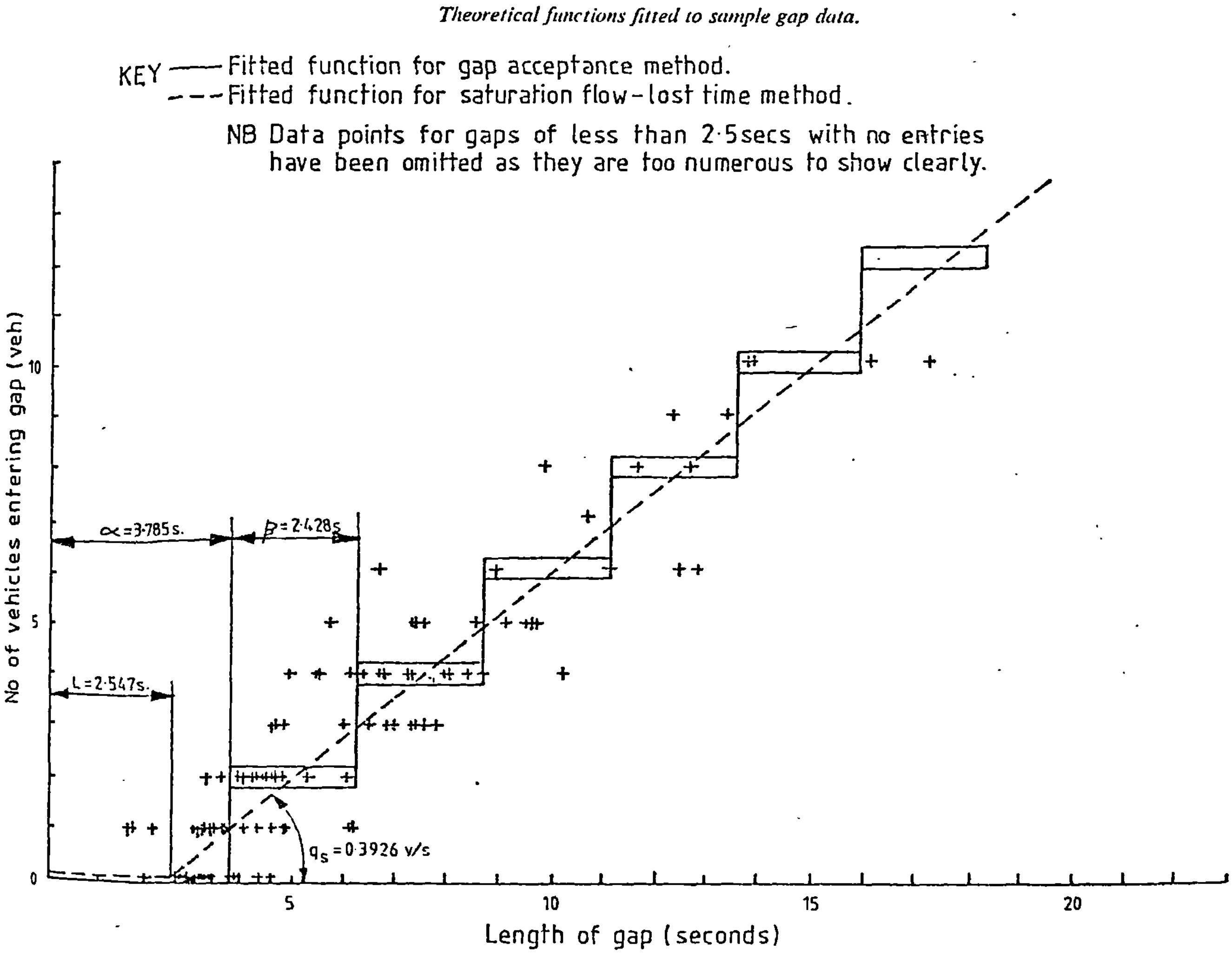


Figure 4.6 Armitage and McDonald's Method

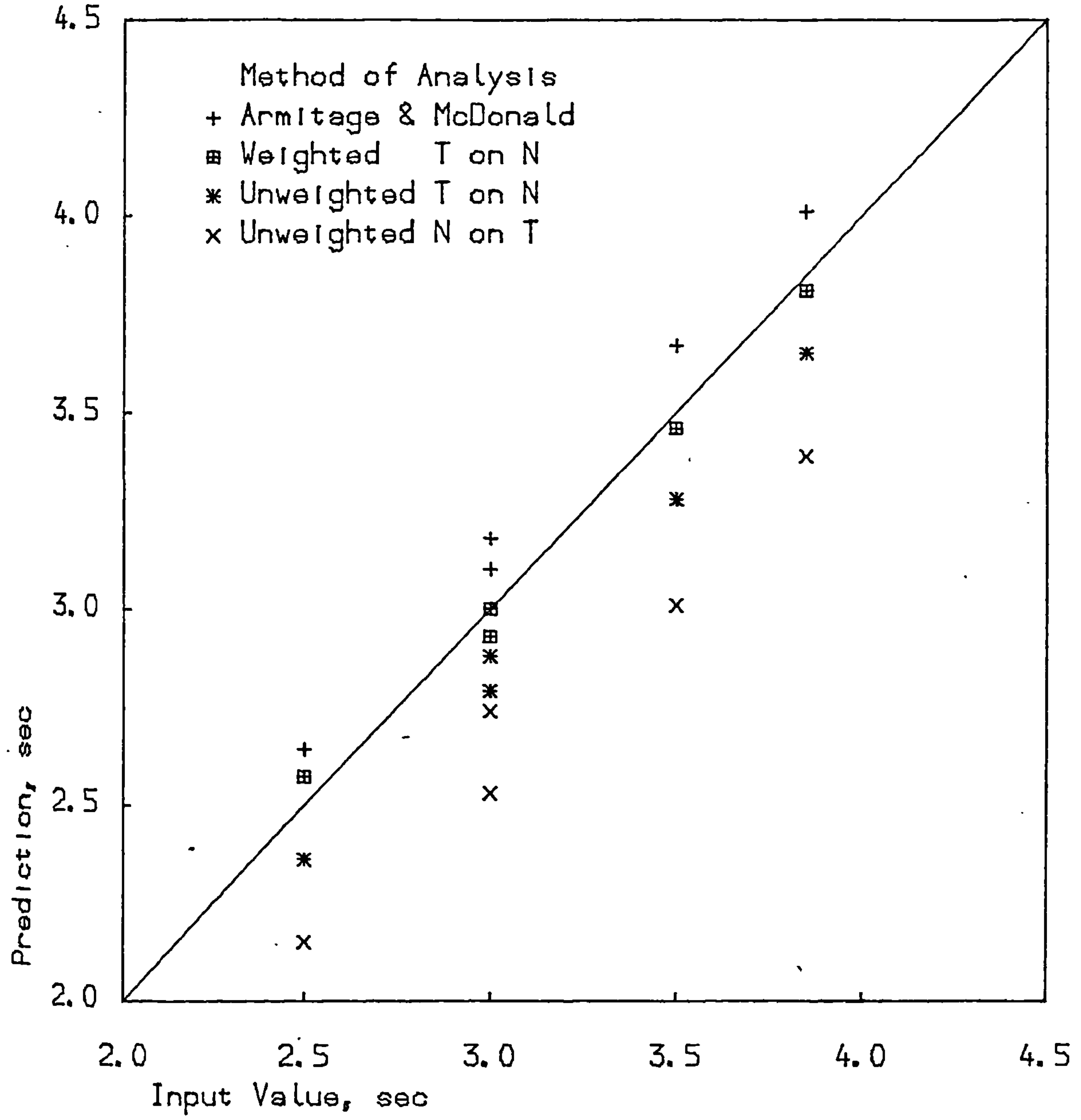


Figure 4.7 *Comparison of Input and Predicted* α *Values*

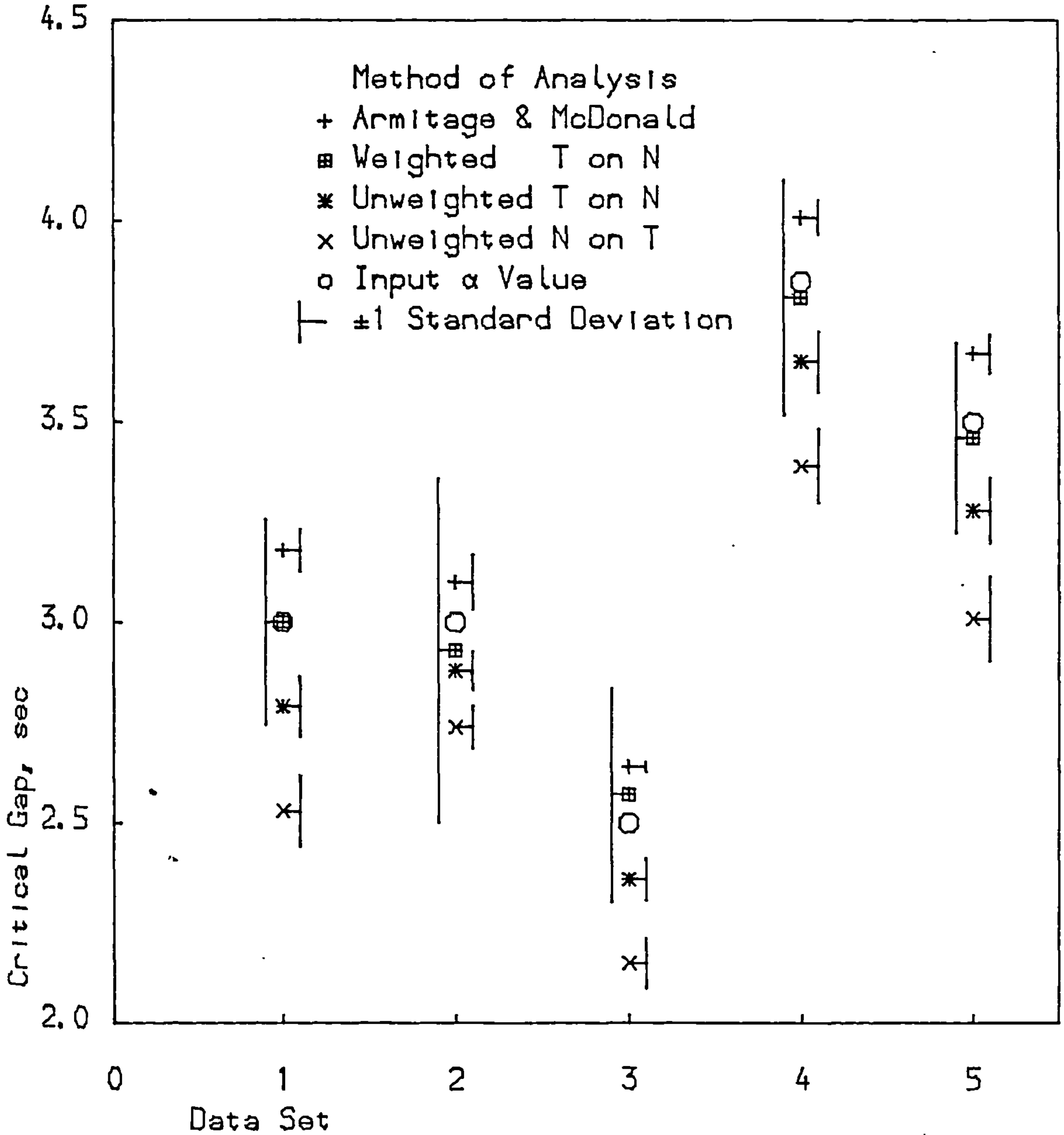


Figure 4.8 *Comparison of Standard Deviations of Predicted α Values*

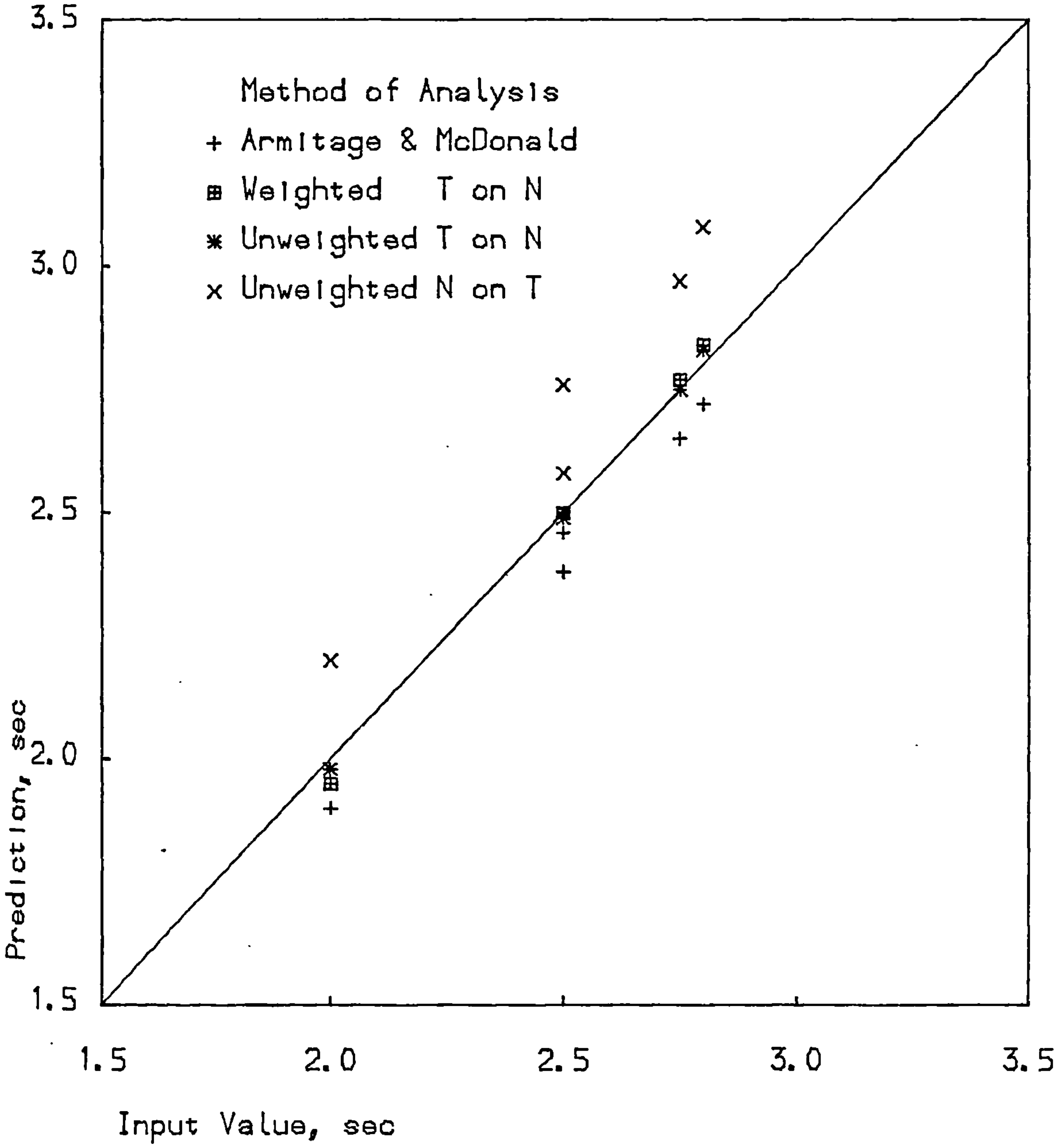


Figure 4.9 *Comparison of Input and Predicted β Values*

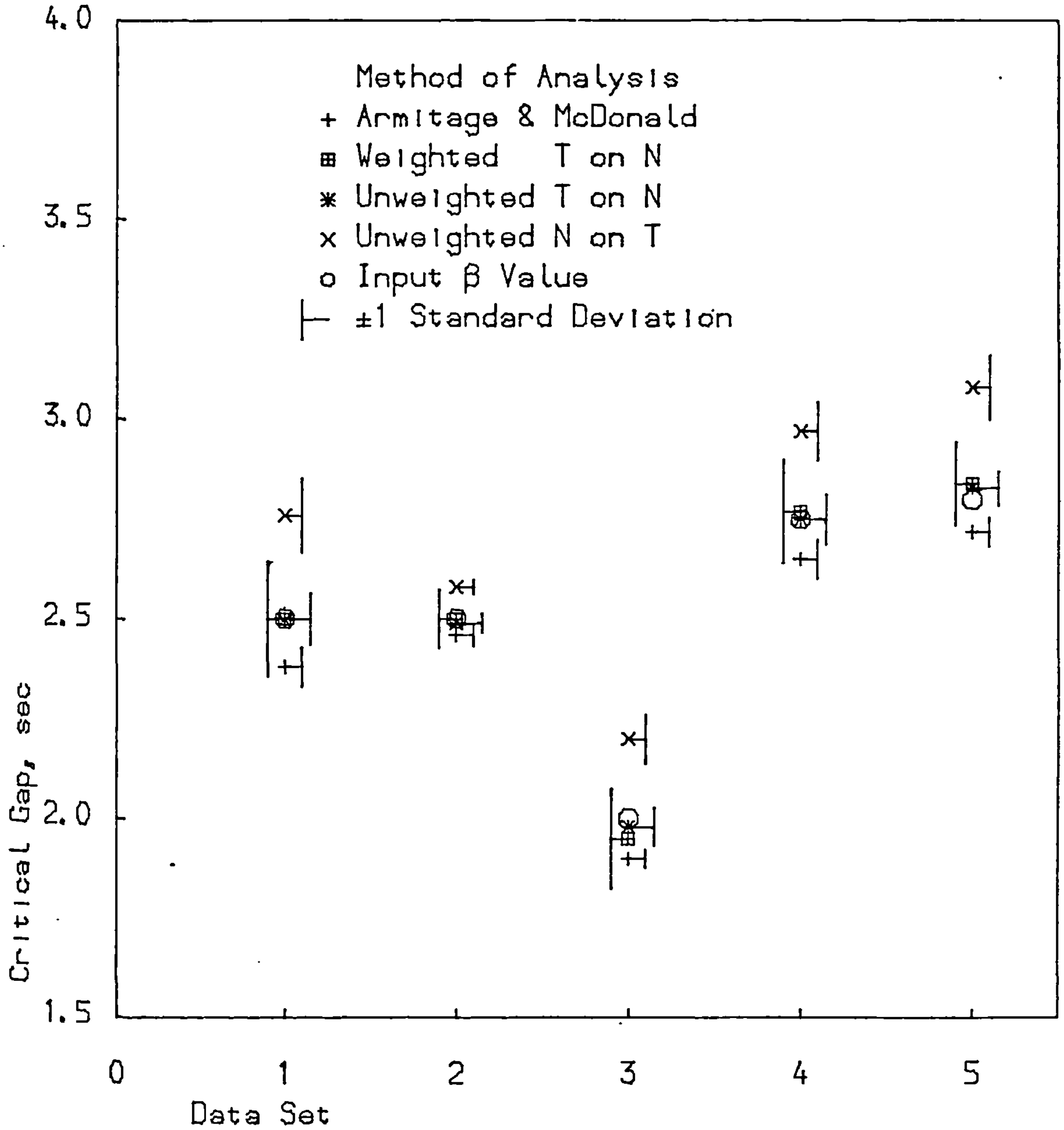
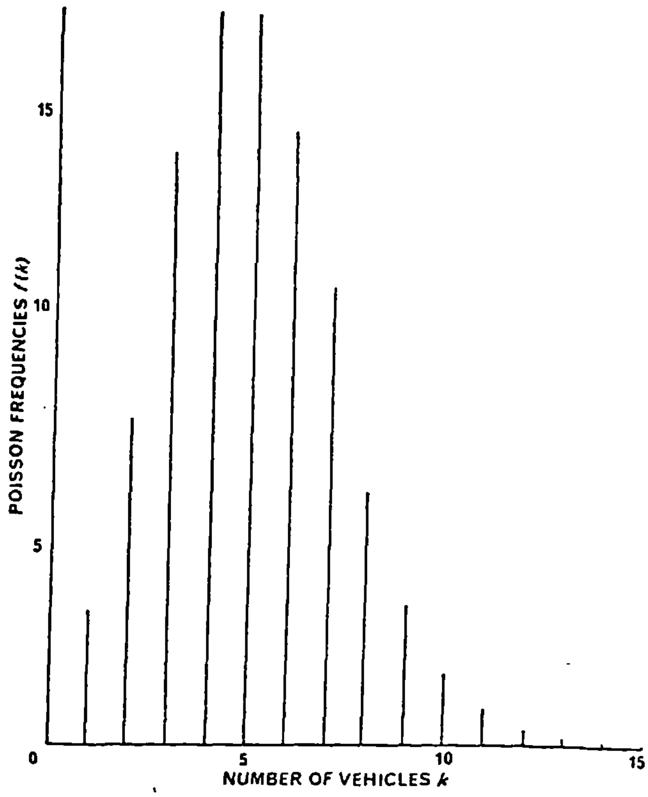
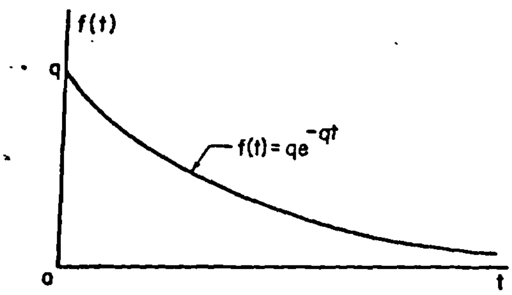


Figure 4.10 *Comparison of Standard Deviations of Predicted β Values*



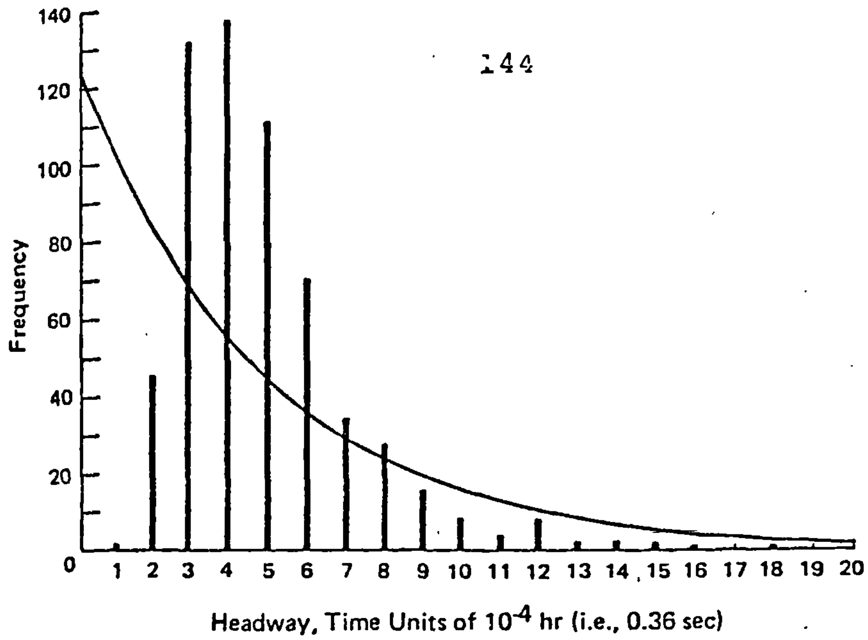
Poisson frequencies $f(k) = \frac{100\,e^{-5}\,5^k}{k!}$

Figure 4.11 Poisson Distribution



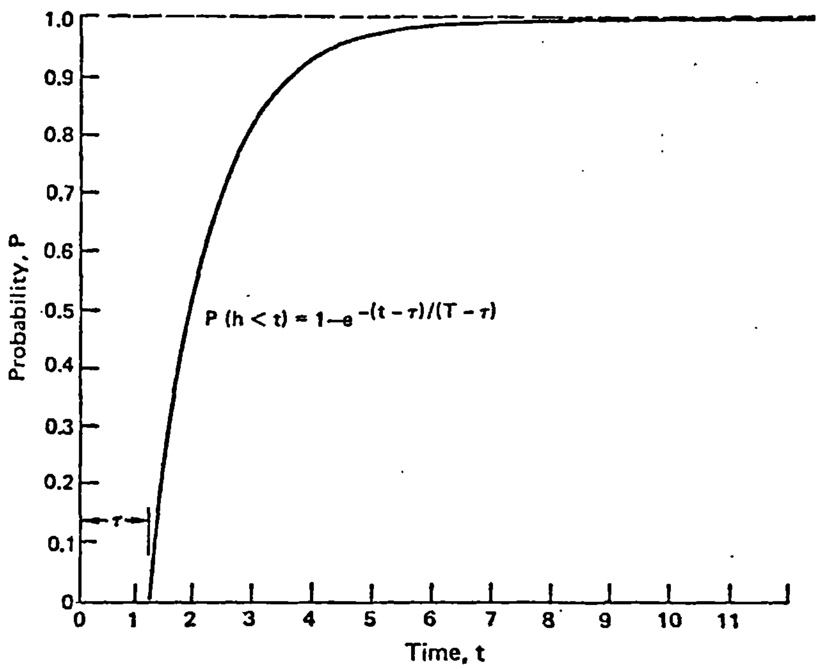
Exponential distribution.

Figure 4.12



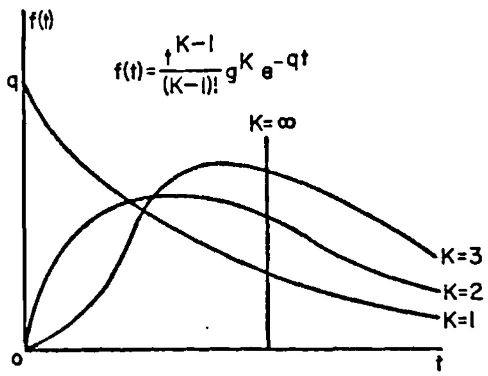
Example of negative exponential frequency curve
Bars indicate observed data taken on sample size of 609.

Figure 4.13



Shifted exponential distribution to represent the probability of headways less than *t* with a prohibition of headways less than τ. (Average of observed headways is T.)

Figure 4.14



. **The Erlang gap distribution.**

Figure 4.15

CHAPTER 5

THE SIMULATION PROGRAM

5.1 Introduction

Computer simulation models have been developed ever since general-purpose computers became readily available in the mid-50's. Short historical summaries are given by Lewis and Michael (1963) and Gerlough and Huber (1975). Lewis and Michael reported that already in 1956, three digital computer simulations were published in the traffic engineering literature.

Drew (1968) defined computer simulation as a dynamic representation of some part of the real world, achieved by building a computer model and moving it through time. The term computer model denotes a model which is not intended to be solved analytically but rather to be simulated on an electronic computer.

Simulation is a working analogy. It involves the construction of a working model presenting similarity of properties or relationships to the real problem under study. Thus complex traffic situations can be studied in the laboratory rather than the field. This allows the study of longer periods than it would be possible in reality; the repetition of certain combinations of relevant parameters with only slight modifications to determine the precise contribution to the problem of each parameter; and the comparison of alternative solutions for specific problems without the expense of in-situ long-term testing.

5.2 Generation of Random Numbers

One of the most important features of simulating traffic is the ability to generate random events. Such a generation takes place in two steps: First, a random number following a uniform (rectangular) distribution is generated. Second, this random number is treated as a probability to substitute into an appropriate distribution function in order to solve for the associated event. (Gerlough and Huber, 1975; Gordon, 1969).

Any phenomenon whose behaviour is not predictable by any obvious deterministic law and whose numerical values satisfy several tests of randomness, to ensure, for example, that each decimal digit occurs with equal frequency without any serial correlation, is accepted as random. Programs for computers can be written which will output a sequence of numbers which satisfy the various statistical tests of randomness that have been devised. Random numbers generated in a non-random fashion are called pseudorandom numbers.

The following is such a process. An assumed starting number, R_o, is multiplied by an appropriate multiplier, k. The remainder of the division by an integer M is the next random nymber, R_1, which is used to generate a subsequent random number. The relationship can be expressed as

$$R_m = k\, R_{m-1} \bmod M \qquad \text{(eq.5.1)}$$

The above will give a sequence of pseudorandom numbers in the range 1 to (M-1). R_o must be an odd integer in the range 1 to (M-1). If the numbers of the sequence are divided by M they will give a sequence of random fractions in the region 0 to 1. The cycle of random numbers is repeated after M/4 operations of equation 5.1.

The following is an example of the operation of the routine. If the initial values of the parameters are: k = 5,

$R_o = 5$, $M = 16$, then

$R_1 = 5 * 5 \bmod 16 = 9$

$R_2 = 5 * 9 \bmod 16 = 13$

$R_3 = 5 * 13 \bmod 16 = 1$

$R_4 = 5 * 1 \bmod 16 = 5$

The length of the cycle can be increased to such a value that would not allow any periodicity to the generated numbers to be observed. However, this value may be limited by the maximum integer value the computer accepts.

One advantage of the pseudorandom processes is that if the same initial number is used the same sequence will result. Thus, exactly the same traffic flow conditions can be tested for each modification of the simulated system.

5.3 Production of the Desired Random Variate

Figure 5.1 shows how the pseudorandom fractions can be converted to the desired distribution. The figure shows the cumulative probability distribution of variable X. The fraction generated by the method described above is interpreted as a probability and is used as an argument to enter the distribution giving the value of X as the function.

As an example, consider the conversion to a shifted negative exponential distribution. The cumulative form $F(t) = P(h \leq t)$ is:

$$F(t) = 1 - e^{-(t-\tau)/(\bar{t}-\tau)} \qquad \text{(eq. 5.2)}$$

where $\bar{t}$: the mean headway (sec)

t: headway (sec)

τ: the shift (sec)

Let $F(t) = r$, the random fraction in the range 0 to 1. Taking logarithms of both sides of equation 5.2

$$-\frac{t - \tau}{\bar{t} - \tau} = \log_e(1 - r) = \log_e R$$

$R = 1 - r$ is equally random in the range 0 to 1.

Solving the above for t

$$t = \tau + (\bar{t} - \tau)(-\log_e R)$$

$$t = \tau + (\bar{t} - \tau)(\log_e \frac{1}{R}) \qquad \text{(eq. 5.3)}$$

Equating τ with 0, equation 5.3 reduces to the negative exponential distribution

$$t = \bar{t} \log_e \frac{1}{R} \qquad \text{(eq. 5.4)}$$

5.4 The Program SIMC

The current project is an improvement of a simulation model built previously (Natsinas, 1979). That model simulated a flared entry to a roundabout but did not take into account vehicle turning movements. This section will describe briefly that model, called SIMC.

The assumptions incorporated in the model SIMC were:

1. The circulating flow is a negative exponential distribution.

2. The layout of the simulated entry consists of a two lane approach road flaring to four lanes at the stop line. Thus the row of vehicles at the stop line has 4 positions, the second row from the stop line has 3 positions, while subsequent rows have 2 positions.

3. There is a 2 sec minimum headway for each lane of the

approach road resulting in a maximum flow of 1800 veh/hr/lane.

4. All entry vehicles have the same critical gap, ALPHA, and the same move-up time, BETA.

5. All vehicles are passenger cars.

6. Vehicles are assigned at the approach lanes without consideration of their turning movements.

7. Queueing vehicles move into the flare only from the immediately adjoining approach lane.

8. Queueing vehicles can move either only forward or forward and sideways simultaneously. They move sideways as many lanes as rows they move forward.

9. When queueing vehicles move sideways through one or more rows they take the same time as when they move only forward through the same number of rows.

10. The available positions in the flare are filled only after the entering flow has stopped, i.e. when the entering flow is not inhibited by circulating vehicles the extra places of the flare are not utilised.

The program of SIMC consisted of a MASTER segment and a SUBROUTINE RANDOM which generates the pseudorandom fractions. The program was written in FORTRAN IV to be run on the ICL 1906S computer at the University of Sheffield.

The MASTER segment consisted of the following main parts, divided according to the function they performed:

1. Generation of circulating and entering vehicles.

2. Assignment of a position in the queue of entering vehicles.

3. Assignment of the earliest departure time.

4. Check of possibility of entry.

5. Entering or alternatively updating of the departure time.

6. Moving-up of vehicles remaining in the queue at the end of a gap.

7. Calculation of parameters of interest, 'figures of merit'.

The output consisted of the capacity, the average delay over the simulated period and the entries per lane.

5.5 The Development of SPHT

The model developed previously, SIMC, did not take into account turning movements. The implication of this assumption is that any generated vehicle could be assigned at any position of the entry, resulting in lower delays and, possibly, higher capacities that expected. Introducing realistic modelling of turning movement could extend the usefulness of the model in the region of flows when the junction operates under or near capacity. It is in that region that average delay is more likely to be affected by turning movements. Further, the inclusion of turning movements would allow differentiation of the delay suffered by each traffic stream. The previous version allowed only one overall average delay estimation. Such a value is likely to be exceeded significantly for vehicles performing specific turning movements.

