Universidade do Minho
Escola de Engenharia

Yoimer Miguel Cassiany Acosta

# Evaluation of traffic performance at roundabouts

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**Professor Doutor Paulo Jorge Gomes Ribeiro**

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*"Those who pass by us, do not go alone, and do not leave us alone; they leave a bit of themselves, and take a little of us."- Antoine de Saint-Exupéry*

Primero que todo, agradezco a Dios por todas las oportunidades, por guiarme siempre y por enseñarme que su tiempo es perfecto; todo es posible para quien cree.

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## DECLARAÇÃO DE INTEGRIDADE

Declaro ter atuado com integridade na elaboração do presente trabalho académico e confirmo que não recorri à prática de plágio, nem a qualquer forma de utilização indevida ou falsificação de informações ou resultados em nenhuma das etapas conducente à sua elaboração.

Mais declaro que conheço e que respeitei o Código de Conduta Ética da Universidade do Minho.

## ABSTRACT

Roundabouts have evolved into one of the most common forms of crossings. Their success in resolving traffic issues and enhancing junction performance has been critical to their adoption. In ancient times, roundabouts were the subject of study by many scientists who transformed the traffic principles by which they were governed and today they are considered one of the most widely implemented alternatives worldwide.

Speed reduction, greater safety, shorter wait times, and higher capacity as compared to any other at-grade junction. These characteristics have resulted in their deployment all over the world.

This dissertation aims to study and compare the performance of various models to assess the capacity of a roundabout. Thus, a comparison of the capacity calculation models developed in different countries to estimate the performance of roundabouts is done, relying its assessment on numerous elements ranging from geometric attributes to driving behaviour. Furthermore, several performance measures such as delay, degree of saturation, and others are evaluated, which will allow comparing empirical, analytical and simulation models.

In this work, a microsimulation model for roundabouts with the application of the PTV VISSIM software is applied. Vissim is considered a tool that simplifies road network research and analysis. allowing to analyze the performance of a roundabout through the evaluation of several indicators in different parts of the roundabout such as entries, exits and conflicting carriageway. Finally, a case study of the roundabout performance in the city of Guimaraes, Portugal, under its geometric and traffic features is done based on real traffic data.

**Keywords:** Intersection, roundabout, performance, model, capacity, microsimulation, traffic and geometric characteristics.

## RESUMO

As rotundas têm evoluído para uma das formas mais comuns de cruzamentos. O sucesso das rotundas na resolução de questões de tráfego e na melhoria do desempenho dos cruzamentos, em termos de eficiência e segurança, tem sido crítico para a sua adopção. No inicio da implementação de interseções giratórias, as rotundas eram objecto de estudo por muitos cientistas que alteraram os princípios de trânsito pelos quais eram governadas, passando por alterar a ordem de prioridade para quem entrava circulava no interior da rotunda, que alterou completamente o projeto, dimensionamento e avaliação do desempenho deste tipo de soluções.

Características como a redução da velocidade, a segurança, a redução dos tempos de espera e a maior capacidade em comparação com outros tipos de cruzamento tornaram as rotundas uma excelente solução para serem implementadas em diversos contextos rodoviários, que resultou na sua disseminação e forte implantação em todo o mundo.

Neste contexto, o objectivo desta dissertação é estudar e caracterizar os modelos mais comuns para avaliar o desempenho deste tipo de interseções. Para esse efeito, é feita uma comparação de modelos de cálculo de capacidade desenvolvidos em diferentes países para estimar o desempenho de rotundas, tendo por base a avaliação de diversos elementos geométricos e de circulação do tráfego. No processo comparativo são avaliadas várias medidas de desempenho, tais como o atraso, grau de saturação, comprimento das filas de espera, entre outras.

Neste trabalho são comparados modelos empíricos e analíticos com os modelos de microssimulação. Para este efeito, foi desenvolvido um estudo de caso numa rotunda da cidade de Guimarães, Portugal, tendo sido utilizado o software PTV VISSIM, que é considerada uma ferramenta que simplifica o estudo e análise do funcionamento de uma rede rodoviária, particularmente ao nível das interseções. Os resultados mostraram uma grande semelhança entre os modelos de capacidade empíricos e analíticos, assim como, destes com o modelo de microssimulação.

**Palavras-chave:** Intersecção, rotunda, desempenho, modelo, capacidade, microssimulação, tráfego e características geométricas.

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# CHAPTER I
# INTRODUCTION

*"Transportation is the center of the world! It is the glue of our daily lives. When it goes well, we do not see it. When it goes wrong, it negatively colors our day, makes us feel angry and impotent, curtails our possibilities*".

**Robin Chase**

## 1.1 Problem statement

Intersections have become one of the main problems of the road network. Road intersections tend to be the most critical points as they constitute a focal point for potential traffic accidents caused by conflicts between the different traffic streams and traffic congestion problems. Because of their great efficiency in terms of performance, particularly in traffic flow and road safety, where they may considerably reduce the number of conflict locations, roundabouts have become an alternative to address the primary problems that impact the road network during the last two decades.

The lack of capacity and the increase of traffic flow induce delays, lowering the level of service supplied by road infrastructure. For the evaluation of roundabout capacity, several models have been developed in several countries to predict the maximum rate of flow at the roundabouts when these are exposed to a specific level of service or set of prevailing roadway and traffic conditions.

The methodologies vary depending on the level of details of the model, in most of the cases, they include parameters to describe the driver behavior as well as the roundabout geometric design features. Empirical and analytical models provide expressions/ formulas for calculating the roundabout capacity of each roundabout entry. In terms of microsimulation, models must be calibrated to accurately reflect the operation of the junction.

Recognizing that constant monitoring of significant road problems and the development of improved technological solutions remains a key task in traffic engineering, particularly regarding road design and operation, namely at intersections such as roundabouts.

The present dissertation approaches an evaluation of the roundabout's performance based on the various available assessment models to estimate its capacity, namely the roundabout's entries capacity. Therefore, a case study of a roundabout near the University of Minho, in Guimaraes, Portugal, was conducted to study, analyze, and compare the results obtained from the different performance methods. This roundabout was chosen because of its importance in the traffic network of the city, but more important than that was the characteristics of the place, especially the variation of characteristics for the different entrance branches of the roundabout in terms of its geometric and traffic flow characteristics.

## 1.2 Objectives

### 1.2.1 Main objective

The principal objective of this work is to select and analyze some of the key empirical and analytical roundabout capacity models used to evaluate its performance, as well as the development of a microsimulation model for a specific case study to be able to compare the results of some key performance indicators provided by microsimulation and the empirical and analytical models.

### 1.2.2 Specific objectives

To achieve the main objective of this dissertation, it will be necessary to fulfill the following secondary objectives:

- Define the roundabouts capacity models, considering empirical, analytical and microsimulation methodologies.
- Determine the key indicators for the analysis of the roundabout performance.
- Apply the empirical, analytical, and micro-simulation models in a case study of roundabout in the city of Guimaraes, Portugal.
- Compare and discuss the results obtained in the different methodologies.

## 1.3 Methodology

To choose and analyze some of the main empirical and analytical models used in the evaluation of the capacity and performance at roundabouts, the following working methodology was followed.

Firstly, based on a literature review, it is intended to define the models for estimating the capacity at roundabout intersections considering different methodologies and calculation parameters proposed in each model (empirical, analytical, and microsimulation). According to their characteristics, parameters as well as calculation procedures, models are going to be studied, analyzed, presented, and compared.

Secondly, taking into consideration the methodologies and procedures provided by the studied models, a case study is going to be carried out applying the previously studied models.

Finally, it is intended to perform a microsimulation of the proposed roundabout using the VISSIM software to simulate the behavior of the different traffic flows and to analyze possible scenarios regarding changes in the geometry and/or the evolution of the traffic stream composition and volumes to compare the results obtained from the detailed data analysis carried out.

### 1.4 Structure of the dissertation

This dissertation consists of four chapters to address the theme of capacity and performance at roundabouts. A summary and description of the contents of each chapter are presented as follows:

This first chapter, **introduction**, briefly presents an introductory overview of intersections and the relevance of this intersection in the traffic and urban environment, additionally, are presented the main objectives, structure, and organization of this dissertation.

In the second chapter, **roundabouts**, the subject is contextualized with a historical review of the typologies, geometric and design principles, implementation, and operation of roundabouts and its safety levels associated with traffic speed and traffic control.

The third chapter, **roundabout performance**, presents the three types of models commonly used to evaluate the roundabout capacity, as well as the methodologies for its estimation/ calculation and other parameters to globally analyze the intersection performance, which will serve as a basis for the analysis carried out in the chapter of the case study.

The fourth, **case study**, presents an evaluation of a roundabout that is located near to the Campus of the University of Minho in the city of Guimarães, Portugal, by applying the empirical, analytical and microsimulation models and presenting the respective comparison of the results obtained for each model.

The last chapter, the **conclusions**, are presented the main conclusions of this work and some recommendations for future works.

# CHAPTER II
# ROUNDABOUTS

*“Our decisions about transportation determine much more than where roads or bridges or tunnels or rail lines will be built. They determine the connections and barriers that people will encounter in their daily lives and thus how hard or easy it will be for people to get where they need and want to go”.*

**Elijah Cummings**

## 2.1 Introduction

Roundabouts are commonly suggested as a type of intersection for inclusion in town and city road networks owing to their features and effect on traffic quality. These intersections have been a part of the road network since the XIX century, but they were not utilized and tested until the beginning of the 20th century to enhance their concept and design. The design improvements made to roundabouts over the years benefit not only traffic but also the environment in which they are located. Thus, their attributes and effect on the traffic nature, roundabouts are viewed as a sort of junction generally prescribed to be introduced in the street organization of towns and urban areas.

This chapter provides a historical overview of roundabouts and their most significant changes throughout the years. The literature search and research process provided a vast amount of information about the different types of roundabouts based on key geometric and traffic flow features and take into consideration the parameters that entail country guidelines in the performance of these intersections.

This chapter also discusses the conditions under which roundabouts are used and how they have been integrated into the road network to minimize the traffic points of conflict. The most critical movements at intersections are defined and a comparison of the advantages of roundabouts with other intersections is made.

The most important geometric aspects associated with the evaluation of roundabout performance have been grouped into three main categories: at the entry, within the roundabout, and at the exits to explain the characteristics of each parameter covered by the analysis in chapters 3 and 4.

## 2.2 History review of roundabouts

In the XIX century, the major European cities were already experiencing congestion traffic problems at roundabouts. These problems started to be transferred to the affluent roads of the intersections, originating further problems such as (Gallardo, 2005):

- Accidents: the number of accidents began to increase significantly due to the lack of regulation at the intersection.
- Capacity: intersections were reached their capacity limits causing delays.

Figure 1 - (a) and (b) View of Columbus Circle, circa 1915 (Jacquemart, 1998)

Local governments of the largest European cities began to be concerned about the organization of traffic circulation and were interested in solving the congestion problems that faced the main streets at junctions (Gallardo, 2005), thereby, the first roundabout concept was introduced in the early 1900s and expanded throughout Europe and America (Wang *et al.*, 2012).

Nowadays, there is a controversy among Americans, Britishers and Frenchs about the introduction of the roundabout. For Americans the first traffic circle or rotaries was introduced for the first time by William Phelps Eno (1858 – 1945), who is known as “The father of the traffic”, since they consider that the concept of the roundabout has been part of the road network with the construction of Columbus Circle in November 1904 in New York City (Jacquemart, 1998). Subsequently, it is believed that several large circles or rotaries were built throughout the U.S. Nonetheless, it seems the merit is for the French architect Eugenie Hénard (1849-1923), who worked in the architecture service of the city of Paris and designed the first urban roundabouts (Gallardo, 2005).

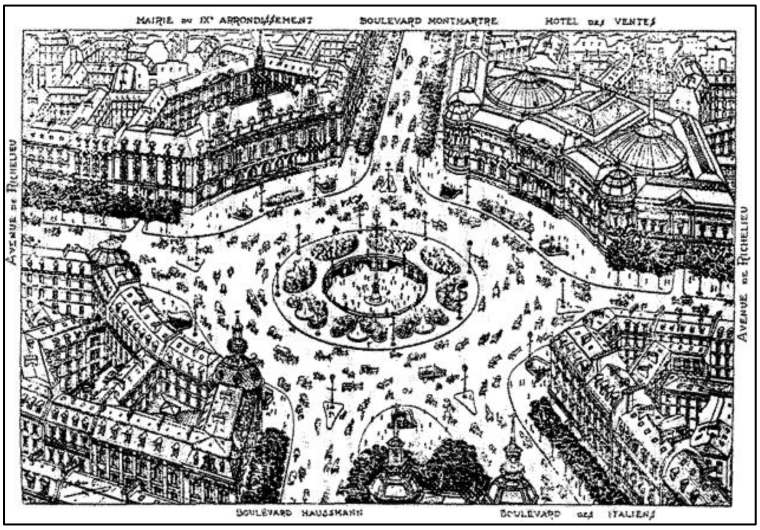



Figure 2- Roundabout project for the “Grands Boulevards” intersection in Paris, designed by E. Hénard (Gallardo, 2005).

Even though it appears that Eno and Hénard arrived at the concept of the gyratory traffic movement independently, an important difference between the two proposals designs was discovered. The suggestion made by both authors regarding the size of the central island for the roundabout was decisive in the debate for choosing which one should be recognized as the gyratory traffic inventor.

On one hand, the strong advocate of one-way streets and gyratory systems William Eno claimed that traffic circles often had relatively small central islands diameter of 1.50 m (*5 ft*) or even less which sometimes consists only of an iron disc and electric lights, or reflectors fitted on the side. On the other hand, Eugenie Hénard stated that the radius should be greater than 8 m, it constituted an important difference from the small island iron disc suggested initially by Eno. In addition to that, Hénard also argued that the issues of intersections are a consequence of conflict points between vehicles trajectories and concluded that is necessary to delete or reduce the number of conflict points as much as is possible (Jacquemart, 1998).

In the design of the first gyratory intersection (traffic circle or rotary), priority was given to the entering vehicle. The geometric features facilitated the circulation entry vehicles to the intersection and consequently, vehicles could enter the intersection at high speeds, generating that traffic circles become congested, and leading to high crash rates. This experience led to traffic circles falling into disuse after the mid-1950s in the United States. In other countries, the experience with these intersections was equally negative and had proven that traffic circles lock up as traffic volumes increased (NCHRP, 2010).

During the interwar period, roundabouts were the only type of intersection in Britain for which there was no rule governing priority. In them, the circulation was operated by crossing or braiding between vehicles driving on the circulatory roadway and those that join or abandon it (Gallardo, 2005)

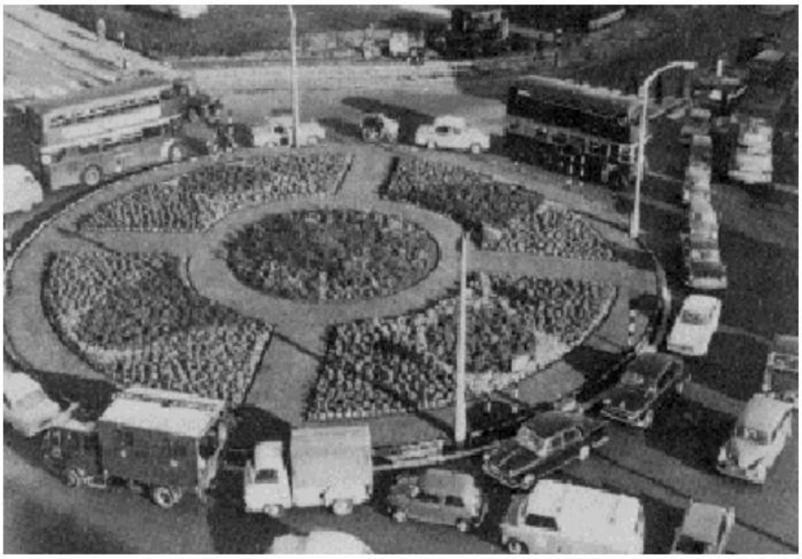


Figure 3 - Congested roundabout in Britain, from: " MIMEE. H." (Gallardo, 2005)

In 1907, the one-way revolving circulation started to operate in two important Parisian squares: *The Plaza de l'Atoile* (now Charles de Gaulle Square) around the *Arc de Triomphe* and the *Place de la Nation*. In 1925, 18 years later, the first English roundabout appeared in Aldwych, central London following the principles set out by Eugenie Hénard (Gallardo, 2005).

Until 1966, English roundabouts were governed by the priority rule on the left (right in other countries). As vehicles approach entering the roundabout at speeds higher than those circulating in the carriageway, this rule forces vehicles that transit on the circulatory roadway to further reduce their speeds. This traffic operation tended to favour the entry of vehicles that come from one of the arms of the roundabout, above those already in it, as they have forced them even to stop. It causes queuing at the carriageway and impedes the entry and exit of vehicles under certain circumstances, such as high levels of traffic circulation, vehicles blocking all movement at the roundabout (Gallardo, 2005).

### 2.2.1 The self-blocking of old roundabouts

The chaos in traffic flows caused by the previous priority rule led the researchers to find the best solution to solve self-blocking problems at a roundabout. From the 1950s to 1960s, English engineers began to test with the reversal of priorities rules, which allowed them to analyse whether the circulation rules improved the traffic conditions or not (Gallardo, 2005).

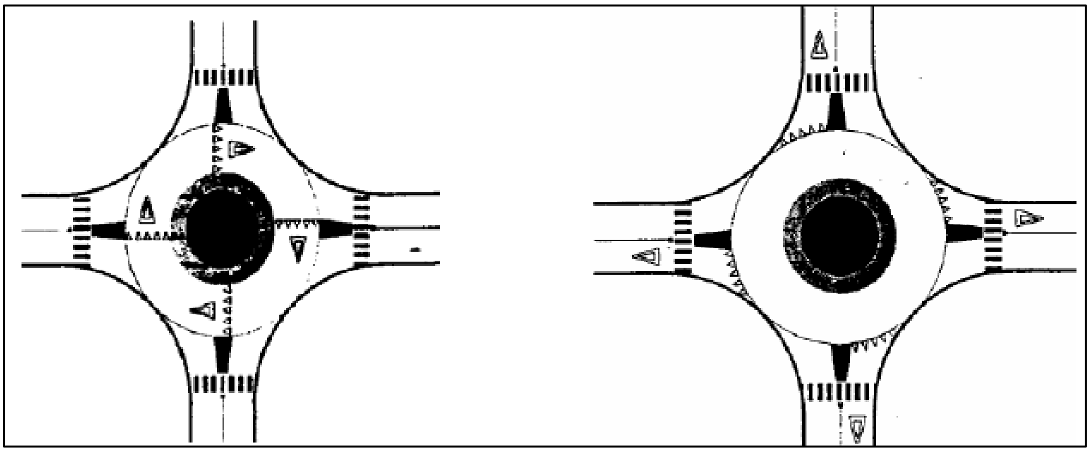


Figure 4 - Scheme of priorities at a roundabout, before and after the intervention proposed by the British (Gallardo, 2005).