In developing SPHT the following assumptions, upon which SIMC was based, were retained: 2 - 5, 8 and 9 (see section 5.4).

The following assumptions were made about turning movements:

1. The entering vehicles can turn right, left or follow a straight through direction. This would imply that the entry forms part of a roundabout with at least 4 arms.

2. Straight through vehicles can use both approach lanes, right turning only the offside, and left turning only the nearside lane.

3. At the flared part of the entry, straight through vehicles can use all lanes at the row before the stop line, but only the same lanes at the stop line, i.e. they can not use the nearside flared position. Right-turning vehicles can use only the offside lane. Left-turning vehicles can use only the extra lanes provided by the flare, i.e. they can use one lane at the row before the stop line and two lanes at the stop line.

4. Each vehicle has a preference in the order of lanes it can occupy at each row (if it is allowed to choose from more than one position). This preference order depends on what turning movement the vehicle is assigned. Right turning vehicles are not affected by this as they are allowed only at the offside lane. For straight through vehicles, it implies that when there is a high right-turning proportion they would use, mostly, the second lane along from the offside. Left-turning vehicles prefer to follow the nearside positions along the whole entry, both at the approach portion and at the flared part.

5. The distribution of circulating traffic was assumed to be shifted negative exponential.

Some further aspects of the model are mentioned below:

(a) The method of simulation was of the event scanning type. This resulted in more complex logic but permitted faster runs on the computer, especially for conditions of high circulating and demand flows.

(b) Three initial numbers for the pseudorandom fraction generating routine were used. This allowed the production of three completely different pseudorandom fractions sequences used to generate the circulating flow vehicles, the demand flow vehicles, and the assignment of turning movement to each demand flow vehicle.

(c) Each demand flow vehicle was assumed to have arrived when it was generated. The time of arrival could be the time of its entry into the circulating flow had there not been any delay due to congestion.

(d) The model does not take into account the delay of the vehicles remaining in the queue at the end of the simulation period. Thus, the average delay estimates refer to the delay of the vehicles which entered during the period of simulation.

(e) The simulated period, during which measurements were taken, was 3600 sec. The complete simulated period was 3900 sec, which allowed 300 sec of initial transient time used to develop the demand and entry queue.

(f) The nature of simulation does not allow the exact production of the requested traffic conditions. The circulating flow generated differs from the one input into the model. Similarly the input proportion of turning movements is different from the proportions as simulated. This aspect of the simulation is developed further in the following Chapter. It should be mentioned that validation of the modelled effect of the turning proportions on roundabout performance is very difficult as such observations in public road sides are almost impossible.

(g) The program was written in FORTRAN IV - 1966 to be run on the PRIME-750 A computer of the University of Sheffield.

The final version of the model, the simulation of 3900 sec of real time would take 10 to 40 sec of CPU time depending on the volume of the circulating and demand flow and on the turning proportions.

5.6 Description of SPHT

A simplified flow chart of the simulation program is shown in Fig. 5.2. SPHT consists of a MAIN segment and subroutine RANDOM.

The MAIN segment can be divided in the following parts according to their function:

(a) Generation of circulating vehicles. The individual and cumulative headways of the circulating vehicles are calculated.

(b) Generation of entry vehicles and turning movements. The turning movement of the vehicle is established using the same random generating algorithm as for generating the circulating and entering vehicles. The individual and cumulative entering headways are calculated.

(c) Assignment of position and departure time. The turning movement of the entering vehicle determines the position of the vehicle in the queue. The position associated with the earliest departure time is the one preferred by the entering vehicle.

(d) Entry check. The departure times are compared with the arrival time of the next circulating vehicle and the number, if any, of possible entries is noted.

(e) Updating of departure times. Any queueing vehicle which refuses an offered gap has its departure time updated to account for the incurred delay.

(f) Moving-up sequence. When the junction operates at or above capacity, most gaps that are accepted by some queueing vehicles are not of such length to allow the complete discharge of the queue. In cases when vehicles remain in the queue this sequence moves them forward and re-assigns them in new positions depending on their turning movement.

(g) Calculations. These are performed in two positions. After each vehicle enters into the circulating flow, its individual delay and other relevant characteristics are computed. The overall figures are calculated after the simulation period has ended.

Subroutine RANDOM generates the pseudorandom fractions required to generate the circulating and entering vehicles as well as the turning movement of the entering vehicles. Each sequence is initiated by a different initial value, therefore each one is different and can be varied independently by the other two.

5.7 Input and Output of SPHT

The input consists of the following parameters:

Q1: the circulating flow (veh/hr);

Q2: the demand flow (veh/hr);

TAU: the minimum circulating headway (sec);

ALPHA: the critical gap (sec);

BETA: the move-up time (sec);

I9, J9, K9: initial numbers for the pseudorandom fraction sequences;

NS: duration of simulation (sec);

AN: the number of position of the first row;

AP: the number of positions per row before the flare;

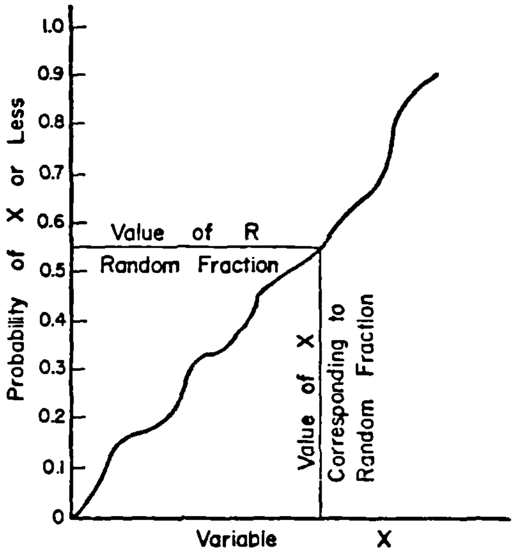
AL: the number of rows over which the flare is developed;

OTL: a parameter, in seconds, which determines whether and at what simulated time detailed output should be produced; detailed output was used to check the working of the program;

PR, PS, PL: the proportions of the turning movements;

LNM: a 3-dimensional array describing the preferred positions per row of each turning movement.

The output from the program could be detailed when any modifications were carried out. This allowed thorough checking of the performance after each modification. The final calculations include the average delay per lane, per turning movement and overall, the capacity of the entry, the number of entries per lane and turning movement, and the number of circulating vehicles.



Generalized cumulative probability distribution.

Figure 5.1

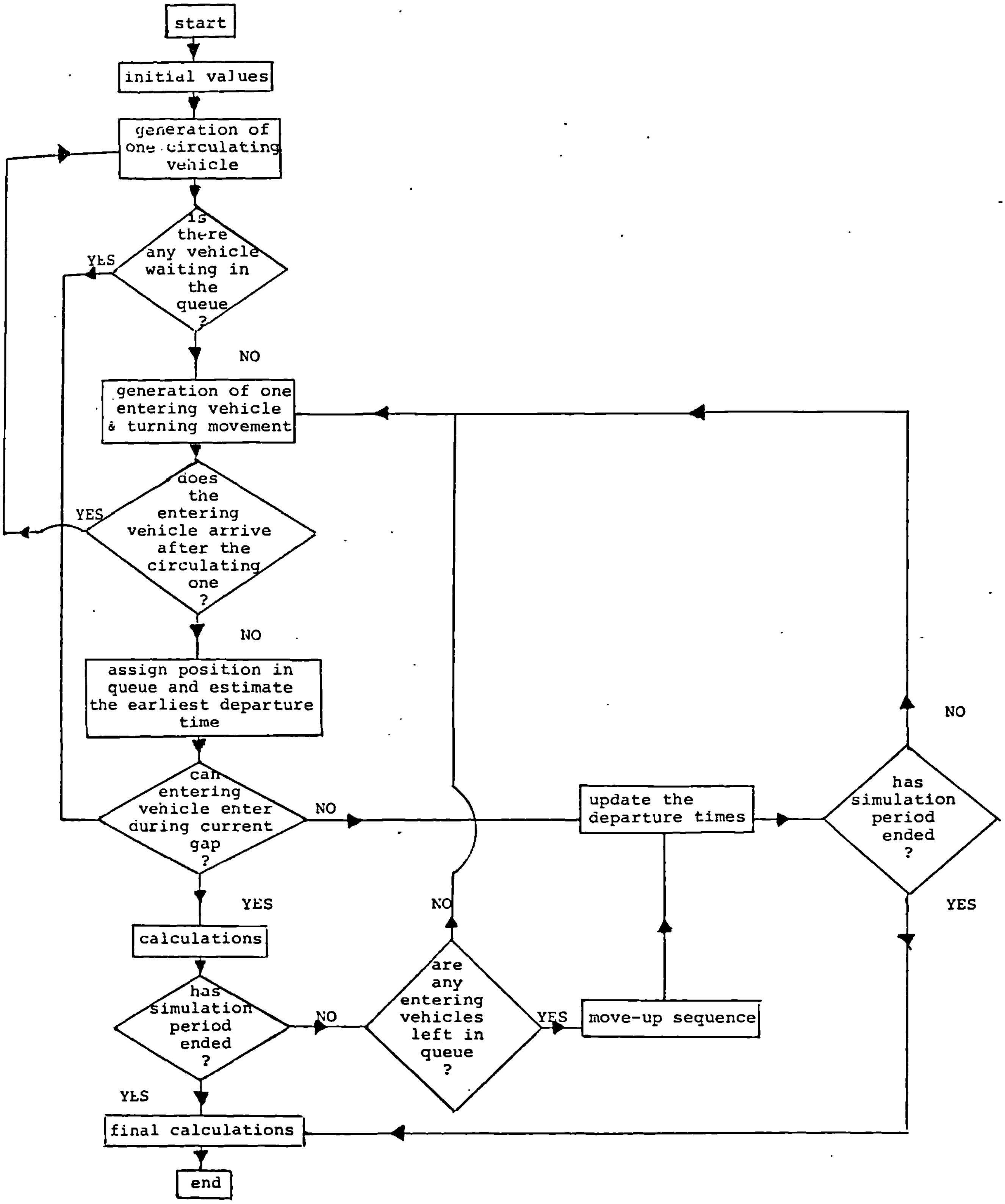


Figure 5.2 Flow chart of SPHT

CHAPTER 6

RESULTS AND COMMENTS

6.1 Introduction

The simulation model SPHT was used to establish the effect of turning proportions on roundabout capacity and delay. It was used to establish the relationship between delay and the flow and gap-acceptance parameters. The performance of flared and straight entries was compared throughout.

The chapter also covers the validation of the model using the observed capacity figures and the relationship between input and simulated circulating flows.

6.2 Validation of the Model

The capacity estimates produced by the computer model developed were compared with observed values of capacity at three entries, to establish how realistically the model behaved. The values of circulating and entering flows abstracted from the videotaped data were used as the observed values. As was explained in Chapters 3 and 4, the observed sites were operating in a way that did not allow the abstraction of a figure of capacity for the whole of the entry. Instead, it was possible to estimate only the capacity of specific lanes. Therefore, the comparison would be valid only if the model simulated the operation of individual lanes. The model has the ability of both simulating a single-lane entry and, also, of providing estimates of the use of each lane. The second case involves the simulation of a flared entry at such conditions that the entry demand flow is greater than the capacity of the entry as a whole. This ensures that the capacity of each lane not directly affected by the flare is equivalent to the capacity of a single-lane entry.

The data collected at the three public road sites in Sheffield provided values for the circulating and entry flows, the gap-acceptance parameters and the minimum circulating headway for each site, (see Chapters 3 and 4). They were used as input for the simulation except for the entering flow. A sufficiently large demand flow was input to ensure continuous queueing and capacity operation of the junction. Therefore, the simulated conditions resembled the observed as close as the model allowed.

Table 6.1 and figure 6.1 demonstrate the agreeement between the observed and simulated values. The largest percentage difference is 15.2% while the average percentage difference (ignoring signs) is 7.4% and the standard deviation of the percentage difference is 5.1%.

A further point of interest is the relationship between the input value for the circulating flow and the ones actually simulated. The agreement is demonstrated in Table 6.2 and figure 6.2.

6.3 Roundabout Performance

The simulation model SP HT was used to model an entry which had two lanes at the approach section and four at the stop line. At times it was modified to allow the simulation of a straight entry having two lanes throughout. The modifications involved changes in the input values and in a DATA statement in the program itself.

The simulation provided estimates of average delays to queueing vehicles and entry flows for a wide range of flow and gap-acceptance parameters. The results were used to study the effect on capacity, entry flow and delay of the gap-acceptance parameters, the turning proportions and the circulating flow. Further, the performance of flared and straight entries were compared. The following sections describe the above in detail.

6.3.1 The Effective Number of Lanes

A measure of the increase in capacity due to flaring that has been proposed (Ashworth & Laurence, 1977; Laurence & Ashworth, 1979) is the effective number of lanes, N_e. If a flared entry has N lanes at the stop line, N_e is defined as the number of non-flared lanes that could have the same capacity as the flared layout. They tentatively suggested that there is a linear relationship between N_e and N:

$$N_e = 0.33N + 1.3 \qquad \text{(eq 6.1)}$$

The simulation model was used to predict the capacity of flared and straight entries which, subsequently, were compared to establish the effective increase in capacity. The comparison was performed over the following ranges of values:

circulating flow (Q_1) = 0.0 - 4000 veh/hr in 500 veh/hr steps,

critical gap (α) = 2.00 - 3.50 sec in 0.50 sec steps,

move-up time (β) = 1.50 - 3.00 sec in 0.50 sec steps.

Throughout it was assumed that $\alpha \geq \beta$.

According to equation 6.1, a flared entry with N = 4 has an N_e = 2.62. The formula does not account for any other parameters. It was found that as capacity is a function of the circulating flow so is the effective number of lanes. Values of N_e were calculated for all combinations of the above range. Figure 6.3 is a plot of all the points obtained together with an envelope within which all such points lie. The common elements of behaviour are that:

(1) For all combinations of the gap-acceptance parameters, and $Q_1 = 0.0$ veh/hr the value of N_e is equal to 2, i.e. the flare is not contributing any extra capacity than a two-lane straight entry.

(2) As Q_1 increases N_e also increases but at differing rates for the various gap-acceptance parameter combinations. The value of $N_e = 3.00$ (i.e. 50% increase in capacity) was achieved by all such combinations for $Q_1 = 2300$ veh/hr approximately, while at $Q_1 = 4000$ veh/hr only the combinations $\alpha = 3.50$ sec, $\beta = 2.50$ sec and $\alpha = 3.50$ sec, $\beta = 3.00$ sec had achieved $N_e = 3.99$ (i.e. almost 100% increase in capacity). The value $N_e = 2.62$ (suggested by Ashworth & Laurence) was achieved by all combinations at $Q_1 = 1525$ veh/hr approximately. See figure 6.3a for comparison with observed values of N_e reported by previous research.

(3) At each Q_1 value, the range of N_e values over all the gap-acceptance parameter combinations differed, the largest range being at $Q_1 = 2000$ veh/hr. The maximum N_e at that value, was 3.52 while the minimum was 2.83, i.e. a difference of 0.69 lanes. At $Q_1 = 0.0$ veh/hr there was no difference, while at $Q_1 = 4000.0$ veh/hr the range was 0.31 lanes.

(4) The effect of the gap-acceptance parameters

on the above range is shown on figures 6.4 and 6.5. Both figures relate to the effective lanes obtained for $Q_1 = 2000$ veh/hr. Figure 6.4 is a plot of N_e vs α while figure 6.5 of N_e vs β. It can be seen that N_e is sensitive to the value of α, also, to the β values for $\alpha = 2.00$ sec but not for $\alpha \geq 2.50$ sec. As can be seen from figure 6.4 N_e is directly related to α, implying that the maximum effective lane number is achieved when drivers have high critical gap values.

The conclusions reached from the above is that flared entries are more beneficial and, therefore, justified for conditions of medium to high circulating flows with driver populations exhibiting slow gap-acceptance behaviour. Before-and-after studies at roundabouts being converted to flared layouts have so far reported only on the change in capacity. It would be of interest to examine whether such changes are accompanied, also, by changes in gap-acceptance parameters. It is conceivable that at the new layouts, due to the capacity increase and hence due to the reduction of pressure on individual drivers, their gap-acceptance parameters would increase in value. This would result in a smaller increase in capacity than would have been expected.

6.3.2 Entry Flow and Turning Proportions

The simulation was used to establish the effect of turning proportions of the entering vehicles on the entry flow. The term entry flow is used instead of capacity to indicate that this section included the study of roundabout operation under such conditions that the entry was not saturated.

The turning proportion was assumed to consist of at least 50% straight ahead traffic, while the other two directions could each achieve a maximum of 50%. The set of proportions used was the following

LT(%)	ST(%)	RT(%)
0	100	0
10	90	0
5	90	5
0	90	10
25	75	0
20	75	5
15	75	10
10	75	15
0	75	25
40	60	0
30	60	10
20	60	20
10	60	30
0	60	40
50	50	0
40	50	10
25	50	25
10	50	40
0	50	50

The above sets were considered that they covered satisfactorily the range of values that could be expected in practice. The above turning proportions were used with two sets of gap-acceptance parameters, which used the values of two of the entry data sets recorded at the Sheffield sites and analysed by the method described in Chapter 4. The values used were

(1) $\alpha = 3.75$ sec, $\beta = 2.60$ sec, lane 1 of Moore Street Roundabout,

(2) $\alpha = 2.80$ sec, $\beta = 1.68$ sec, lane 1 of Castle Square Roundabout.

The values of the circulating flow, Q_1, and of the entry demand flow, Q_2, were:

Q_1: 500, 1000, 2000, 3000 veh/hr

Q_2: 500, 1000, 1500, 2000 veh/hr.

Thus the effect of the turning proportions was examined using two sets of gap-acceptance parameters which represented the higher and lower possibilities of observed values. The above analysis was carried out for flared and straight entries.

Figures 6.6 to 6.9 show the results of the above analysis for four selected sets of the possible combinations for both the flared and straight entries. The results of the remaining sets were similar, hence this selection was considered adequate. The first and second sets represent operation of below saturation flow levels (figures 6.6 and 6.7), while the third and fourth sets represent operation with the entry saturated, (figures 6.8 and 6.9). The conditions of figures 6.6 and 6.7 allowed virtually the whole of the demand flow generated to enter. In such below capacity cases it can be seen that variation in turning proportion is not affecting the entering flow for both flared and straight entries.

At-capacity operation differs between the two types of layout. Turning proportion does not affect capacity of straight entries, while for flared the capacity is directly related to the left-turning proportion, (figure 6.8a and figure 6.9a). The explanation to this lies on the use of the lanes. At flared layouts it was assumed, and supported by observations, that the extra lanes near the stop line are used by left-turners, (see Chapters 3 and 5). At 0% left turning proportion, the nearside lane which is exclusively used by left-turning vehicles, is not used at all. Therefore the entry becomes equivalent to a three-lane entry. The gradual decrease in capacity with a reduction in the left-turning proportion is also associated with the difficulty such vehicles have in reaching the flared area, since they share the nearside approach lane with straight-through vehicles.

The entering flows for the flared approach, of figure 6.6, have a difference of the extreme values of 9 veh/hr (2.0%), whilst for the straight entry, it is 5 veh/hr, (1.1%). Similarly for figure 6.7 the flared approach difference was 7 veh/hr (0.7%) and for straight 20 veh/hr (2.1%). As can be seen the variation is small, further there is no discernable pattern in the variation over the left-turning percentage.

The flows of the straight approach for figures 6.8b and 6.9b present similar values, respectively 20 veh/hr (3.9%) and 6 veh/hr (3.7%). However, the flared approach represents a much wider range. The difference between the extreme values is 196 and 79 veh/hr respectively (24.4% and 30.7%). The effective numbers of lanes of the maximum and minimum values respectively are 3.65 and 3.05 for figure 6.8, while for figure 6.9 they are 3.91 and 3.11.

6.3.3 Delay and Turning Proportion

The simulation model estimates values for the average delays incurred by vehicles of the entering flow while queueing to join the circulating flow. The simulation runs over which delay was estimated lasted one hour. Over this period the values of the circulating and entry demand flows were assumed constant. Therefore, the delays estimated by the model increased rapidly as the entry became saturated and entry capacity was exceeded. If the simulation period was increased the delays would increase indefinitely. From this point of view, the model has the same disadvantages as equilibrium state prediction formulae, and which are overcome by time-dependent methods, (see Chapter 2). Thus, the model is only suitable for studying delays suffered at conditions below and around capacity. The effect of turning proportion, flow and gap-acceptance parameters on delay has been studied for both flared and straight approaches. The relationship between delay and turning proportion is dealt with in this section, the next section covers the relationship between delay and the flow and gap-acceptance parameters. It should be noted that the results presented here are a selection of the values produced.

The data relevant to this section are presented in figures 6.10 - 6.25. The first eight, figures 6.10 - 6.17, present the data for the flared layout only, while figures 6.18 - 6.25 repeat the above data together with the data for the straight entry allowing direct comparisons to be made.

From figures 6.10 - 6.17 an overall pattern of the variation of delay over the range of percent of left-turn used is emerging. Maximum delay is obtained for the combination: left-turn = 0%/straight = 50%/right-turn = 50%. For each straight proportion, delay is a maximum when left-turn = 0%. Delay decreases as the left-turn proportion increases; however, the graphs for the smaller straight proportions exhibit a minimum average delay with 30 - 40% left-turn proportion; while for 50% straight proportion, there is an increase in average delay as the left-turn proportion approaches its 50% maximum value.

The maximum delay at 0% left-turn is caused by the effective reduction of the entry from four to three lanes. The rise of delay at maximum left-turn is associated with the smallest straight through values used. At such conditions the effective number of lanes is reduced again as the high number of left-turners prevent the full use of certain positions at the stop line.

Comparing the performance of flared and straight approaches (figures 6.18 - 6.25) the percentage increase in delay with straight entries has been determined for all data points. They are included in tables at the Appendix. From the eight cases presented, seven produce differences which on average are above 60%, i.e. the straight entries have on average delays exceeding the ones of flared entries by 60% or more. The averages are over all the turning proportion for each set of flow and gap-acceptance parameters. The only exception to the above relationship is the delays associated with the following parameters: Q_1 = 500 veh/hr, Q_2 = 500 veh/hr, α = 2.80 sec, β = 1.68 sec. In this case, although the average difference was only 10.6% the straight entry delays were consistently higher than the flared entry ones.

The conclusion of this study is that conversion of straight to flared entries is associated with delay reductions of 40% or more in most cases for operation below and around capacity. Ashworth & Mattar (1974) studied delays at Brook Hill roundabout before and after flaring. They reported the following delay savings: a.m. peak = 34.5%, off-peak = 28.6% and p.m. peak = 22.6%. It should be noted that these figures refer to delay savings for traffic at all five approaches of the roundabout (shown in figure 3.4).

6.3.4 Delay and Flow and Gap-Acceptance Parameters

The analysis to determine the effects of the above parameters was based on estimates produced by the simulation model assuming a constant set of turning proportions (left-turn = 20%/straight = 60%/right-turn = 20%), while varying the other parameters over the following range

Q_1 = 500, 1000, 2000 and 3000 veh/hr,

Q_2 = 500, 1000, 1500 and 2000 veh/hr,

α = 2.00, 2.50, 3.00 and 3.50 sec,

β = 1.50, 2.00, 2.50 and 3.00 sec.

The figures that are included relevant to this section are only a small selection demonstrating the points described below. They are typical of the results not included.

For all combinations delay increases at fast rates as the entry approaches and exceeds capacity. Figures 6.26 - 6.29 demonstrate the relationship between delay, the circulating flow and the demand flow. Figures 6.30 - 6.33 show the relationship between delay and the critical gap, while figures 6.34 - 6.37 show the relationship between the delay and move-up time.

Figures 6.26 - 6.29 indicate that the delay increases only slowly with the circulating flow as long as the entry is not operating at or near capacity; also at below capacity operation average delay is not greatly affected by the volume of the entry demand flow. Similarly, the insensitivity of delay to the move-up time parameter at below-capacity operation is indicated by figures 6.30 - 6.37.

Site	Lane	Observed Capacity (veh/hr)	Simulated Capacity (veh/hr)	Circulating Flow (veh/hr)	Gap-Acceptance Parameters	
					α	β
Moore St Roundabout	1	413	405	2596	2.80	1.68
	3	380	390		2.82	1.77
Castle Sq Roundabout	1	518	551	1059	3.75	2.60
	2	590	657		3.22	2.59
Park Sq Roundabout	1	453	438	1980	3.50	1.89
	2	435	369		3.42	2.18
	3	495	438		3.10	2.17

Table 6.1 Agreement between observed and simulated capacity of entry lanes

Input Q_1	Simulated Q_1 Random Number Generator Initial Number		
	11113	3	13
500	476	499	499
1000	996	1004	965
1500		1506	1450
2000	2012	2000	1958
2500		2506	2460
3000	3029	2993	2979
3500		3491	3518
4000		4010	4011

Table 6.2 Agreement between input and simulated circulating flow

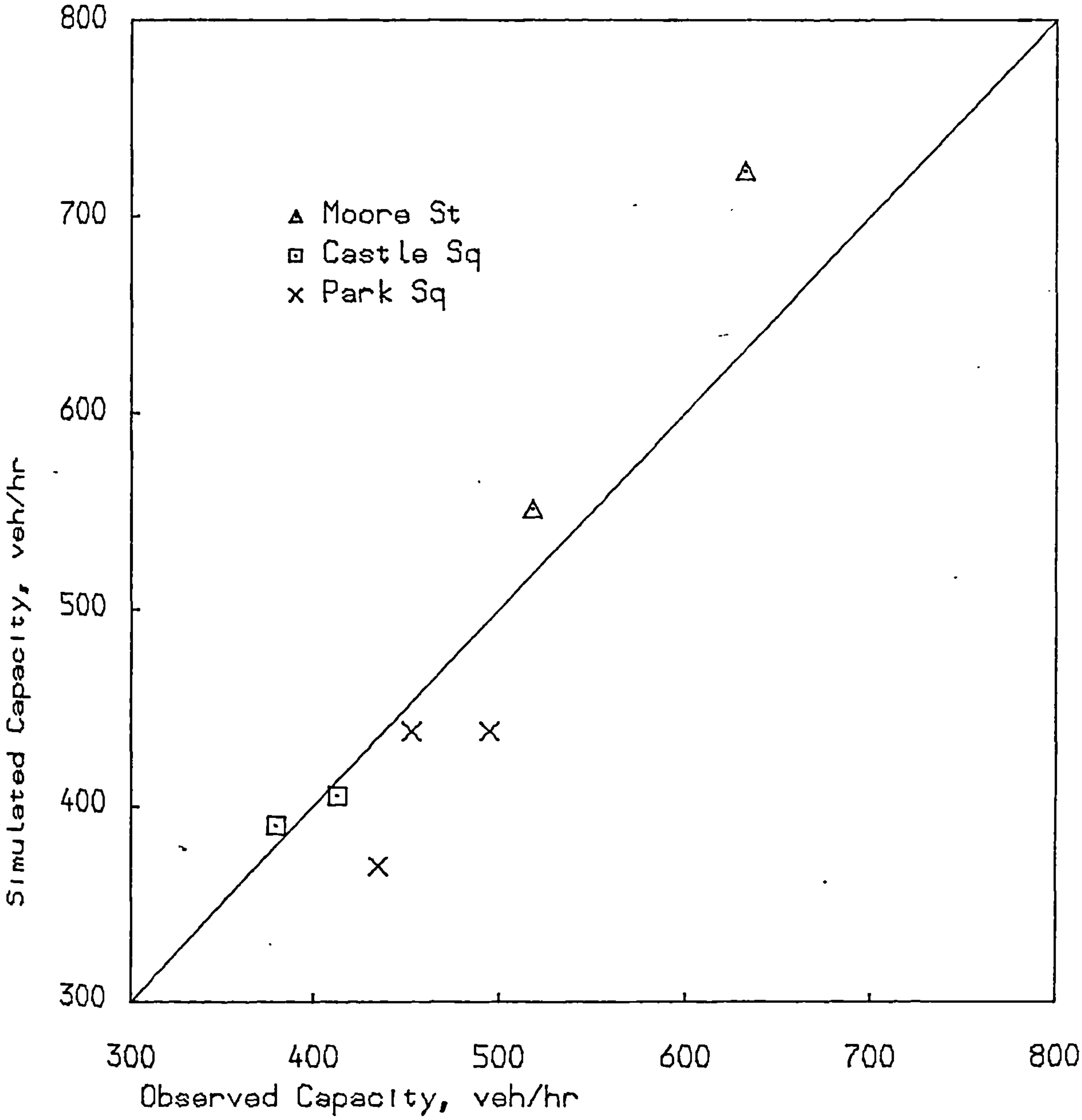


Figure 6.1 *Comparison of Observed and Simulated Capacities*

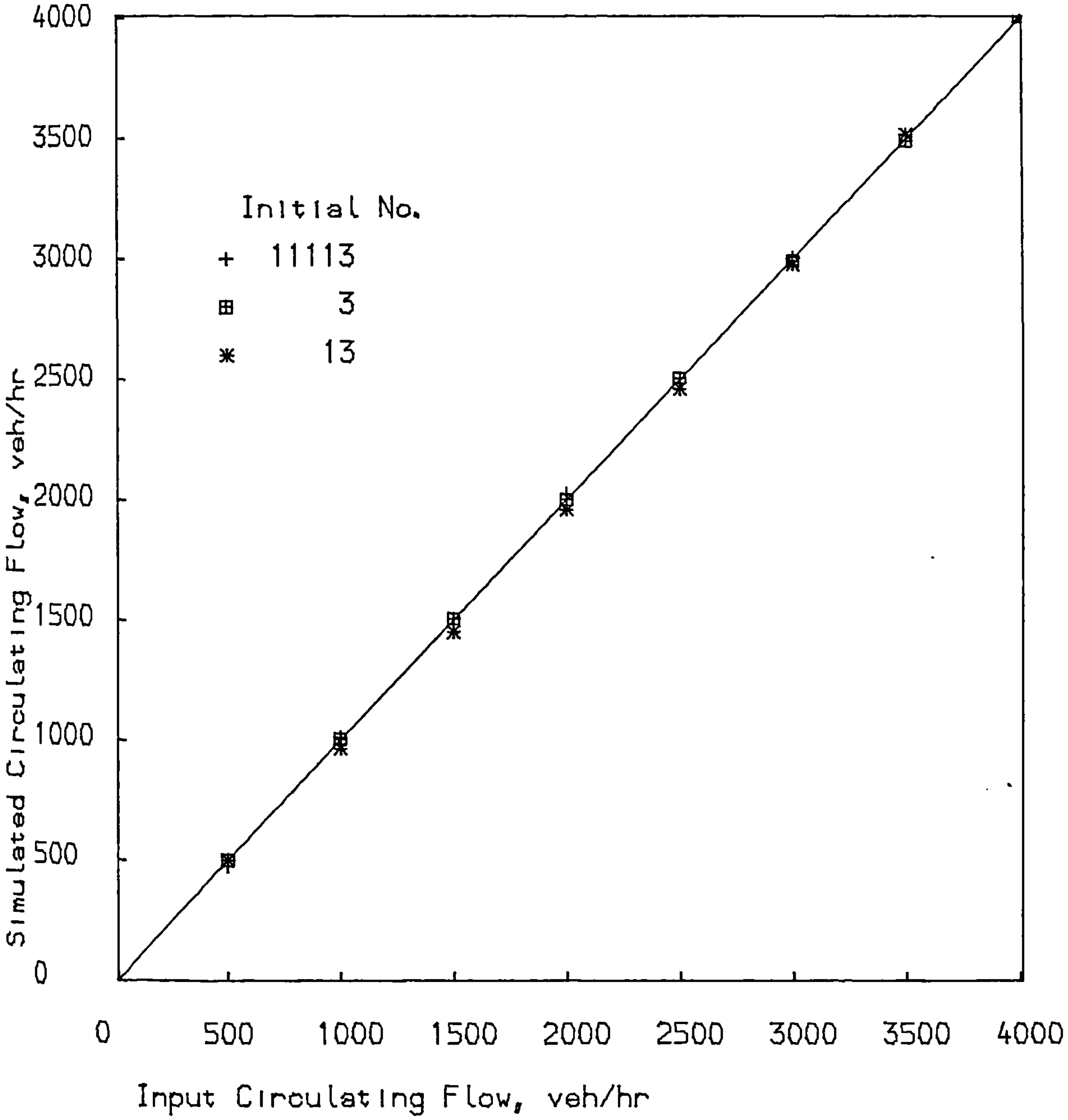


Figure 6.2 *Comparison of Input and Simulated Q1 Values*

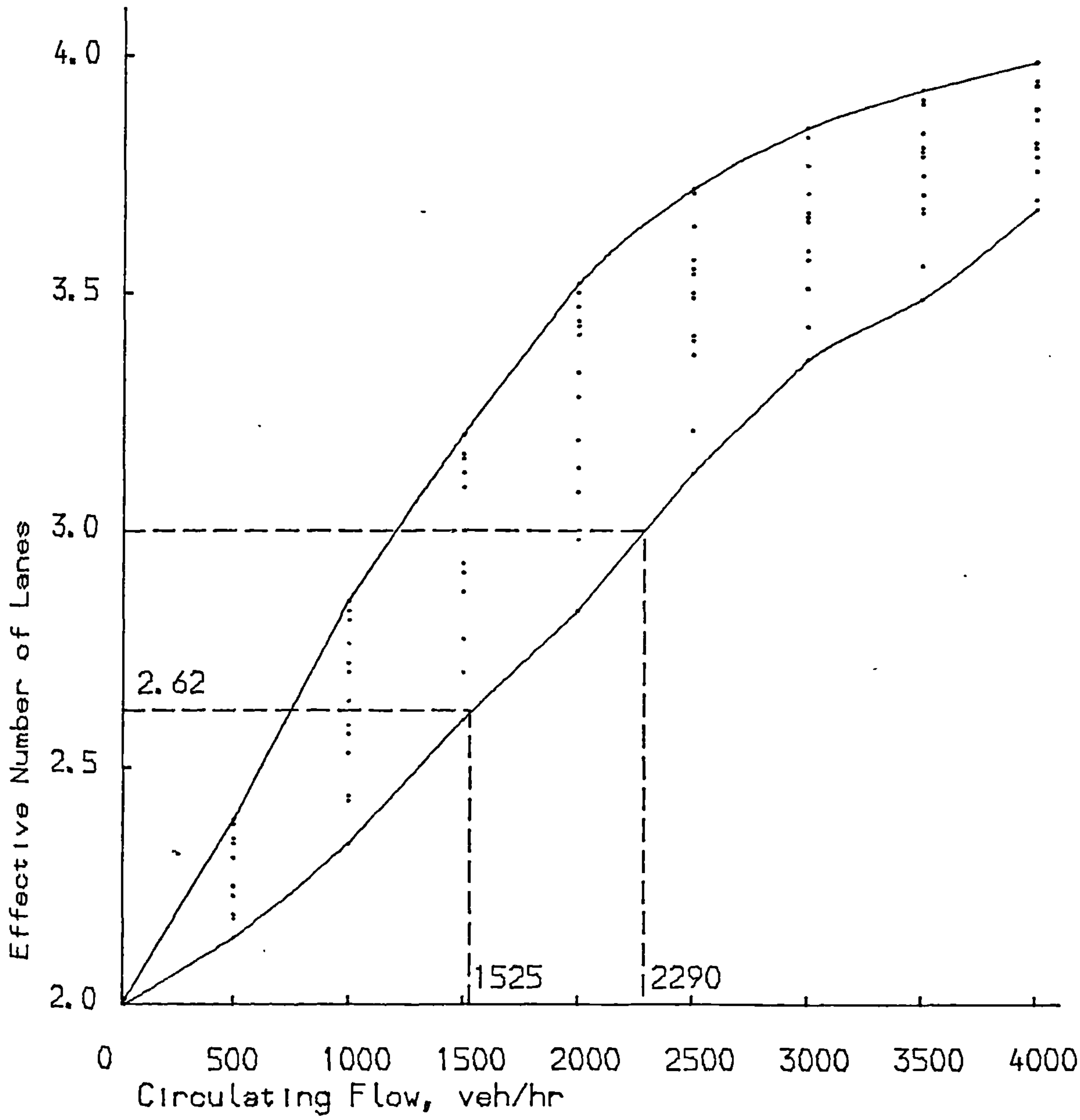


Figure 6.3 *Effective Number of Lanes and Circulating Flow*

1. Ashworth and Laurence (1977): The value of circulating flow is the mean of the values observed before and after flaring
2. Fang (1976): One effective number of lanes was arrived per site, corresponding to a range of circulating flows, i.e. it was assumed that the circulating flow did not influence the effective number.

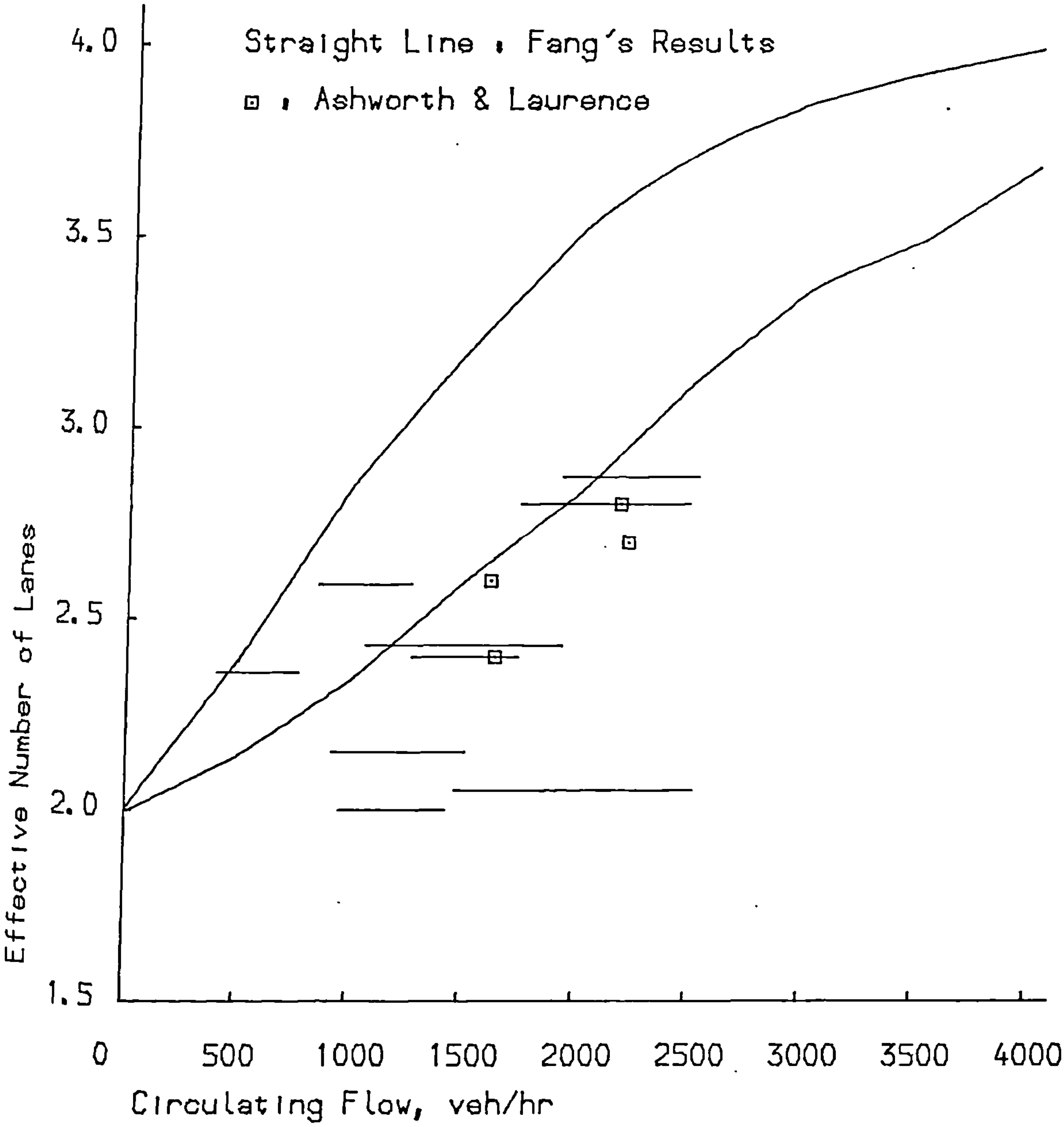


Figure 6.3a *Effective Number of Lanes and Circulating Flow*

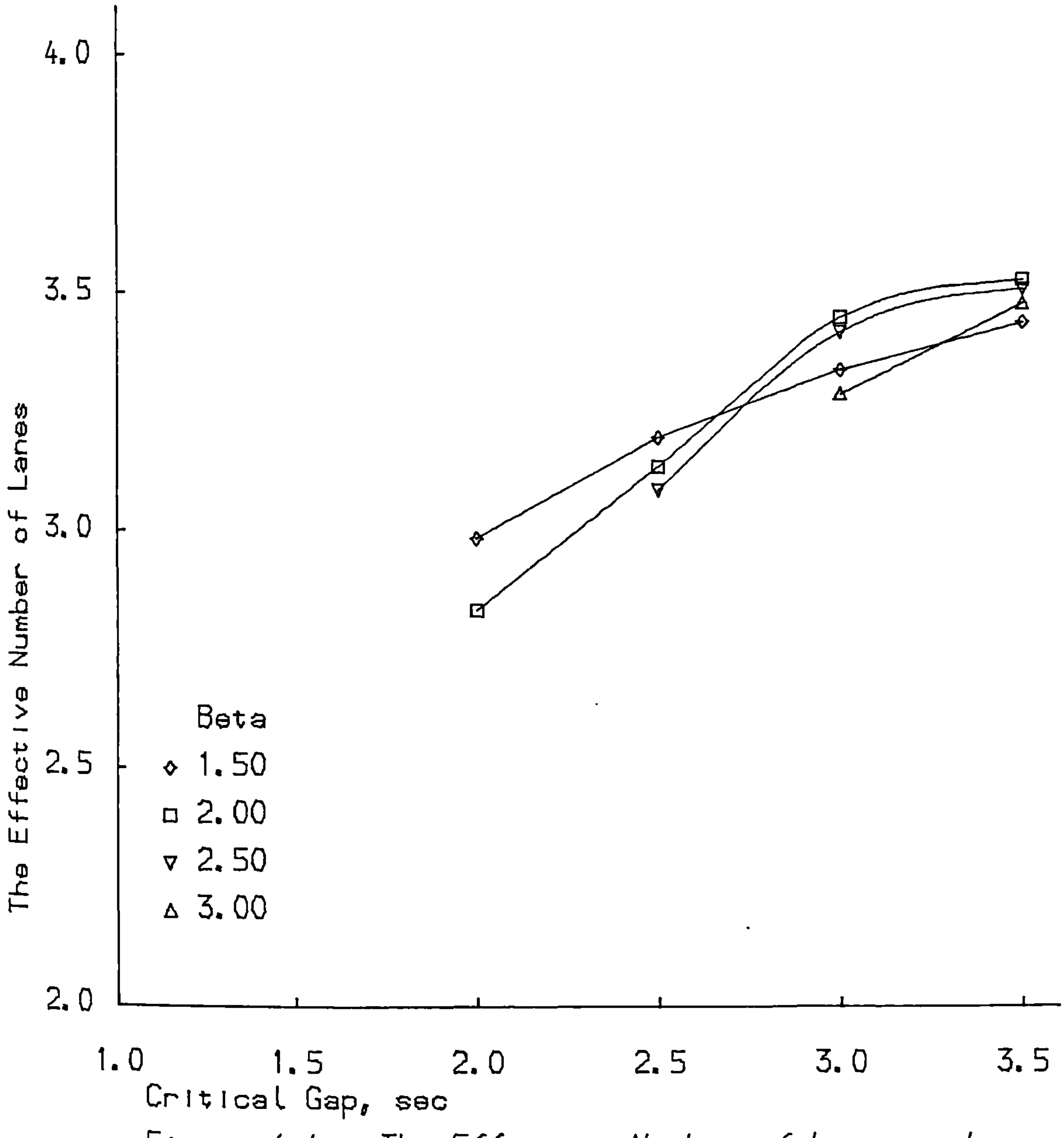


Figure 6.4 *The Effective Number of Lanes and α for Q1 = 2000 veh/hr*

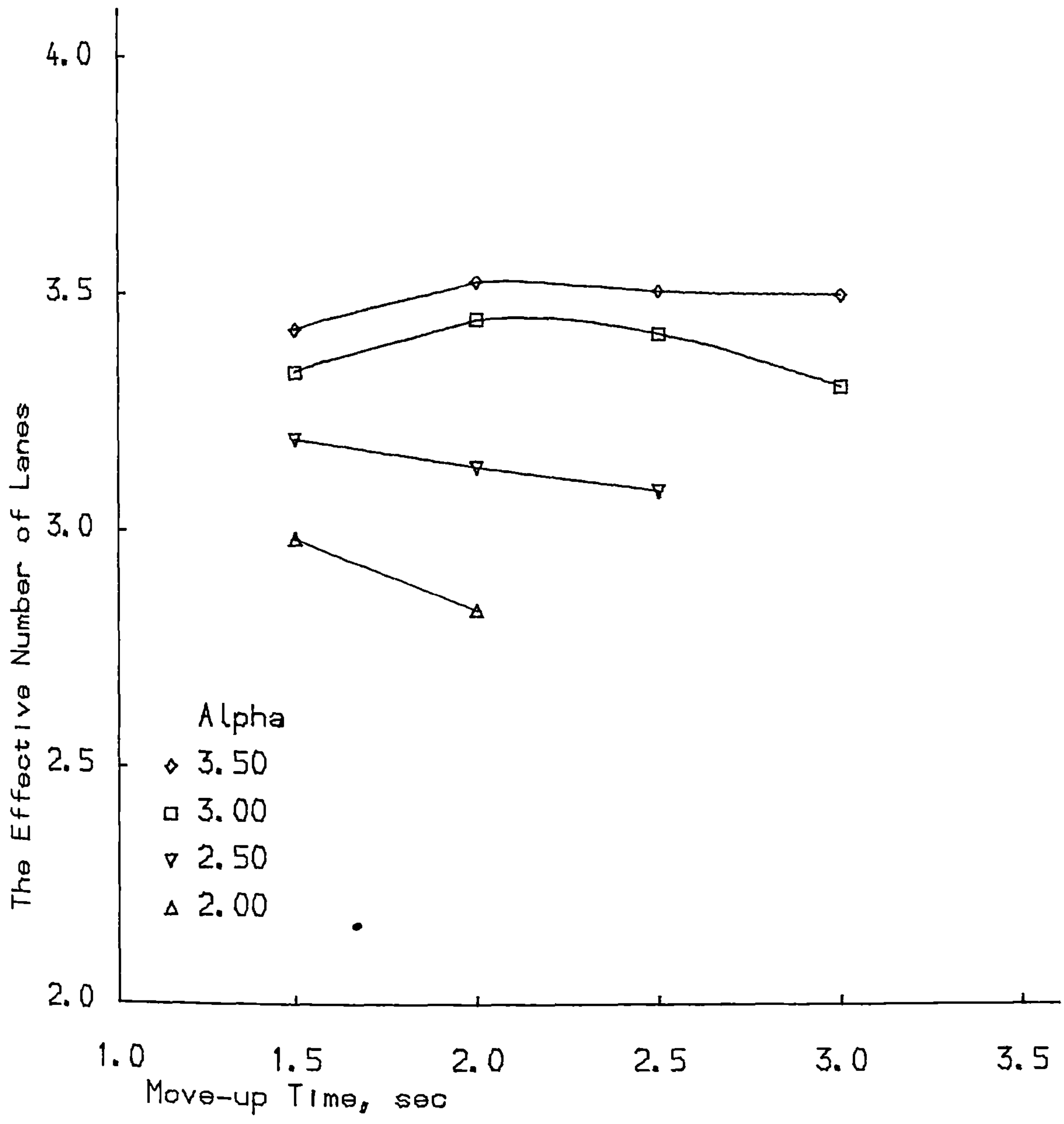


Figure 6.5 *The Effective Number of Lanes and β for Q1 = 2000 veh/hr*

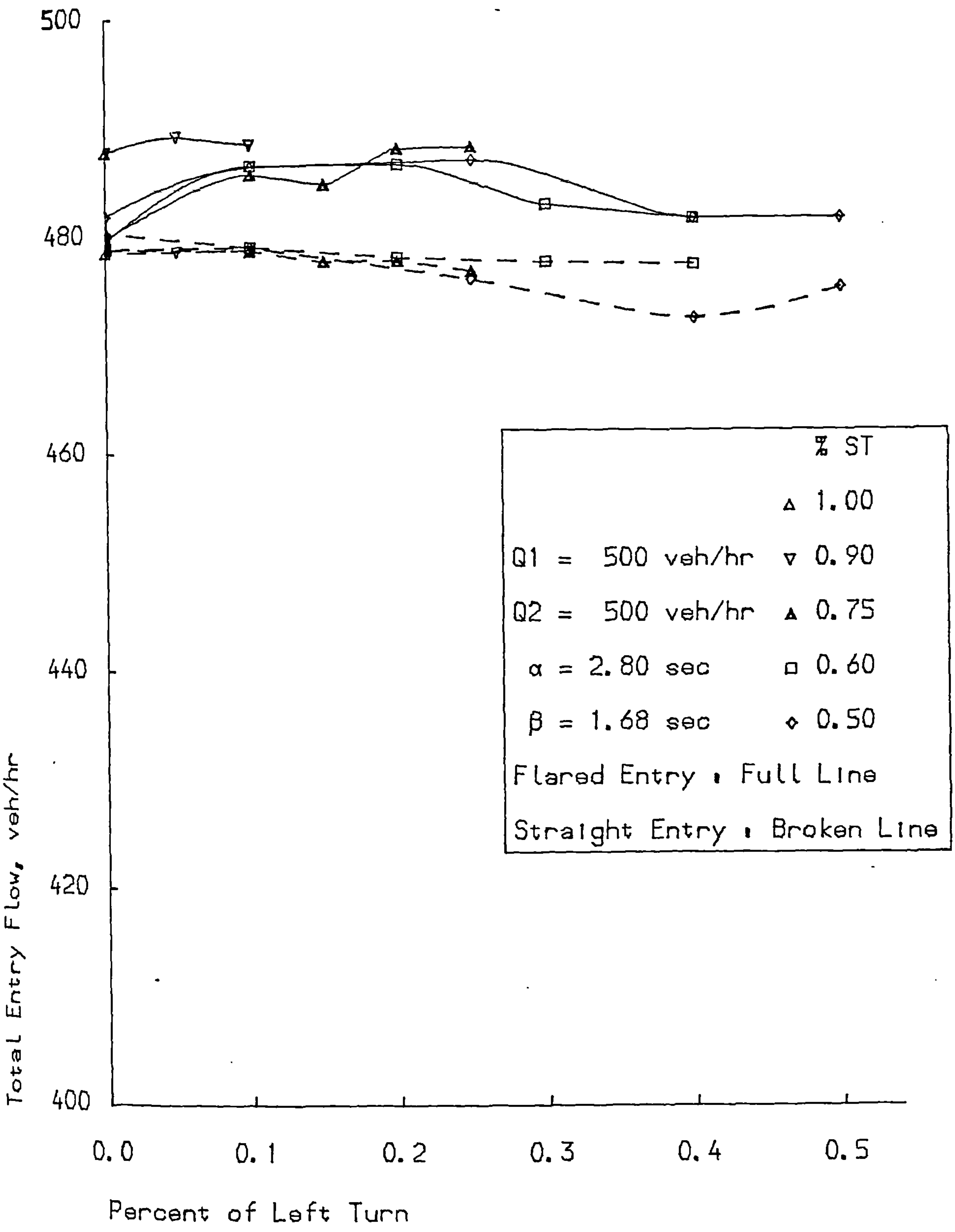


Figure 6.6 *Entry Flow and Turning Proportion Comparison of Flared and Straight Entries*

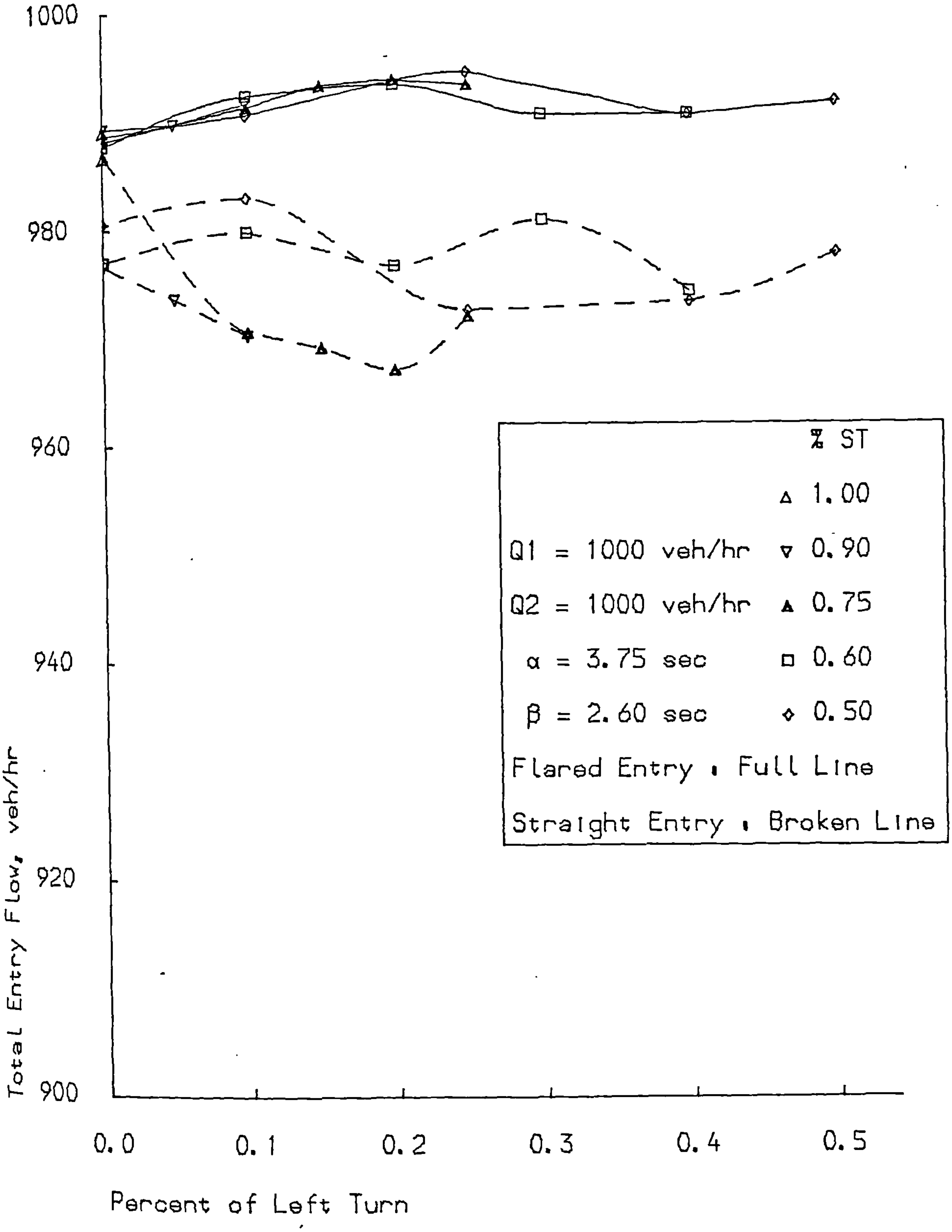


Figure 6.7 *Entry Flow and Turning Proportion Comparison of Flared and Straight Entries*