In 1956, in collaboration with the authorities of numerous locations that were suffering from the traffic collapse of their roundabouts. The Road Research Laboratory (RRL), in the United Kingdom, began a series of trials consisting of observing the traffic operation of roundabouts before and after their introduction. The experimental study carried out was focused on the analysis of the offside-priority rule of the roundabouts, and the results could not have been more favourable. Thus, in November 1966 and after another series of tests on 83 roundabouts, was establish the Offside Priority Rule at gyratory intersections (Gallardo, 2005).

Since the implementation of the new priority system, the roundabouts have become a sequence of "T" junctions, which replaced the previous system. In this new system, the distance between successive entry and exits is reduced, resulting in significantly smaller roundabout sizes. The design of the roundabouts changed, the central island become smaller and the entries and exits are sketched (Gallardo, 2005).

This change of conception led to what is now known as modern roundabouts, which facilitate the generalization of roundabouts by allowing for the design of more compact roundabouts, with much less required surface area, and which can even be implemented at existing city intersections with no important dimensions that impact the space conditions in the built environment (Gallardo, 2005).

### 2.2.2 Modern roundabout

The modern roundabout or the roundabout concept was developed in 1966 in the United Kingdom to rectify problems associated with traffic circles, generally having a much smaller circumference than rotaries or traffic circles. Roundabouts had substantially lowered operating speeds than other circular intersections due to their smaller circumferences and curved entries. Whereas a rotary or traffic circle may be constructed for operating speeds ranging from 40 to 65 km/h, a roundabout is typically intended for speeds ranging from 25 to 35 km/h (Turner, 2011).

According to the department of transportation of New York (DPT, 2021), there are three basic principles to differentiate the modern roundabout from a traffic circle:

- Yield lane: English roundabouts follow the "yield-at-entry" rule, which states that vehicles approaching must wait for a gap in the circulation flow before entering the carriageway. In traffic circles, the vehicles already circulating in the carriageway must yield those vehicles entering. Some traffic circles also use signals to control vehicle entry.
- Deflection: At a roundabout, the safety operation is influenced by an adequate deflection, the traffic is channelled with a curved from the entry path to the circulatory roadway. In general, a roundabout should be constructed to limit vehicle speeds to 50 km/h or fewer by adjusting the geometry of the roundabout, the entry alignment, splitter island, central island, and exit alignment to ensure that the vehicle's trajectory is suitably deflected.
- Geometric curvature: The geometric design of modern roundabouts forces users to slow down their speed by imposing radius and angles at both entry and circulatory roadways, as small diameters and deflected (curved) entries enable traffic speeds at the intersection to be controlled. In the design of traffic circles, the entries have larger diameters, and tangential or straight criteria promote vehicles to circulate at high speed when performing merging manoeuvres.

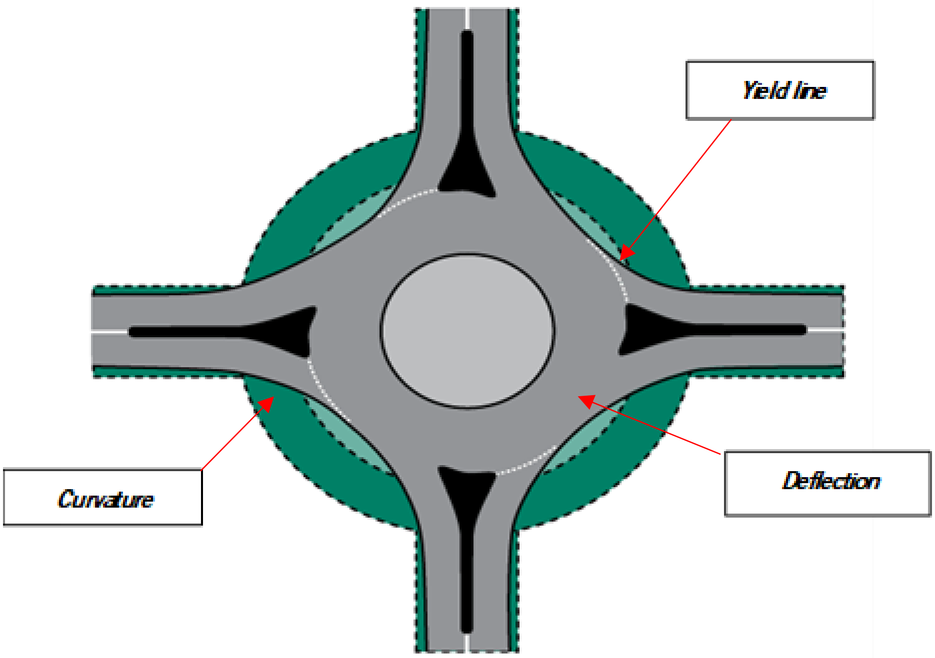



Figure 5 - Key features of modern roundabouts (Spack, 2021)

### 2.3 Definition of roundabout

Roundabouts are a subset of a wide range of circular intersections, typically with a circular shape, where traffic flows counterclockwise (in right-hand traffic countries) around a central island where vehicles entering the circulatory roadway must yield to vehicles already circulating. These gyratory junctions are constructed with special geometric elements and traffic control features, to encourage divers to approach them at a desirable speed, while ensuring the safety and efficiency of the intersection (Nikou *et al*., 2010; NCHRP, 2010).

According to NCHRP (2010), not all gyratory intersections must be classified as roundabouts; three forms or categories are identified: rotaries or traffic circles, signalized traffic circles, and neighbourhood traffic circles. The following are the features of each type of gyratory intersection:

- Traffic circles or rotaries***:*** As previously stated, they are an old-style circular junction characterized by having large diameters of their central island (IDC), often more than 100 m. Because of their large size, rotaries required a considerable space between arms for the weaving section. As a result, the intersection is projected to self-locking. Figure 6 – A, illustrated two roundabouts of this kind in the United States.

- Signalized traffic circles: These traditional circular junctions govern one or more entry–circulating points using traffic lights. Signalized traffic circles differ significantly from yield-controlled roundabouts in terms of operating features, queue storage within the circulatory roadway and progression of signals required. Signalized traffic circles are distinct from roundabouts with pedestrian signals, as the entry–circulating point at a roundabout is still governed by a yield sign (Figure 6 – B and C).

- Neighborhood traffic circles: These are traffic circles that are often built at street crossings or in the middle of residential crossroads for traffic calming or aesthetic reasons. The intersection approaches can be uncontrolled (Figure 6 – D) or stop-controlled (Figure 6 – E), normally do not have a raised channelization to guide the users when approaching the circulatory roadway. These intersections can limit the ability of larger vehicles, particularly left-turning actions of larger vehicles that tend to occur in front of the center island, generating a conflict with other circulating traffic.

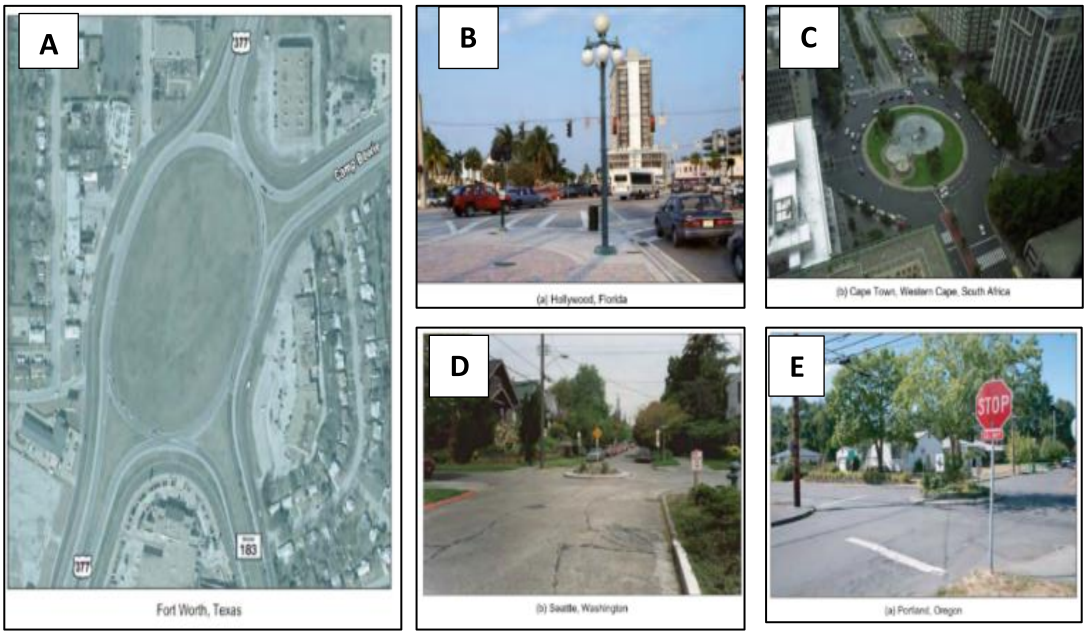


Figure 6– Examples of gyratory intersections (NCHRP, 2010).

## 2.4 Types of roundabouts

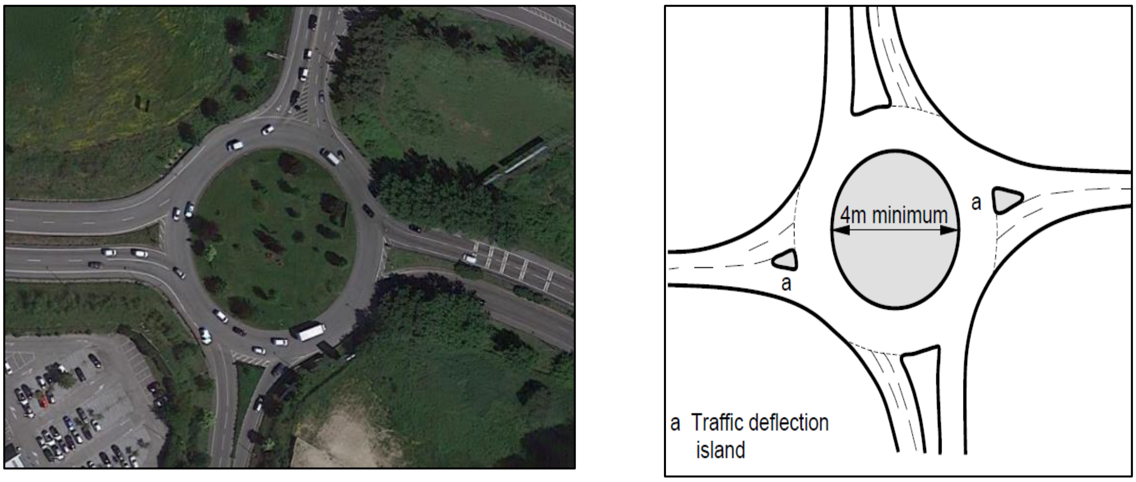
The roundabout is classified mostly based on geometric factors such as the shape and proportions of the IDC. The English Department of Transportation Manual (Highways Agency, 2007) and Bastos *et al*., (2008) describe four main basic typologies of roundabouts; Normal and Compact roundabout (with truck apron), mini roundabouts and grade-separated roundabout. Others four types such as, ring junctions, signalised roundabouts, and hamburger roundabouts are classified as variants of these four basic types.

In Portugal, the classification has been adjusted following the national reality, thus, five categories were established taking into account their overall size and basic geometrical characteristics (Bastos *et al.*, 2008).

The following is a description of each type of roundabout and an analysis of its potential use as an intersection solution according to its functionality and geometric characteristics.

### 2.4.1 Normal roundabout

Normal roundabout is a one-way circulatory carriageway around a central island and is the most common type of roundabout on the national road network. Geometrically, normal roundabouts are characterized by having a central island with a diameter equal to or greater than 4 meters, an inscribed circle diameter *(ICD)* greater than 28 m (Figure 7) and flared approaches to allow multiple vehicle entries. The dimension attributed to the traffic circle is normally defined to suit the operational needs of any type of vehicles, as the central island is impassable under normal traffic conditions *(*Highways Agency, 2007*;* Bastos *et al., 2008).*




*a)* *b)*

Figure 7 – (a) example of normal roundabout in Portugal (Google Earth, 2021) / (b) Layout of a normal roundabout (Highways Agency, 2007)

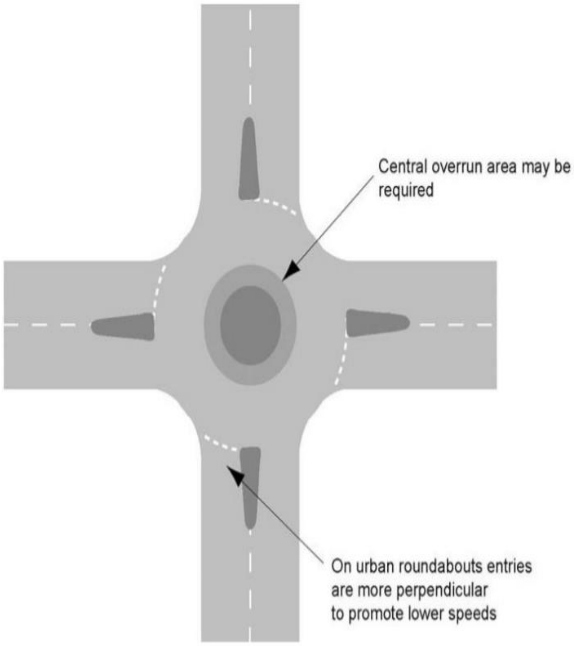
Normal roundabouts are generally employed at settlement exits or where there is a distinct series of T-junctions. The advantages and disadvantages of this type of roundabout are particularly related to their large size, as it ensures continuous flows and steady speeds which led to being considered a high-capacity roundabout. Nevertheless, its large size comes at a great disadvantage because when they have very large ICDs, more land area is required for its construction, in the great majority of the cases the location is conditioned by the space available.

### 2.4.2 Compact roundabout (with truck apron)

Compact roundabout has a kerbed central island and an ICD between 28 and 40 m and the central island. This is an impassable intersection contoured with a strip of material of contrasting color to the roadway and preferably of irregular texture, commonly with pebbles or granite cubes. All of these geometrical and functional characteristics allows to be used by vehicles, especially lane heavy ones to make turns sufficiently uncomfortable, while discouraging its use by light vehicles *(*Bastos *et al., 2008).*

*(a.1)*

*(a.2)*




*(b)*

Figure 8 - Examples of a Compact roundabouts with truck apron (NCHRP, 2010).

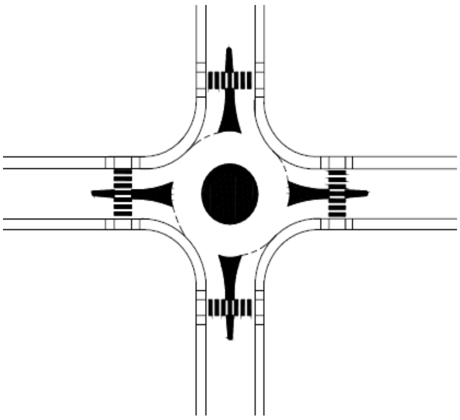
This type of roundabout is recommended to be implemented as a solution in places where it is necessary to improve the deflection imposed on light vehicles while maintaining heavy truck operability. Therefore, its application is particularly effective in the presence of low flows of heavy vehicles and whenever, for safety reasons, it is essential to ensure the moderation and control of speeds associated with light vehicles (Bastos *et al.*, 2018).

### 2.4.3 Mini-roundabout

Mini roundabouts have been widely used in several countries, especially in Australia and the UK where it was first introduced in the early 1970s. In Germany, for instance, its implementation started in 1995, with an experiment in which 13 unsignalized intersections were converted to mini-roundabouts. The success was overwhelming, the results of the study developed by Germans showed that in terms of capacity mini-roundabouts achieved up to 17.000 vehic/day without generating major delays for vehicles (Brilon, 2011; Bastos *et al.*, 2008).

These roundabouts have a diameter smaller than 4 m and ICD between 14 and 28 m, the central island that can be materialized or can be simply demarcated on the pavement as a horizontal sign. This solution is only applicable to IDC longer than 18 m and should have a maximum height of 12 cm in the center to accommodate the maneuvering needs of heavy vehicles, as they have total freedom to maneuver around the central island, namely during left turns (Bastos *et al.*, 2008).

*(a)*



*(b)*

Figure 9- (a) Example of a Mini-roundabouts in Germany (Brilon, 2011) / (b) layout of a Mini-roundabout (FHWA, 2010)

A positive aspect of mini-roundabouts is that they require little space and are relatively inexpensive to implement, which makes them a feasible solution for urban and suburban intersections at low-speed two-lane road intersections. In most cases, mini roundabouts can fit within the limits of existing traffic lanes. Thus, all channelisation can be added within the limits of the existing roadway. However, their use should be restricted to residential areas, where the presence of heavy vehicles is exceptionally (Bastos *et al.*, 2008; Brilon, 2011a).

### 2.4.4 Grade separated roundabout

This type of roundabout has one or more approaches coming from a road at a different level. As a solution can be employed at motorway junctions, but can also be used to link underpasses, flyovers, and other multi-level intersections (Highways Agency, 2007). The main objective of the grade separated roundabouts is to channel the movements of the minor roads and the change of direction at the junction (Bastos *et al*., 2018).

According to the Highway Agency (The Highway Agency, 1993), two bridge roundabout and dumbbell roundabout are the two most common forms of roundabout used at at-grade intersections, below is a description of each category:

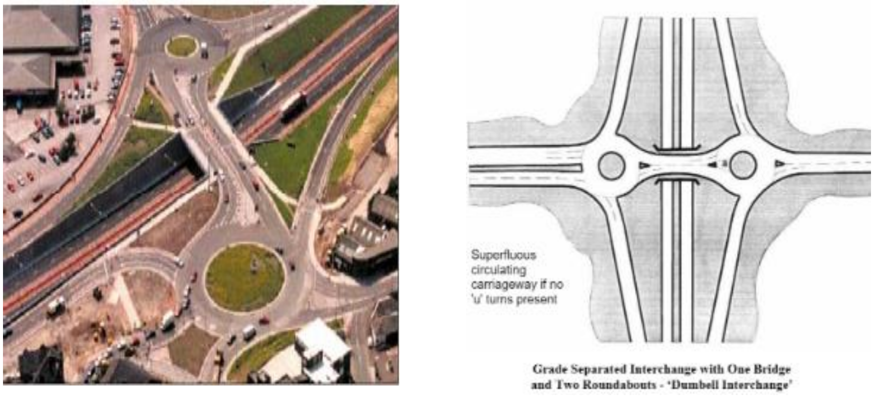
- <u>Two bridge roundabouts:</u> This layout is commonly associated with a large roundabout (Figure 10). It is characterized by being a large solution in which there is a central roundabout to access the intersecting secondary road. The construction of this solution requires a significant amount of space which makes it costly. In terms of safety, its large size permits high circulatory speeds, which can cause problems for drivers trying to enter the system. Thus, the accident rate for this type of roundabout is higher than in other solutions.