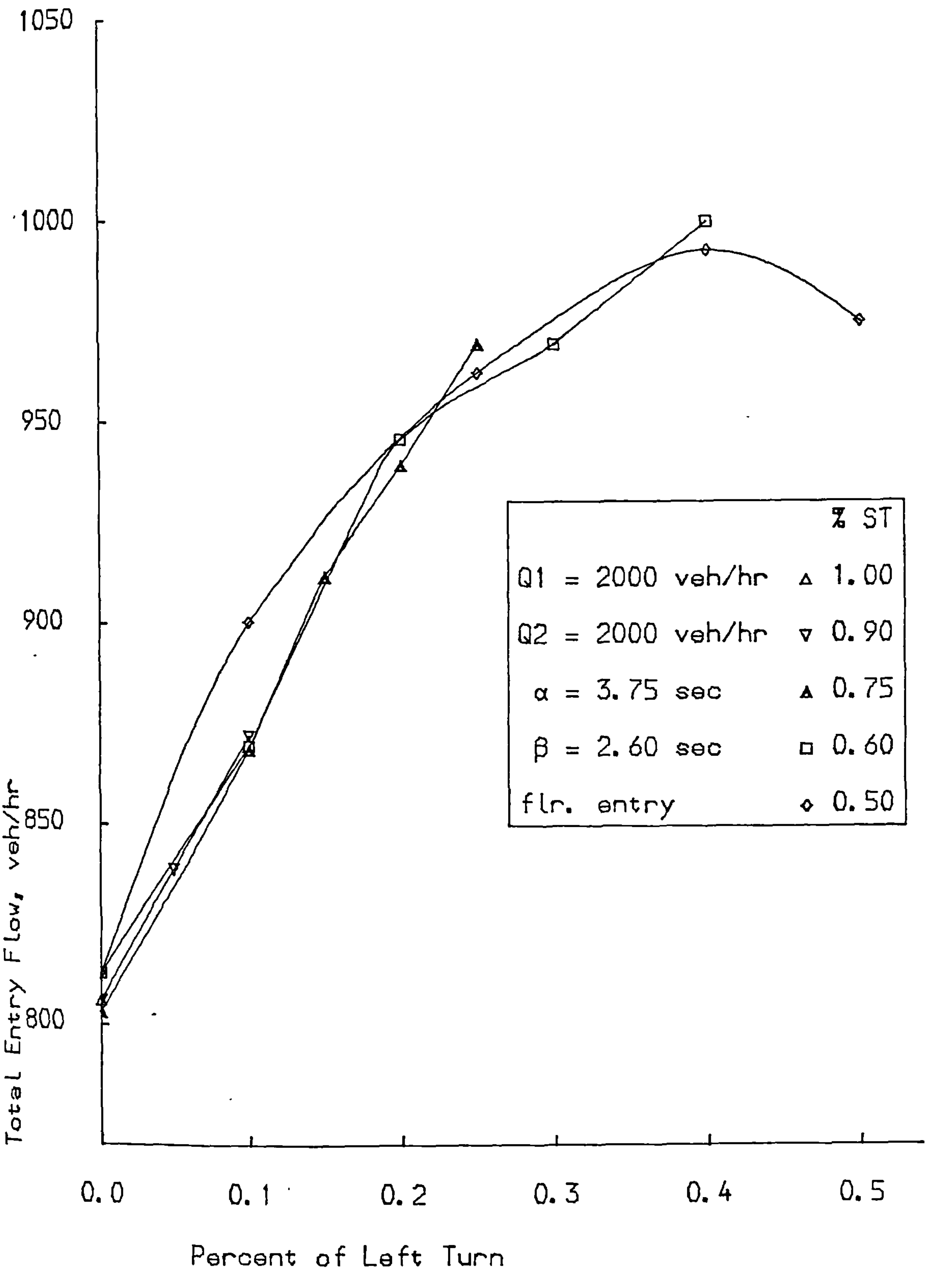


Figure 6.8a *Entry Flow and Turning Proportion*

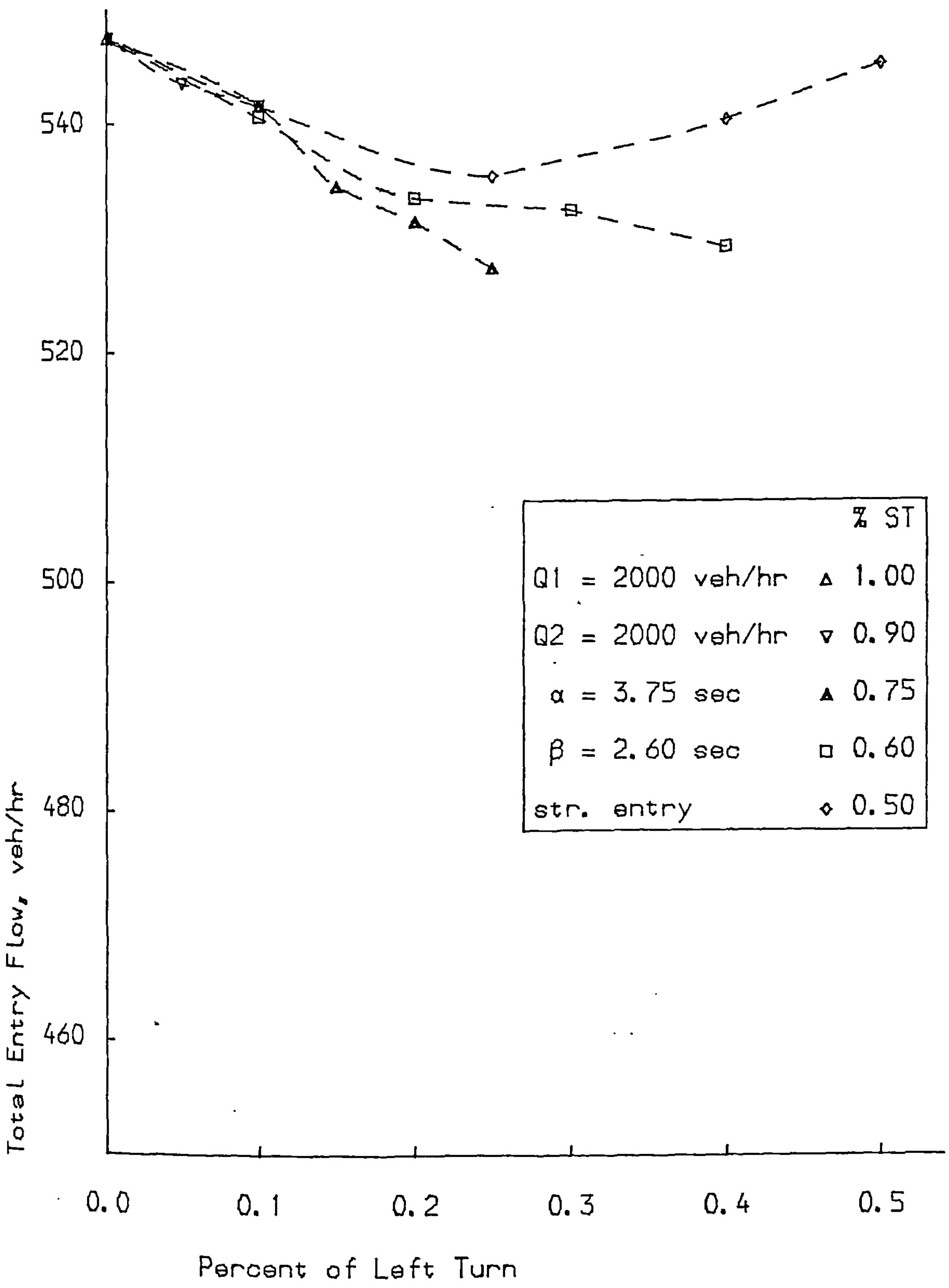


Figure 6.8b *Entry Flow and Turning Proportion*

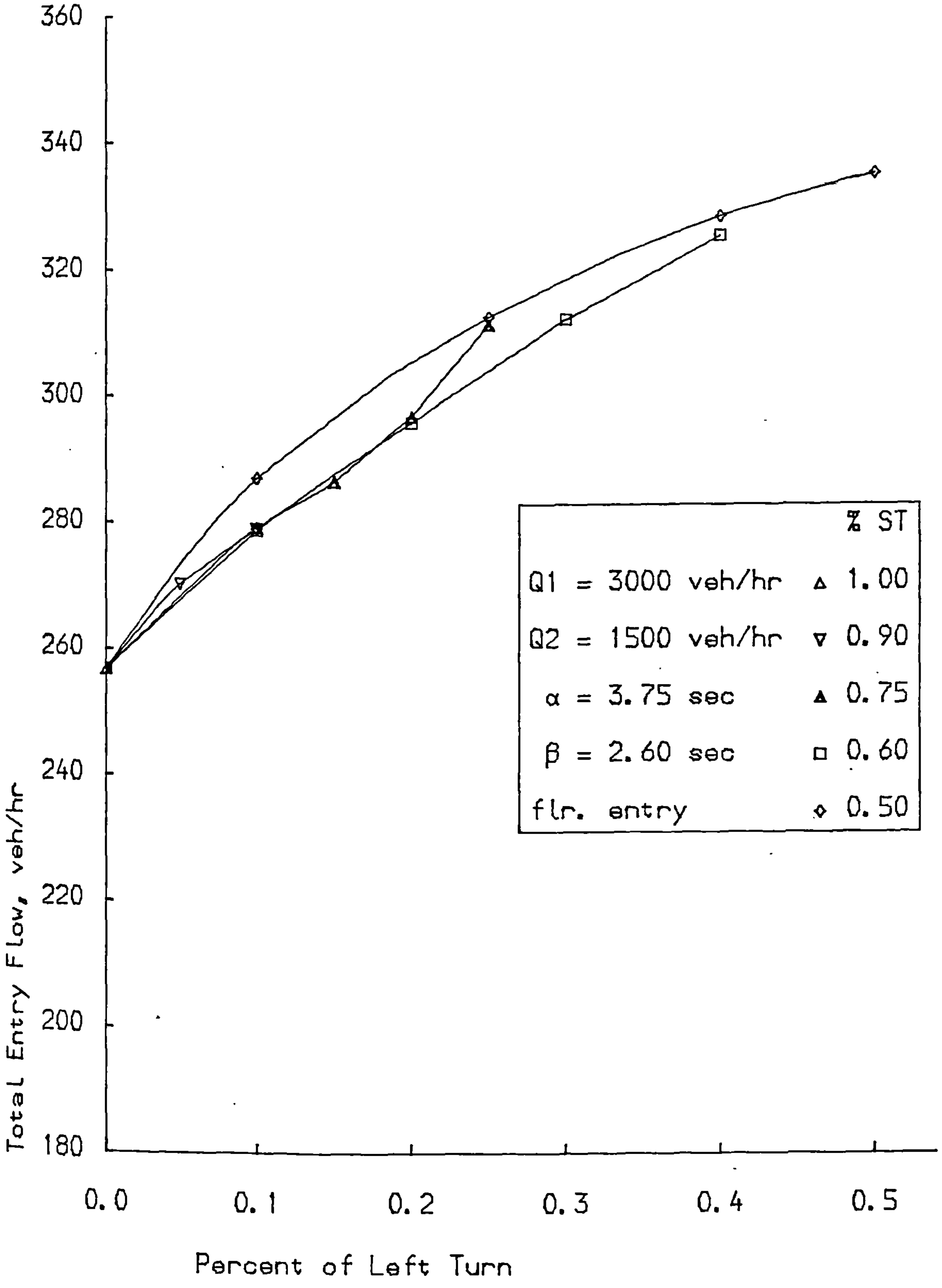


Figure 6.9a *Entry Flow and Turning Proportion*

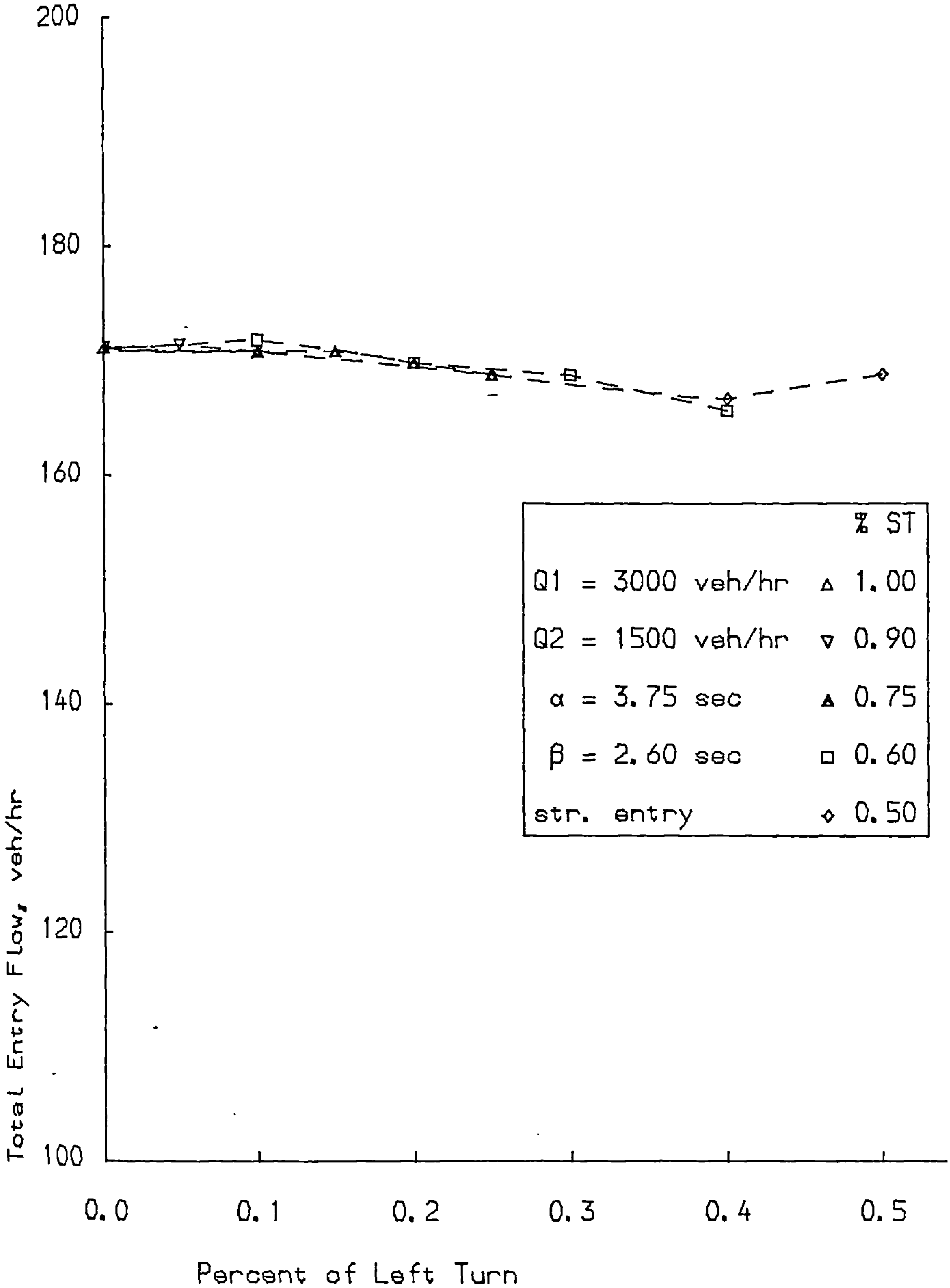


Figure 6.9b *Entry Flow and Turning Proportion*

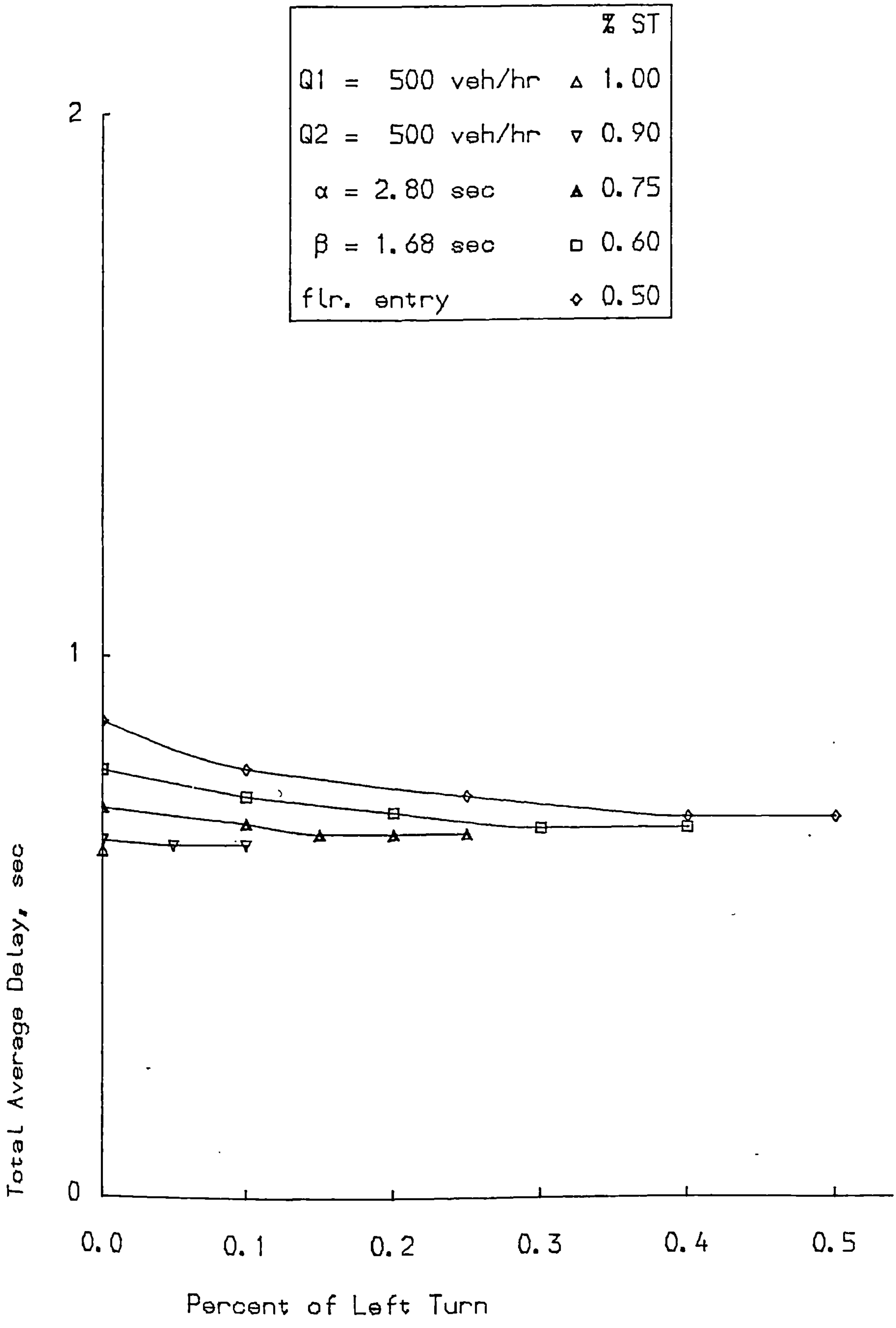


Figure 6.10 *Delay and Turning Proportion*

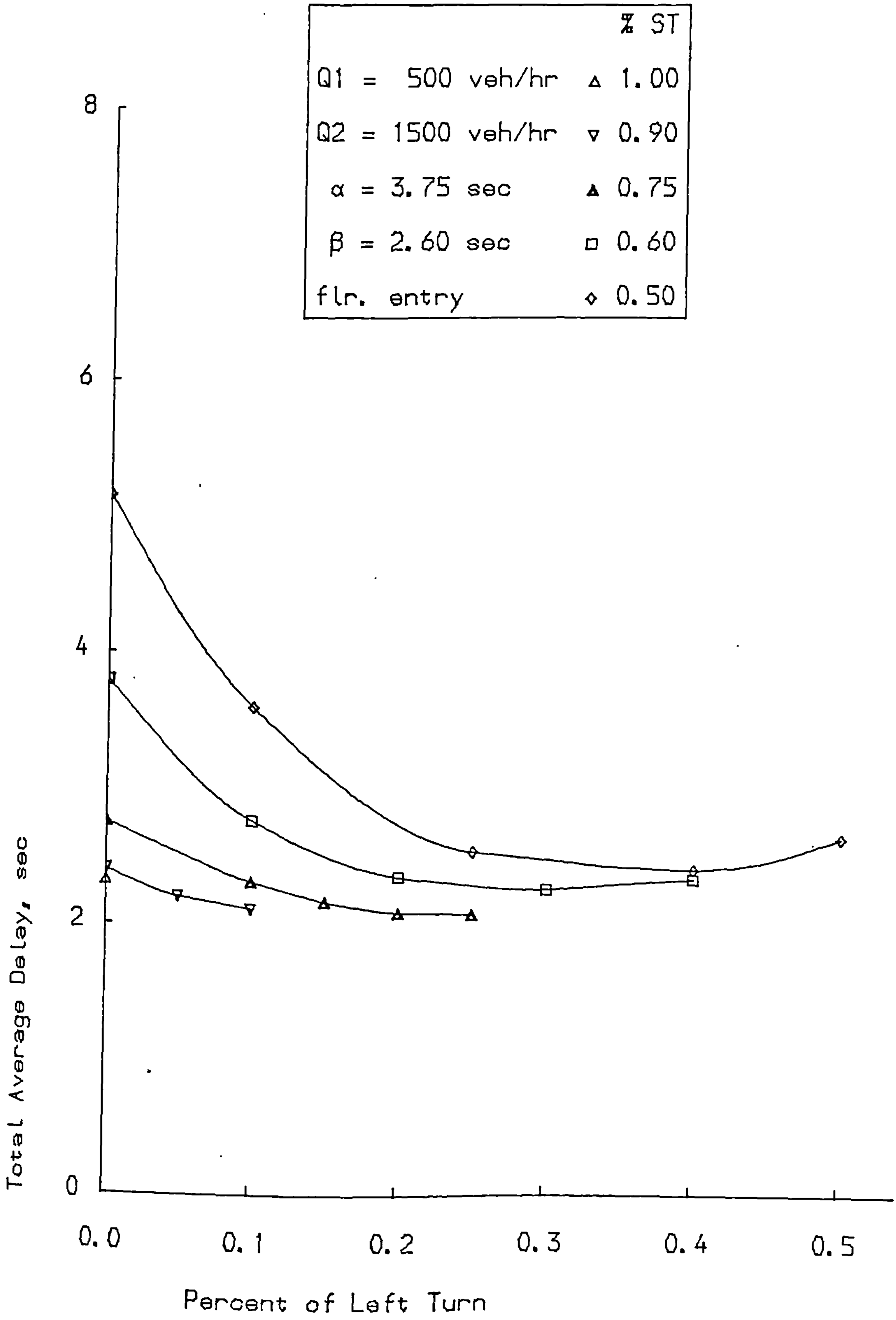
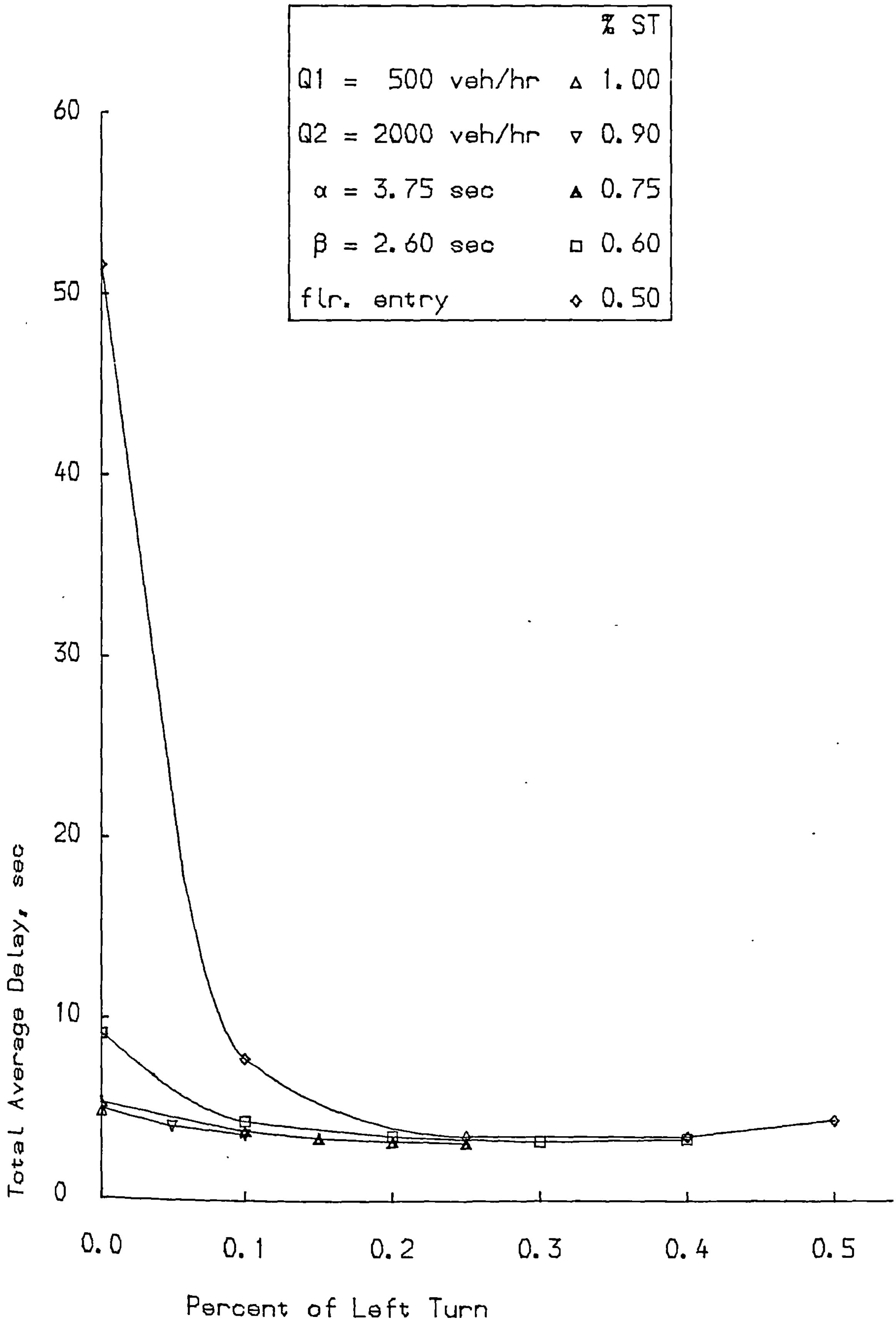


Figure 6.11 *Delay and Turning Proportion*



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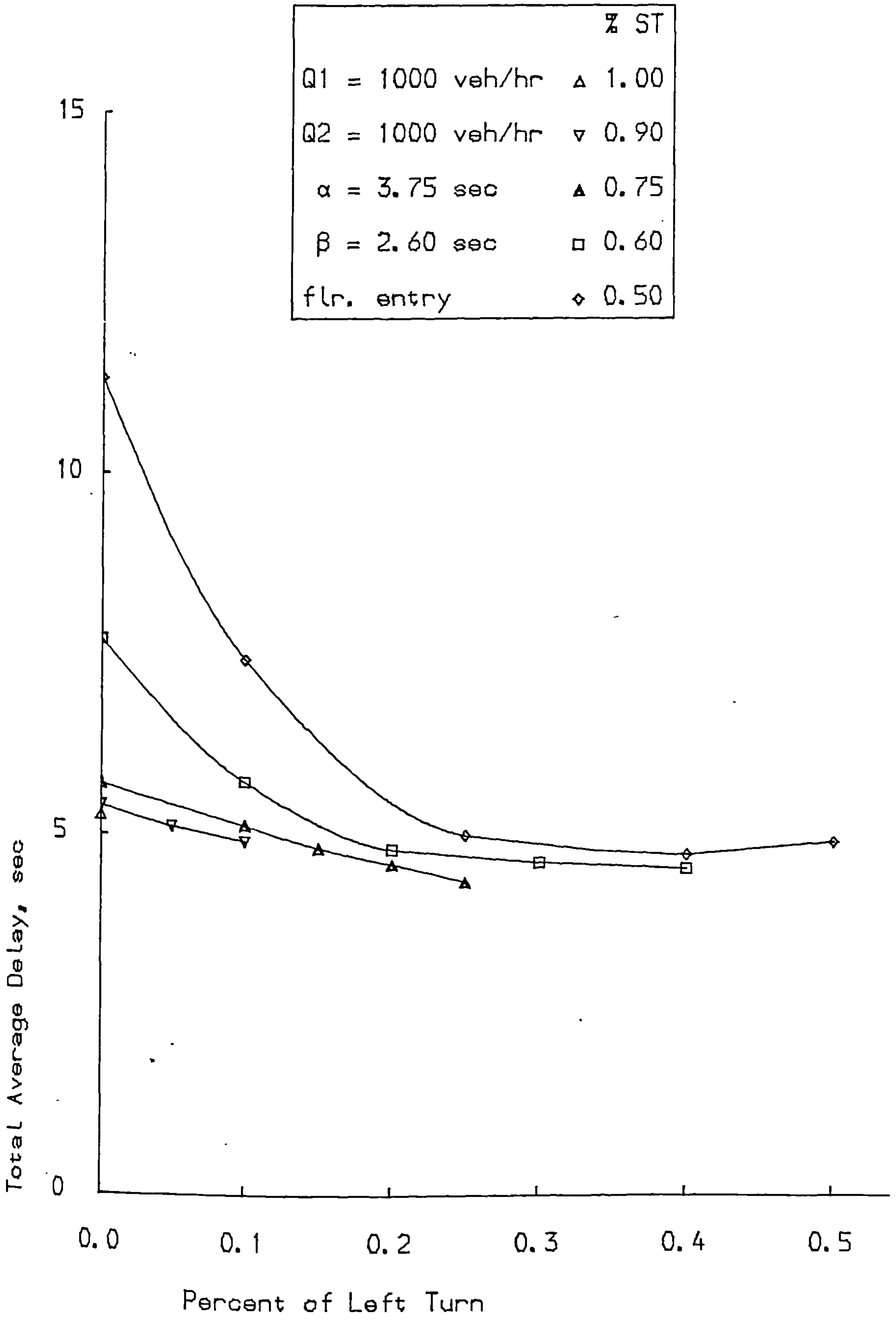

Figure 6.13 *Delay and Turning Proportion*

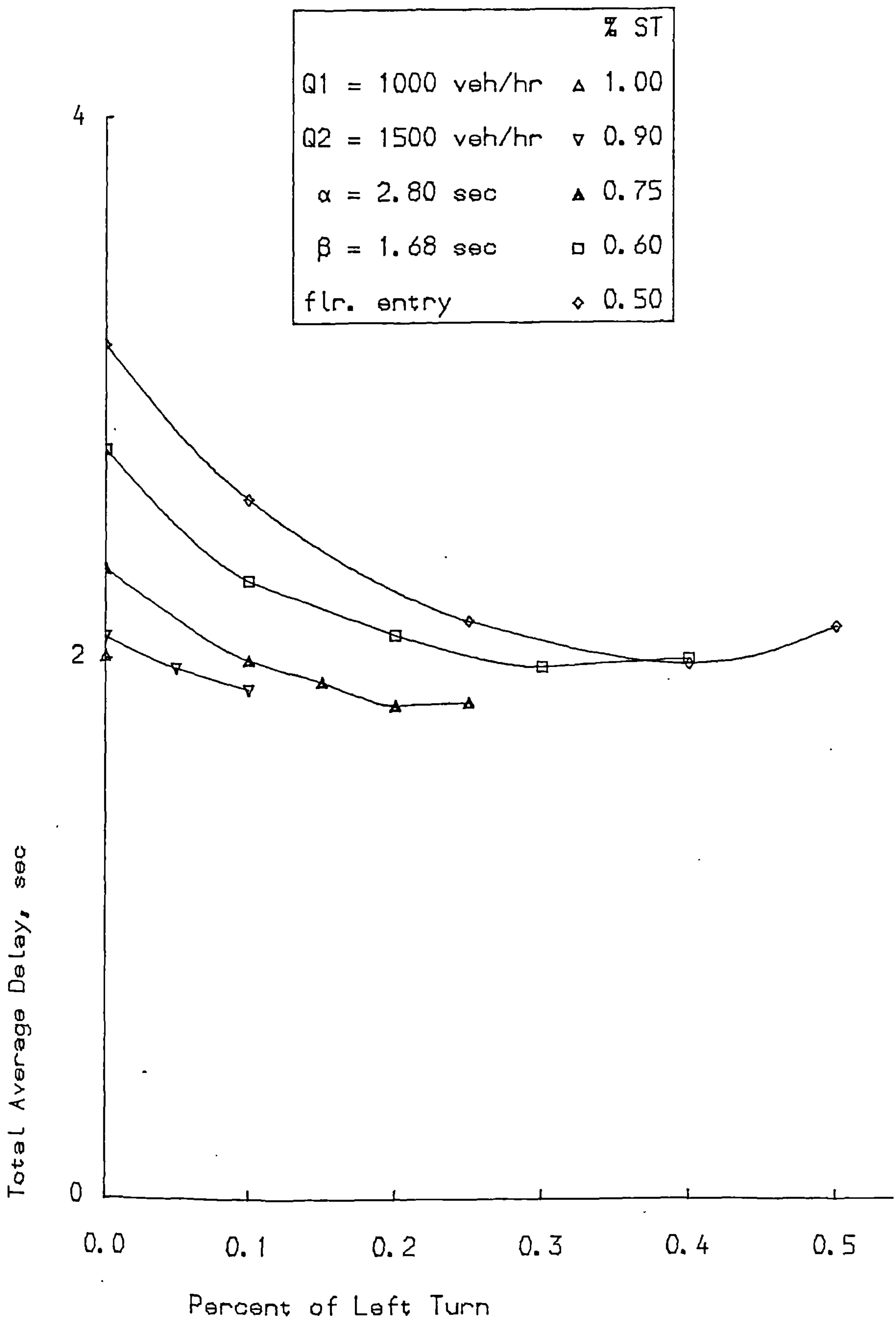

Figure 6.14 *Delay and Turning Proportion*

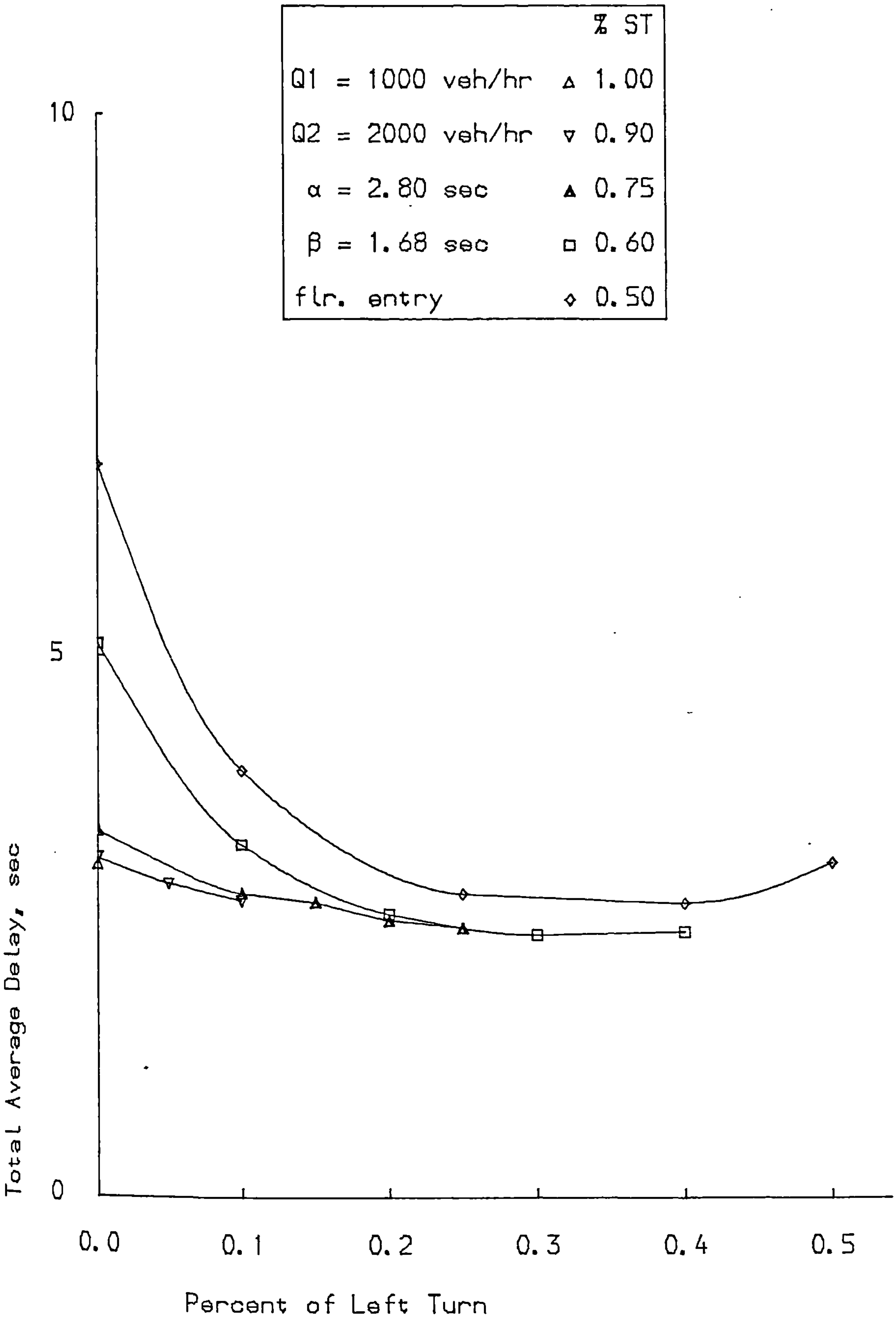

Figure 6.15 *Delay and Turning Proportion*

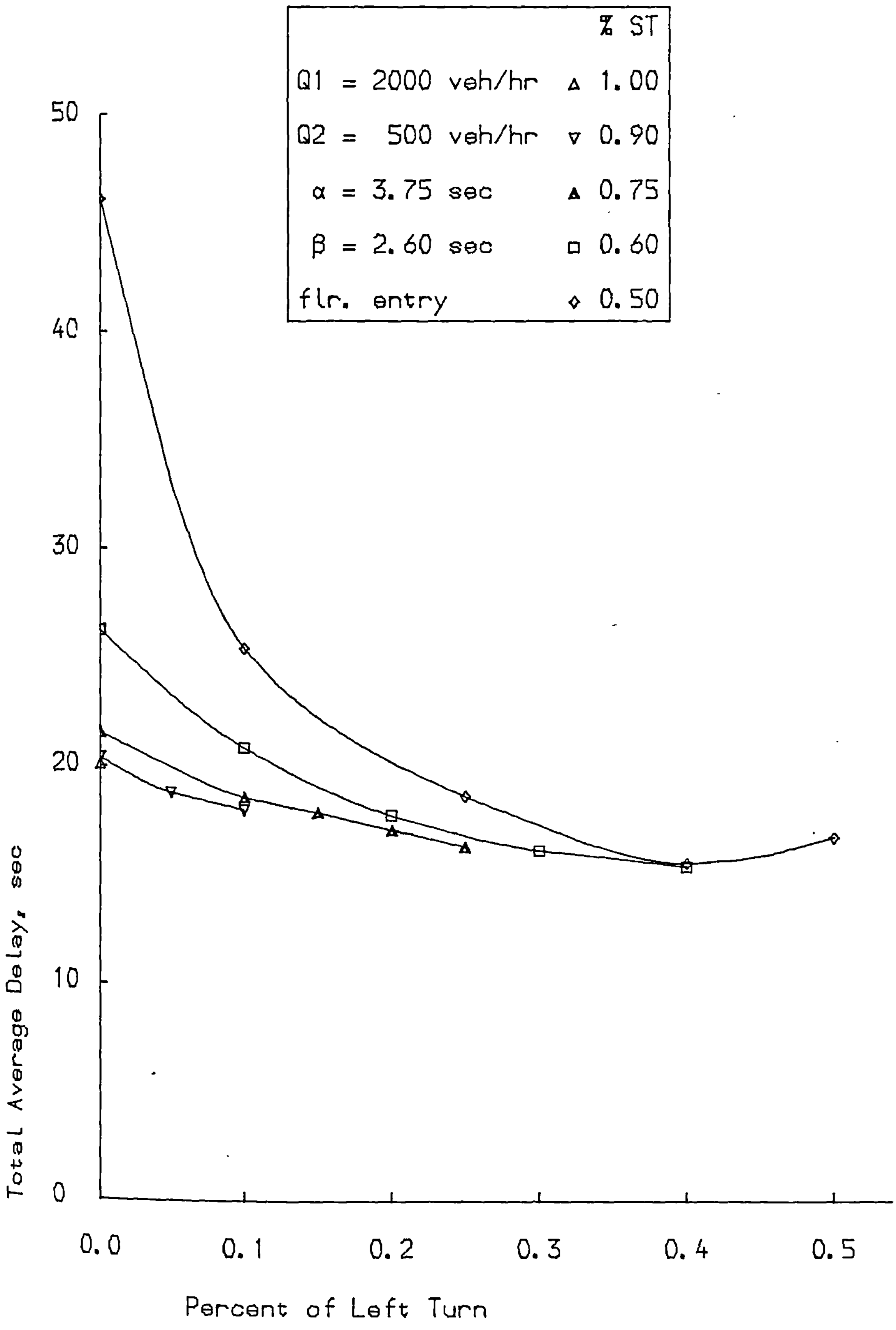

Figure 6.16 *Delay and Turning Proportion*

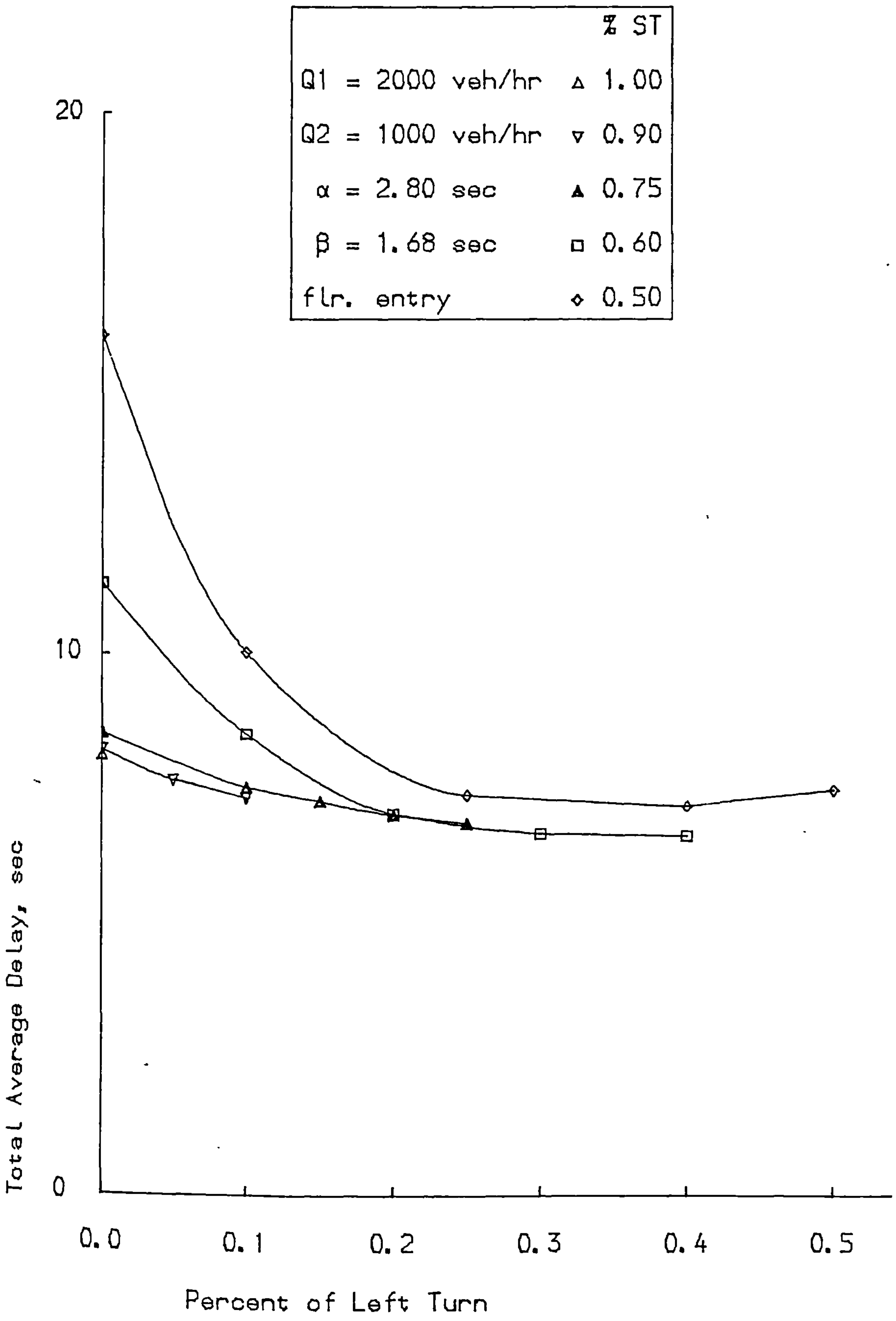

Figure 6.17 *Delay and Turning Proportion*

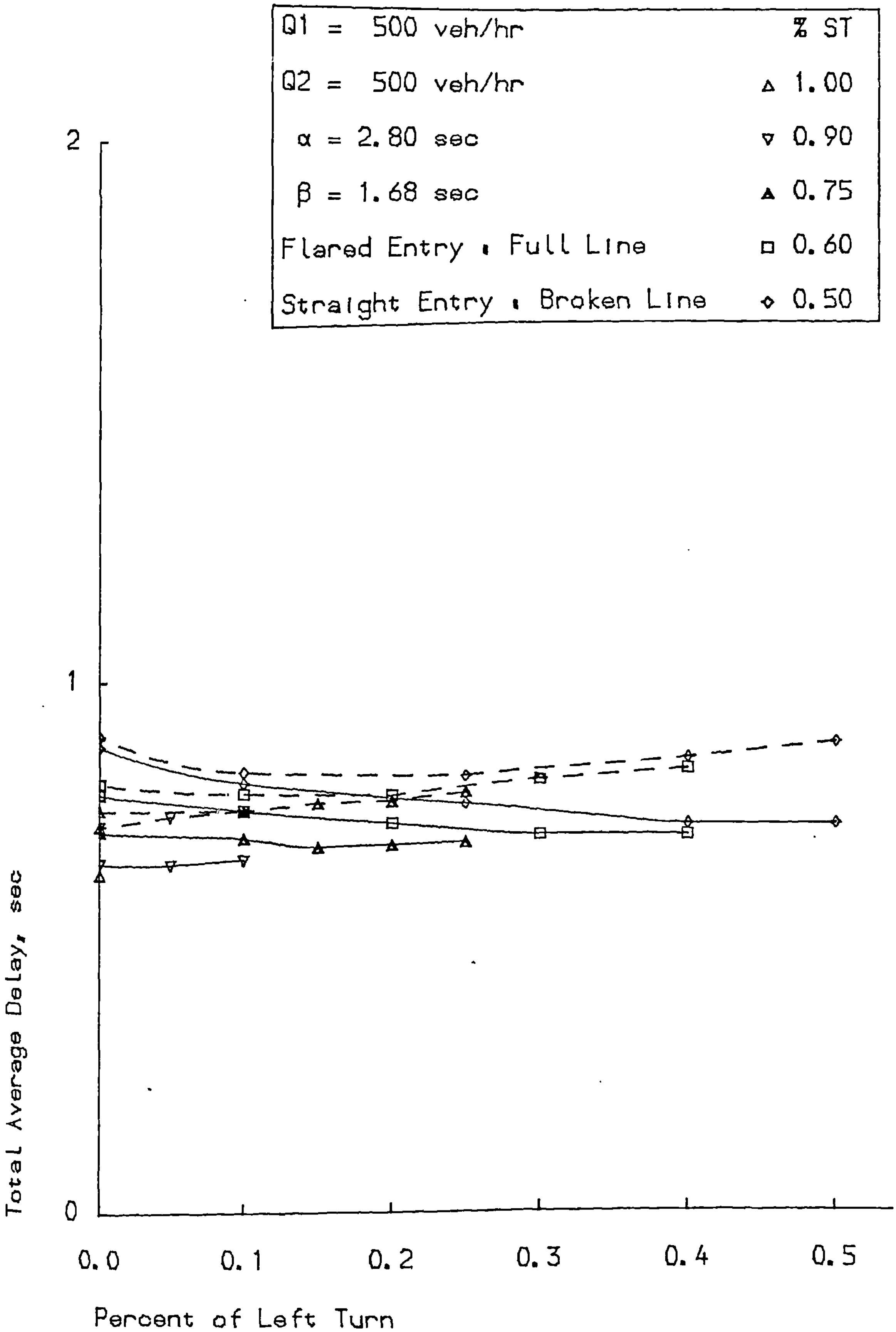

Figure 6.18 *Delay and Turning Proportion Comparison of Flared and Straight Entries*

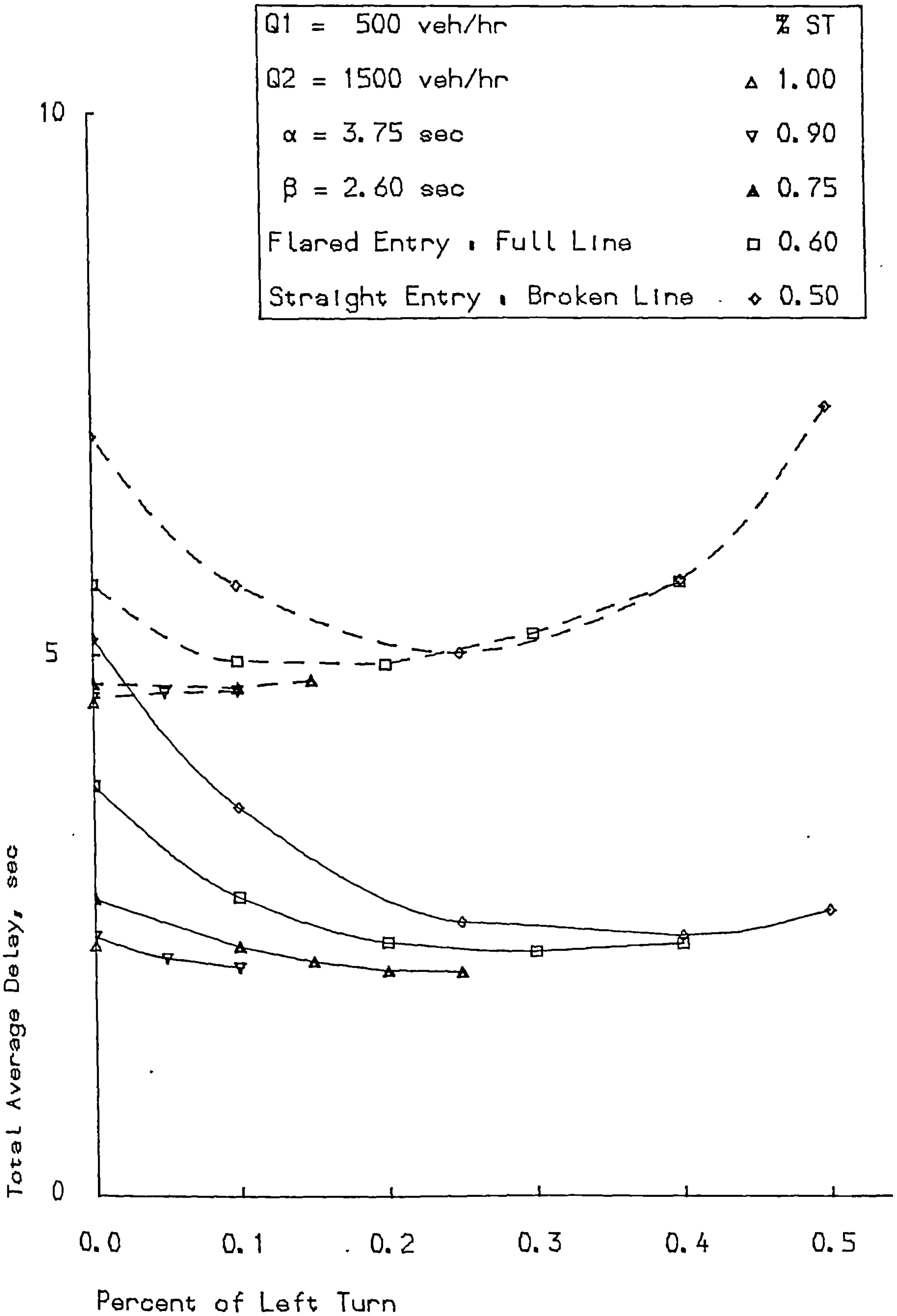

Figure 6.19 *Delay and Turning Proportion Comparison of Flared and Straight Entries*

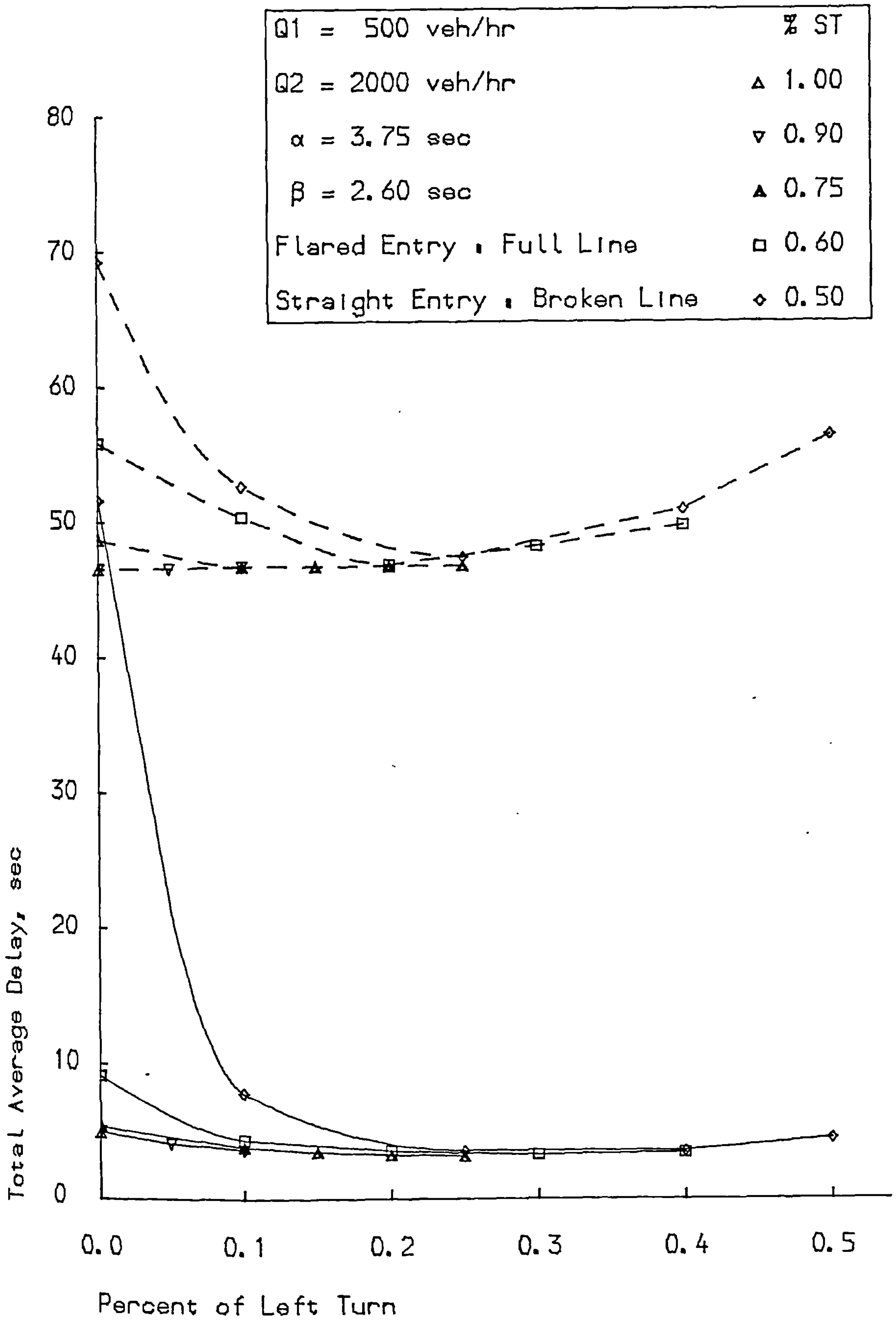

Figure 6.20 *Delay and Turning Proportion Comparison of Flared and Straight Entries*

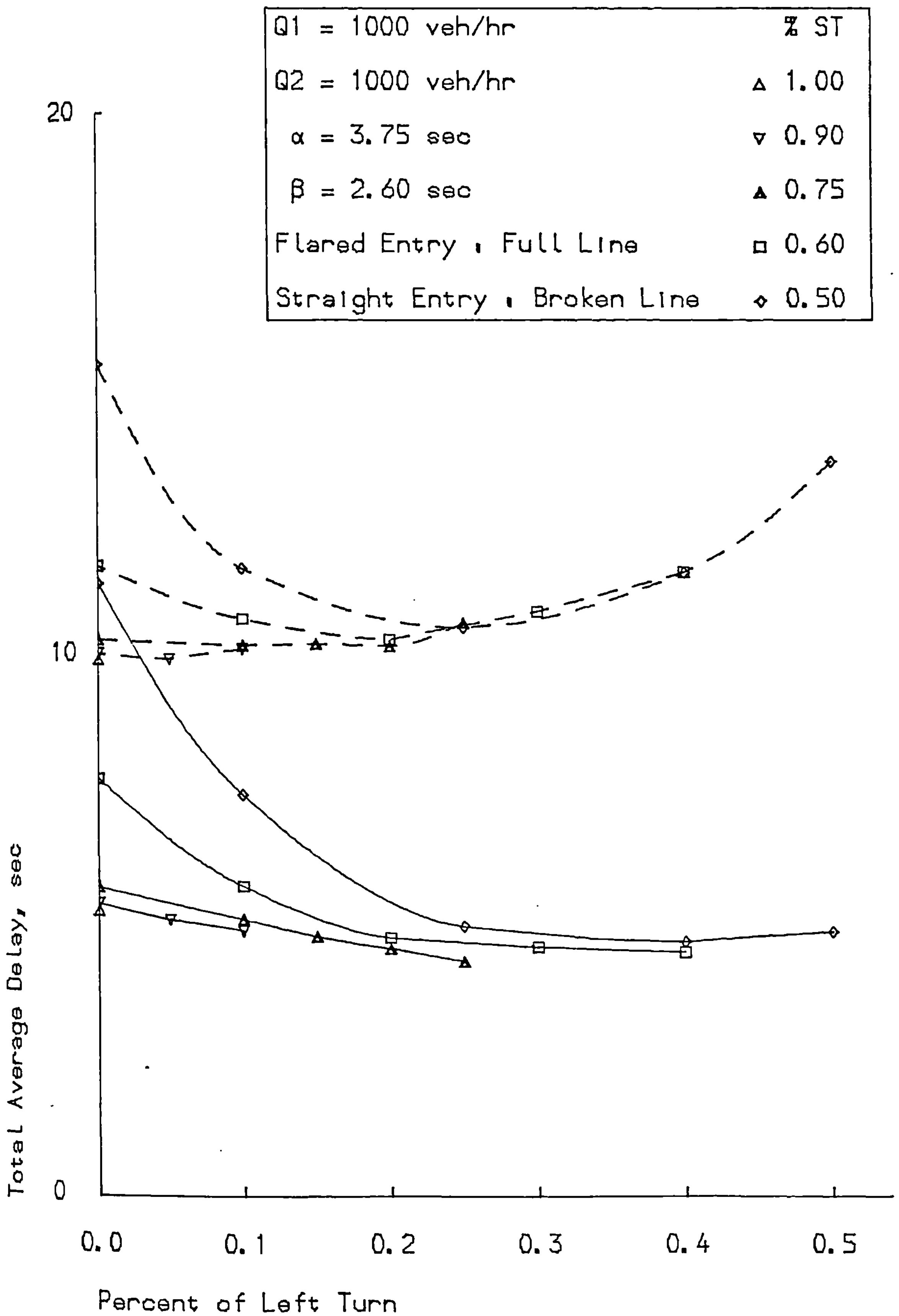

Figure 6.21 *Delay and Turning Proportion Comparison of Flared and Straight Entries*

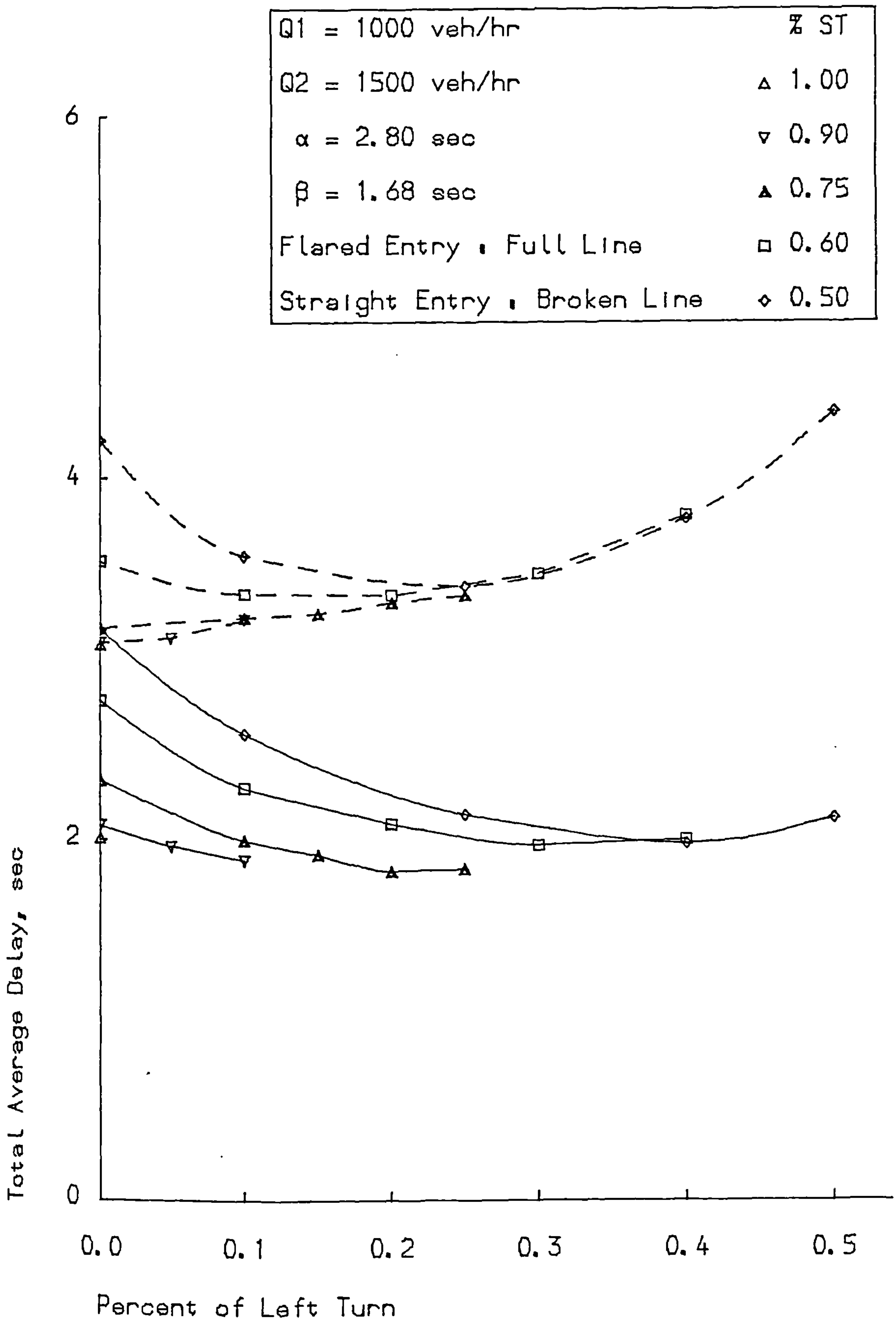

Figure 6.22 *Delay and Turning Proportion Comparison of Flared and Straight Entries*

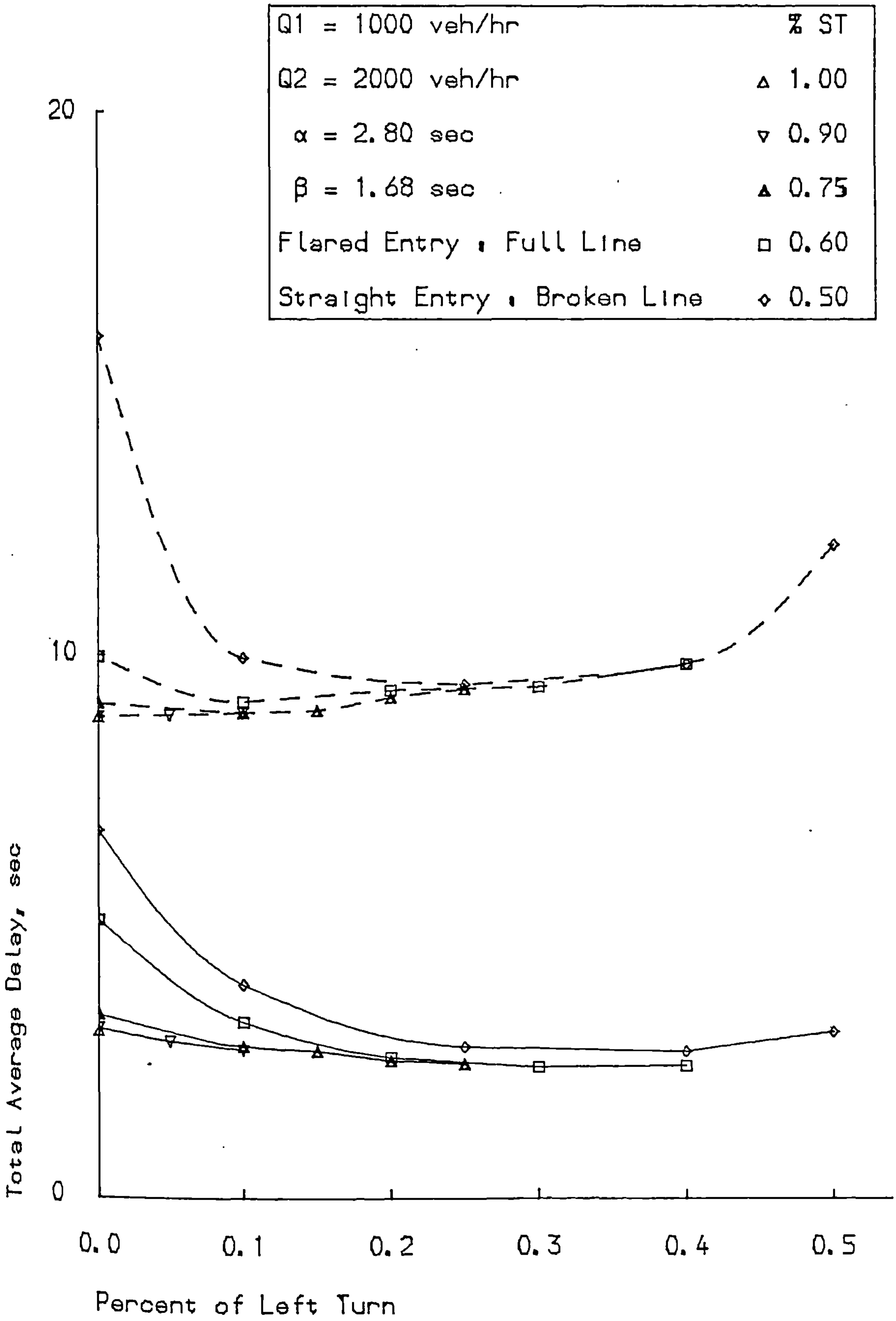

Figure 6.23 *Delay and Turning Proportion Comparison of Flared and Straight Entries*

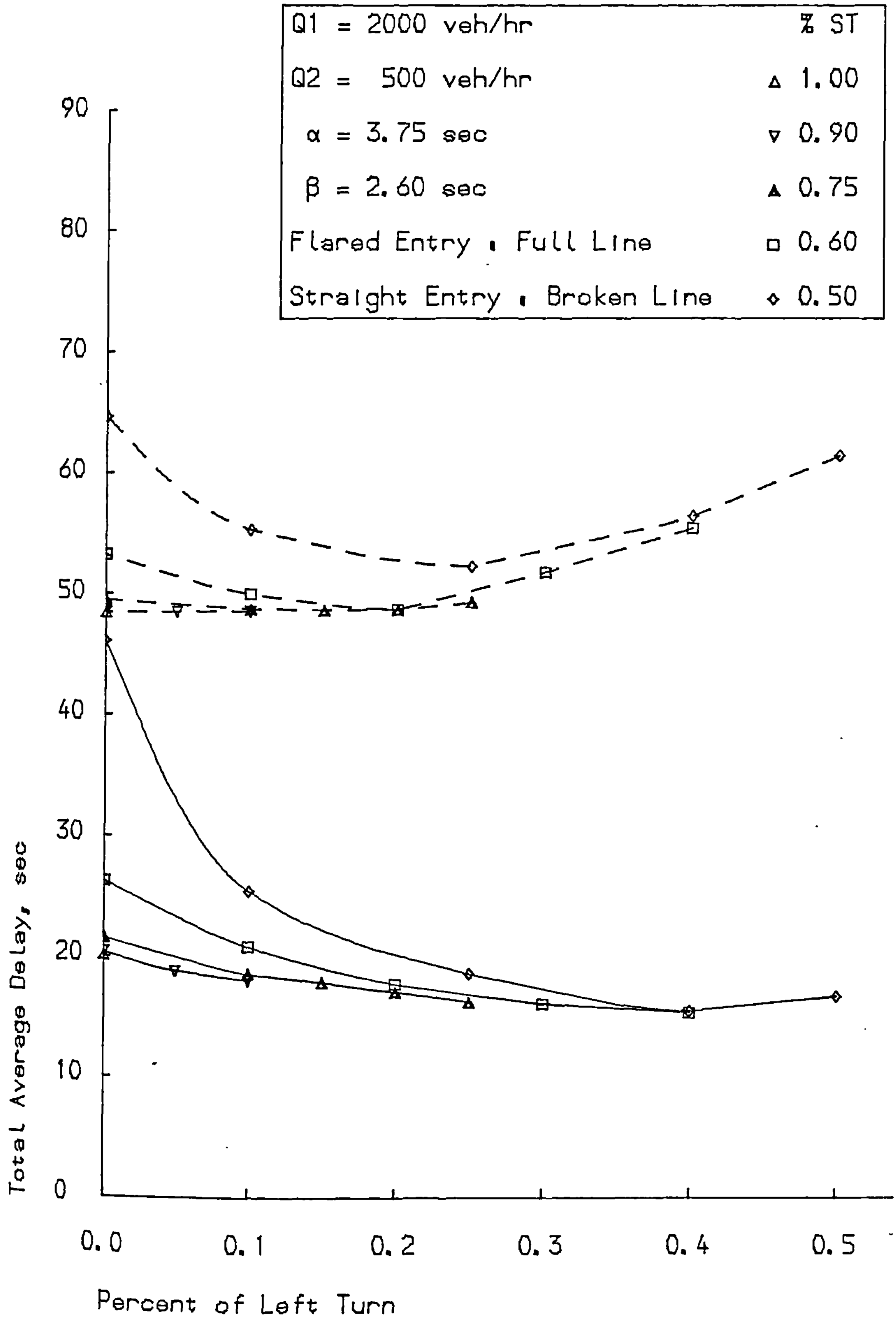

Figure 6.24 *Delay and Turning Proportion Comparison of Flared and Straight Entries*