*(a)*


*(b)*

Figure 10 – (a) Example of Two Bridge Roundabout (FHWA, 2010)/ (b)- Layout of a Two Bridge Roundabout at Grade Separated Interchange (The Highway Agency, 1993).

- <u>Dumbbell roundabout:</u> This type of layout is compound by two compact roundabouts which are connected by a central viaduct, located laterally in relation to the carriageway of the priority lane (figure 11). It is considered a more advantageous layout than the previous one, not only because it requires less space for its construction, but also by its low construction cost. In terms of performance, a dumbbell roundabout allows analyzing the capacity of the roundabout and avoids blocking the priority lane by forming queues at the entries to the roundabout.




*(a)* *(b)*

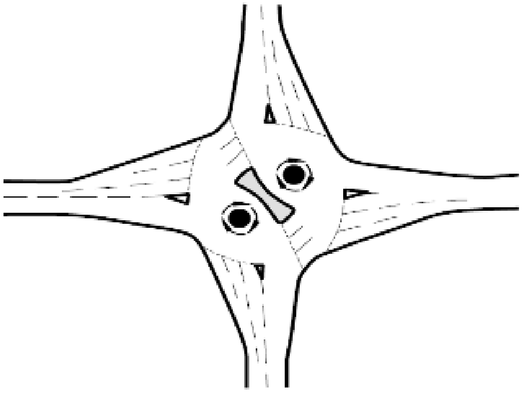
Figure 11 – (a) Example of a Dumbbell Roundabout (FHWA, 2010) / (b) Layout of a Grade Separated Interchange with One Bridge and Two Roundabouts – “Dumbbell Interchange” (The Highway Agency, 1993).

### 2.4.5 Double roundabout

These are the less common type of roundabout, compared to the other solutions presented above, their complexity and loss of legibility make them solutions of more restrictive use (Bastos *et al., 2008)*. Double roundabouts are characterized by being an individual junction with two normal or mini roundabouts, they can be either contiguous or connected by a central link road or curbed island with a reduced dimension (Figure 12).

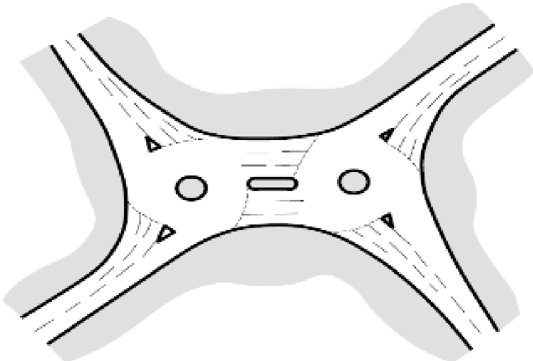
This type of roundabout is particularly suitable for locations where the use of a normal roundabout would lead to an oversized solution, or in extremely long squares to resolve conflicts between the junction of two parallel roads and to minimize the effects of left turns and U-turns on the capacity of different entries (MOPU, 1995).

*(a.1)*



*(b.1)*

*(a.2)*



*(b.2)*

Figure 12 – Typical layouts of double roundabouts (a) Contiguous Double Roundabout and (b) Double Roundabout with Short Central Link road (The Highway Agency, 1993).

### 2.4.6 Ring junctions

Ring junctions are a combination of traffic islands with a junction arrangement allowing two-way traffic on the circulation system. It replaces the usual clockwise one-way circulation of vehicles around a large island because require drivers on the circulatory system to give way. In this solution, usually, each entry arm with the circulatory carriageway relate to three arm mini roundabouts, or it may be signalized (The Highway Agency, 1993).

The unusual traffic circulation at these roundabouts could make it more difficult for drivers to interpret them, particularly those who are not familiar with them. Thus, it is imperative to be careful and contemplate aspects such as clarity, conciseness, and unambiguous signals at the design stage to achieve a successful operation (The Highway Agency, 1993).

In countries less familiar with the operation of this type of roundabouts, its use should be conditioned, as is still the case in Portugal. They should preferably be limited to the treatment of areas that allow considerable distances between the different arms and, in turn, to a large radius of the central island (Bastos *et al.*, 2018).

Figure 13 - Ring junctions (Kittelso, 2021)

### 2.4.7 Signalized roundabout

An intersection governed by a signalized system, even if its geometric shape resembles a roundabout, should not be considered as such, since the design principles are clearly different (Bastos, 2018).

Because of growth of the traffic flow, this sort of solution is useful for addressing capacity issues at roundabouts where some of the entry ways do not operate properly. A signalized intersection could also be a favorable when self-regulating nature problems are evident, and it is likely to experience an overloading, an unbalanced flow, or it is also needed to control traffic speeds, to ensure the safety of the more vulnerable road users. In all those cases, a traffic signals can be installed at the roundabout to alleviate the problems of the intersection. The operation of the traffic signals can be either continuous or part-time, at some or all the entries, or even at certain times of the day (The Highway Agency, 1993; Bastos, 2018).

*(a)*

*(b)*

Figure 14 – (a) Example of a signalized - Roundabout Marquez de Pombal, Lisbon – Portugal (Trip advisor, 2021)/ (b) Layout of a signalized roundabout (Duarte *et al*., 2004)

### 2.4.8 Hamburger roundabout

This solution is commonly known as “hamburger‘‘ in which the two halves of the central island look like the ’’bread’’, and the splitter island between two roads represents the ’’meat’’. The central island is crossed by a road considered to be a priority, subdividing it at the same time, into two lateral semicircles (Tollazzi, 2014).

Although this typology has been widely used in some countries such as Spain, Portugal, the U.K. and Canary Islands, its use is mostly associated with high accident rates and accident severity due to the difficulty for non-regular drivers to read it. The conflicts point at 90° (figure 15), are the main reasons why have been transformed over time into normal roundabouts or signalised intersections.

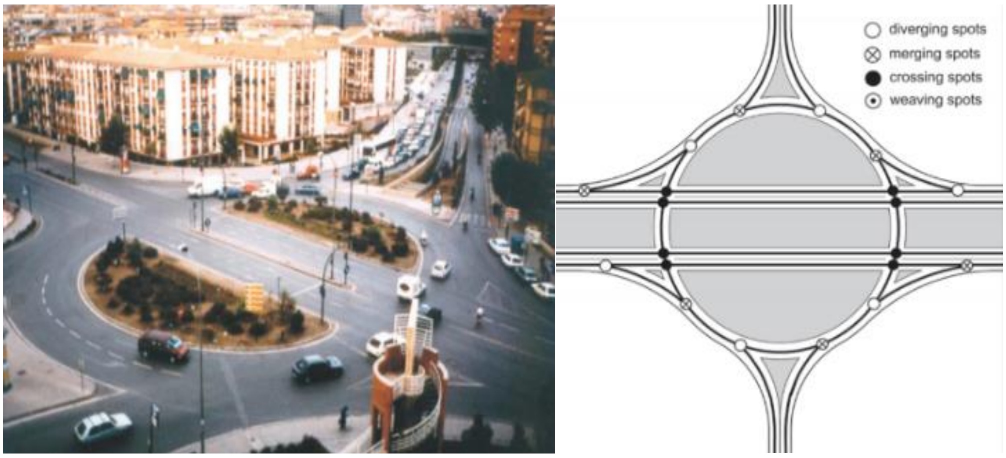



Figure 15 – At-grade hamburger roundabout and conflict points (Tomaz Tollazzi, 2014)

While these solutions do not impose delays on the main movements, they can significantly reduce the levels of service and safety associated with the secondary movements that require crossing the mainstream (Tollazzi, 2014; Bastos, 2018). The operation of this solution differs significantly from the principle of roundabouts operation; therefore, they are not usually included in a set of types of roundabouts.

### 2.4.9 Turbo roundabouts

This form of roundabout was developed by Mr. L.G.H. Fortuijn, Professor at Delft University of Technology. The turbo roundabout corresponds to an innovative solution of a two-lane roundabout in which the geometrical characteristics are modified. In the Netherlands standard two-lane roundabouts are no longer utilized since the introduction of the turbo roundabout in 1998s (Engelsman *et al*., 2007).

Figure 16 - Typical Turbo Roundabout in the Netherlands (Baranowski *et al.,* 2017)

Entering and exiting a typical two-lane roundabout can be complicated for some drivers, leading to collisions due to lane changes within the roundabout. For this reason, turbo roundabouts eliminate some of the most serious conflict points of a roundabout and reduce the need to change lanes. As illustrated in Figure 17. A standard turbo roundabout has 10 vehicles conflict points, whereas a two-lane roundabout has 24. This represents 60% more conflict points, including four weaving conflicts and two exit conflicts, implying a greater accident risk for a two-lane roundabout (Baranowski *et al.,* 2017).

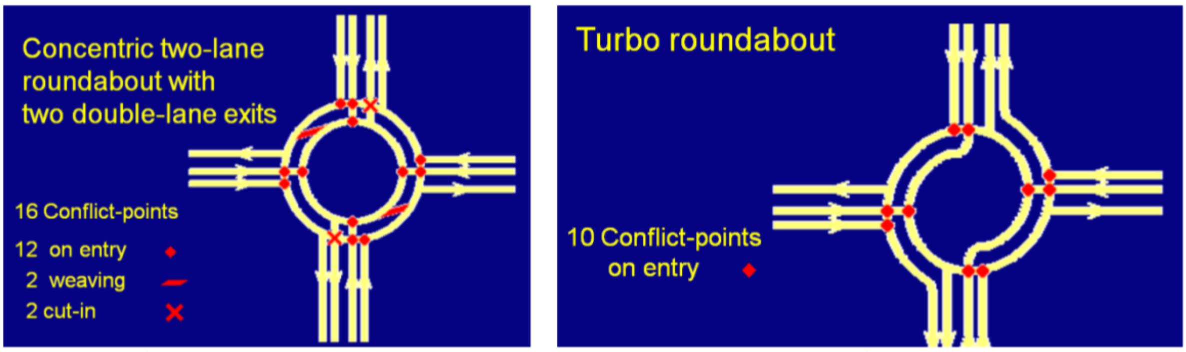



Figure 17 - Two-lane Roundabout vs. Turbo Roundabout conflict points Comparison (Baranowski *et al.,* 2017)

## 2.5 Conditions for application and potential of roundabouts

According to (Bastos *et al.*, 2008), the low cost of roundabouts associated with high levels of capacity and safety has led to their application in various parts of the road network, sometimes in inadequate situations. The application of the roundabout solutions is conditioned by the individual characteristics of the junctions and can vary significantly. These prevailing characteristics of the location are typical, the existing topography, the type of intersected road, the road environment, and the features of entry traffic flows. Thus, the decision to adopt one solution or not only depends on investment and maintenance costs but on the characteristics of application that such a type of roundabout can offer.

In some scenarios, the type of roundabout is chosen according to the space available for its construction and the traffic volume expected to flow through it. In Germany, for instance, the selection of the types of roundabouts is based on the size and traffic volumes. Figure 18 exhibits the application of each type of roundabouts in Germany, regarding their inscribed circle diameter (IDC) and their maximum capacity in terms of average daily traffic (ADT) described by Brilon (2011).

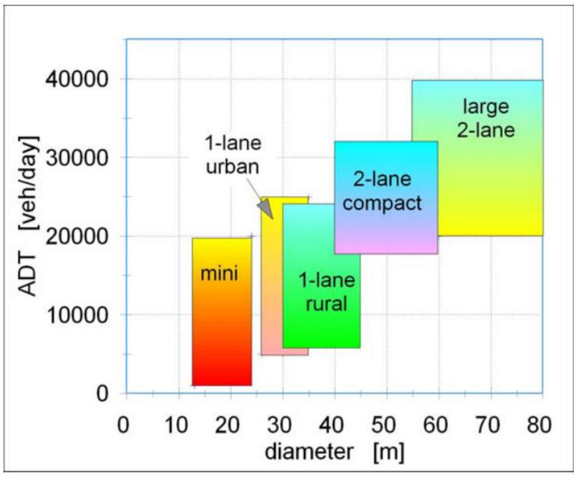



Figure 18 – Types of roundabouts by IDC and ADT (Brilon, 2011b)

Bastos *et al.*, (2008) maintain that in the road environment, roundabouts provide good levels of capacity and safety for both urban and interurban areas. However, in urban areas, their performance is mainly limited by the presence of vulnerable users such as pedestrians and cyclists and local conditions (Hydén *et al.*, 2000).

In urban areas, roundabouts are frequently located at the entries of residential zones or central spaces to modify the characteristics of the surroundings where it is required a sudden change to the drivers' behavior (Bastos, 2018). As is summarized in Table 1, for its geometric simplicity, normal and grade separate roundabouts are suitable to improve the performance at most of the existing junctions or conflicts spaces in urban areas. Nevertheless, in expressways, for instance, systematic adoption of roundabouts is not recommended because it can lead to significant delays on the main traffic streams. It can also notice that local streets could require no solutions with high potential performance or capacity, as they usually have very low traffic demand.

Table 1– Applicability of roundabouts according to the functional classification of intersecting roads in urban area (Bastos, 2008).

| Type of road | Expressways | Arterial Streets | Collector Streets | Local Streets |
|---|---|---|---|---|
| **Expressways** | A (Rd/Rn) | A (Rd) /a (Rn) | A (Rd) /A (Rn) | X |
| **Arterial Streets** | -- | A (Rn) | A (Rn) | a (Rn) |
| **Collector Streets** | -- | -- | A (Rn) | a (Rn) |
| **Local Streets** | -- | -- | -- | a (Rn) |
| **A** - suitable in most cases; **a** - suitable in some cases; **X** - link to be avoided; **Rn** - Normal roundabout; **Rd** - Grade separate roundabout | | | | |

Although in interurban areas the road environment incentive the practice of high traffic speeds, the adoption of roundabouts should be assumed as possible and desirable (Bastos, 2018). As is illustrated in Table 2, in IP and IC, it is more convenient to adopt grade separate intersections, as an at-grade solution as normal roundabouts could result in a substantial reduction of traffic speeds which can increase delays at the intersection. Hence, for the treatment of junctions, it is suitable to implement normal or at-grade roundabouts at the EN/ER and EM.

Table 2 – Applicability of roundabouts according to the classification of the intersected roads in inter-urban areas (Bastos, 2018).

| Type of road | IP | IC | EN/ER | EM |
|---|---|---|---|---|
| **IP** | N | a (Rd) | A (Rd) | A (Rd)* |
| **IC** | -- | a (Rd) | A (Rd) | A (Rd)* |
| **EN/ER** | -- | -- | a (Rd) /A (Rn) | a (Rd) /A (Rn) |
| **EM** | -- | -- | -- | A (Rn) |
| **N** - Not normally suitable; **a**- Suitable in some cases; **A** - Suitable in most cases; **Rn** - Level roundabout; **Rd** - Grade separate roundabout; ***** - according to JAE P5/90 is a junction to be avoided | | | | |

Turning junctions into roundabouts can improve safety and traffic flow, several traffic studies in the U.S and Europe have found that roundabouts perform better than other intersections form (Transport, 2009; NHTSA, 2021; Jacquemart, 1998). The presence of signalized intersections may increase the frequency of traffic conflicts since signalizations interrupt the traffic flow by imposing stop-and-go movements (Chromosomes *et al.*, 2009; Wang *et al.*, 2012).

Table 3 – Annual crash frequencies before and after roundabout construction (NCHRP, 2010)

| Country | Mean Reduction (%) | |
|---|---|---|
| | All Crashes | Injury Crashes |
| Australia | 41 - 61% | 45 - 87% |
| France | - | 57 - 78% |
| Germany | 36% | - |
| Netherlands | 47% | - |
| United Kingdom | - | 25 - 39% |

### 2.5.1 Movements and conflict points at the roundabout

A well-designed roundabout contributes to road safety and the disaggregation of traffic impacts at an intersection, the crash rates are reduced by experiencing a substantial reduction in the number of injury accidents, fatal accidents, and serious injury accidents. The injury accidents depend on the number of arms and the previous form of traffic control (Transport, 2009; Jacquemart, 1998).

This has aroused the interest of numerous countries to decide to incorporate roundabouts as a potential solution to deal with problems at the intersections, especially at four-arm intersections, where is evident a reduction and even the elimination of some vehicles movements that are deemed for users, e.g., crossing movements and left turns, which probably are the most hazardous due to the necessity of drivers to select a gap from two directions (Summary, 2011).

At a conventional intersection, at least four types of conflict points can be found, these conflict points require special attention. The NCHRP (2010) classifies conflict points into three basic categories by taking into account the degree of severity of the potential crashes of each movement:

- **<u>Queuing conflicts:</u>** These conflicts are caused when a vehicle run into the back of a vehicle queue on an approach, in this case, crashes involve the most protected parts of the vehicle and the relative speed difference between vehicles is less than in others, for this reason, they are least severe conflict points.
- **<u>Merging and diverging conflicts:</u>** These conflicts are originated by the joining (*Merge*) or separating (*diverge*) of two traffic streams at the carriageway of the roundabout. On one hand, merge conflicts are ranked as being more severe than diverge movements since are caused by the joining of two traffic streams. The likelihood of collision side to side with another vehicle is higher in this type of manoeuvres, the less protected parts of the vehicle namely, the front and rear, are implicated during the crash. On the other hand, diverge conflicts are caused by the separation of two traffic streams, if the speed

of an individual movement differs significantly from the other, the resulting speed differential increase the risk of a rear-end collision.

- **<u>Crossing conflicts:</u>** These are the most severe of all conflicts and the most likely to involve injuries or fatalities since crashes are right-angle crashes and head-on crashes.