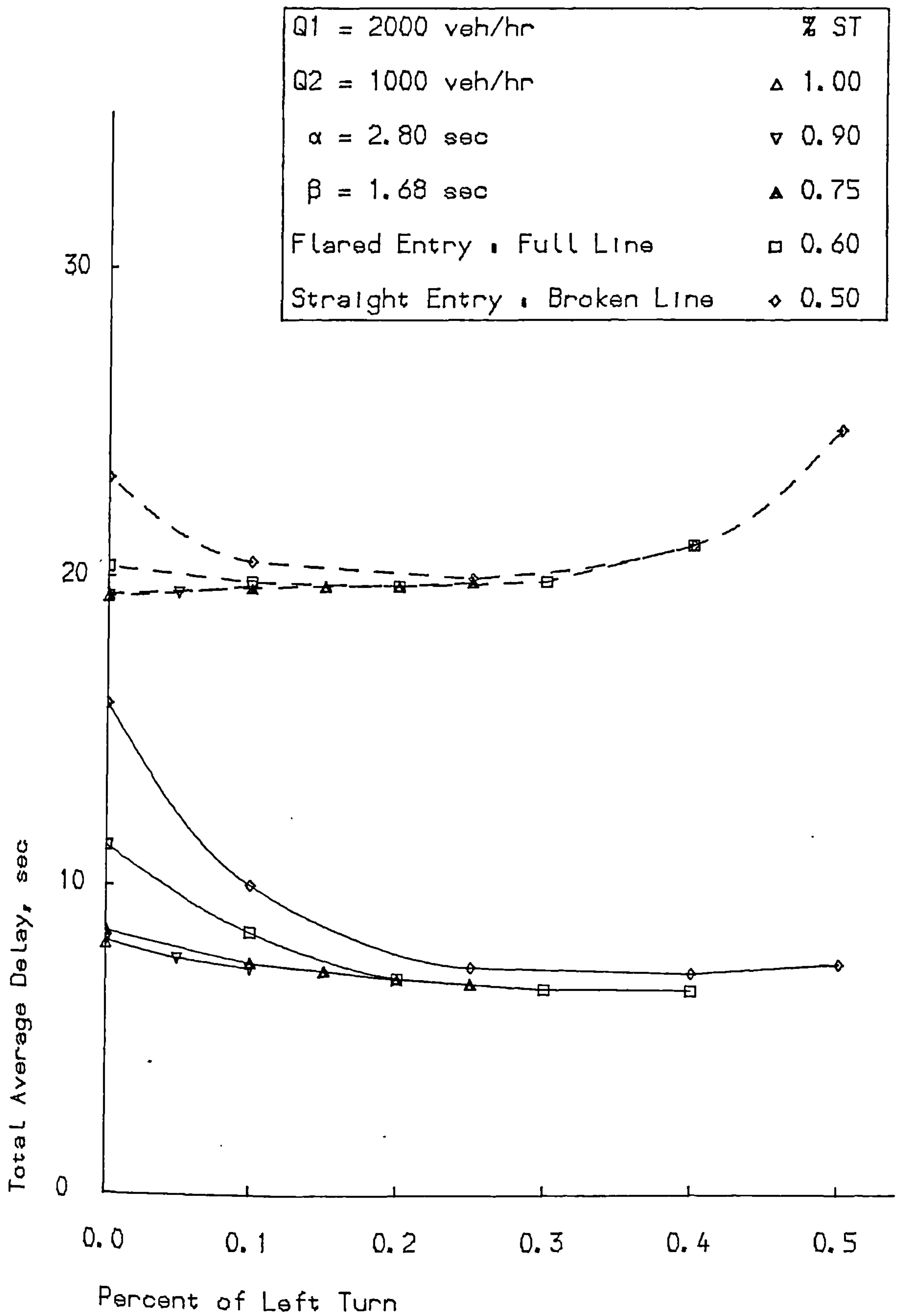

Figure 6.25 *Delay and Turning Proportion Comparison of Flared and Straight Entries*

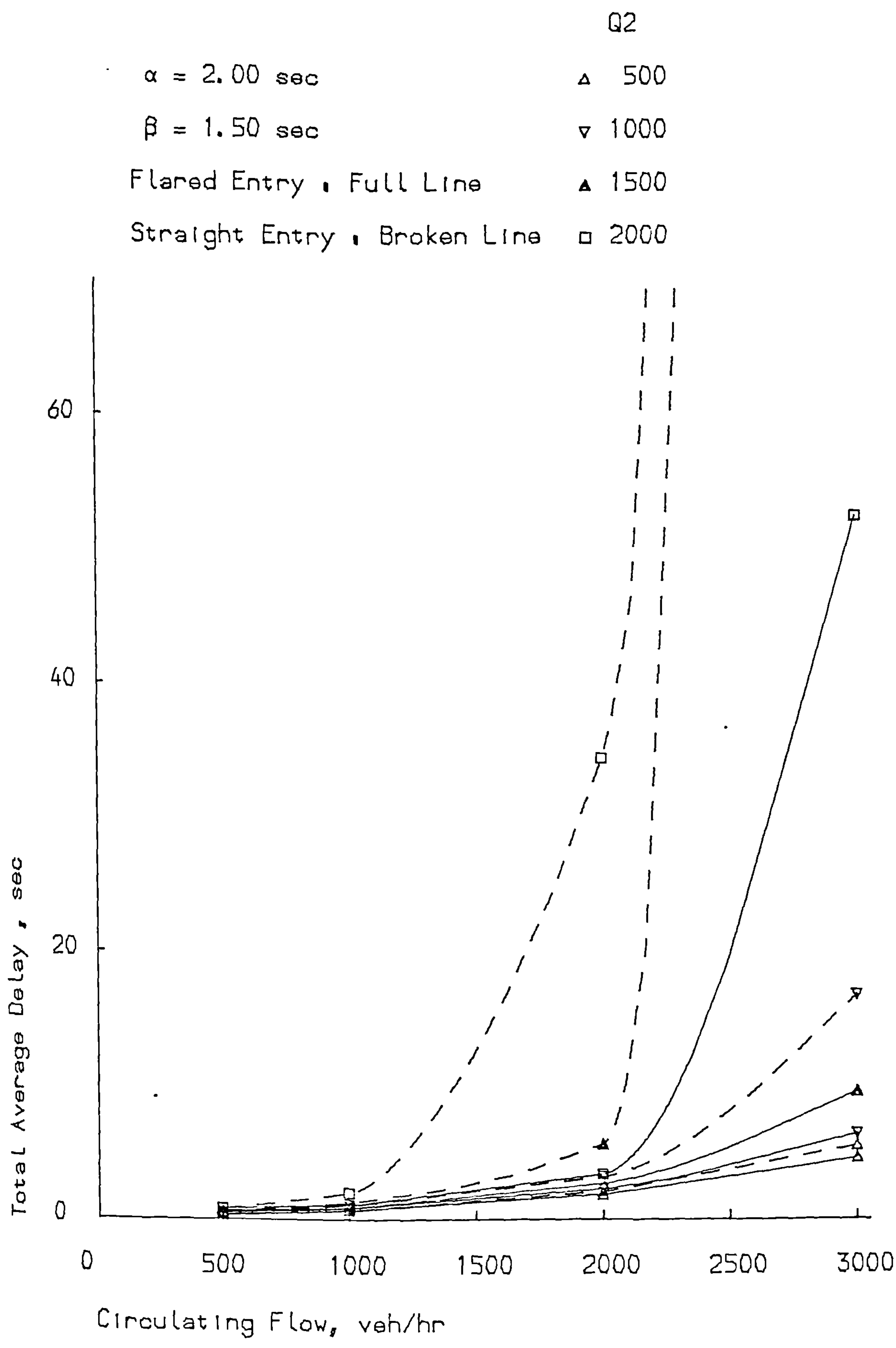

Figure 6.26 *Delay and Circulating Flow*
Comparison of Flared and Straight Entries

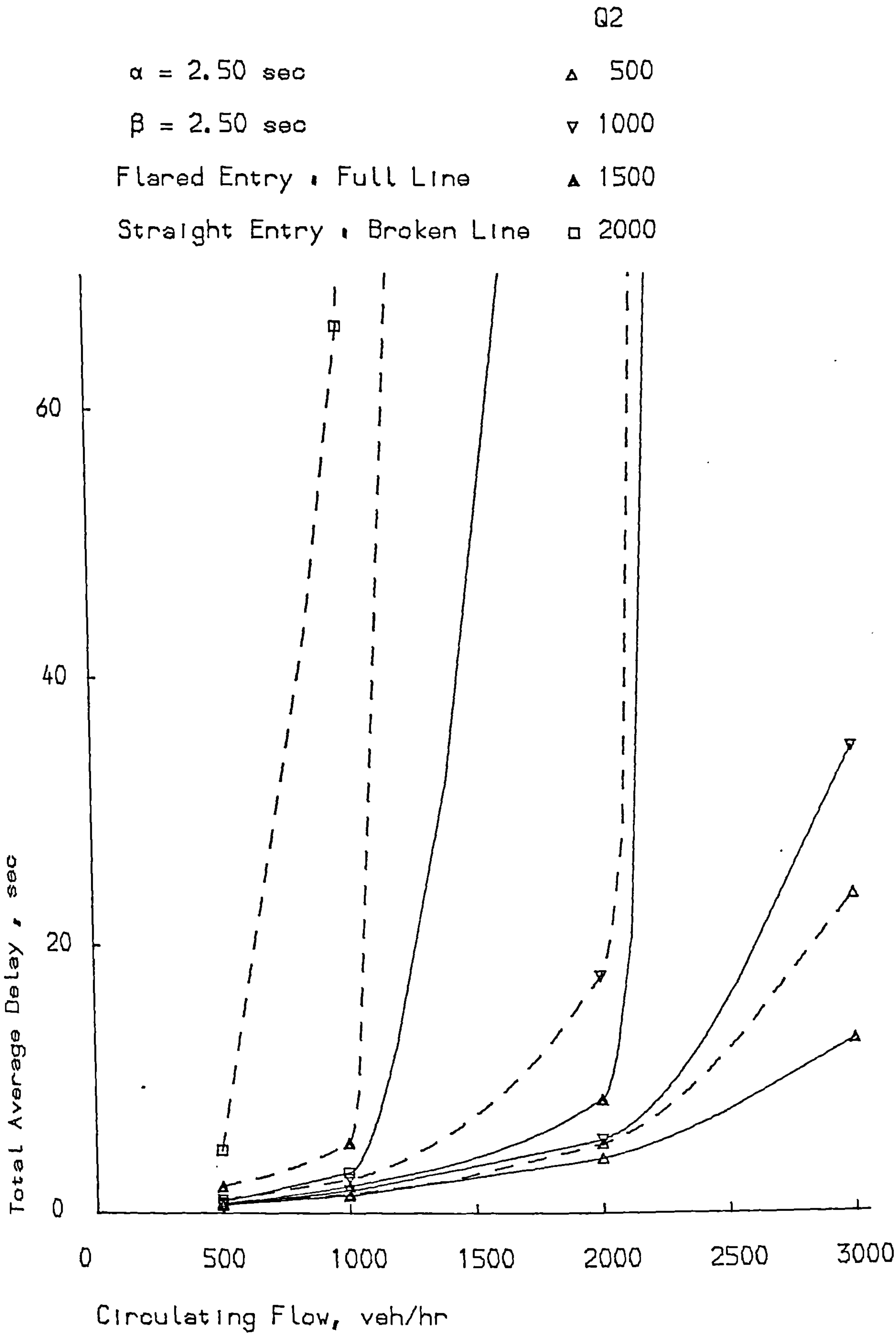

Figure 6.27 *Delay and Circulating Flow*
Comparison of Flared and Straight Entries

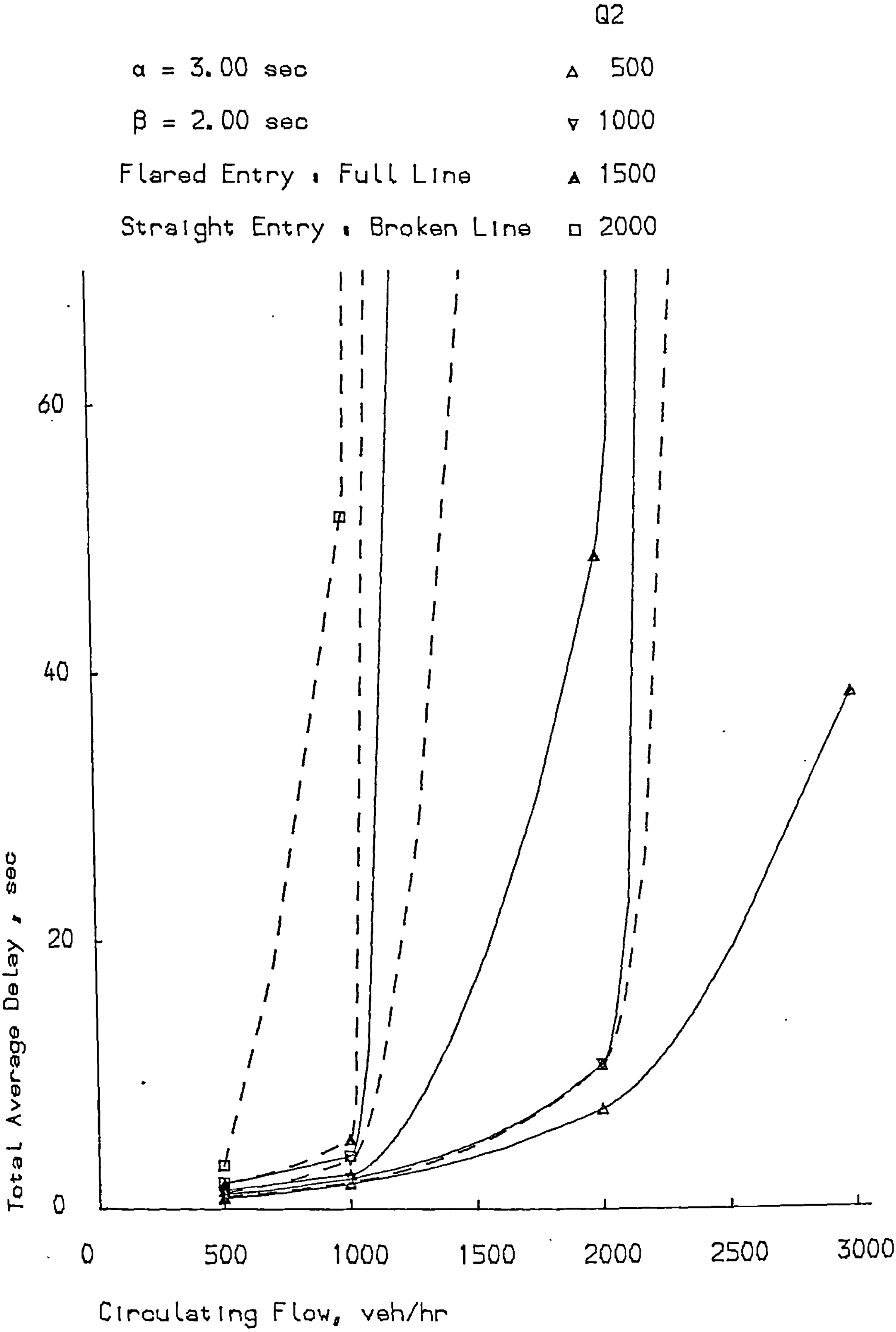

Figure 6.28 *Delay and Circulating Flow*

Comparison of Flared and Straight Entries

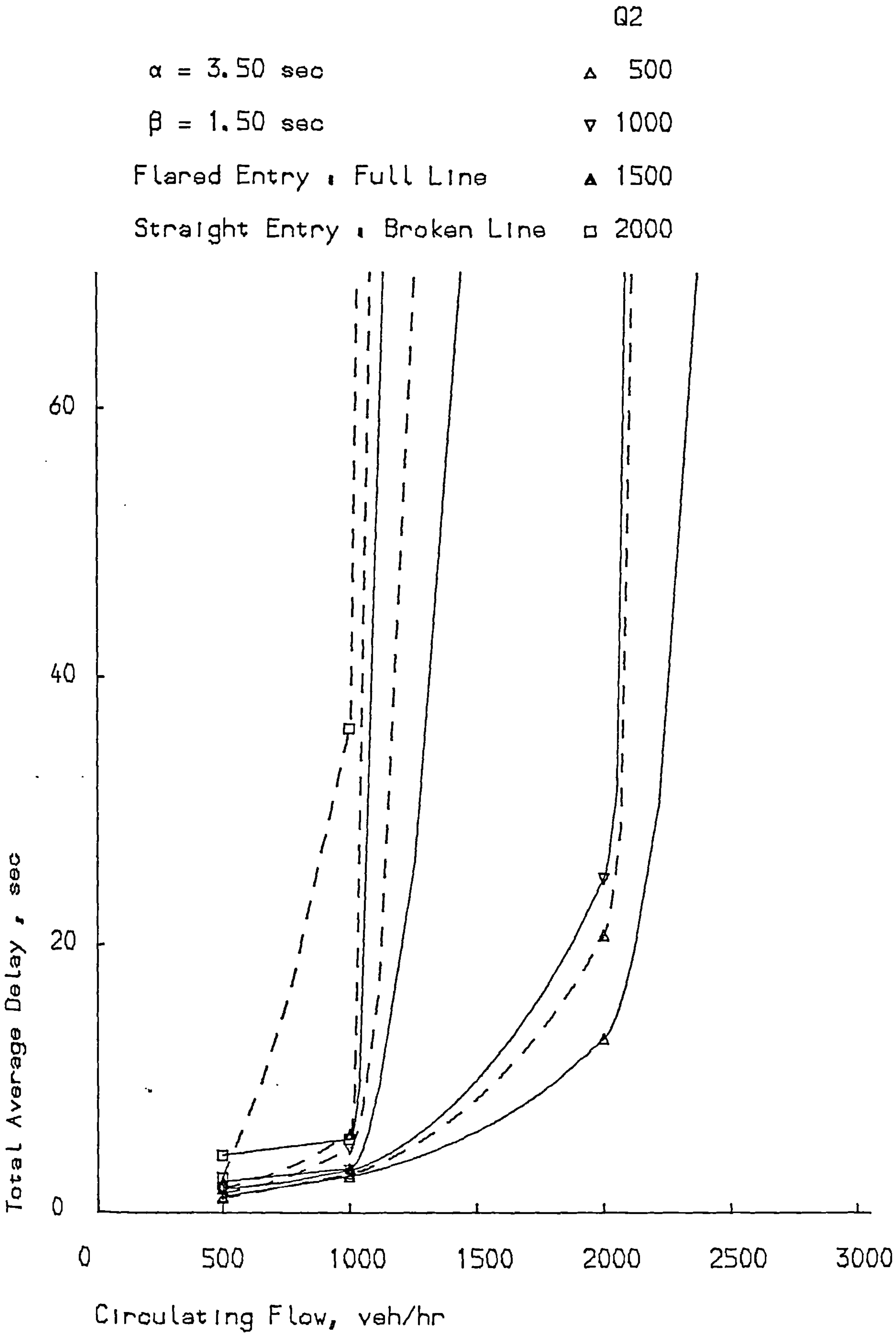

Figure 6.29 *Delay and Circulating Flow*
Comparison of Flared and Straight Entries