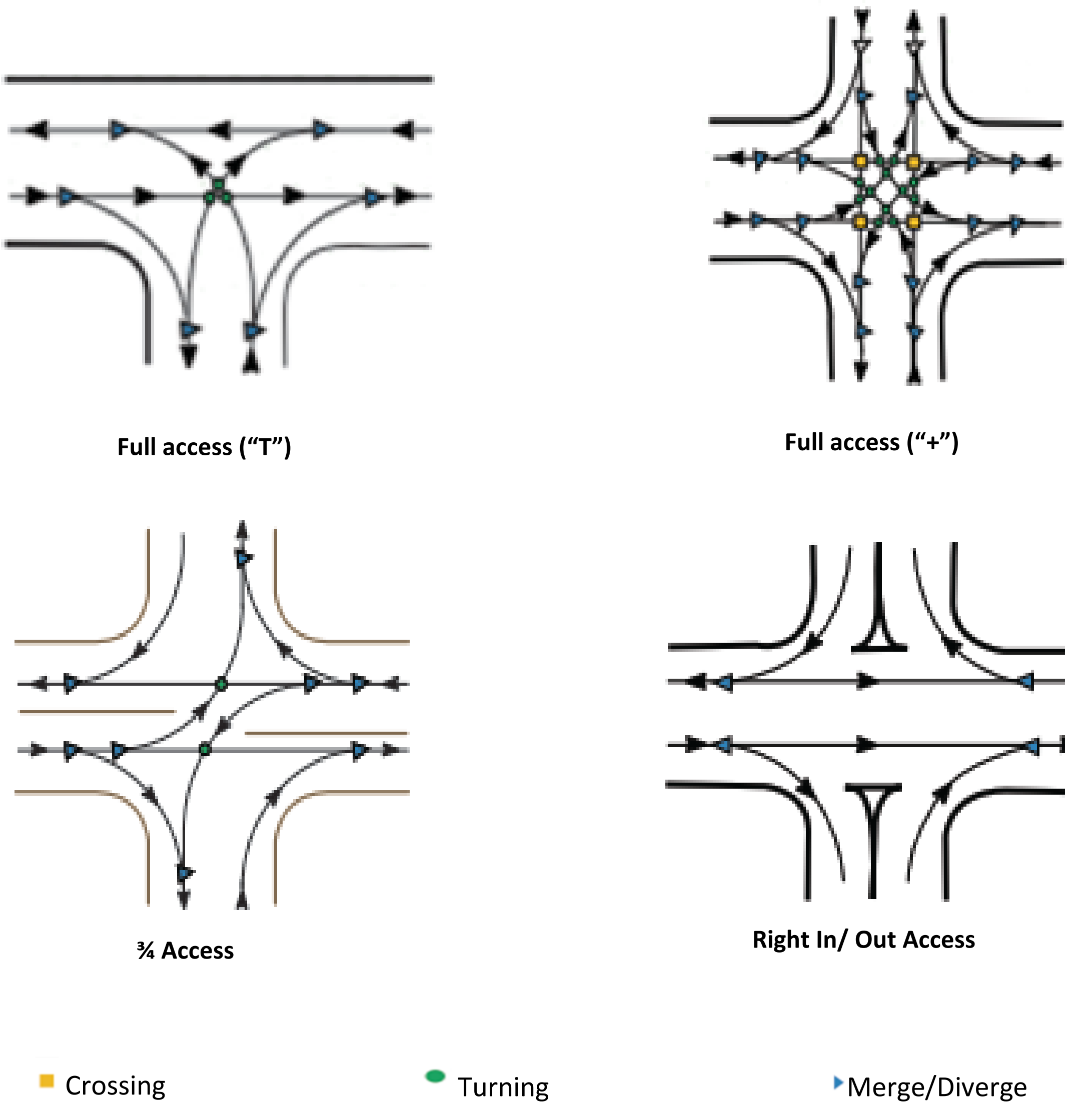



Figure 19 – Five types of junctions and conflict types at junctions (DTM, 2021).

Table 4 summarize five types of junctions taking into consideration the number of accesses and the points where vehicles cross, turn, merge, or diverge, and figure 19 illustrates the number of conflict points by intersection type.

In a four-arm intersection, there are 32 conflicts: 4 crossing, 12 turning, and 16 converging and diverging movements. In contrast, roundabouts eliminate crossing and turn-left conflict points, which are the most hazardous and treatment guarantee a reduction of 50% merging and diverging conflicts for a total of 8 conflicts being comparable to a succession of "T" intersections.

Table 4 – Conflict points by intersection type (DTM, 2021).

| | **Crossing** | **Turning** | **Merge/Diverge** | **Total** |
|---|---|---|---|---|
| **Full access ( + )** | 4 | 12 | 16 | 32 |
| **Full access ( T )** | 0 | 3 | 6 | 9 |
| **3/4 Access** | 0 | 2 | 8 | 10 |
| **Right-in/out Access** | 0 | 0 | 4 | 4 |
| **Roundabout** | 0 | 0 | 8 | 8 |

The elimination of crossing and turning conflict points ensure good levels of capacity allowing an acceptance of shorter critical intervals. Figure 20 shows the difference between a conventional junction with conflict points between a conventional junction.

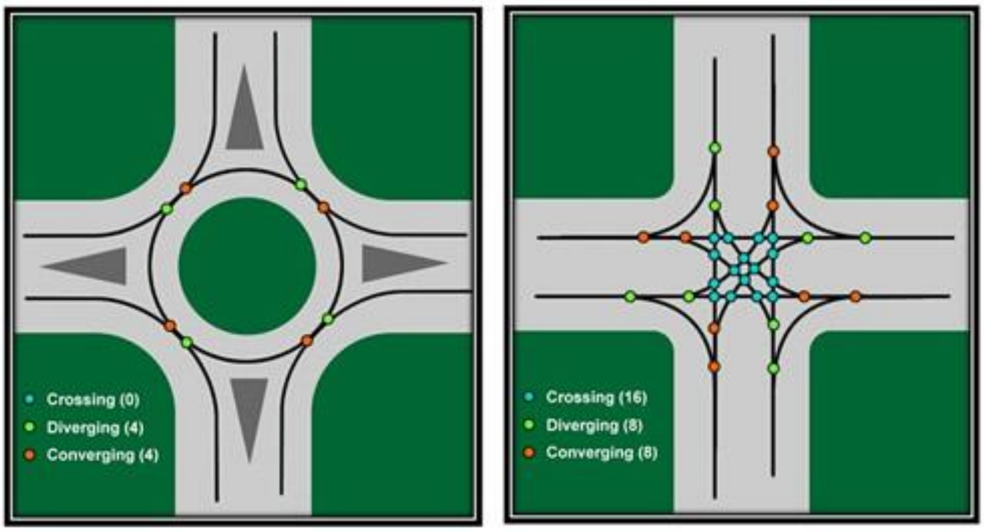



Figure 20 –Four-arm and “T” roundabout comparison (Streets & ROW, 2021)

## 2.6 Basic elements for the measurement of roundabout performance

The maximum entry flow at a roundabout depends mainly on factors such as the conflicting flow at the roundabout and conflicts with the entry flow, the exit flow, and the geometric elements of the roundabout (Al-madani, 2016).

Some key geometric aspects must be taken into consideration when designing a roundabout to provide adequate speed control and good levels of capacity and achieve the performance desired. Geometrical characteristics are divided into three main groups, at the entry, at the circulatory roadway and at the exit are defined as follows (WDT, 2021; Silva & Seco, 2004).

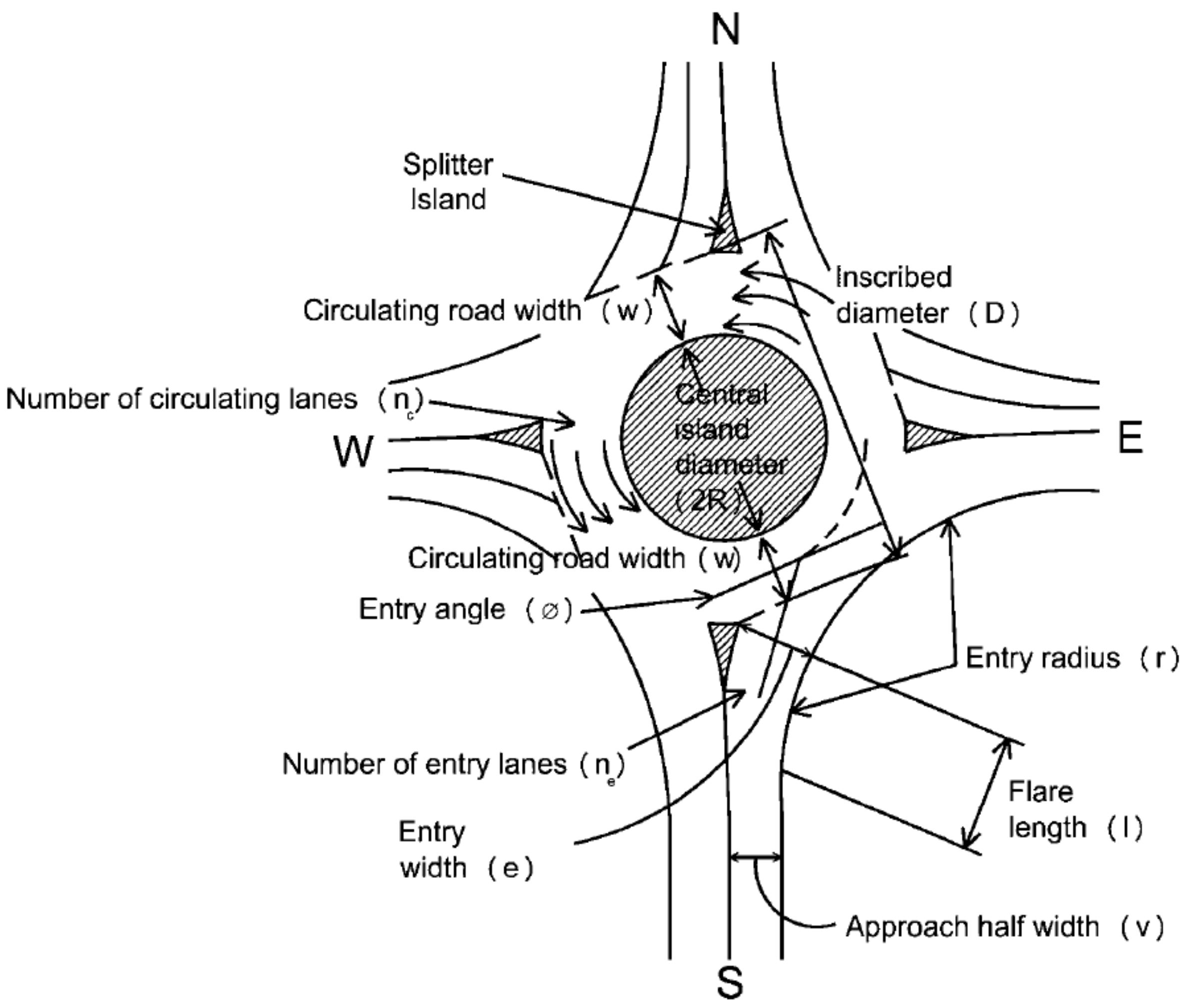



Figure 21 – Key geometric parameters for determining roundabout performance (Al-madani, 2016).

- Entry Angle: The purpose of the entry angle is that vehicles do not collide sideways. The range of angle values that are desirable to achieve the proper amount of deflection for each approach to a roundabout. This range is between 20° and 60°, but it is recommended entry angles between 20° and 40°.

- Entry width: This parameter is determined by the turning template of the design vehicle turning through the entry curve at the desired entry speed and depends on the functionality of the intersecting roads and the characteristics of the traffic. It is recommended that the maximum radius not exceed 50 meters, ideally, be closer to 20-30 meters.

- Number and width of entry lanes: For safety and operational reasons, the maximum number of entry lanes should preferably be limited to three. The minimum width of the entry lane is determined by the operational requirements of the larger vehicles being recommended lanes graters than 3.0 meters, it should be as small as possible to minimize weaving at the intersection. The effective width is calculated for each entry, being recommended that values of between 4 and 12 meters.

- Splitter island: Raised or painted within the entry and exit of an arm of the roundabout used to separate entering from exiting traffic, deflect and slow entering traffic, and provide a refuge for pedestrians crossing the road in two stages.

- Visibility at the entry: This parameter is based on entry visibility criterion to assure that any vehicle in the vicinity of the yield line can see at a considerable distance to any priority vehicle and/or pedestrians at crossings.

- Circulatory roadway width: A curved path where vehicles circulate counterclockwise around the central island. The circulatory roadway width is mainly conditioned by the number of entry lanes at the roundabout and the deflection of the carriageway. To facilitate the circulation of vehicles, its dimension should be between 5 and 12 m and should be between 1 and 1.2 times the width of the largest entry.

- Inscribed Diameter Circle (IDC): The largest diameter that can be inscribed inside the roundabout (including berms) and which passes tangentially to the boundary of the entry under study.

- Central island: A raised area in the centre of a roundabout around which traffic circulates, its dimension is determined by the width of the circulatory roadway and the size of the Inscribed circle diameter. Geometrically, the shape of the central island necessarily does not require to be circular.

- Truck apron: Required on smaller roundabouts to accommodate the wheel tracking of large vehicles. An apron is the mountable portion of the central island adjacent to the circulatory roadway, the truck apron is generally paved to delineate the apron from the normal vehicle path by contrasting colours that allow drives to identify it easily.

- Exit radius: The purpose of this parameter is to ensure users have a moderate speed for exiting the intersection. The exit radius should be larger than the entry radius of the roundabout, dimensions of less than 20 m and greater than 100 m are not allowed.

- Exit width: In general, the exit width must ensure the continuity of the number of lanes assigned to the entry and the circulatory roadway of the roundabout, its dimension should be preferably 5 m (plus berms) and never be less than 4 meters.

# CHAPTER III
# ROUNDABOUT PERFORMANCE

*"The reality about transportation is that it is future-oriented. If we are planning for what we have, we are behind the curve".*

**Anthony Foxx**

### 3.1 Introduction

This chapter explores models developed in various countries for capacity calculation of roundabout, which enable the evaluation of the changes in system behavior. It is imperative to highlight that, while capacity is the most critical factor in defining the performance of a roundabout, it does not entirely describe its operation. Additional variables such as delays, saturation level, queuing, and environmental issues such as pollutant emissions and fuel consumption influence the performance of this type of intersection.

In general, roundabouts are characterized by having a circulatory roadway in which different traffic streams (approaches) converge. All approaches must follow the priority rule, which results in significant time loss (delays) for vehicles attempting to enter the intersection. Because of the interruption in the traffic flow at the approach of the roundabout, drivers must wait until finding a gap to decide to enter the intersection and yield to those already circulating in the circulating lane.

Entry capacity is the first indicator of the performance of the roundabout, it has been the subject of investigation in many countries since the implementation of modern roundabouts in the 1980s.

The remarkable aspect, as Wu & Brilon (2018) explains, is that each country has attempted to find its own solution. The traffic engineers created numerous models to assess capacity at roundabouts. The procedure for determining this parameter is often established based on a new system, in which roundabouts operate as a sequence of "T" junctions, which implies that the capacity study must be carried out individually for each of the entries.

Models for evaluating capacity are grouped into three types, each of which differs in the approach throughout the intersection analysis process:

- Empirical
- Analytical
- Microsimulation

## 3.2 Capacity

Roundabout performance is mainly determined by its capacity. The term capacity refers to the maximum hourly rate of vehicles passing through a lane in a road section over a period of time and under prevailing roadway, traffic, and control conditions (TRB, 2000).

In roundabouts, capacity is determined for each entry and not for the entire intersection. The Roundabout Informational Guide (FHWA, 2010) defines the concept of capacity at roundabouts as:

> *"The capacity of each entry to a roundabout is the maximum rate at which vehicles can reasonably be expected to enter the roundabout from an approach during a given time period under prevailing traffic and roadway (geometric) conditions".*

In addition to the geometric qualities of the road, the entry capacity is strongly influenced by the priority traffic that travels through it (conflicting traffic flow). As a result, the primary objective of roundabout design is to provide capacity to the intersection; it is a measure of how efficiently a road system serves demand. From an analysis of the perception of drivers and through the level of service (LOS), is also possible to know the quality of operation of the roundabout (Cal & Mayor, 2019).

The manual for the Application and design of roundabouts of Netherlands of *Ministry of Transport & Roads (2009)* states that the capacity of an intersection depends strongly on at least four parameters:

- the traffic volumes per direction on the major and minor road (in peak hour).
- the number of travel lanes on the major and minor roads.
- the speeds in practice on the major road.
- the traffic composition.

The models for estimating the roundabout capacity are a function of traffic flow parameters such as the conflicting traffic flow, the driver behaviour on the approaches at the entries, as well as the geometry of the intersection (FHWA, 2010).

The roundabout analysis may be divided into three main categories based on the parameters utilized for the evaluation, the modelling approach used in each of them, the geometric and traffic features of the roundabout, and the computational complexity, Yap *et al.* (2013) categorized these roundabout models as follow:

- Empirical models: Methods based on relationships between geometry and actual measured capacity.
- Analytical models: Methods based on understanding driver behaviour.
- Micro-simulation: Methods based on modelling of vehicle kinematics and interactions.

The following sections present the most important models used to evaluate the capacity and performance of the roundabouts through the description of the calculation methodology and the respective parameters that influence entry's capacity.

### 3.3 Empirical models

Empirical or statistical models use field data to establish relationships between geometric design features and performance measures such as capacity and delay (TRB, 2000). They were originated in the UK in the late 1970s, after many roundabouts observations in periods of oversaturation to describe the system performance from a field data statistical analysis.

Several empirical models have been proposed worldwide. However, the most widely recognised procedure was developed by Kimber (1980), a British professional in the management of the Transportation Research Laboratory (TRL) in London (Piña *et al.*, 2012).

The empirical models are generated through multivariate statistical regression analysis to fit the mathematical relationships between the conflicting flow $(Q_c)$, the measured entry capacity $(Q_e)$, and the other dependent variables that significantly affect the entry capacity.

In general, it is assumed that the relationship between $Q_e$ and $Q_c$ is linear, and is given as follow:

$$Q_e = \alpha - \beta * Q_c \tag{1}$$

or exponential.

$$Q_e = \alpha * e - \beta * Q_c \tag{2}$$

Models which use these calibration relationships are the most widely known form of roundabout capacity modelling (Yap *et al.*, 2013).

The following subsections describe six empirical models for roundabout entry capacity calculation, such as the UK model (TRL), the French models (SETRA and CERTU), the Portuguese model (FCTUC), the German exponential and linear models, as well as the Colombian and Dutch models.

### 3.3.1 TRL

The TRL model is known as the most complete empirical model, it was developed by the Transport and Road Research Laboratory (TRRL, now TRL) in the U.K. The equations obtained to calculate the capacity of each entry using this model are the result of an analysis of 88 roundabouts, in which more than 11.000 were recorded, and over 500.000 vehicles were observed in a wide range of geometric designs and traffic conditions (Peirce, 1998).

Kimber model has been incorporated into a software package widely known as RODEL and ARCADY (Yap *et al.*, 2013). According to Kimber (1980), the model involves roundabout geometry (geometric requirements) and conflicting flow (traffic flow parameters), and capacity calculation is made for each entry independently, this methodology required over six parameters to estimate the entry capacity.

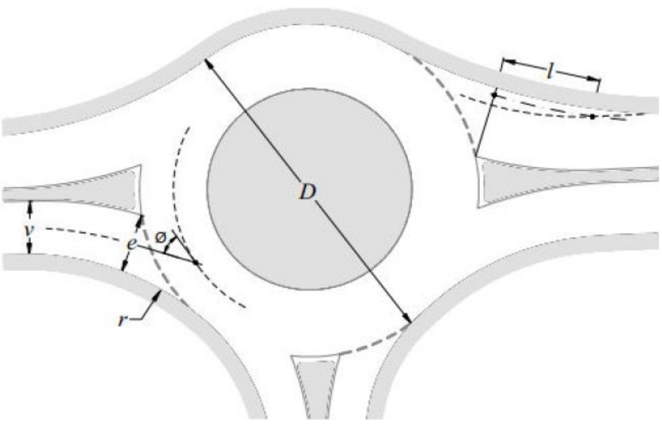



Figure 22- Geometric parameters for calculating capacity through TRL model (Vasconcelos *et al.,* 2013*)*

Where:

- ***v:*** Approach width, expressed in (m)
- ***e:*** Entry width, expressed in (m)
- ***l´:*** Effective flare length, expressed in (m)
- ***r:*** Entry radius, expressed in (m)
- ***D:*** Inscribed circle diameter, expressed in (m)
- Ø: Entry angle, expressed in degrees (°)

The entry capacity defined by the TRL model is computed through the following equations:

$$Q_e = K \times (F - f_c \times Q_c) \quad \text{when} \quad f_c \times Q_c < F \tag{3}$$

$$Q_e = 0 \quad \text{when} \quad f_c \times Q_c \geq F \tag{4}$$

Where:

- $\boldsymbol{Q_e}$: Capacity of an entry, expressed in pcu/h.
- $\boldsymbol{Q_c}$: Conflicting flow, expressed in pcu/h.

$K, F$ and $f_c$ are calibrated parameters as a function of the geometric characteristics of the roundabout, which are estimated as follow:

- <u>Accumulation factor</u> $(K)$*:*

$$K = 1 - 0.00347\ (\emptyset - 30) - 0.978 \times \left\{ \frac{1}{r} - 0.05 \right\} \tag{5}$$

- <u>Maximum storage capacity</u> $(F)$*:*

$$F = 303 \times X_2 \tag{6}$$

- <u>Correction factor</u> $(f_c)$:

$$f_c = 0.21 \times t_p \times (1 + 0.2 \times X_2) \tag{7}$$

- Potential for accumulation $(t_p)$:

$$t_p = 1 + \frac{0.5}{1 + M} \tag{8}$$

With:

$$M = \exp\left\{\left(\frac{D - 60}{10}\right)\right\} \tag{9}$$

$$X_2 = v + \frac{(e - v)}{1 + 2S} \tag{10}$$

$$S = 1.6 \times \frac{(e - v)}{l'} \tag{11}$$

Where:

- $X_2$ is a constant depending on e, v, and S.
- $S$ is the sharpness of flare (m/m).

This capacity equation has an accuracy of approximately 15%, the intersections were carefully selected to collect a considerable quantity of data, and the measurements were made at peak hours to obtain information about various roundabout parameters (Kimber, 1980). Statistical data analysis was made to determine which parameters were significant and the respective effect. This research also grouped geometric features of the roundabout into four categories, where they were classified according to the relevance of the geometric effects and its variations on the roundabout entry capacity. Thus, six statistical parameters of the roundabout geometry that were identified in the analysis (Table 5).

Table 5 - Geometric parameters measured and range of values observed for which the Kimber linear model is valid (Kimber, 1980)

| **Geometric parameters** | **Symbol** | **Range** |
|---|---|---|
| Entry width | *e* | 3.6 - 16.5 m |
| Approach width | *v* | 1.9 - 12.5 m |
| Effective flare length | *l'* | 1 - infinity (m) |
| Sharpness of flare | *S* | 0 - 0.29 m |
| Inscribed circle diameter | *D* | 13.5 - 171.6 m |
| Entry radius | *r* | 3.4 - infinity (m) |
| Entry angle | *Ø* | 0 - 77 ° |

It was found that the most influential parameters are the entry width (e) and the approach width (v), whereas the Inscribed Circle Diameter (IDC) provides a moderate influence on capacity performance. The influence of the entry angle and the entry radius (r) appreciable and the other geometric parameters of the roundabout were the least influence on the capacity performance of each entry.

### 3.3.2 SETRA

This model was published in 1987 by SETRA (*Service d'Etudes Techniques des Routes et Autoroutes*). SETRA model is a simple method for calculating the roundabout entry capacity, the methodology was based on a major observation campaign that was carried out on French interurban roundabouts in a congested state (Gallardo, 2005). This calculation methodology of the entry capacity is used specifically for roundabouts located in rural or peripheral areas with an IDC greater than 45 m (Pratelli *et al.,* 2018).

In contrast to the TRL approach, SETRA includes an impeding factor that depends on the approach width, the conflicting and the exiting traffic flow. For this methodology, the entry width is the most important parameter. According to Gallardo (2005), each additional meter over a standard entry width of 3.5 meters implies a 10% increase in the entry capacity. Other coefficients such as the circulatory roadway, the central island dimensions and the types of vehicles have a lower incidence into the capacity.

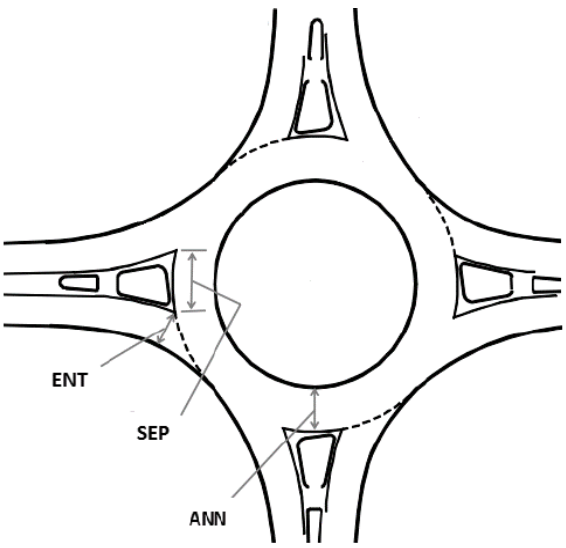



Figure 23- Geometric requirements in SETRA equation (Gallardo, 2005)

Where:

- $\boldsymbol{ENT}$: entry width, expressed in *m*
- $\boldsymbol{ANN}$: circulatory roadway width, expressed in *m*
- $\boldsymbol{SEP}$: splitter island width in *m*.

The entry capacity in pcu/h, is calculated with the following equation:

$$\boldsymbol{C_e} = \left(1330 - 0.7 \times\ Q_g\right) \times [\ 1 + 0.1 \times (ENT - 3.5)] \tag{12}$$

Where:

- $Q_g$ is the impeding traffic and is computated as follows:

$$\boldsymbol{Q_g} = \left(Q_c + \frac{2}{3} \times\ Q_u^*\right) \times [\ 1 - 0.085 \times (ANN - 8)] \tag{13}$$

For impeding traffic flow calculation, two parameters of the traffic flow are included and, additionally, a dependent factor on the splitter island dimension.

- $\boldsymbol{Q_c}$: conflicting flow, expressed in pcu/h
- $\mathbf{Q_u}$: exiting flow, expressed in pcu/h

The factor ${Q^*}_u$ is strongly associated with the exiting flow and the splitter island width of the approaches, it is calculated as follows:

$$\boldsymbol{Q_u^*} =\ Q_u * \frac{15 - SEP}{15}\ , \qquad Q_u^* = 0 \quad if \quad SEP\ \geq 15\, m \tag{14}$$

When analyzing the detailed factors and effects on entry capacity through SETRA model, can be noticed that the splitter island width (SEP) has a higher influence on the entry capacity, where values of less than 15 m could cause a significant reduction in the capacity. Different from the TRL model, for SETRA, the IDC has a lower influence on the entry capacity of the roundabout, while exiting traffic influences the capacity performance of the roundabout.

### 3.3.3 CERTU

The CETUR (*Centre d'Etudes des Transports Urbains*), now called CERTU, is a body attached to the French Ministry of Infrastructure and Transport and is the urban institution equivalent to SETRA. In 1986, it was proposed a simplified capacity calculation method based on the CETE d'Aix studies. Thereby, CERTU was created in Lyon in February 1994.

This model is a traditional method to estimate the capacity at the entry of French roundabouts (CERTU, 2021). The methodology provided by CERTU is suitable when the sum of entering and disturbing traffic flow is less than 1500 pcu/h. The equation has been calibrated for the measurement of the capacity performance of roundabouts located in urban or peripheral areas of cities. The procedure for calculating the entry capacity of a roundabout by the means of this model is presented as follows.

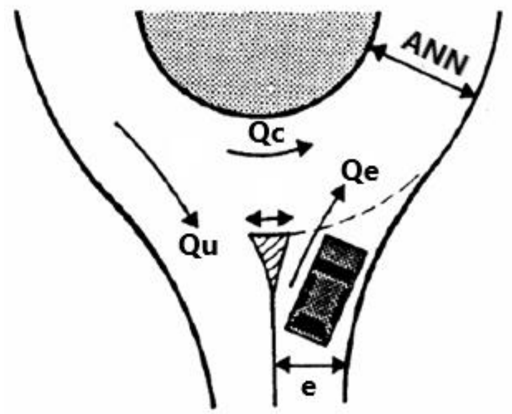



Figure 24- Geometric requirements in CERTU equation (Gallardo, 2005)

The formula for finding the entry capacity of a roundabout by using CERTU model is the following:

$$\boldsymbol{Q_e} = \gamma \times (1500 - 0.83 \times \ Q_g) \qquad (15)$$

$$\boldsymbol{Q_g} = \alpha \times \ Q_c + 0.2 \ \times \ Q_u \qquad (16)$$

Where:

- $\boldsymbol{Q_e}$: Entry capacity, expressed in *pcu/h*
- $\boldsymbol{Q_g}$: Impeding flow, expressed in *pcu/h*
- $\boldsymbol{Q_c}$: Conflicting flow, expressed in *pcu/h*
- $\boldsymbol{Q_u}$: Exiting flow, expressed in *pcu/h*

*"0.2"* is a typical value in function of the central island *(IDC)*

The factors $\gamma$ and $\alpha$ are attributed according to the following geometrics criteria provided in table 6 and table 7.

Table 6 – Factor $\gamma$ related to the number of entry lanes (Pratelli *et al*,. 2018)

| | |
|---|---|
| $\gamma = 1$ | 1 entry lane |
| $\gamma = 1.5$ | 2 or more entry lanes |

Table 7- Factor $\alpha$ related to the ANN and IDC dimensions (Pratelli *et al*,. 2018)

| | |
|---|---|
| $\alpha = 1$ | If ANN < 8 m |
| $\alpha = 0.9$ | If ANN $\geq$ 8 m and IDC < 40 m |
| $\alpha = 0.7$ | If ANN $\geq$ 8 m and IDC $\geq$ 40 m |

### 3.3.4 FCTUC

This model was developed in 1988 by the Faculty of Science and Technology of the University of Coimbra (FCTUC, for its acronym in Portuguese). The empirical Portuguese model is an adaptation of the TRL model proposed by the United Kingdom in which some geometrical factors were adjusted to suit formulas to the traffic network and behavioural conditions of the drivers in Portugal, in other words, the coefficients of the equation were calibrated to Portuguese realities.

The calibration process of the FCTUC model was made through an analysis of eight national roundabouts, six of which correspond to four roundabouts located in the urban area and the others in the peri-urban areas of the country. The period of analysis of the study carried out was approximately 952 minutes with a variance of the data collected close to 61.7%. Hence, the current Portuguese guideline recommends forecasting roundabout capacity from the TRL model due to the limited sample of data collected. The evaluation and the applicability of the TRL to the Portuguese conditions were conducted in the 1990s (Bastos, 2010).

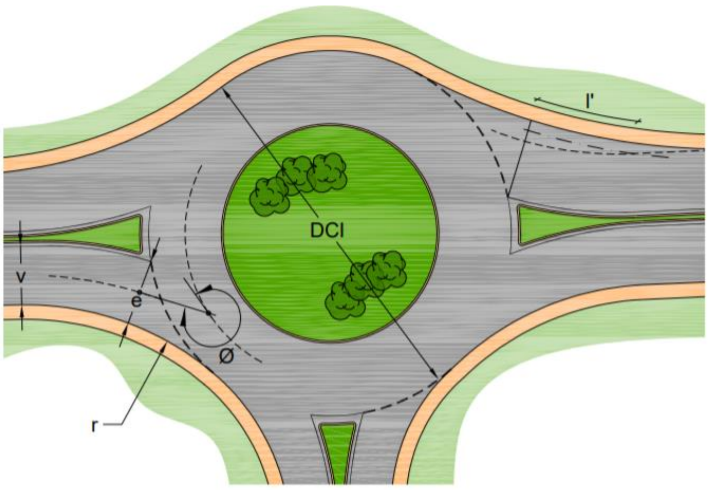



Figure 25- Geometric requirements in FCTUC model (Silva & Seco, 2004)

The methodology for the capacity calculation by the means of the FUTUC is given as follow:

$$Q_e = K \times (F - f_c \times Q_c) \quad \text{when} \quad f_c \times Q_c < F \tag{17}$$

$$Q_e = 0 \quad \text{when} \quad f_c \times Q_c \geq F \tag{18}$$

Where:

- $\boldsymbol{Q_e}$: The capacity of an entry, expressed in pcu/h
- $\boldsymbol{Q_c}$**:** Conflicting flow, expressed in pcu/h.

$\boldsymbol{K}, \boldsymbol{F}$ *and* $\boldsymbol{f_c}$: Calibrated parameters as a function of the geometrical features of the entry and the roundabout, these values are calculated by the following equations. Each of the following parameters has the same meaning as in the TRL method. However, the factor that affects the parameters were adjusted to the Portuguese conditions.

$$\boldsymbol{K} = 1 - 0.00163(\emptyset - 30) - 3.431 \times \{\frac{1}{r} - 0.05\} \tag{19}$$

$$\boldsymbol{F} = 335.47 \times X_2 \tag{20}$$

$$\boldsymbol{f_c} = 0.611 \times t_p \times (-0.457 + 0.2 \times X_2) \tag{21}$$

$$\mathbf{t_p} = 1 + \frac{0.983}{\mathrm{M}} \tag{22}$$

With:

$$\boldsymbol{M} = exp\,\{\left(\frac{D-60}{10}\right)\} \tag{23}$$

$$\mathbf{X_2} = \mathrm{v} + \frac{(\mathrm{e}-\mathrm{v})}{1+2\mathrm{S}} \tag{24}$$

$$\mathbf{S} = 1.6 \times \frac{(\mathrm{e}-\mathrm{v})}{\mathrm{l}'} \tag{25}$$

### 3.3.5 German empirical models

Germans investigated both statistical and analytical models. The model follows a similar approach to the U.K model (TRL), it establishes a correlation between the conflicting and the entering flow. Contrary to the UK linear approach, an exponential regression was used to explain this relationship based on the theory determined by Siegloch in 1973.

The German statistical model is based on a study of 10 roundabouts with diameters between 28 and 100 m. For computing the capacity, each of the entries must be classified whether they are single or multi-lane entry roundabout. The model also included a constant dependent on the demand of the entry ensured by the existence of a queue at the entries (Iguel *et al.*, 2014).

The exponential capacity is calculated by the following equation:

$$\boldsymbol{Q_e} = A \times e^{\frac{-B \times Q_C}{10000}} \tag{26}$$

Where:

- $\boldsymbol{Q_e}$**:** Entry capacity, expressed in *pcu/h*
- $\boldsymbol{Q_c}$**:** Conflicting flow, expressed in *pcu/h*

$\boldsymbol{A}$ and $\boldsymbol{B}$ Empirical parameters

The parameters A and B in this equation have been determined separately from the measurements by regression calculation as is shown in table 8. Brilon & Stuwe (1993*)* grouped the data collected in four classes of roundabouts in concordance with the number of lanes in the circulatory roadway and the entries. For the conversion of vehicles into pcu, single-unit trucks were rated as 1.5, for trucks and trailers the equivalent factor was 2.0 and motorbikes as 0.5.

Table 8- Parameters A and B for the Calculating Capacity (Brilon & Stuwe, 1993).

| **Number of lanes** | | **Regression parameters** | | **Number of Circular measurers** |
|---|---|---|---|---|
| **Entry** | **Circulating Roadway** | **A** | **B** | |
| 3 | 2 | 2018 | 6.68 | 295 |
| 2 | 2 | 1553 | 6.69 | 4574 |
| 2-3 | 1 | 1200 | 7.30 | 867 |
| 1 | 1 | 1089 | 7.42 | 1060 |

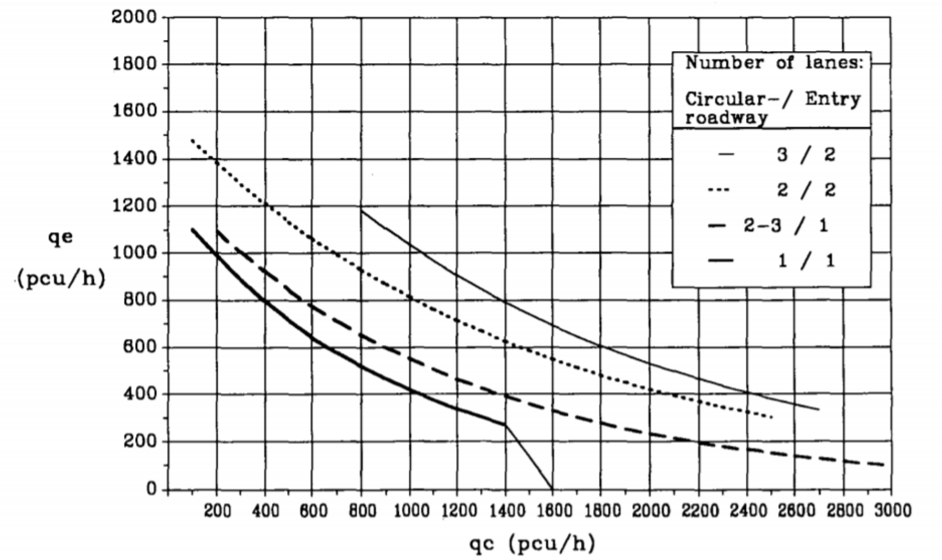



Figure 26- Regression curves for determination of roundabout capacity by the German exponential model (Brilon & Stuwe, 1993).