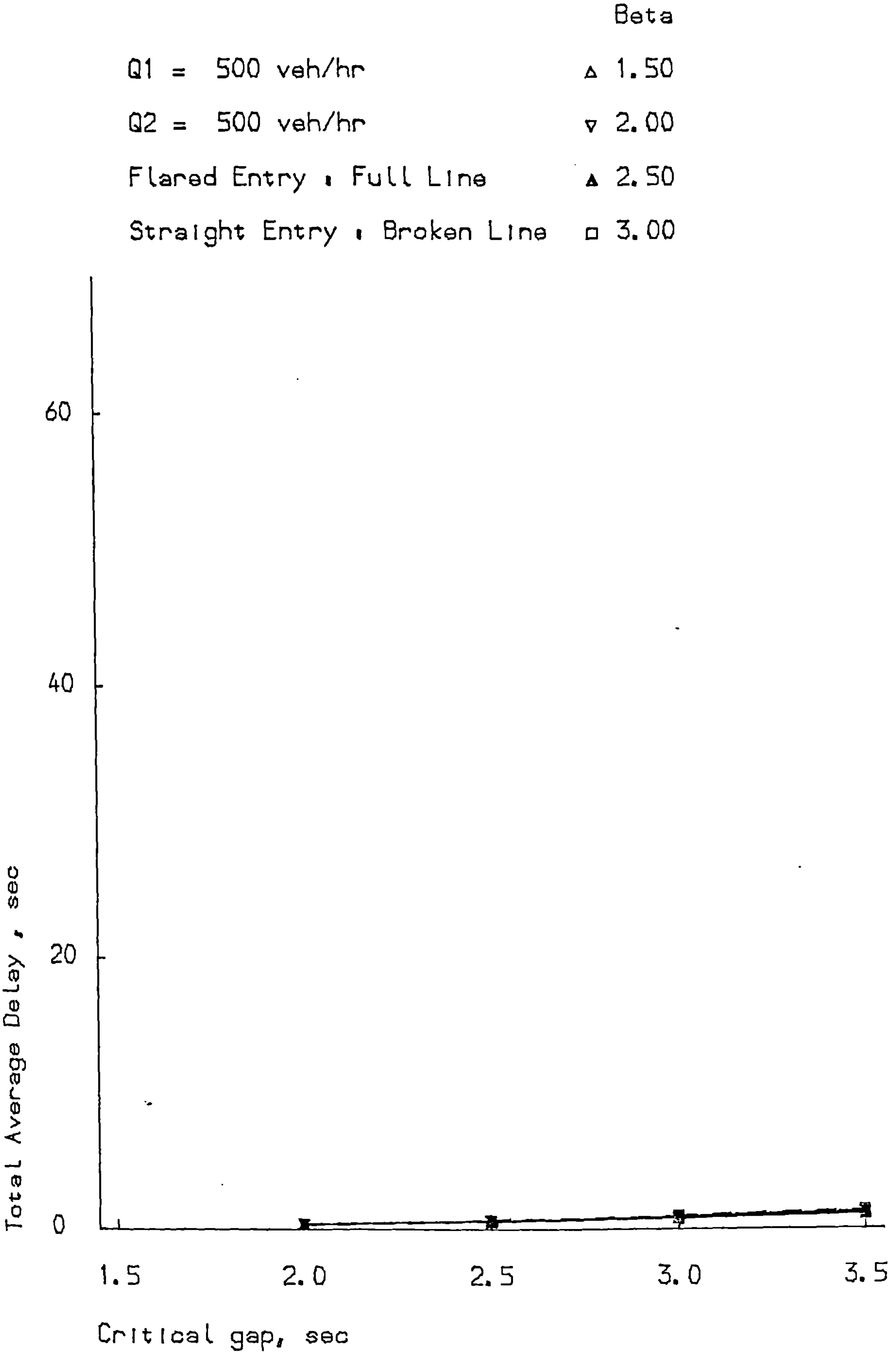

Figure 6.30 *Delay and Critical Gap Comparison of Flared and Straight Entries*

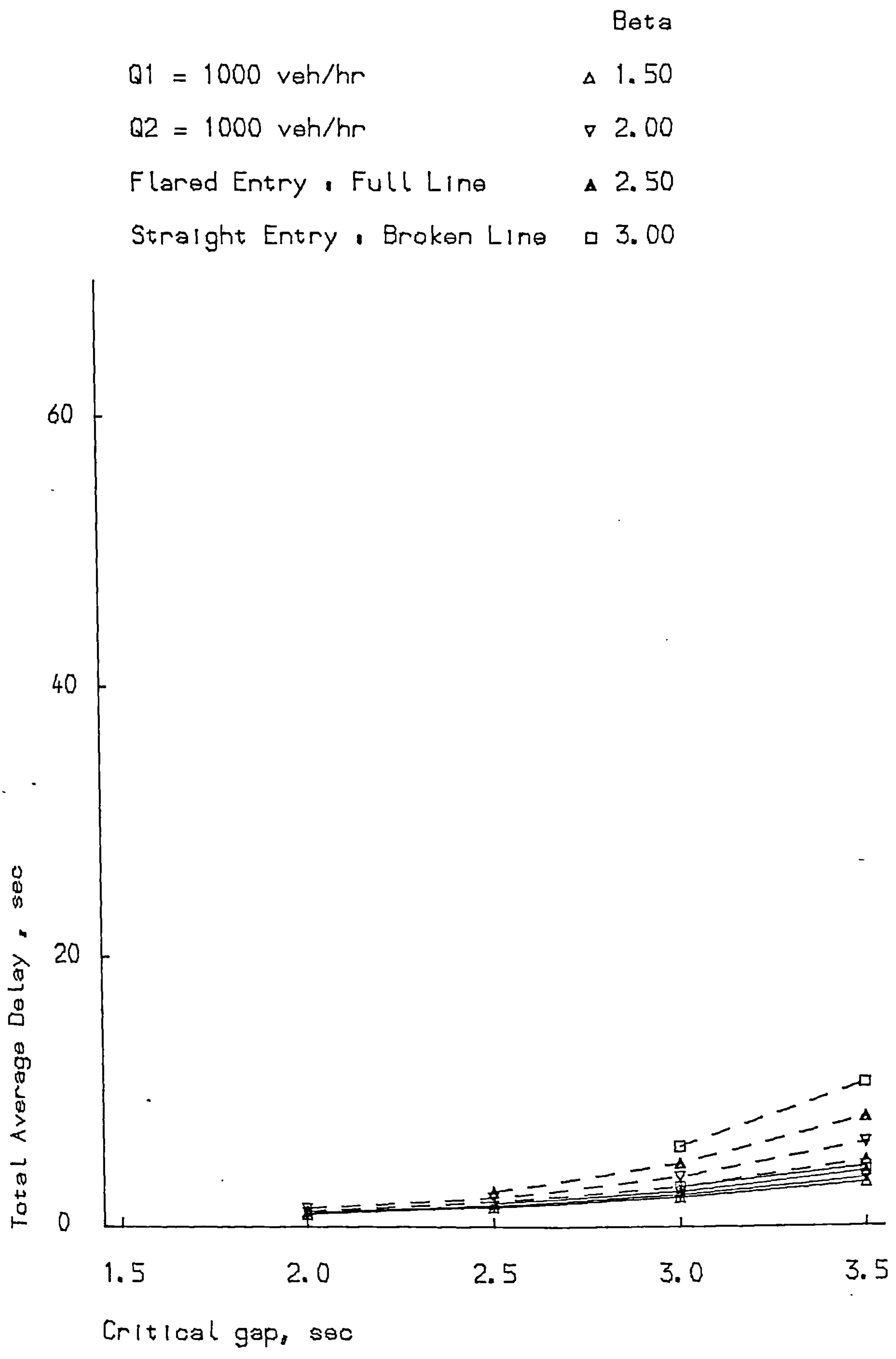

Figure 6.31 *Delay and Critical Gap*
Comparison of Flared and Straight Entries

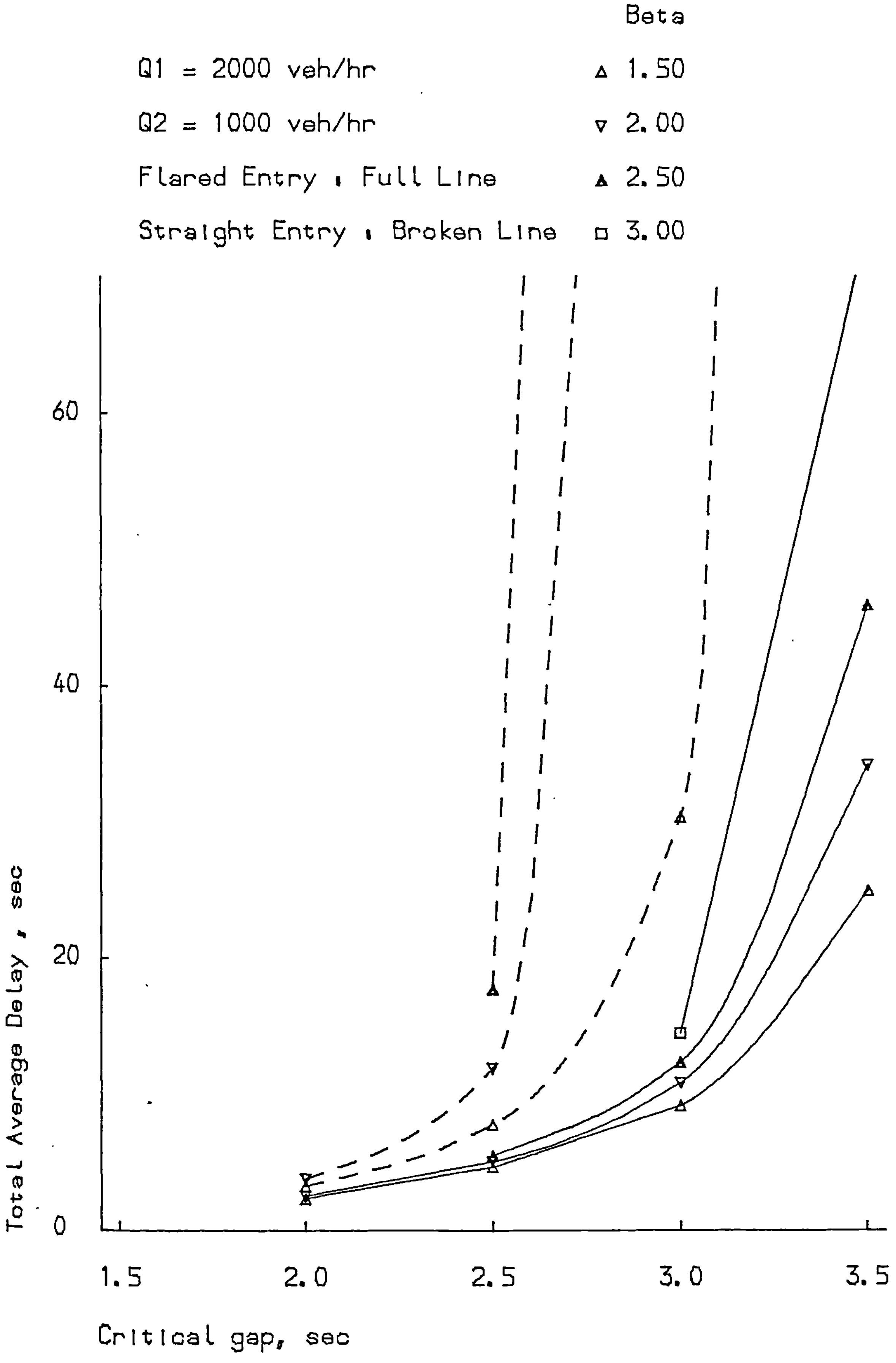

Figure 6.32 *Delay and Critical Gap*
Comparison of Flared and Straight Entries

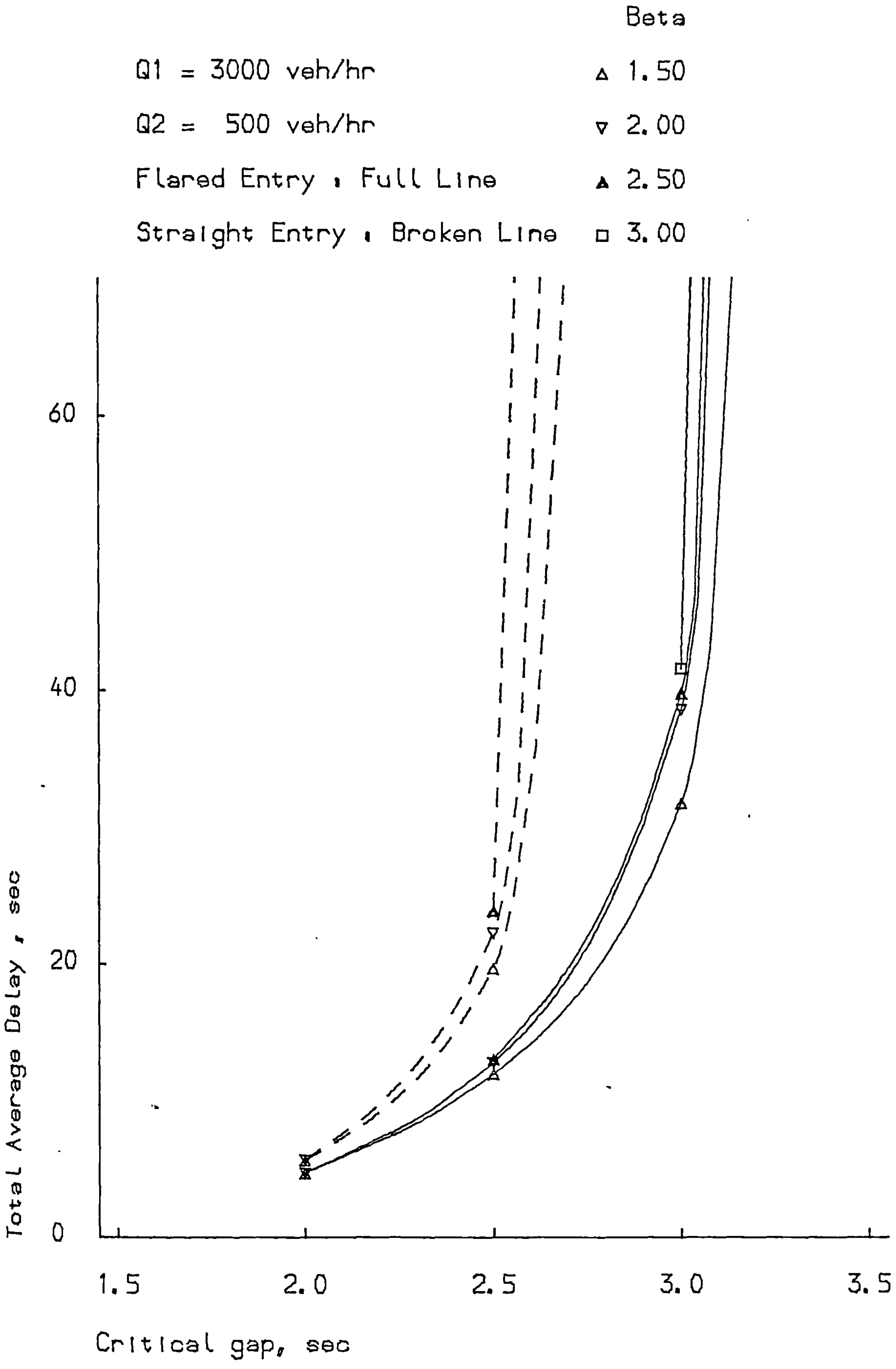

Figure 6.33 *Delay and Critical Gap Comparison of Flared and Straight Entries*

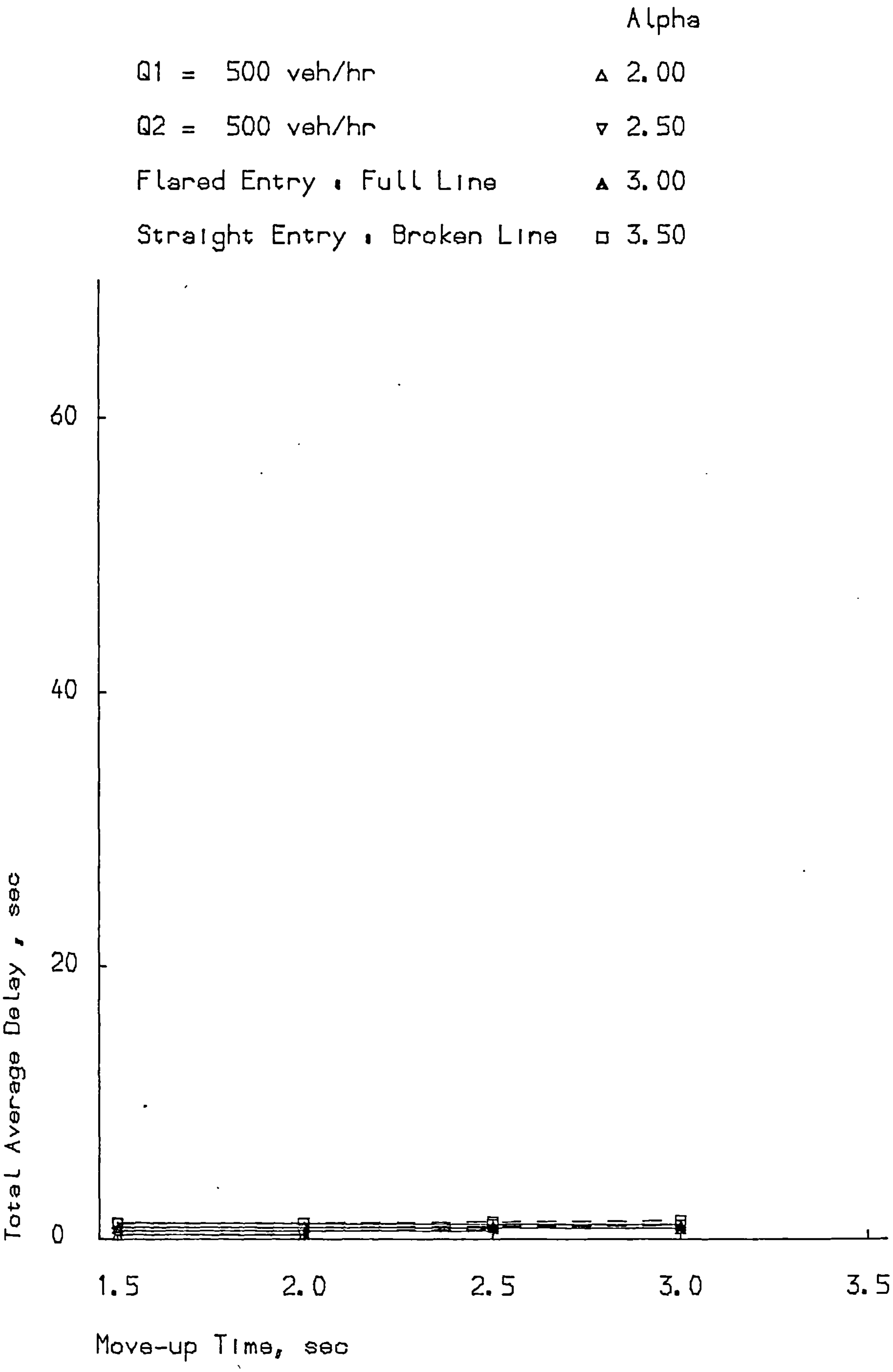

Figure 6.34 *Delay and Move-up Time Comparison of Flared and Straight Entries*

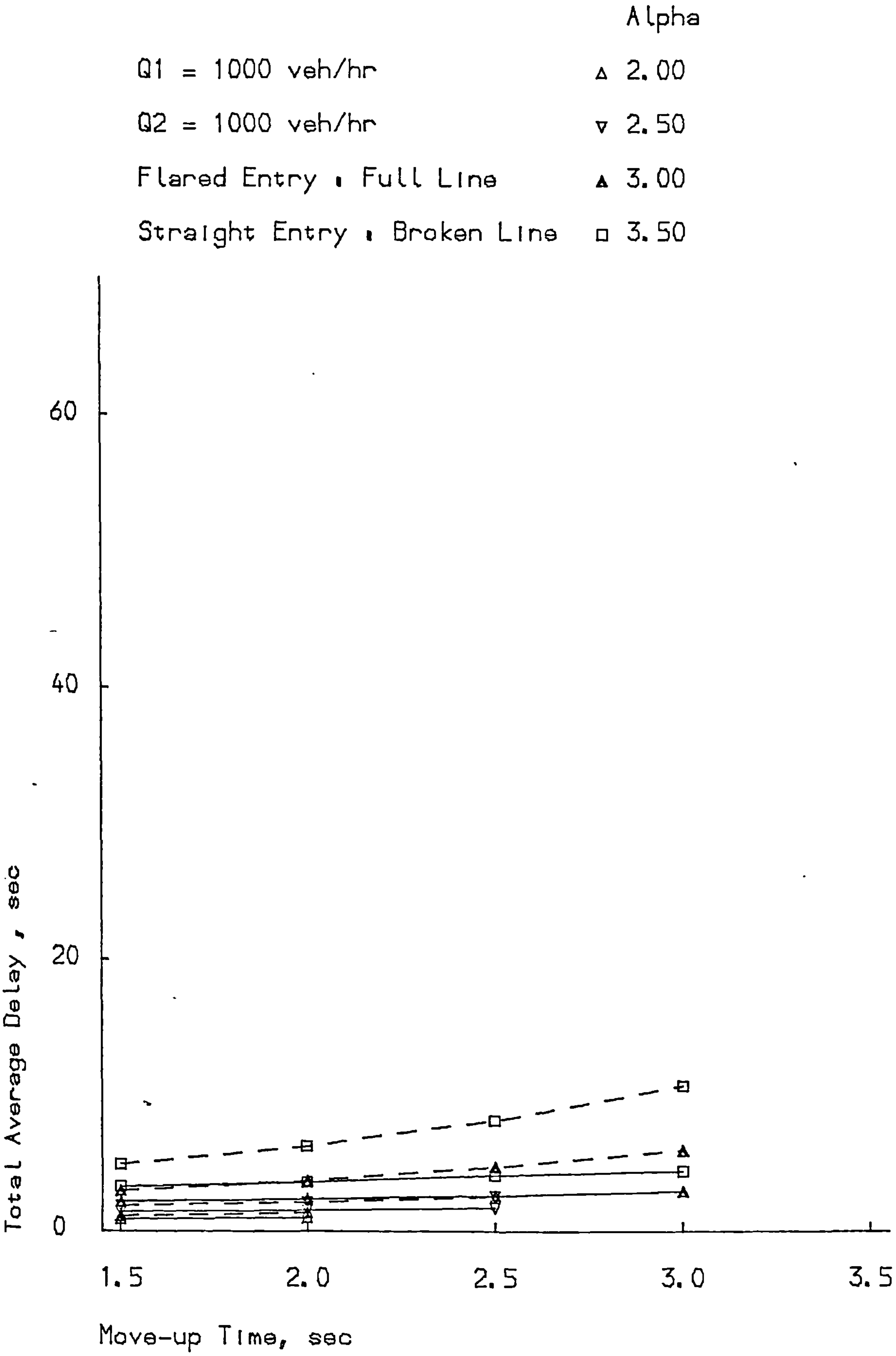

Figure 6.35 *Delay and Move-up Time*
Comparison of Flared and Straight Entries

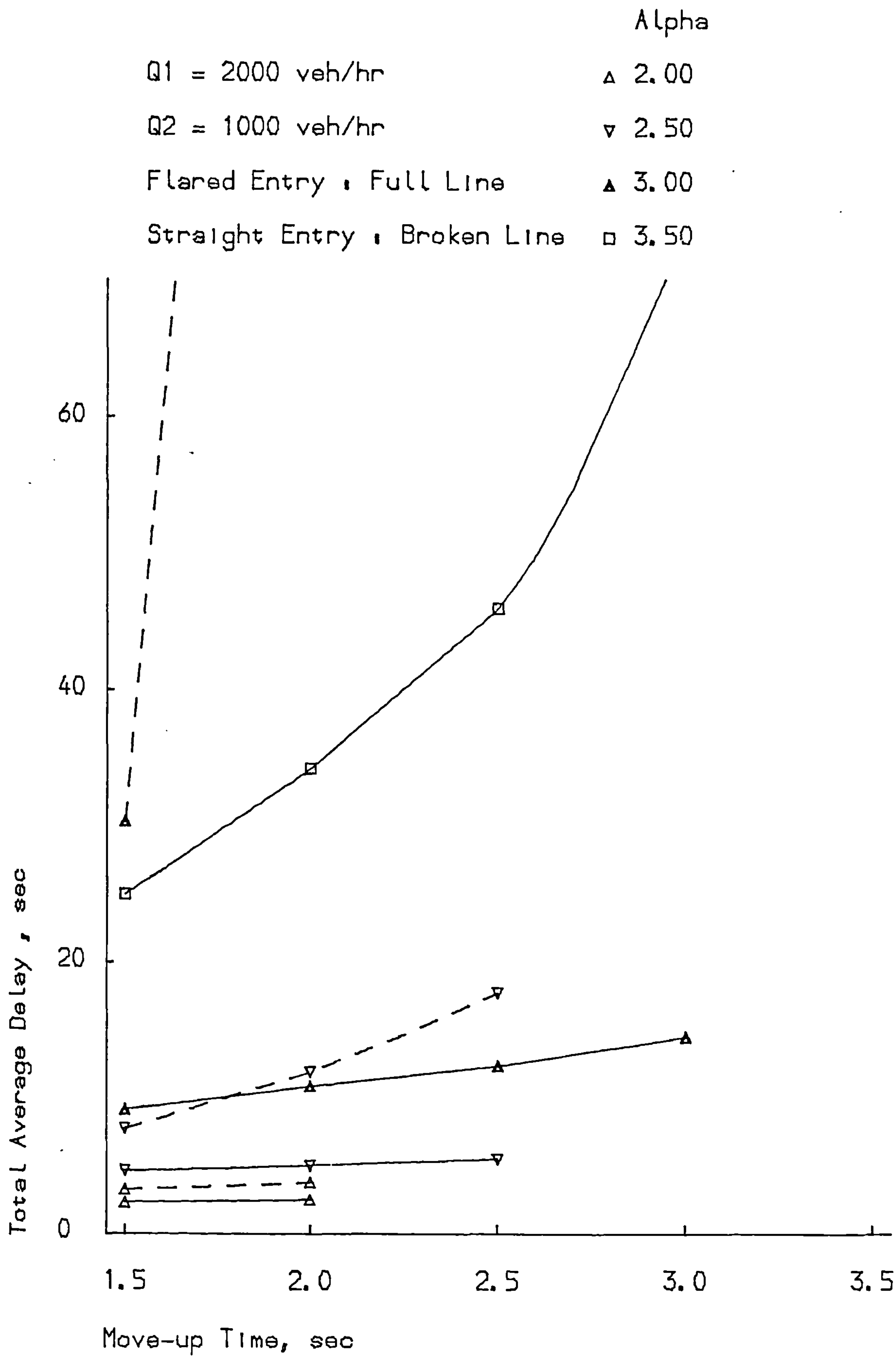

Figure 6.36 *Delay and Move-up Time*
Comparison of Flared and Straight Entries

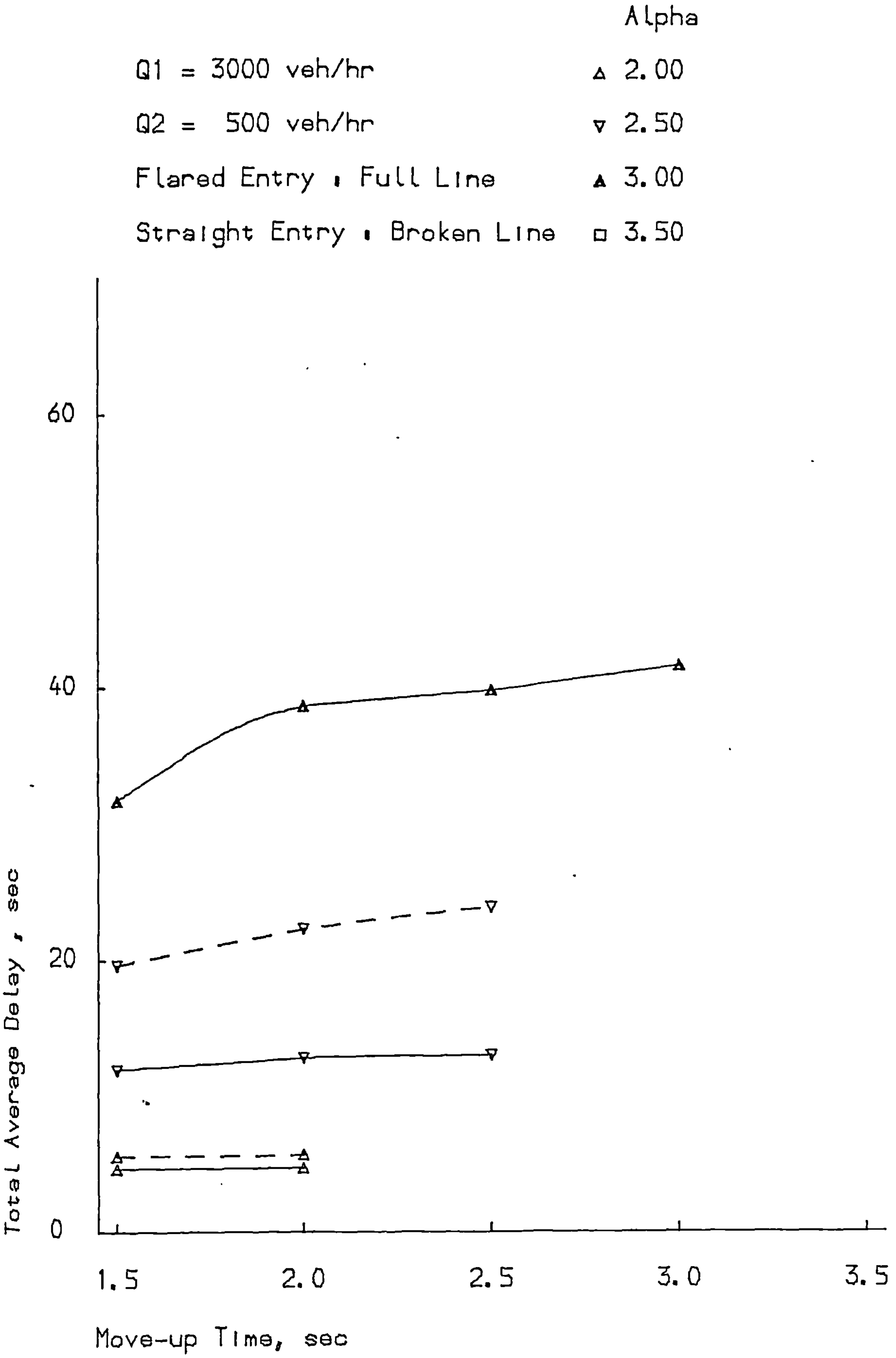

Figure 6.37 *Delay and Move-up Time Comparison of Flared and Straight Entries*

CHAPTER 7

CONCLUSIONS

A computer simulation model has been developed to study the performance of entries to roundabouts. It allowed the comparison of straight and flared entries and the study of the effect of turning movements, flow characteristics, and the gap acceptance parameters on delay and capacity associated with such entries. Further, the abstraction of gap-acceptance parameters from data collected at roundabouts was examined and a method for such abstraction was proposed.

1. Several methods for estimating the gap-acceptance parameters were tested. It was concluded that a model which fits a single line to the accepted gaps only, predicts adequately both the critical gap and the move-up time.

2. The predictions of capacity by the computer simulation model were compared with observed values. That gap-acceptance parameter values used as input to the model were abstracted from the observed data. While the agreement was not exact, it was though to be sufficient and the predictions did not exhibit consistent overestimation or underestimation.

3. The concept of the effective number of lanes was used as a measure of the increase in capacity associated with the conversion of a two-lane entry from straight to flared. It was found that the effective number of lanes is a function of the circulating flow and, secondarily, of the critical gap. The full use of the extra lanes provided by flaring, is only achieved at very high circulating flows. A value of 50% increase in capacity has been achieved by all studied conditions for a circulating flow of 2300 veh/hr approximately.

4. Reduction to delay associated with conversion from straight to flared entries is on average 40% or more for

operation below and around capacity.

5. Delay increases only slowly with the circulating flow as long as the entry is not operating at or near capacity.

6. The turning proportions of the entering flow were introduced into the simulation model and the effect on entry flow and delay was studied. In below-capacity operation the turning proportion is not affecting the entering flow for both flared and straight entries. At above-capacity operation, turning proportion does not affect the capacity of straight entries but it does affect the capacity of flared entries. Highest capacity is obtained for the maximum left-turning proportion and lowest for zero left-turn proportion; the difference being of the range 25 - 30%. Turning proportions affect delays for both flared and straight entries. Minimum delay was obtained for combinations which include a left-turning proportion of 30 - 40% and maximum delay for zero left-turn.

7. This work could not conclude in the production of a mathematical relation between the parameters studied. Further it did not study the effects of the proportion of heavy goods vehicles in the flows or of gradient on roundabout performance. Such study would entail the collection of more data and has to be left for further work in this area. Finally, it must be noted that despite the advantages computer simulation has (mentioned in Chapter 5), its major disadvantage is that one is never certain that such a model is consistently behaving as it

should. Therefore any change in input parameters outside the tested range can make evident shortcomings of the model.

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APPENDIX 1

THE OBSERVED DATA COLLECTED AT THREE ROUNDABOUTS IN SHEFFIELD

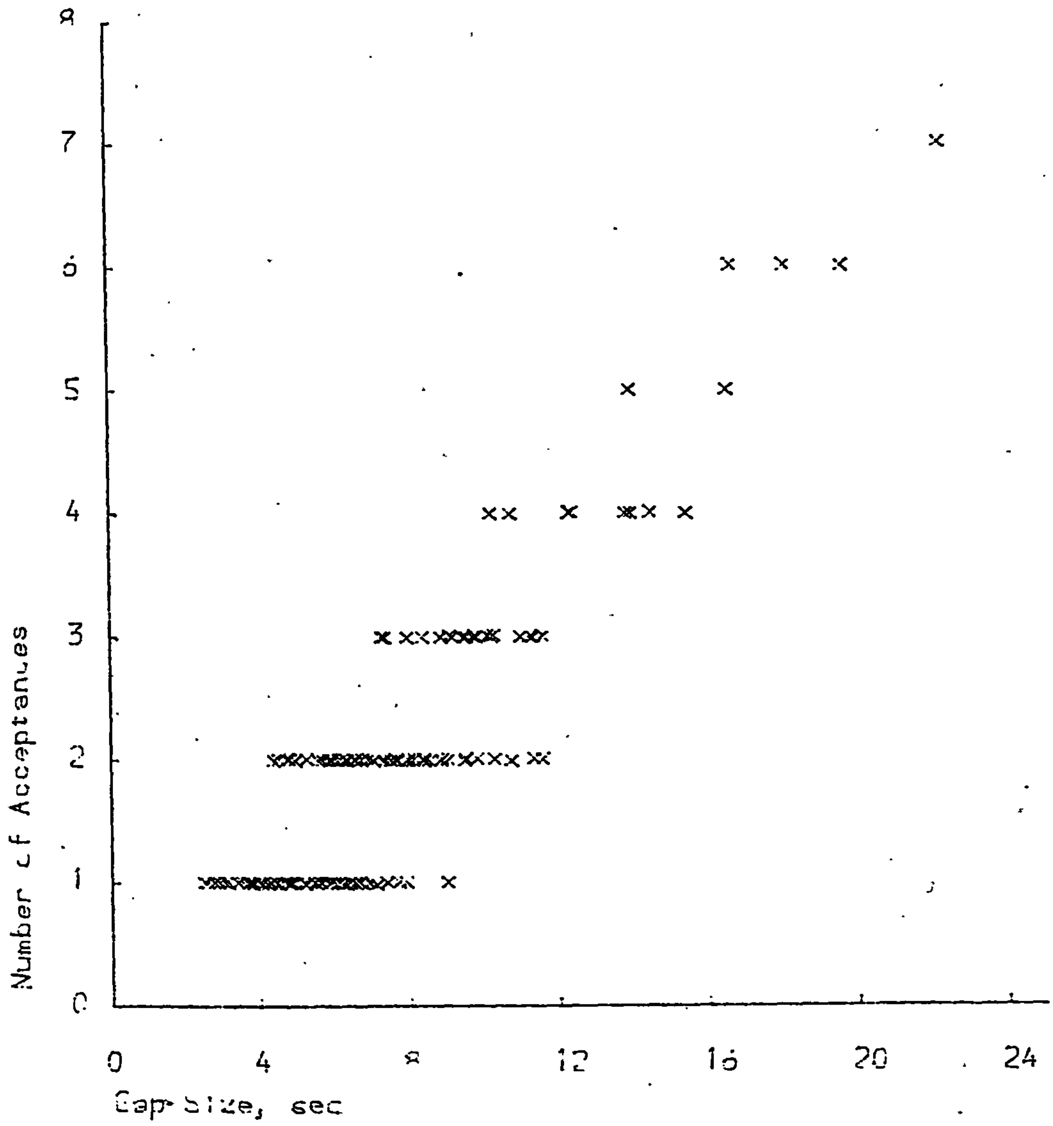

Figure A.1 *Observations at Moore St. Roundabout*

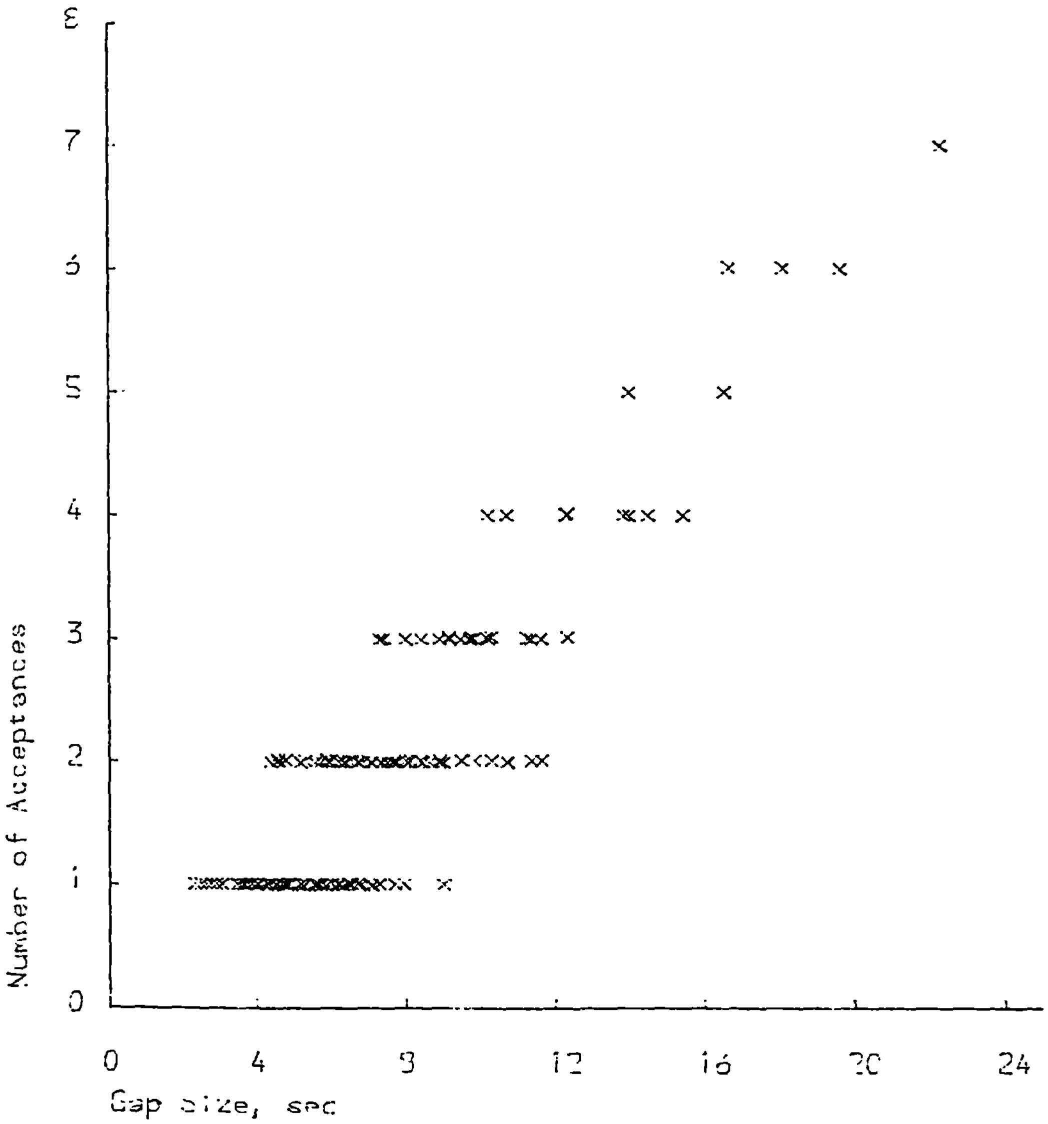

Figure A.2 *Observations at Castle Sq. Roundabout*

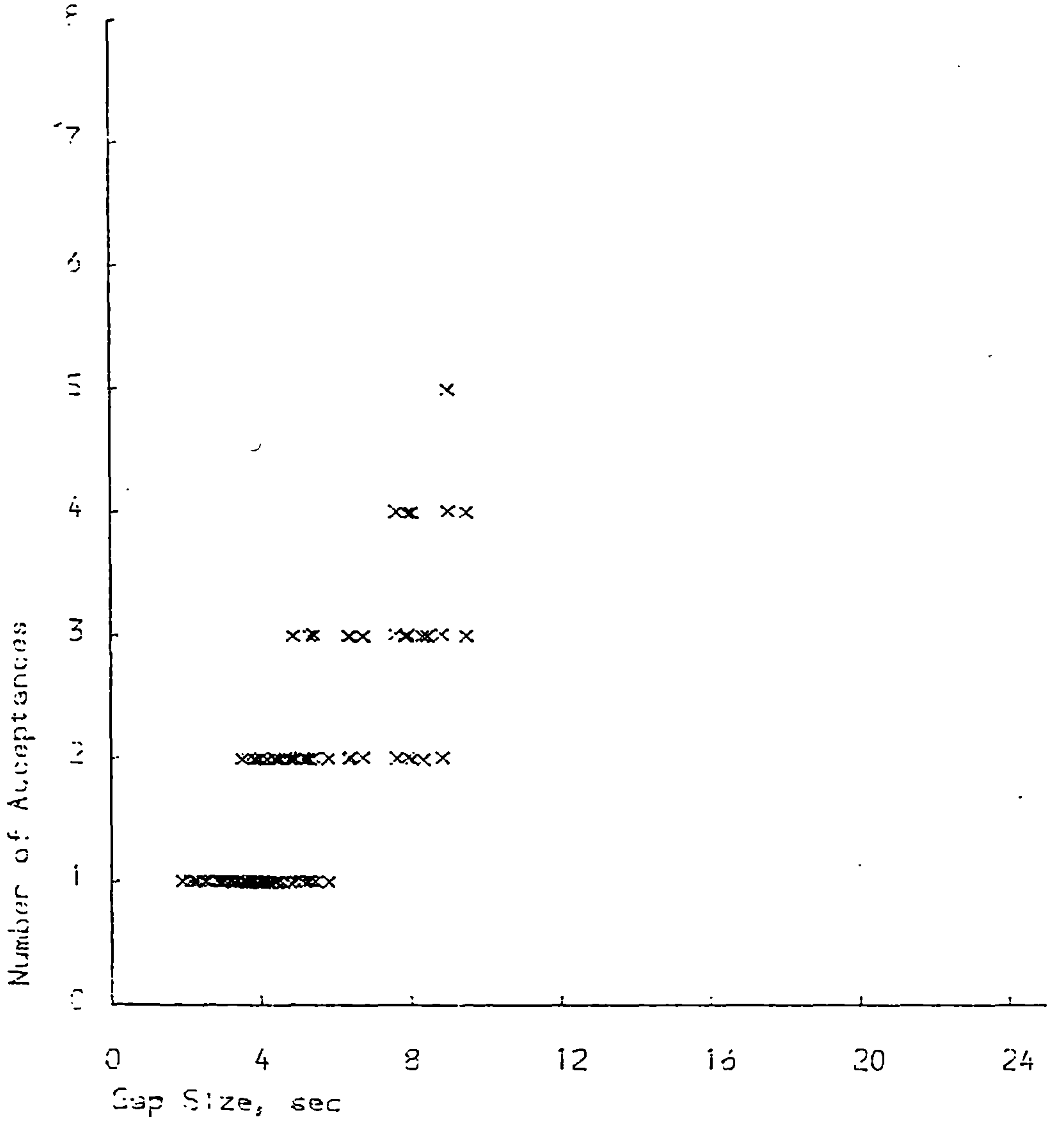

Figure A.3 Observations at Park Sq. Roundabout

APPENDIX 2

THE COMPUTER SIMULATION PROGRAMS

APPENDIX 2a

```
      DOUBLE PRECISION R
      INTEGER*4 I
      A = 2.95
      B = 2.42
      I = 4.91
      Q = 0.44
      TAU = 1.0
1     CALL RANDOM (I,R)
      ITT = ITT + 1
      H = TAU + (1./Q - TAU) * ALOG(1/R)
      IF(H.LT.A)GOTO 5
      IF(H.GE.A.AND A.LT. A+B) IE = 1
      IF(C.GE. A+B) GOTO 3
      GOTO 4
3     FH = H - A
      FD = FH/B
      IE = FD + 1
4     W = EXP(Q*(T-TAU)/(1.-Q*TAU) )
      MTA = MTA + 1
      IF(MTA.GT.500)MTA = 1
      WRITE(6,2)IE,H,W,ITT,MTA
2     FORMAT(I6,F6.2,F7.2,2I7)
5     IF(ITT.GT.16834.AND.MTA.EQ.500)GOTO 6
      GOTO 1
6     CALL EXIT
      END
      SUBROUTINE RANDOM
      DOUBLE PRECISION R
      INTEGER*4 I
      I = 125+I
      I = MOD(I,65536)
      R = I/65536
      RETURN
      END
```