As is reported by Brilon & Stuwe (1993), various departments of transportation research centers measured capacity data, defined as 1-minute entry flow under saturated conditions. The comparisons made revealed that the linear functions explained the variance of the data slightly better than the exponential function. The linear capacity formula of the German model is:

$$Q_e = C + D \times Q_c \tag{27}$$

The parameters C and D are also affected by the number of lanes and the circulation highway, Table 9 displays these results.

Table 9 - Parameters C and D for calculating capacity- German linear model (Brilon & Stuwe 1993)

| Number of lanes | | Parameters | | |
|---|---|---|---|---|
| No. entry lanes | Circulating roadway | C | D | N (Sample Size) |
| 1 | 1 | 1218 | 0.74 | 1504 |
| 1 | 2 – 3 | 1250 | 0.53 | 879 |
| 2 | 2 | 1380 | 0.50 | 4574 |
| 2 | 3 | 1409 | 0.42 | 295 |

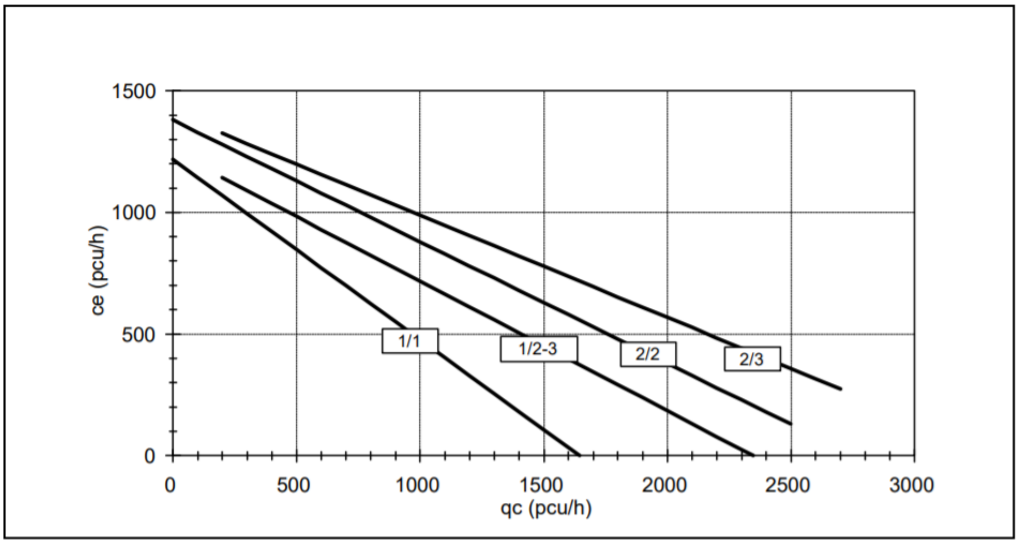



Figure 27- Regression curves for determination of roundabout capacity by the German exponential model (Brilon *et al.*, 1997)

### 3.3.6 Colombian model

In Colombia, the geometric design manual for highways (INVIAS, 2008*)* for the capacity calculation is based on Wardrop's 1973 theory. According to Wardrop's theory, the cross section of the roundabout reflects the capacity of the roundabout. Thus, the model computes the entry capacity as function of cross-section.

The Colombian model is primarily dependent on geometric design elements (traffic flow characteristics are not employed in this method). Contrary to other models, capacity is not determined for each entry, but rather a global capacity of the roundabout is computed. As a result, capacity is determined according to equation 28:

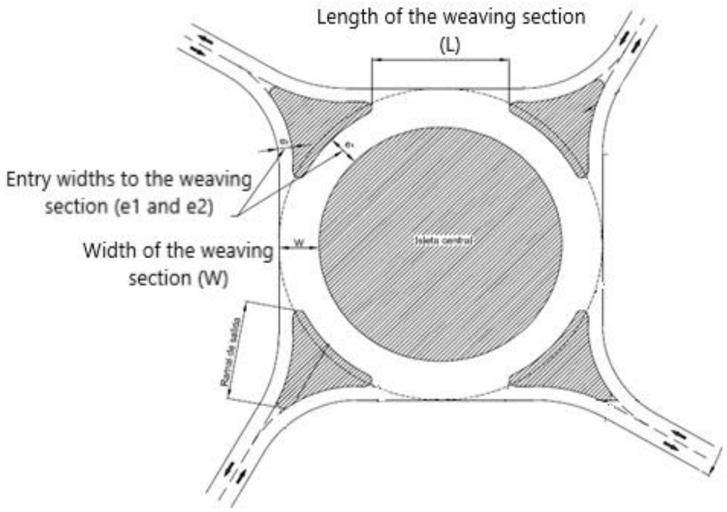



Figure 28- Geometric parameters for capacity calculation using the Colombian model (INVIAS,2008)

The equation for calculating weaving section capacity is the following:

$$\boldsymbol{Q_p} = \frac{160\,W\,\left(\frac{1+e}{W}\right)}{1+\frac{W}{L}} \tag{28}$$

$$\boldsymbol{e} = \frac{e_1 + e_2}{2} \tag{29}$$

Where:

- $\boldsymbol{Q_p}$**:** Capacity of the weaving section, expressed in pcu.
- $\boldsymbol{W}$**:** Width of the weaving section, expressed in m.
- $\boldsymbol{e}$**:** Average width of the entries to the weaving section, expressed in m.
- $\boldsymbol{e_1}$**,** $\boldsymbol{e_2}$**:** Width of each entry to the weaving section, expressed in m.
- $\boldsymbol{L}$**:** Length of the weaving section (Figure 27), expressed in m.

Table 10 - Design criteria for roundabouts (INVIAS, 2008)

| Description | Measurement Unit | | Range |
|---|---|---|---|
| Minimum diameter of the central island | m | | 25 |
| Minimum diameter of the inscribed circle | m | | 50 |
| W/L ratio (cross-section) | | | 0.25 - 0.40 |
| Width of the weaving section (W) | m | | max. 15 |
| Minimum lower radius at the entries | Entry | m | 30 |
| | Exit | m | 40 |
| Ideal entry angle | | | 60 |
| Ideal exit angle | | | 40 |

### 3.3.7 Dutch model

Different from the other countries which have based their capacity analysis studies on two types of roundabouts such as single-lane and multi-lane, the Dutch guidelines provide two different capacity calculation models according to whether the roundabout is a single-lane roundabout or a turbo roundabout. The entry capacity of this model is estimated with an empirical method for single-lane roundabout while for turbo roundabouts an acceptance gap model is used. It is because the experience that the Netherlands had in terms of the safety of multi-lane roundabouts proved to be quite negative as they generated a disappointing performance in terms of capacity and road safety.

The studies carried out shows that a relatively high number of accidents were registered due to the weaving and people crossing near the exits. Therefore, the Netherlands opted to stop the construction of multi-lane roundabouts and instead converted them into turbo roundabouts. In 2007 over 70 turbo roundabouts were in operation. Currently, multilane roundabouts are no longer built-in Netherland (MT, 2009).

The empirical formula for calculating entry capacity of a single-lane roundabout with single-lane entries and without cyclists having priority on the roundabout through the Dutch model is defined according to equation 30:

$$\boldsymbol{A_{entry}} = 1.500 \ - \ B_{roundabout} \ - \ 0.3 \times \ C_{exit} \qquad (30)$$

Where:

- $\boldsymbol{A_{entry}}$: Entry capacity, expressed in pcu/h
- $\boldsymbol{1.500}$: Maximun conflict load
- $\boldsymbol{B_{roundabout}}$ : Conflicting flow, expressed in pcu/h
- $\boldsymbol{C_{exit}}$ : Exiting flow, expressed in pcu/h

Dutch guidelines don't include geometric parameters for the entry capacity estimation. The effects of traffic flow parameters influence the evaluation of the roundabout capacity estimation process.

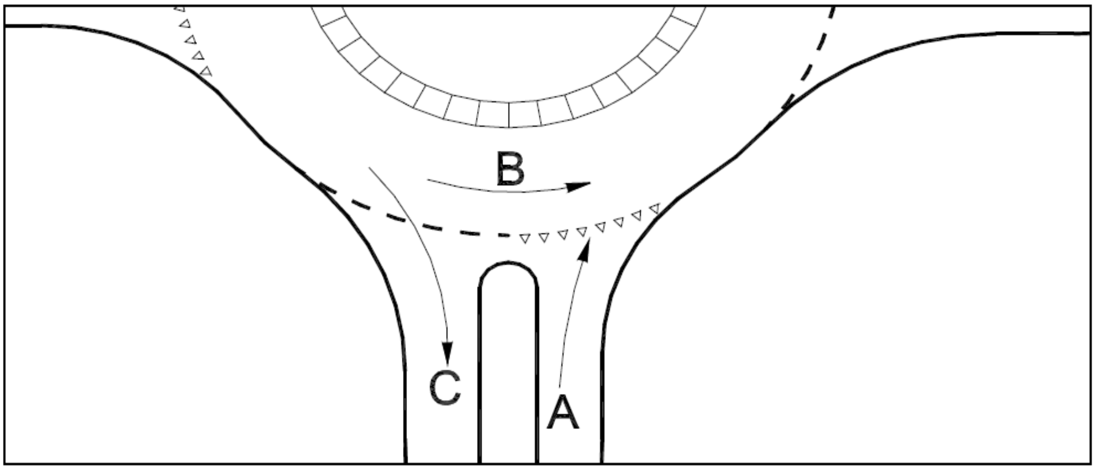



Figure 29 – Conflicting flows to determine entry capacity of a single lane roundabout in Dutch model (MT, 2009).

### 3.4 Analytical models

Analytical traffic models are estimated based on key parameters to explain the vehicles interaction at the roundabout and how drivers can utilize gaps (driver behaviour), the distribution of headways within the circulating flow, the critical headway, and the follow-up time (Dahl, 2011).

In a roundabout, drivers must decide when and look for a safe opportunity or a “gap” to make a manoeuvre to finally enter the intersection in the safest manner (Troutbeck & Brilon, 2001). This decision must be taken respecting the priority rule, which indicates that drivers attempting to access the intersection (major stream) must yield to those already circulating in the circulatory roadway (minor stream). The advantage of accepting the gap is to avoid further delays in the roundabout (gap-acceptance) and rejecting means not accepting the gap that will produce dangerous actions and endanger safety. Therefore, the gap is related to the safety of the roundabout entry and the minimum delay.

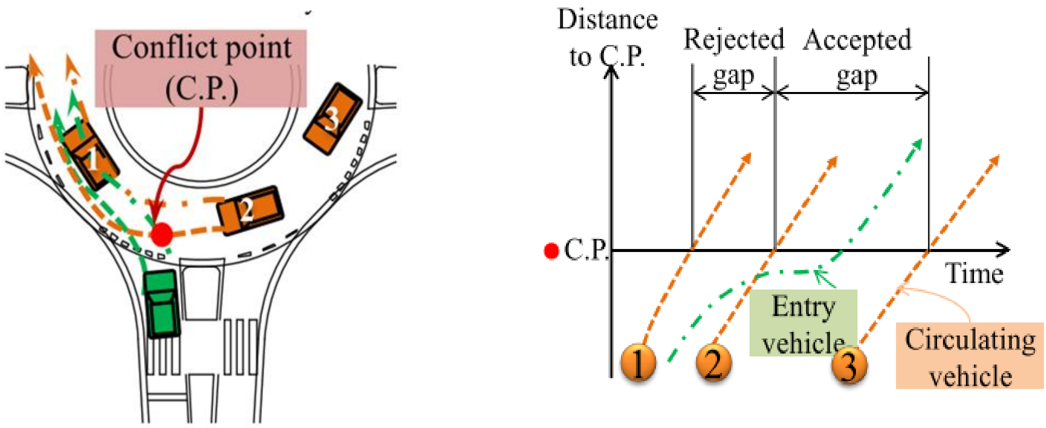



Figure 30 – Gap acceptance at roundabouts (Kang *et al.*, 2012)

As is pointed out by Silva *et al.*, (2013), mathematically is reasonable to argue that downstream vehicles cross the yield line if the time gap between the upstream vehicles is less than the critical gap, *tc*. A vehicle may also advance if the time available to enter the roundabout is less than *tc+tf*, where *tf* is the time needed for the second vehicle in the queue to reach the yield line (follow-up time).

In the presence of a time interval, it may be assumed that non-priority vehicles cross the yield line. This interval (Int) is theoretically given by:

$$Int = t_c + (n-1)t_f \text{ or, } Int = t_c + nt_f$$

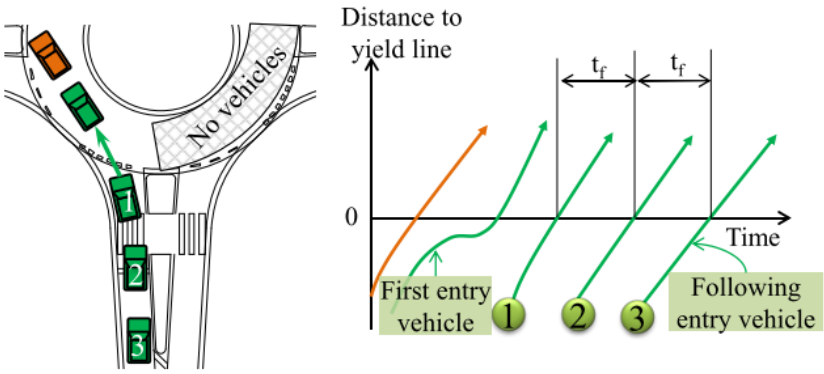



Figure 31 – Follow-up time (Kang *et al.,* 2012)

The parameters of driver behaviour are defined in the next subsections, each of which addresses aspects relevant to the evaluation of roundabout performance based on analytical or gap acceptance models, particularly for roundabouts.

### 3.4.1 Headways

Headway is a time between consecutive vehicles passing the conflict. Commonly, gaps acceptance models assume arrival patterns for headways of the circulating flow.

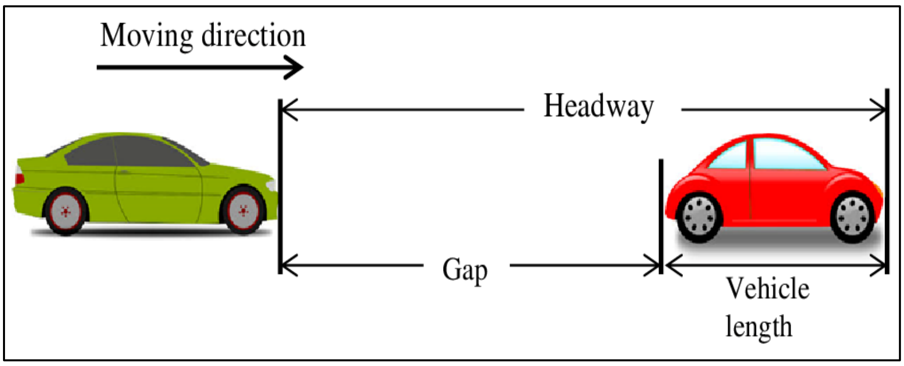



Figure 32 – Vehicles headways *(Hassan et al., 2017)*

There is three specific headway distribution under different assumptions of arrival patterns that have been applied to understand drivers behaviour (Dahl, 2011; Cowan, 1975):

- Exponential (M1)
- Displaced exponential (M2)
- Bunched exponential (M3)

Exponential Headways (M1)

An exponential headway distribution assumes that vehicles arrive randomly without depending on the arrival time of the previous vehicle, which means that all arriving vehicles are free flow vehicles. Several authors have shown that the distribution of intervals in the priority stream tends towards a negative exponential distribution *(*Kang *et al.,* 2012*;* Silva *et al.*, 2013) because of that, it is simply termed the *"negative exponential distribution".* The exponential headway is described with the following equation:

$$F(t) = 1 - e^{-\lambda t}\, for\ t > 0 \tag{31}$$

$$\lambda = q_c \tag{32}$$

Where:

- $q_c$ is the circulating traffic, expressed in *vehicle/h*

This distribution can be drawn from the assumption that the probability of a vehicle arrival in a small-time interval $(t, t + \delta t)$ is a constant and can also be obtained from the cumulative distribution function (Troutbeck & Brilon, 2001).

Luttinen (2004) found two main limitations of the exponential models: The model allows unrealistic distances and does not describe platooning, and it becomes more distorted as flows increase. Thus, the exponential distribution can be considered as a realistic headway model only under very low flow conditions, approximately $q < 150\ veh/h$. Additionally, headways of free vehicles which are not driving in platoons can also be described through the negative exponential distribution, it indicates that empirical headway distributions have an "exponential tail".

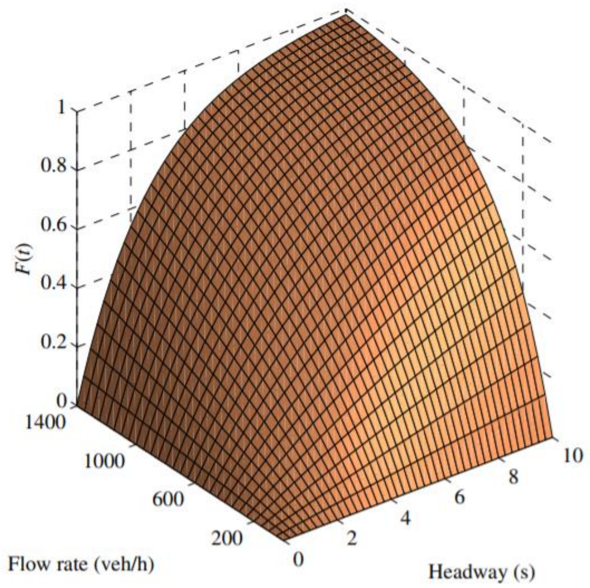



Figure 33 - Negative exponential cumulative distribution function for vehicle headways (Luttinen, 2004)

<u>Shifted exponential distribution (M2)</u>

The shifted or displaced exponential distribution assumes that there is a minimum headway between vehicles, $t_m$, this time can be considered the space around a vehicle that no other vehicle can intrude divided by the traffic speed (Silva *et al.,* 2013). This assumption is based on one part of intra-bunched vehicles with the minimum headway and the other are free flow vehicles (Kang *et al.*, 2012). When headways are larger than $t_m$the exponential distribution is called a shifted exponential distribution, the accumulative function obtained for headways is given by:

$$F(t) = 1 - e^{-\lambda(t-t_m)}\ for\ t \geq t_m \tag{33}$$

Scale parameter $\lambda$ is expressed in terms of flow rate as follow:

$$\lambda = \frac{q_c}{1 - t_m * q_c} \tag{34}$$

The sifted exponential distribution (M2) is conceptually better than the negative exponential distribution (M1) because avoids the problem of extremely short distances model, but it does not consider the platooning that can occur in a stream with higher flows. Thus, a dichotomized headway distribution will provide a more accurate fit (Luttinen, 2004; Silva *et al.,* 2013).

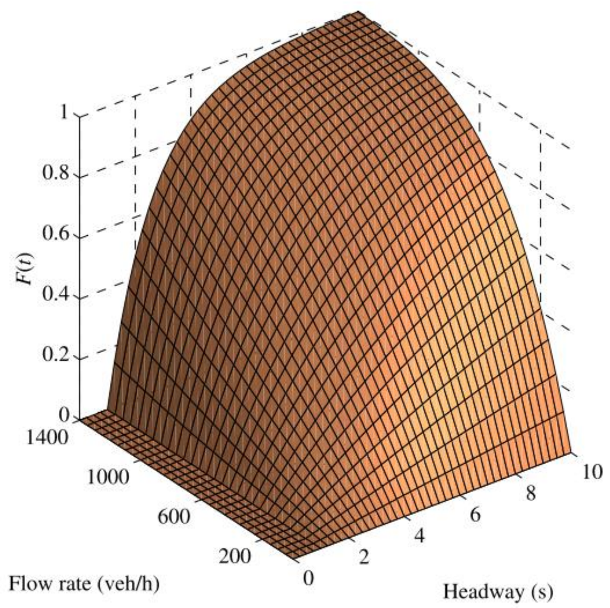



Figure 34 - Shifted exponential cumulative distribution function for a minimum headway of 1 second (Luttinen, 2004)

<u>Bunched distribution (M3)</u>

The M3´s Cowan headway distribution assumes that circulating vehicles arrive as a mixture of free flow and bunching vehicles, for which proportions are α and 1-α respectively. In this scenario the distances in the platoons are supposed to be constant ($t_m$) and the distances of the free vehicles follow a shifted exponential distribution (Cowan, 1975).

The accumulative function obtained for headways of M3´s distribution is described as follow:

$$F(t) = 1 - \alpha e^{-\lambda(t-t_m)} \; for \; t \geq t_m \tag{35}$$

Where $\alpha$ is the proportion of free-vehicle headways, and $\lambda$ is the scale parameter calculated with the following equation:

$$\lambda = \frac{\alpha * q_c}{1 - t_m * q_c} \tag{36}$$

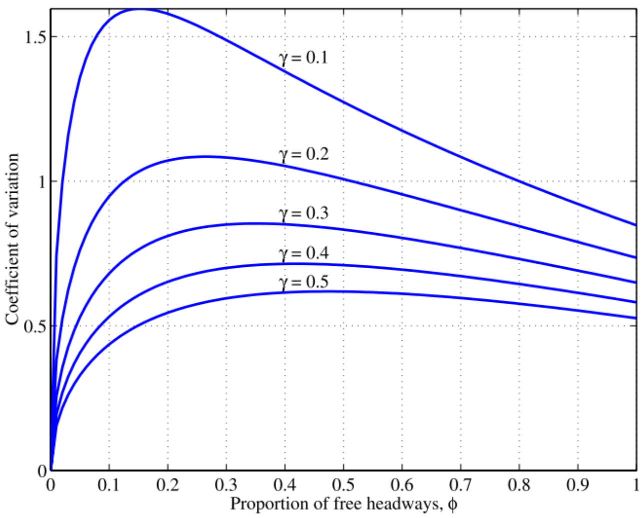



Figure 35 - Coefficient of variation of Cowan's M3 distribution for a distances in the platoons of 1.8 seconds (Luttinen, 2004)

According to Luttinen (2004), *a* M3 distribution is a suitable traffic flow model when short gaps no need to be modelled accurately, that is the case for unsignalized intersection analysis. The M3 distribution reproduces very similar data with moment characteristics to real headway distributions and gives good results in the analysis of the capacity of unsignalized intersection.

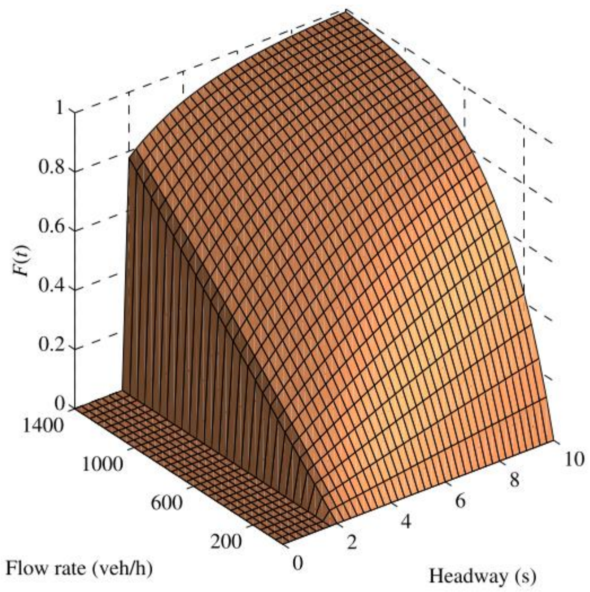



Figure 36 - Cumulative distribution function of headways from an M/D/1 queuing process with service time of 2 second (Luttinen, 2004)

### 3.4.2 Critical gaps (tc)

This parameter constitutes an important criterion in the gap acceptance theory for capacity calculation and the delay of minor traffic streams at roundabouts. A critical gap represents the minimum gap that a driver attempting to enter the intersection is willing to accept to manoeuvre into conflicting traffic flow or major stream (Kusuma *et al*., 2011; Guo *et al.,* 2014).

For roundabouts a gap is assumed to be accepted when the headway of the conflicting flow is larger than the critical gap, whereas the gap is rejected when the headway of the conflicting traffic is smaller than the critical gap. Various models are used for the estimation of this parameter, from which has been concluded that the average gap relates to the conflicting traffic speed, particularly to the speed at which the vehicles move on the carriageway of the roundabout, the number of the entry lane and proportion of turn-left lane (Guo *et al.*, 2014).

Although this is a factor that differs from driver to driver, Guo *et al*., (2014) claims that certain features of the roundabout such as pavement markings and entry angle may influence the decisions of the drivers. For instance, if there are no yield line-markings, vehicles may be confused about where to stop, forcing them to choose a place further away from the line to avoid interfering with the conflicting traffic flow, in this scenario, the critical gap is longer. On the other hand, a greater entry angle would provide better visibility to drivers entering the roundabout which facilitate drivers' judgement of the gap. Therefore, a shorter critical gap may occur.

Similarly, the truck apron marking that is used to ensure heavy vehicles can easily turn at the roundabout. In the absence of such markings, the width of the traffic roadway becomes wider and, consequently, the likelihood of parallel travelling of the conflicting vehicle increases. The judgement of the space for entering drivers may be difficult under these conditions. As a result, short gaps are rejected by drivers, which leads to an increase in the critical gap.

### 3.4.3 Follow-up times (ft)

The follow-up is the headway of entry vehicles defined as the average time gap in which multiple vehicles that are queued at the approach of the roundabout can enter to conflicting flow using the same gap. This time is related to the entry speed, which can be affected by some features of the roundabout such as the physical curb, the distance between the stop line and the yield line.

The physical curb at the entry of the roundabout increases the follow-up time, since forces drivers to reduce entry speed thereby limiting driver traffic. By contrast a longer distance between the stop line and the yield line increases the likelihood that drivers entering will speed up until reaching the yield line, leading to a higher entry speed and, as result, a shorter follow-up time can be expected under this condition (Kang, 2012). These parameters depend on the local conditions for each study area and is linked to driver behavior, traffic volume, traffic composition, and geometry. The low traffic speed that a roundabout can give facilitates acceptance at the entry and thus increases its capacity.

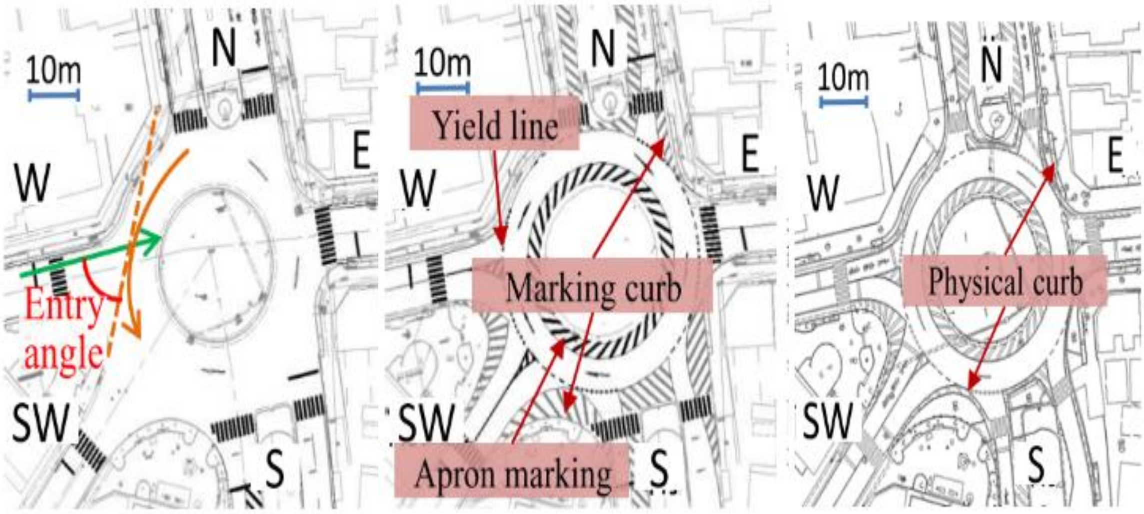



Figure 37 – Key roundabout features for roundabouts (Kang, 2012).

### 3.4.4 HCM 2010

The analytical method presented in the Highway Capacity Manual (HCM 2010) is an update of the method presented in the 2000 edition. The model is formulated from a study of roundabout performance through an evaluation at 31 sites in the United States *(*Boundaries *et al.*, 2010).

The capacity calculation is depending on the configuration of the entry and the number of lanes in conflict with the travel lane. The model allows estimating the capacity for one-lane and two-lane roundabouts. The American Guideline provides an exponential expression generalized for the calculation of the capacity, this general equation (equation 37) is based on the gap acceptance and includes two parameters, A and B, that depend on the critical gap and follow-up time as follows:

$$C = A \times e^{-B \times v_c} \tag{37}$$

Where:

$$A = \frac{3600}{t_f} \tag{38}$$

$$B = \frac{t_c - \frac{t_f}{2}}{3600} \tag{39}$$

- $t_f$: Critical gap, expressed in sec
- $t_c$: Follow-up time, expressed in sec

Table 11 – Capacity formulas for roundabouts (Boundaries *et al.*, 2010)

| Number of entry lanes | Number of conflicting lanes | Capacity model |
|---|---|---|
| One-entry Lane | One conflicting lane | $c = 1.130 \times e^{(-0.0010 v_{c.pce})}$ |
| | Two conflicting lanes | $c = 1.130 \times e^{(-0.0007 v_{c.pce})}$ |
| Two entry lanes with two conflicting lanes | Left entry lane | $c = 1.130 \times e^{(-0.00075 v_{c.pce})}$ |
| | Right entry lane | $c = 1.130 \times e^{(-0.0007 v_{c.pce})}$ |

Where:

- ***Ce,pce***: Lane capacity, adjusted for heavy vehicles, expressed in *pcu/h*.
- ***Vc,pce:*** Conflicting flow rate, expressed in *pcu/h*.

The capacity and efficiency of a roundabout are very sensitive to the $t_f$ and $t_c$ values, so they must be adjusted to local driver characteristics, geometry, and traffic flow conditions. The U.S.A guideline state that as drivers become more familiar with driving at these types of intersections generating higher capacities (Iguel *et al*., 2014). Table 12 summarizes the factors obtained from the study carried out by the Transportation Research Board in the U.S.A. factors are presented according to the configuration of the roundabout.

Table 12 – Critical gaps, $\boldsymbol{t_c}$ , and follow-up times $\boldsymbol{t_f}$ (Boundaries *et al.*, 2010)

| Number of entry lanes | Number of conflicting lanes | $\mathbf{t_f}$ | $\mathbf{t_c}$ |
|---|---|---|---|
| One-entry Lane | One conflicting lane | 5.19 | 3.19 |
| | Two conflicting lanes | 4.11 | 3.19 |
| Two entry lanes with two conflicting lanes | Left entry lane | 4.29 | 3.19 |
| | Right entry lane | 4.11 | 3.19 |

### 3.4.5 Australian model

The Australian model for capacity calculation was developed by the National Association of Australia State Road Authorities (NAASRA). To formulate this model, it was used the theoretical basis suggested by Tanner in 1962, in which two flow streams were considered, one major and one minor, the arrival to the intersection of both flows was assumed to be random and the entries at the intersection by the minor stream was considered occurring with a minimum vehicle-to-vehicle interval of T seconds (Iguel *et al.*, 2014 ; Tollazzi, 2015).

Following Tanner's approach, the following equation is used to estimate the capacity for multi-lane roundabouts by the Australian model (Autoroads, 2016):

$$Q_e = \frac{3600 \times q_c(1 - q_c \times \Delta) \times e^{-(q_c(T - \Delta))}}{1 - e^{(-q_c \times T_0)}} \tag{40}$$

Where:

- $\boldsymbol{Q_e}$: entry capacity, expressed in pcu/h
- $\boldsymbol{q_c}$: conflicting flow, expressed in pcu/sec
- $\boldsymbol{T}$: the critical gap, expressed in sec

- $\boldsymbol{T_0}$: the follow-up time, expressed in sec
- Δ: the minimum headway in the conflicting streams, which is 1 second for multilane and 2 seconds for single lane (Figure 38).

Horman and Turnbull based on Australian conditions suggested that the factors *T = 3-4 s*, $T_0$ *=2 s*, and *Δ = 1 or 2* are the most suitable for single-lane roundabouts, whereas for two-lane traffic flow the factors *T = 4 s*, $T_0 = 2$ s and *Δ = 0* are considered as providing a satisfactory prediction for entry capacity for conflicting traffic flows in the 300-2000 pcu/h range. Subsequently, Avent and Taylor verified the data calculated by Tollazzi through a study of three roundabouts in Brisbane and proposed that the data that best fit the characteristics of roundabouts in Australia should be $T$ *=2.5 s*, $T_0 = 2.1$ s, and *Δ = 2.2* for single-lane and two-lane roadways (Tollazzi, 2015).

| Number of effective circulating lanes | **One** | **More than one** |
|---|---|---|
| Average headway between bunched vehicles , Δ (s) | 2.0 | 1.0 |
| Circulating flow (veh/h)<br>0 | 0.250 | 0.250 |
| 300 | 0.375 | 0.313 |
| 600 | 0.500 | 0.375 |
| 900 | 0.625 | 0.438 |
| 1200 | 0.750 | 0.500 |
| 1500 | 0.875 | 0.563 |
| 1800 | 1.000 | 0.625 |
| 2000 | | 0.667 |
| 2200 | | 0.708 |
| 2400 | | 0.750 |
| 2600 | | 0.792 |

Figure 38 – Minimum headway in the conflicting streams (Kang, 2012)

The calculation of the capacity calculation also can be estimated through figure 39, where capacity is calculated as relationship between the entry and conflicting traffic volumes.

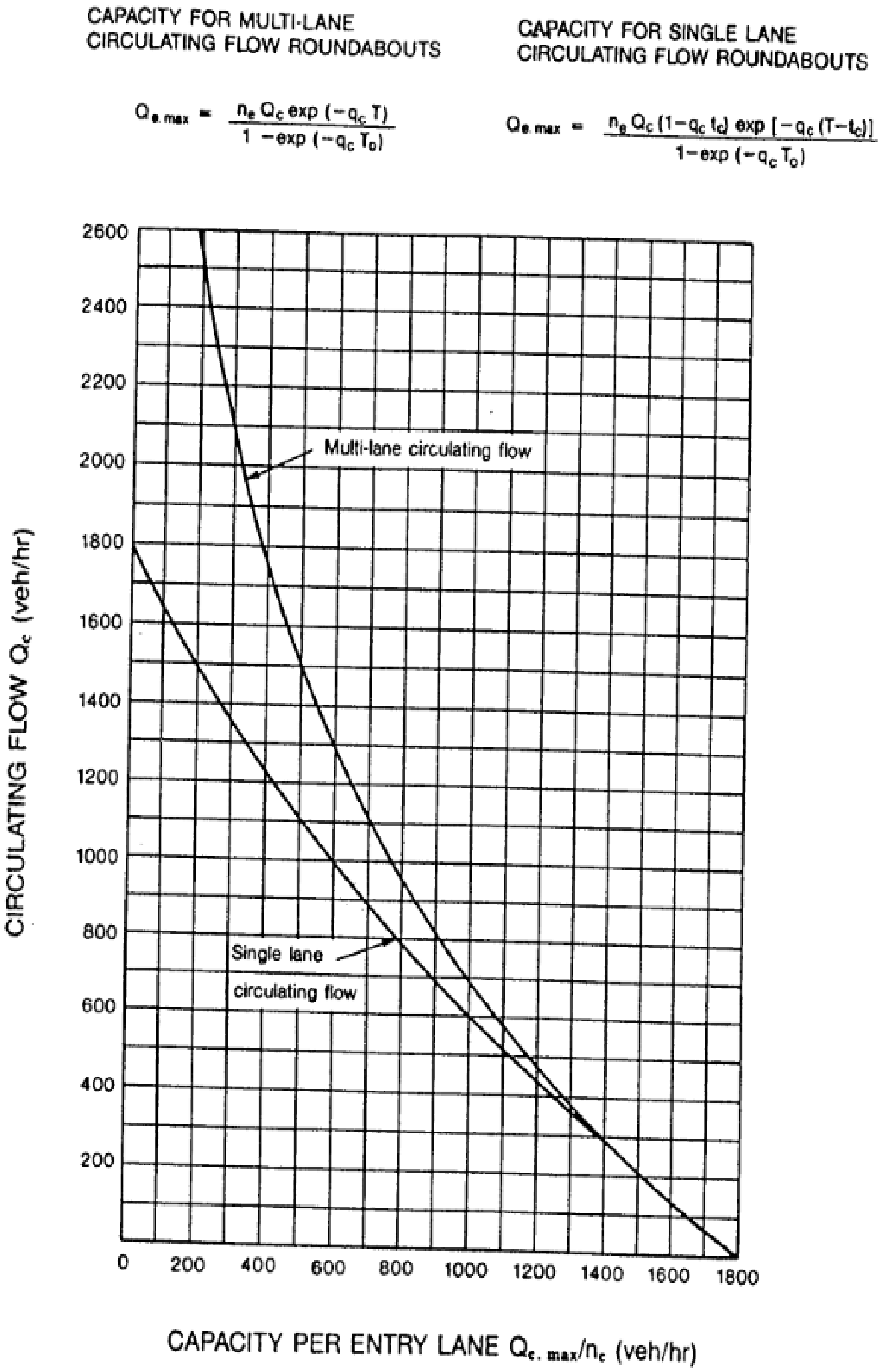



Figure 39 – capacity calculation by NAASRA model (Tollazzi, 2015).