Computer Simulation Program for Single Entry to Roundabout

APPENDIX 2b

Computer Simulation Program SFHT

```
      DIMENSION K1(20,4),T21(20,4),DELAY(20,4),T3(20,4),T4(4),
     1 T21L(4),T2AP(4),EN(4),LN(20,4),GPNO(4),GAPP(4,500,2),
     2 CHGP(4),IDLA(4),IDGP(4,3),MT(3),MR(3),PF(3,4,3),LNM(3,3,2),
     3 WW(4),CA(4),NT(4),T3L(4),T3S(4),LCSUM(3),TDLN(4),TDTM(3),
     4 AVDLN(4),AVDTM(3),CLPP(3),STOP(2)
      REAL*8 RJ,NT
      INTEGER*4 I9,J9,K9,T1CT
      INTEGER SX,A1,AN,AP,AL,WW,AX,AP1,A2,ALL,GPNO,GPJ,PF,STOP
      PARAMETER (MIN=1,MAX=2,NTO=10,NTO1=5)
      DATA  K1/42*1,18*0,1,19*0/,SX/1/,MCH/0/,T1CT/0/
      READ (5,501) Q1,Q2,TAU,ALPHA,BETA,I9,J9,K9,NS,AN,AP,AL,OTL
      WRITE (6,502) Q1,Q2,TAU,ALPHA,BETA,I9,J9,K9,NS,AN,AP,AL
501   FORMAT(2F5.0,3F5.2,4I6,3I2,F5.0)
502   FORMAT(/' Q1   Q2    TAU ALPHA  BETA     I     J     K    NS'/
     1'V/H  V/H    SEC   SEC   SEC',21X,'SEC'//2F5.0,3F6.2,4I6/
     2 //'The configuration of the entry is '/
     3 'AN = ',I2,' AP = ',I2,' AL = ',I2/)
      IF (NS.GT.4000) GOTO 120
      READ (5,503) PR,PS,PL,(((LNM(IZ,JZ,KZ),IZ=1,3),JZ=1,3),KZ=1,2)
     1 ,(MT(IZ),IZ=1,3),(MR(IZ),IZ=1,3)
     2 ,(((PF(IZ,JZ,KZ),JZ=1,4),IZ=1,3),KZ=1,3)
      WRITE(6,504) PR,PS,PL,(((LNM(IZ,JZ,KZ),IZ=1,3),JZ=1,3),KZ=1,2)
     1 ,(MT(IZ),IZ=1,3),(MR(IZ),IZ=1,3)
     2 ,(((PF(IZ,JZ,KZ),JZ=1,4),KZ=1,3),IZ=1,3)
503   FORMAT(3F8.5,18I2/6I2/(4I2))
504   FORMAT(//'The proportions of the turning movements are'/
     1 'Right turn',F6.2,'   Straight',F6.2,'   Left turn',F6.2//
     2 'the  lanes they can use are'/
     3 'Minimum    ',9I4/'Maximum    ',9I4/
     4 'MT matrix = ',3I3,'  MR matrix = ',3I3//
     5 'Preference Matrices for each turning movement'/
     6 ' Right turn   Straight     Left turn'/(2X,4I2,5X,4I2,5X,4I2))
10    CALL RANDOM(I9,RJ)
      H1 = TAU+(3600/Q1-TAU)*ALOG(1/RJ)
      T1 = T1+H1
      IF (T1.GT.301.AND.T1.LT.NS) T1CT = T1CT+1
      IF (T1.GT.OTL) MCH=1
      IF (H1.LT.ALPHA) GOTO 11
      NGO = 0
      DO 12 M3=1,AP
```

```
      GPNO(M3) = GPNO(M3)+1
      NOG = GPNO(M3)
      GAPP(M3,NOG,1) = T1-H1
      GAPP(M3,NOG,2) = T1
      IF (NOG.GT.NGO) NGO = NOG
12    CONTINUE
      IF (MCH.EQ.1) WRITE(6,511)
     1 (((GAPP(IZ,JZ,KZ),KZ=1,2),IZ=1,2),JZ=1,NGO)
11    IF (MCH.EQ.1) WRITE(6,510) H1,T1,T1CT
510   FORMAT('H1  ',F8.4,'  T1  ',F10.2,'  T1CT  ',I6)
511   FORMAT(//'Gaps greater than alfa'/(2F7.2,3X,2F7.2))
      DO 20 M2=1,AN
        IF (K1(1,M2).EQ.2) GOTO 56
20    CONTINUE
      IF (MCH.EQ.1) WRITE(6,531) IGEN
531   FORMAT('IGEN = ',I4)
31    IF (IGEN) 30, 30, 44
30    CALL RANDOM(J9,RJ)
      H2 = (3600/Q2)*ALOG(1/RJ)
      T2 = T2+H2
      IF (T2.GT.NS) GOTO 120
      IF (T1.GT.OTL) MCH=1
      DO 32 M5=1,AP
        LPP = GPNO(M5)
        LPH = 0
        IF (LPP.EQ.0) GOTO 35
        DO 33 M4=1,LPP
          IF (GAPP(M5,M4,2).GT.T2) GOTO 35
          LPH = LPH+1
33      CONTINUE
35      IF (LPH.EQ.0) GOTO 32
        DO 34 M6=1,LPP
          GAPP(M5,M6,1) = GAPP(M5,M6+LPH,1)
          GAPP(M5,M6,2) = GAPP(M5,M6+LPH,2)
34      CONTINUE
        GPNO(M5) = GPNO(M5)-LPH
        CHGP(M5) = GAPP(M5,1,2)
32    CONTINUE
      IGEN = 1
      CALL RANDOM(K9,RJ)
```

```
      IF (RJ.LE.PR) LAN=1
      IF (RJ.GT.PR.AND.RJ.LE.PR+PS) LAN=2
      IF (RJ.GT.PR+PS) LAN=3
      IF (MCH.EQ.1) WRITE(6,530) M2,T2,LAN,T1
530   FORMAT(4X,'M2',8X,'T2 LAN',8X,'T1'/F6.4,F10.2,I4,F10.2)
44    DO 40 M2=1,NTO
        M5 = M2
        IF (M2.GT.AL) M5 = AL
        MIA = LNM(M5,LAN,MIN)
        MAA = LNM(M5,LAN,MAX)
        IF (MCH.EQ.2) WRITE(6,542) LAN,M2,MIA,MAA
542     FORMAT('turn row   min max lanes'/2I4,2X,2I4)
        MXR = MAA-MIA+1
        DO 40 M4=1,MXR
          M3 = PF(M5,M4,LAN)
          MST = M3
          IF (MST.GT.AP) MST = AP
          IF (STOP(MST).EQ.1.AND.K1(NTO,MST).EQ.2) GOTO 47
          IF (K1(M2,M3).NE.1) GOTO 40
          IF (M3.LT.AP.OR.M2.GE.AL) GOTO 46
          IF (K1(M2+1,M3).NE.1.AND.K1(M2+1,M3-1).NE.1) GOTO 40
46        IF (MCH.EQ.1) WRITE(6,540) (T4(IZ),IZ=1,AN)
540       FORMAT('T4 MATRIX'/4F8.2)
          IF (M2.GT.1.OR.MXR.EQ.1) GOTO 41
          TMIN=T4(M3)
          IQ = M3
          DO 42 N2=1,MXR
            A2 = PF(M2,N2,LAN)
            IF (T4(A2).GE.TMIN) GOTO 42
            TMIN = T4(A2)
            IQ = A2
42        CONTINUE
          M3 = IQ
41        SX = M2
          A1 = M3
          GOTO 43
40     CONTINUE
45     IGEN = 1
       GOTO 10
47     GOTO 30
```

```
43    T21(SX,A1) = T2
      IF (A1.GE.AP) AX = AP
      IF (A1.LT.AP) AX = A1
      IF (T2.GT.90000.AND.T21(SX,A1).LT.T2AP(AX)+2.)
     1 T21(SX,A1) = T2AP(AX)+2.
      IF (T21(SX,A1).GT.T1) GOTO 45
      M1 = M1+1
      K1(SX,A1) = 2
      LN(SX,A1) = LAN
      T2AP(AX) = T21(SX,A1)
      IF (A1.GE.AP) LAP = LAP+1
      IGEN = 0
      IF (MCH.EQ.1) WRITE(6,541) SX,A1,K1(SX,A1),LN(SX,A1),T21(SX,A1)
541   FORMAT('POSITION ASSIGNED TO EV'/
     1'   S   A  K1(S,A)  LN(S,A)  T21(S,A)'/2I4,2I9,F10.2)
      IF (SX.EQ.1) GOTO 51
      IF (A1.LT.AP) GOTO 50
      IF (A1.GT.AP.OR.SX.GE.AL) GOTO 158
      DO 157 M7=SX,AL
        IF (K1(M7,A1).EQ.2) GOTO 50
157   CONTINUE
158   IF (T21(SX,A1).LT.T3LAP+BETA) T3(SX,A1) = T3LAP+BETA
      IF (T21(SX,A1).GE.T3LAP+BETA) T3(SX,A1) = T21(SX,A1)
      IF (T3(SX,A1).LT.T3(SX-1,A1)+BETA) T3(SX,A1) = T3(SX-1,A1)+BETA
      T3LAP = T3(SX,A1)
      GOTO 151
50    IF (T21(SX,A1).LT.T3(SX-1,A1)+BETA) T3(SX,A1) = T3(SX-1,A1)+BETA
      IF (T21(SX,A1).GE.T3(SX-1,A1)+BETA) T3(SX,A1) = T21(SX,A1)
      GOTO 151
51    IF (T4(A1).GE.T21(SX,A1)) T3(SX,A1) = T4(A1)
      IF (T4(A1).LT.T21(SX,A1)) T3(SX,A1) = T21(SX,A1)
      IF (A1.LT.AP) GOTO 54
      IF (A1.GT.AP) GOTO 52
      DO 152 M8=2,AL
        IF (K1(M8,A1).EQ.2) GOTO 156
152   CONTINUE
      GOTO 52
156   IF (K1(1,A1-1).EQ.1) GOTO 153
      M00 = 1
      DO 154 M0=2,AL
```

- - -

```
          IF (K1(M0,A1-1).EQ.2) GOTO 154
          M00 = M0-1
          GOTO 155
154     CONTINUE
155     IF (T3(M00,A1-1).GT.T3(1,A1)) T3(1,A1) = T3(M00,A1-1)+BETA
        GOTO 52
153     IF (T4(A1-1).GT.T3(1,A1)) T3(1,A1) = T4(A1-1)
52      DO 53 M2=AP,AN
          IF (T4(M2).LT.T21(SX,A1)+BETA) T4(M2) = T21(SX,A1)+BETA
53      CONTINUE
54      T4(A1) = T4(A1)+1000
151     M3 = A1
        IF (M3.GT.AP) M3 = AP
150     IF (GAPP(M3,1,2).EQ.0.0) GOTO 55
        IF (T3(SX,A1)+ALPHA.GT.GAPP(M3,1,2)) GOTO 59
        IF (GAPP(M3,1,1).GT.T3(SX,A1)) T3(SX,A1) = GAPP(M3,1,1)
        IF (M3.GE.AP.AND.T3(SX,A1).GT.T3LAP) T3LAP = T3(SX,A1)
        GOTO 55
59      GPJ = GPNO(M3)
        IF (GPJ.EQ.0) GPJ = 1
        DO 58 NJ=1,GPJ
          GAPP(M3,NJ,1) = GAPP(M3,NJ+1,1)
          GAPP(M3,NJ,2) = GAPP(M3,NJ+1,2)
58      CONTINUE
        GPNO(M3) = GPNO(M3)-1
        GOTO 150
55      DO 57 IZ=1,NT01
          IF(MCH.EQ.1)WRITE(6,601)(K1(IZ,JZ),JZ=1,AN),(LN(IZ,JZ),JZ=1,AN)
57      CONTINUE
56      IF (MCH.EQ.1) WRITE(6,551) T3(SX,A1),T3LAP,(T4(IZ),IZ=1,AN)
551     FORMAT('T3(S,A)=',F8.2,' T3LAP=',F8.2/'T4 MATRIX = ',4F8.2)
        DO 60 M2=1,NTO
          DO 60 M3=1,AN
            IF (K1(M2,M3).NE.2) GOTO 60
            IF (T1.LT.T3(M2,M3)+ALPHA) GOTO 60
            IC = IC+1
            K1(M2,M3) = 3
            IF (T3L(M3).LT.301.OR.T3L(M3).GT.NS) GOTO 60
            LNA = LN(M2,M3)
            LCSUM(LNA) = LCSUM(LNA)+1
```

```
          DELAY(M2,M3) = T3(M2,M3)-T21(M2,M3)
          TOTD = TOTD+DELAY(M2,M3)
          TDLN(M3) = TDLN(M3)+DELAY(M2,M3)
          TDTM(LNA) = TDTM(LNA)+DELAY(M2,M3)
          NT(M3) = T3L(M3)
          IF (LNA.NE.3.OR.M2.EQ.1) GOTO 61
          EN(AN) = EN(AN)+1
          GOTO 60
61        EN(M3) = EN(M3)+1
60      CONTINUE
        IF (IC.GT.0) GOTO 80
70      DO 71 M2=1,NTO
          DO 71 M3=1,AN
          IF (K1(M2,M3).NE.2) GOTO 71
          IF (M2.NE.1) GOTO 72
          IF (T1.GT.T3(M2,M3)) T3(M2,M3) = T1
          IF (T3(M2,M3).GT.NS.AND.M3.LE.AP) STOP(M3) = 1
          IF (T3(M2,M3).GT.NS.AND.M3.GT.AP) STOP(AP) = 1
          GOTO 71
72        IF (M3.LT.AP) GOTO 76
          DO 75 N3=M3,AN
            IF (LN(M2-1,N3).EQ.LN(M2,M3)) GOTO 74
75        CONTINUE
          N3 = M3
74        IF (T3(M2,M3).LT.BETA+T3(M2-1,N3)) T3(M2,M3) =
     1     T3(M2-1,N3)+BETA
          GOTO 71
76        IF (T3(M2,M3).LT.BETA+T3(M2-1,M3)) T3(M2,M3) =
     1     T3(M2-1,M3)+BETA
71      CONTINUE
        IF (MCH.EQ.1) WRITE(6,570) T3LAP,((T3(IZ,JZ),JZ=1,AN),IZ=1,NT01)
570     FORMAT('T3LAP=',F8.2/'MATRIX OF UPDATED T3'/4F8.2/(4F8.2))
        DO 77 II=1,AP
          IF (STOP(II).EQ.0) GOTO 78
77      CONTINUE
        GOTO 120
78      IF (MCH.EQ.1) WRITE(6,531) IGEN
        IF (IGEN) 30, 30, 44
80      JB = 0
        ID = 0
```

```
      IF (MCH.EQ.1) WRITE(6,580)
580   FORMAT('RECORD AND CALCULATE PARAMS FOR ALL EVS'/
     1'  JB  ID       T3       T2 DELAY       TOTD',
     2 '  ROW LANE    LN')
      DO 81 M2=1,NTO
        DO 82 M3=1,AN
          IF (K1(M2,M3).NE.3) GOTO 82
          JB = JB+1
          K1(M2,M3) = 1
          ID = ID+1
          T21L(M3) = T21(M2,M3)
          LZ = LN(M2,M3)
          N3 = M3
          IXG = 1
          IF (M3.GT.AP) N3 = AP
          IF (T3(M2,M3).GT.GAPP(N3,1,2)) IXG = 2
          IF (DELAY(M2,M3).EQ.0.00) GOTO 182
          IF (CHGP(N3).EQ.GAPP(N3,IXG,2)) GOTO 88
          CHGP(N3) = GAPP(N3,IXG,2)
182       DO 180 LMZ=1,3
            IDGP(N3,LMZ) = 0
180       CONTINUE
          IDGP(N3,LZ) = 1
          IDLA(N3) = 1
          GOTO 89
88        IDGP(N3,LZ) = IDGP(N3,LZ) + 1
          IDLA(N3) = IDLA(N3)+1
89        IF (M3.LT.AP.OR.AP.EQ.AN) GOTO 83
          IF (IDGP(N3,LZ).LE.MT(LZ).AND.IDLA(N3).LE.MR(N3)) GOTO 83
          AP1 = AP+1
          DO 84 ILF=AP1,AN
            T4(ILF) = T3(M2,M3)+BETA
            IF (T4(ILF)+BETA.GT.T1) T4(ILF) = T1
84        CONTINUE
83        T4(M3) = T3(M2,M3)+BETA
          IF (T4(M3)+BETA.GT.T1) T4(M3) = T1
          T3L(M3) = T3(M2,M3)
          IF (MCH.EQ.1) WRITE(6,581) JB,ID,T3(M2,M3),T21(M2,M3),
     1     DELAY(M2,M3),TOTD,M2,M3,LN(M2,M3),IDLA(N3),IDGP(N3,LZ)
581       FORMAT(2I4,2F8.2,F6.2,F10.2,2I5,I5/
```

```
    1     'sum entries of lane for gap :',I5/
    2     'sum entries of turn for gap :',I5)
          T3(M2,M3) = 0.0
          T21(M2,M3) = 0.0
          LN(M2,M3) = 0
          IF (ID.EQ.IC) GOTO 85
82      CONTINUE
        JB=0
81    CONTINUE
85    IC=0
      IF (MCH.EQ.1) WRITE(6,582) (EN(IZ),IZ=1,AN)
582   FORMAT('Lane totals'/4F10.2)
      DO 181 N3=1,AN
        IF (WW(N3).EQ.0.AND.T3L(N3).GE.301) GOTO 86
        GOTO 181
86      WW(N3) = 1
        T3S(N3) = T3L(N3)
181   CONTINUE
87    IF (MCH.EQ.1) WRITE(6,570) T3LAP,((T3(IZ,JZ),JZ=1,AN),IZ=1,NT01)
      IF (MCH.EQ.1) WRITE(6,583) ((LN(IZ,JZ),JZ=1,AN),IZ=1,NT01)
583   FORMAT('Matrix of turn moves'/4I2/(4I2))
      IF (T2.GT.NS) GOTO 120
      IF (PR.EQ.1.00.OR.PL.EQ.1.00.OR.PS.EQ.1.00) GOTO 92
      DO 94 LPF=1,3
        IF (LPF.EQ.1) PC = PR
        IF (LPF.EQ.2) PC = PS
        IF (LPF.EQ.3) PC = PL
        IF (PC.EQ.0.00) GOTO 94
        DO 90 N3=1,AN
          LQQ = PF(1,N3,LPF)
          IF (LQQ.EQ.0) GOTO 90
          IF (MCH.EQ.1) WRITE(6,590) (T3L(IZ),IZ=1,AN)
590       FORMAT('T3L of lanes 1,2,3,4 : ',4F10.2)
          IF (T3L(LQQ).LT.NS) GOTO 91
90      CONTINUE
94    CONTINUE
      GOTO 120
92    DO 93 N3=1,AN
        IF (T3L(N3).GT.NS) GOTO 120
93    CONTINUE
```

```
91    IF (M1-ID.GT.0) GOTO 110
      M1 = 0
      LAP = 0
      SX = 1
      GOTO 31
110   ALL = NTO-1
      DO 103 M2=1,ALL
        DO 103 M3=1,AN
          IF (K1(M2,M3).NE.1) GOTO 103
          NN2 = M2+1
          N3 = M3
          DO 102 N2=NN2,NTO
117         IF (K1(N2,N3).EQ.1) GOTO 102
            IF (K1(N2,N3).EQ.0.AND.M3.NE.LNM(M2,3,2)) GOTO 103
            IF (K1(N2,N3).EQ.0.AND.M3.EQ.LNM(M2,3,2)) GOTO 116
            LC = LN(N2,N3)
            M5 = M2
            M4 = M3
            IF (M2.GT.AL) M5 = AL
            IF(M3.LT.LNM(M5,LC,1).OR.M3.GT.LNM(M5,LC,2))GOTO 103
            IF (LC.NE.3.OR.N2.NE.2) GOTO 104
            IF (K1(1,4).EQ.1) M4 = 4
            GOTO 104
116         N3 = N3-1
            IF (N3.LT.1) GOTO 103
            GOTO 117
102       CONTINUE
          GOTO 103
104       K1(M2,M4) = 2
          K1(N2,N3) = 1
          T21(M2,M4) = T21(N2,N3)
          T3(M2,M4) = T3(N2,N3)
          LN(M2,M4) = LN(N2,N3)
          T21(N2,N3) = 0.0
          T3(N2,N3) = 0.0
          LN(N2,N3) = 0
103   CONTINUE
      DO 109 M2=2,NTO
        DO 109 M3=1,AN
          IF (K1(M2,M3).NE.2) GOTO 109
```

```
          GOTO 107
108       IF (N3.LT.NMN) LCH = LCH+1
          IF (N3.GT.NMX) LCH = LCH+1
          IF (LCH.EQ.2) GOTO 109
106     CONTINUE
109   CONTINUE
      DO 101 IZ=1,NTO1
      IF(MCH.EQ.1)WRITE(6,601)(K1(IZ,JZ),JZ=1,AN),(LN(IZ,JZ),JZ=1,AN)
601   FORMAT(4I2,4X,4I2)
101   CONTINUE
      M1 = M1-ID
      LAP = 0
      DO 121 M2=1,AN
        IF (K1(1,M2).EQ.1) GOTO 121
        IF (M2.GE.AP) LAP = LAP+1
        T4(M2) = T3(1,M2)+1000+BETA
        IF (M2.LT.AP) GOTO 121
        DO 122 N2=M2,AN
          IF (K1(1,N2).EQ.2) GOTO 122
          DO 123 N3=2,NTO
            IF (K1(N3,M2).EQ.1) T4(N2) = T3(1,M2)+BETA
            IF (K1(N3,M2).EQ.2) T4(N2) = T3(N3,M2)+BETA
            IF (T4(N2)+BETA.GT.T1) T4(N2) = T1
123       CONTINUE
122     CONTINUE
121   CONTINUE
      GOTO 70
120   AVERD = TOTD/(EN(1)+EN(2)+EN(3)+EN(4))
      DO 126 M2=1,AN
        IF (EN(M2).EQ.0) GOTO 126
        AVDLN(M2) = TDLN(M2)/EN(M2)
126   CONTINUE
      DO 127 M2=1,3
        LCSTO = LCSTO+LCSUM(M2)
        IF (LCSUM(M2).EQ.0) GOTO 127
        AVDTM(M2) = TDTM(M2)/LCSUM(M2)
127   CONTINUE
      DO 125 N3=1,AN
        IF ((NT(N3)-T3S(N3)).EQ.0.0) GOTO 125
        CA(N3) = EN(N3)*3600./(NT(N3)-T3S(N3))
```

```
      LB = LN(M2,M3)
      MM2 = M2
      MM3 = M3
      N2 = M2
107   N2 = N2 - 1
      IF (N2.LT.1) GOTO 109
      N5 = N2
      IF (N2.GT.AL) N5 = AL
      NMN = LNM(N5,LB,1)
      NMX = LNM(N5,LB,2)
      NDN = MM3-(MM2-N2)
      NDX = MM3+(MM2-N2)
      IF (NMN.LT.NDN) NMN = NDN
      IF (NMX.GT.NDX) NMX = NDX
      LCH = 0
      MAN = AN*2
      DO 106 LFP=1,MAN
        IF (N2.EQ.1.AND.LB.EQ.3) GOTO 112
        N3 = MM3+LFP/2*(-1)**(LFP)
        IF (N3.LT.NMN.OR.N3.GT.NMX) GOTO 108
        GOTO 113
112     N3 = AN+1
114     N3 = N3-1
        IF (N3.LT.NMN) GOTO 109
        IF (K1(N2,N3).NE.1) GOTO 114
        GOTO 115
113     IF (K1(N2,N3).NE.1) GOTO 106
115     K1(N2,N3)= 2
        K1(MM2,MM3) = 1
        T21(N2,N3) = T21(MM2,MM3)
        T3(N2,N3) = T3(MM2,MM3)
        LN(N2,N3) = LN(MM2,MM3)
        T21(MM2,MM3) = 0.0
        T3(MM2,MM3) = 0.0
        LN(MM2,MM3) = 0
        MM2 = N2
        MM3 = N3
        DO 111 IZ=1,5
     IF(MCH.EQ.1)WRITE(6,601)(K1(IZ,JZ),JZ=1,4),(LN(IZ,JZ),JZ=1,4)
111     CONTINUE
```

```
        CAP = CAP+CA(N3)
125   CONTINUE
      DO 128 MX5=1,3
        CLPP(MX5) = FLOAT(LCSUM(MX5))/FLOAT(LCSTO)
128   CONTINUE
      WRITE (6,622) T1CT
622   FORMAT ('The total number of circulating vehs :',I5)
      WRITE (6,623)
623   FORMAT ('Per Lane',12X,'Per Turn'/
     1 'Lane   Delay  Capac.  Turn  Veh.     %   Delay')
      DO 124 M2=1,AN
        IF (M2.GT.3) GOTO 129
        WRITE(6,620) M2,AVDLN(M2),CA(M2),M2,LCSUM(M2),CLPP(M2),AVDTM(M2)
        GOTO 124
129     WRITE(6,620) M2,AVDLN(M2),CA(M2)
620     FORMAT(I4,F8.2,F8.2,2I6,F6.3,F8.2)
124   CONTINUE
      WRITE (6,621) AVERD,CAP,LCSTO
621   FORMAT(' Tot',2F8.2,I12)
      STOP
      END
      SUBROUTINE RANDOM(I9,RJ)
      REAL*8 RJ
      I9=125*I9
      I9=MOD(I9,65536)
      RJ=I9/65536.
      RETURN
      END
```

APPENDIX 3

DIFFERENCE IN DELAY OBTAINED AT STRAIGHT AND FLARED ENTRIES

			Q_1=500veh/hr, Q_2=500 veh/hr			Q_1 = 500 veh/hr, Q_2= 1500veh/hr		
Turning Proportion			Delay (sec)		Difference	Delay (sec)		Difference
RT	ST	LT	Straight	Flared	%	Straight	Flared	%
0.00	1.00	0.00	0.73	0.64	14.1	4.57	2.32	97.0
0.00	0.90	0.10	0.74	0.65	13.8	4.66	2.11	120.9
0.05	0.90	0.05	0.74	0.65	13.8	4.65	2.20	111.4
0.10	0.90	0.00	0.73	0.66	10.6	4.61	2.40	92.1
0.00	0.75	0.25	0.76	0.67	13.4	4.93	2.07	138.2
0.05	0.75	0.20	0.75	0.67	11.9	4.82	2.09	130.6
0.10	0.75	0.15	0.75	0.67	11.9	4.76	2.17	119.4
0.15	0.75	0.10	0.74	0.69	7.2	4.69	2.31	103.0
0.25	0.75	0.00	0.75	0.72	5.6	4.74	2.75	72.4
0.00	0.60	0.40	0.80	0.68	17.6	5.65	2.33	142.5
0.10	0.60	0.30	0.78	0.68	14.7	5.17	2.25	129.8
0.20	0.60	0.20	0.76	0.71	9.9	4.90	2.35	108.5
0.30	0.60	0.10	0.77	0.74	4.1	4.93	2.76	78.6
0.40	0.60	0.00	0.81	0.79	2.5	5.65	2.79	102.5
0.00	0.50	0.50	0.85	0.70	21.4	7.27	2.64	175.4
0.10	0.50	0.40	0.82	0.70	17.1	5.67	2.40	136.3
0.25	0.50	0.25	0.79	0.74	6.8	5.00	2.53	97.6
0.40	0.50	0.10	0.81	0.79	2.5	5.63	3.59	56.8
0.50	0.50	0.00	0.90	0.88	2.3	7.02	5.15	36.3

Table A3.1 Delays at flared and straight entries

			Q_1 = 500 veh/hr, Q_2=2000 veh/hr			Q_1=1000 veh/hr, Q_2 = 1000veh/hr		
Turning Proportion			Delay (sec)		Difference	Delay (sec)		Difference
RT	ST	LT	Straight	Flared	%	Straight	Flared	%
0.00	1.00	0.00	46.53	4.86	857.4	9.91	5.27	88.0
0.00	0.90	0.10	46.65	3.66	1174.6	10.10	4.90	106.1
0.05	0.90	0.05	46.53	4.10	1034.9	9.92	5.11	94.1
0.10	0.90	0.00	46.53	4.99	832.5	10.03	5.41	85.4
0.00	0.75	0.25	46.68	3.19	1363.3	10.58	4.33	114.3
0.05	0.75	0.20	46.65	3.30	1313.6	10.16	4.58	121.8
0.10	0.75	0.15	46.60	3.49	1235.2	10.20	4.80	112.5
0.15	0.75	0.10	46.65	3.87	1105.4	10.16	5.12	98.4
0.25	0.75	0.00	46.70	5.36	771.3	10.28	5.71	80.0
0.00	0.60	0.40	47.60	3.42	1291.8	11.52	4.52	154.9
0.10	0.60	0.30	47.06	3.25	1348.0	10.79	4.60	134.6
0.20	0.60	0.20	46.76	3.59	1202.5	10.28	4.78	115.1
0.30	0.60	0.10	47.29	4.39	977.2	10.65	5.72	86.2
0.40	0.60	0.00	47.84	9.13	424.0	11.64	7.71	51.0
0.00	0.50	0.50	56.25	4.47	1158.4	13.44	4.90	176.5
0.10	0.50	0.40	47.80	3.55	1246.5	11.51	4.71	144.4
0.25	0.50	0.25	47.30	3.57	1224.9	10.49	4.98	110.6
0.40	0.50	0.10	47.53	7.84	506.3	11.53	7.41	56.3
0.50	0.50	0.00	69.25	51.61	34.2	15.35	11.32	35.6

Table A3.2 Delays at flared and straight entries

			Q_1=1000veh/hr, Q_2=1500veh/hr			Q_1 = 1000veh/hr, Q_2=2000veh/hr		
Turning Proportion			Delay (sec)		Difference	Delay (sec)		Difference
RT	ST	LT	Straight	Flared	%	Straight	Flared	%
0.00	1.00	0.00	3.08	2.01	53.2	8.86	3.07	188.6
0.00	0.90	0.10	3.21	1.88	70.7	3.92	2.73	226.7
0.05	0.90	0.05	3.11	1.96	58.7	8.89	2.89	207.6
0.10	0.90	0.00	3.09	2.08	48.6	8.86	3.13	183.1
0.00	0.75	0.25	3.34	1.83	82.5	9.36	2.48	278.6
0.05	0.75	0.20	3.30	1.82	81.3	9.21	2.55	259.8
0.10	0.75	0.15	3.24	1.91	69.6	8.97	2.72	229.8
0.15	0.75	0.10	3.22	1.99	61.8	8.92	2.80	218.6
0.25	0.75	0.00	3.17	2.33	36.1	9.11	3.38	169.5
0.00	0.60	0.40	3.78	1.99	89.9	9.82	2.45	300.8
0.10	0.60	0.30	3.46	1.96	76.5	9.40	2.42	288.4
0.20	0.60	0.20	3.34	2.08	60.6	9.34	2.61	257.9
0.30	0.60	0.10	3.35	2.28	46.9	9.12	3.25	180.6
0.40	0.60	0.00	3.54	2.77	27.8	9.96	5.11	94.9
0.00	0.50	0.50	4.35	2.11	106.2	12.01	3.08	289.9
0.10	0.50	0.40	3.76	1.97	97.0	9.82	2.71	262.4
0.25	0.50	0.25	3.39	2.13	74.6	9.44	2.79	238.4
0.40	0.50	0.10	3.56	2.58	53.5	9.93	3.93	152.7
0.50	0.50	0.00	4.21	3.16	33.2	15.86	6.76	134.6

Table A3.3 Delays at flared and straight entries

			Q_1=2000veh/hr, Q_2= 500 veh/hr			Q_1=2000 veh/hr,Q_2= 1000 veh/hr		
Turning Proportion			Delay (sec)		Difference	Delay (sec)		Difference
RT	ST	LT	Straight	Flared	%	Straight	Flared	%
0.00	1.00	0.00	48.46	20.11	141.0	19.35	8.13	138.0
0.00	0.90	0.10	48.53	17.98	169.9	19.68	7.33	168.5
0.05	0.90	0.05	48.47	18.78	158.1	19.46	7.66	154.0
0.10	0.90	0.00	48.46	20.40	137.5	19.35	8.23	135.1
0.00	0.75	0.25	49.25	16.26	202.9	19.82	6.85	189.3
0.05	0.75	0.20	48.63	17.06	185.1	19.69	7.01	180.9
0.10	0.75	0.15	48.60	17.87	172.0	19.68	7.28	170.3
0.15	0.75	0.10	48.51	18.57	161.2	19.61	7.53	160.4
0.25	0.75	0.00	48.48	21.56	124.9	19.41	8.54	127.3
0.00	0.60	0.40	55.34	15.38	259.8	21.00	6.63	216.7
0.10	0.60	0.30	51.71	16.09	221.4	19.83	6.68	196.9
0.20	0.60	0.20	48.64	17.72	174.5	19.68	7.04	179.5
0.30	0.60	0.10	49.94	20.82	139.9	19.81	8.51	132.8
0.40	0.60	0.00	53.23	26.26	102.7	20.32	11.30	79.8
0.00	0.50	0.50	61.37	16.75	266.4	24.70	7.49	229.8
0.10	0.50	0.40	56.34	15.52	263.0	21.00	7.18	192.5
0.25	0.50	0.25	52.25	18.59	181.0	19.94	7.38	170.2
0.40	0.50	0.10	55.25	25.40	117.5	20.46	10.02	104.2
0.50	0.50	0.00	64.66	46.10	40.3	23.20	15.88	46.1

Table A3.4 Delays at flared and straight entries