### 3.4.6 German model

In Germany, an analytical model was proposed by Brilon and Wu, they adjusted Tanner's equation for calculating roundabout capacity to the requirements for the analysis of German roundabouts (Tollazzi, 2015; Wu & Brilon 2018) . This method is the official procedure used to calculate capacity for each entry ($G$) for a double-lane roadway and double-lane entries. The capacity is for each entry is estimated as follows by equation 41:

$$G = 3600 \times \left(1 - \frac{t_{min} \times q_k}{n_k \times 3600}\right)^{n_k} \times \frac{n_z}{t_f} \times e^{-\frac{q_k}{3600} \times \left(t_g - \frac{t_f}{2} - t_{min}\right)} \quad (41)$$

Where:

- $\boldsymbol{G}$: Capacity of one entry lane, expressed in pcu/h
- $\boldsymbol{q_k}$: Conflicting traffic flow, expressed in *pcu/h*
- $\boldsymbol{n_k}$: Number of conflicting lanes
- $\boldsymbol{n_z}$: Number of entry lanes
- $\boldsymbol{t_g}$: Critical gap, expressed in sec
- $\boldsymbol{t_f}$: Follow up time, expressed in sec
- $\boldsymbol{t_{min}}$: Minimun gaps beteween succeding vehicles on the circle, expressed in sec

Tollazzi (2015) suggests that the values for the critical gap, follow-up time and minimun gaps beteween succeding vehicles on the circulatory roadway with conflicting flows in the range of 300-2000 *pcu/h* which suit the realities of German drivers on the roundabout are: $t_g = 4.12$ *s*. $t_f = 2.88$ *s* and $t_{min}$= *2.10 s,* respectively.

Additionally, the German Highway Capacity Manual (HBS) developed a graph to compute the capacity calculation taking into account the number of lanes at the entry and at conflicting roadway of the roundabout (Brilon, 2011a). Figure 40 indicates the numbers of lanes at entry/circulatory roadway.

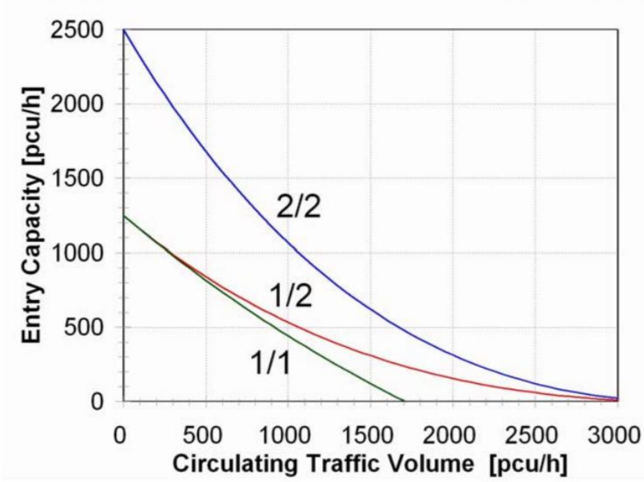



Figure 40 – Entry capacity of roundabouts according to the German Highway Capacity Manual (Brilon, 2011a)

Table 13 and 14 summarize the parameters for the calculation of capacity by each of the methodologies.

Table 13 – Parameters for capacity evaluation of empirical models

| PARAMETERS | MODELS | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | TRL | SETRA | CERTU | FCTUC | GERMAN EXPONENTIAL | GERMAN LINEAR | COLOMBIAN | DUTCH |
| Approach width | ✓ | | | ✓ | | | | |
| Entry width | ✓ | ✓ | | ✓ | | | ✓ | |
| Effective flare length | ✓ | | | ✓ | | | | |
| Entry radius | ✓ | | | ✓ | | | | |
| Inscribed circle diameter | ✓ | | ✓ | ✓ | | | | |
| Entry angle | ✓ | | | ✓ | | | | |
| Circulatory roadway width | | ✓ | ✓ | | | | ✓ | |
| Splitter island width | | ✓ | | | | | | |
| Number of entry lanes | | | ✓ | | ✓ | ✓ | | |
| Number of conflicting lanes | | | ✓ | | ✓ | ✓ | | |
| Length of the weaving section | | | | | | | ✓ | |
| Conflicting flow | ✓ | ✓ | ✓ | ✓ | | | | ✓ |
| Exiting flow | | ✓ | ✓ | | | | | ✓ |

Table 14 – Parameters for capacity evaluation of analytical models

| PARAMETERS | MODELS | | |
|---|---|---|---|
| | HCM 2010 | AUSTRALIAN | GERMAN |
| Number of entry lanes | ✓ | ✓ | ✓ |
| Number of conflicting lanes | ✓ | | ✓ |
| Critical gap | ✓ | ✓ | ✓ |
| Follow-up time | ✓ | ✓ | ✓ |
| Conflicting flow | ✓ | ✓ | ✓ |
| Minimum headway in the conflicting streams | | ✓ | |
| Minimum gaps between succeeding vehicles in conflicting streams | | | ✓ |

## 3.5 Testing roundabout capacity models for a specific test scenario

This section is a test model scenario that evaluates the calculation procedures of the models described in sections 3.3 and 3.4 to determine the main difficulties in the model calculation procedures that are going to be evaluated in the case study in Chapter 4.

The data utilized for this examination corresponds to a roundabout with no saturation, *i.e.*, the volume-to-capacity ratio is less than 1.0 and preferably less than 0.85.

In the empirical analysis, the Dutch model has not been evaluated since it only allows for the analysis of single-lane entries and has a simple calculation procedure that incorporates no roundabout geometric elements.

### 3.5.1 Base scenario

For calculation purposes is considered a four-arm roundabout with the following geometric characteristics where for calculation purposes all four entries are considered to have the same geometrical characteristics (Figure 41). The traffic flows data for the base scenario are displayed in below.

- <u>Base geometric features</u>

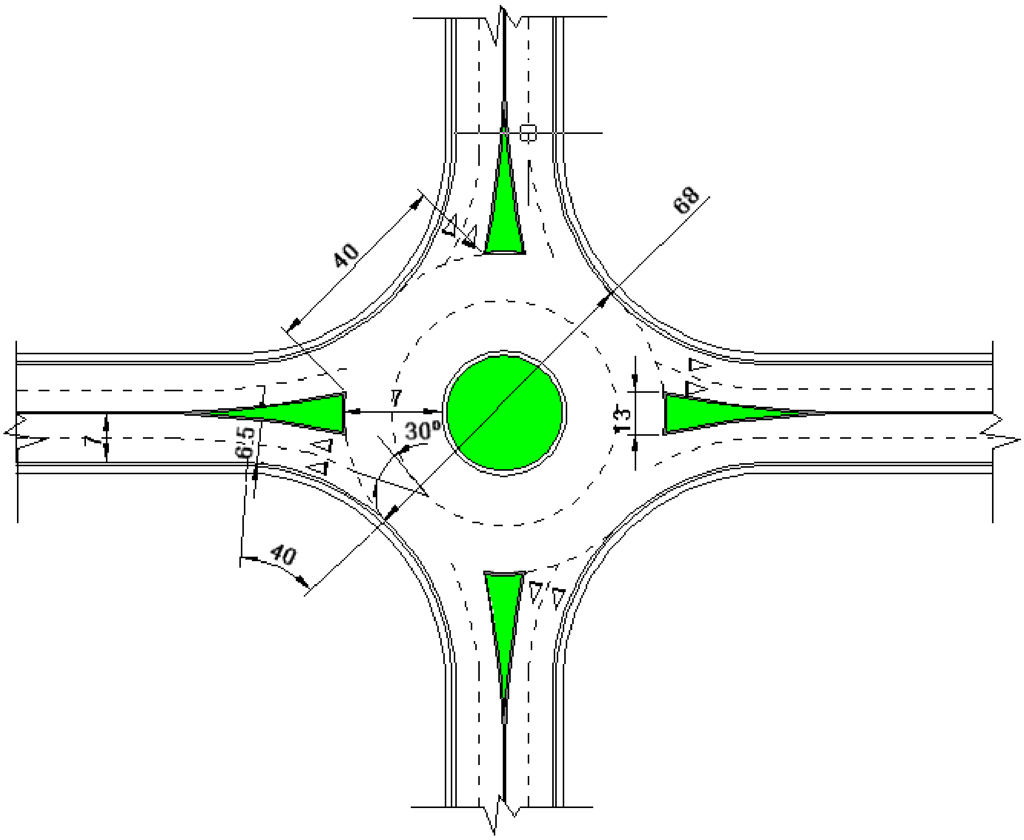



Figure 41 – Base geometric parameters for testing models (Author)

- <u>Traffic flow</u>

The traffic data used to test each of the roundabout approaches are shown in the O/D matrix (Table 15).

Table 15 – Proportions of traffic for testing models in pcu/h

| O/D | A | B | C | D |
|---|---|---|---|---|
| **A** | 0 | 120 | 98 | 104 |
| **B** | 35 | 0 | 74 | 135 |
| **C** | 240 | 131 | 0 | 101 |
| **D** | 140 | 234 | 250 | 0 |

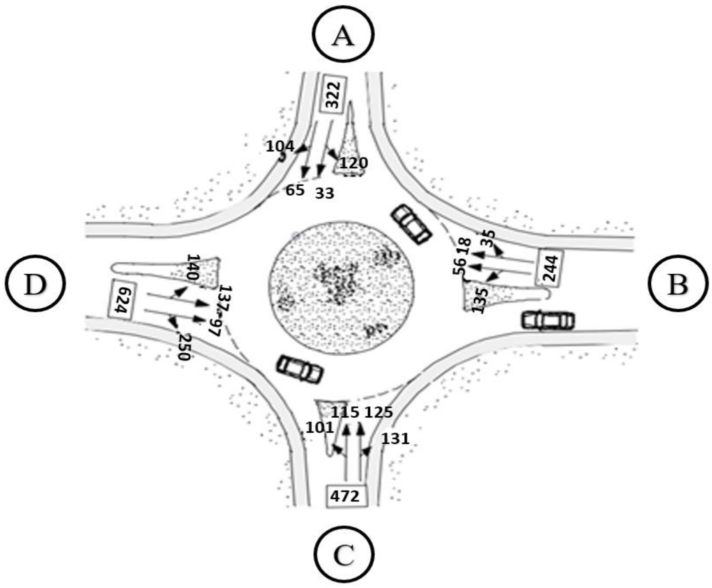



Figure 42 – Distribution of the traffic flow at each of the entries (Adapted from: Bastos, 2008)

As models requires some specific traffic flow parameters to estimate the entry capacity of the roundabout, the conflicting and exiting flow is computed for each of the entries (See appendix A). The results obtained for each of the models are shown below:

Table 16 – Results of capacity model testing

| **Capacity models** | | | | |
|---|---|---|---|---|
| **Model** | **Entry A** | **Entry B** | **Entry C** | **Entry D** |
| TRL | 1864 | 1765 | 1758 | 1871 |
| SETRA | 1387 | 1212 | 1204 | 1411 |
| CERTU | 1877 | 1449 | 1714 | 1911 |
| FCTUC | 2094 | 1916 | 1903 | 2213 |
| GERMAN EXPONENTIAL | 1262 | 1126 | 1116 | 1277 |
| GERMAN LINEAR | 1535 | 1621 | 1627 | 1526 |
| COLOMBIAN | 1055 | | | |
| DUTCH | No tested | | | |
| HCM 2010 | 1340 | 1306 | 1223 | 1125 |
| AUSTRALIAN | 1505 | 1394 | 1386 | 1517 |
| GERMAN | 1968 | 1710 | 1691 | 1996 |

From the results obtained for the geometric traffic flow conditions presented, it can be observed that the Colombian model shows the lowest capacity levels of all the models tested, except for the model that was not tested since it does not provide a methodology for the estimation of multi-lane roundabouts.

In the empirical models the highest capacity results are obtained through the Portuguese method (FCTUC), furthermore the SETRA and the German exponential approach for the given conditions reflect similar capacity data. When compared to the other empirical models analysed, the German exponential model estimates lowest capacities.

From the analytical models tested, the German analytical model estimates the highest capacity results of all models. In addition, the HCM 2010 methodology provides comparable results to those of the empirical SETRA methodology.

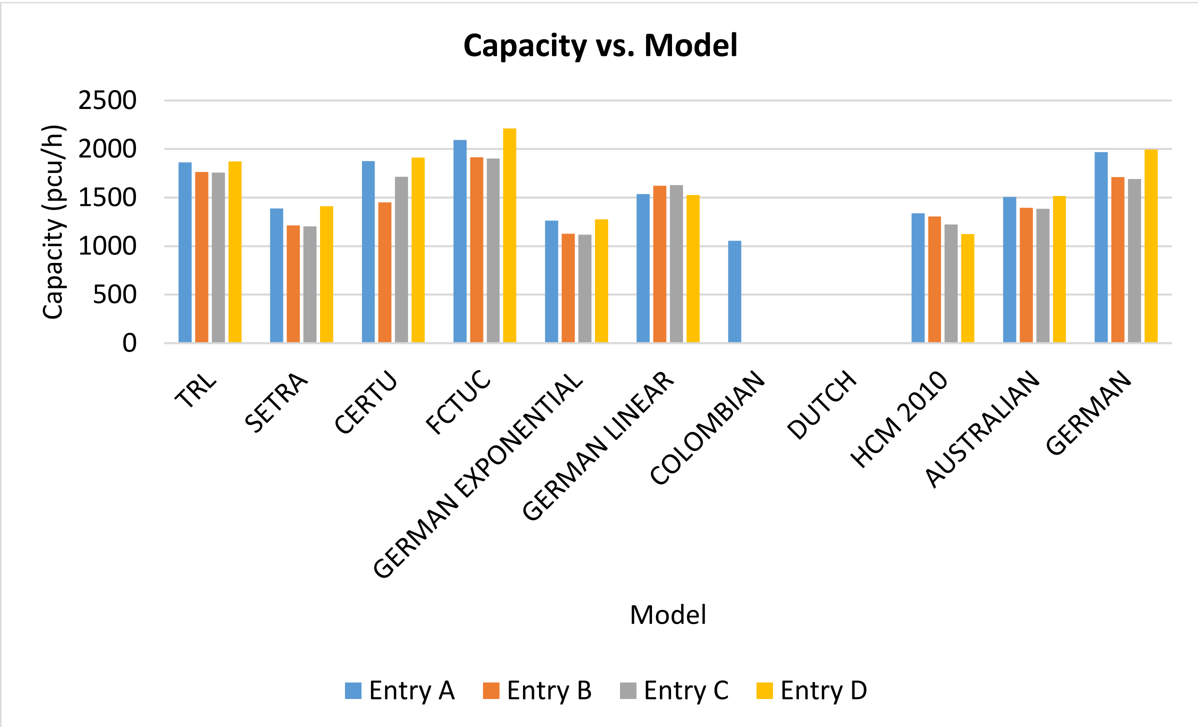



Figure 43 – Results of model testing

### 3.6 Performance indicators

Although capacity is normally used as a decisive indicator in evaluating the performance of roundabouts, other measures, such as delay, queue length and degree of saturation, can be used to analyze traffic performance at these intersections in more detail. As mentioned by Humoody (2007), the quality of traffic service offered by a roundabout under certain traffic conditions and geometric features can be validated using these parameters as a measure of effectiveness. However, a capacity estimation must be obtained for the roundabout entries before the calculation of one of these performance measures. A brief definition of the performance indicators at roundabouts is summarized in Table 17.

Table 17 - Performance indicators (Russell *et al.*, 2000)

| Performance indicators | Description |
|---|---|
| 95% Queue Length | Total length of the queues for all approaches at the 95% confidence level. Queue lengths were based on a vehicle length of 8 meters. |
| Average Intersection Delay | Average vehicle delay for all entering vehicles (sec/veh). |
| Maximum Approach Delay | Average vehicle delay for the approach with the highest average vehicle delay (sec/ veh). |
| Proportion Stopped | Proportion of entering vehicles that are required to stop due to vehicles already in the intersection. |
| Maximum Proportion Stopped | Proportion of entering vehicles that are required to stop due to vehicles already in the intersection on the approach with the highest proportion stopped value. |
| Degree of Saturation | Amount of capacity that is consumed by the current traffic loading (commonly referred to as the v/c ratio). |

### 3.6.1 Delay

Delay is a standard parameter for evaluating intersection performance (Humoody, 2020). As reported by (Akgüngör, 2008). Delay is one of the most important parameters for measuring effectiveness and determining service level (LOS) which can be measured in the field or using analytical methods. This parameter can be measured in the field or using analytical methods.

Delays experienced by drivers at roundabouts occur due to different factors that are associated with the traffic control, its geometry, the traffic flows, and accident occurrence, hence several definitions of delays must be explained to distinguish the type of delays when comparing delay models (Akçelik, 2017).

In concordance with Luttinen (2004), when analyzing unsignalized intersections, delays due to vehicle interaction (traffic delays) are included in the control delays. Furthermore. the consequences of occurrences, such as road maintenance, are examined individually. Some approaches for determining delay effects at roundabouts have been proposed in the literature; nevertheless, entry capacity must be estimated before calculating delay because it is a key parameter in all delay methods. There are mainly three types of delays at roundabouts: queuing, geometric, and control delay.

- **Control delay**

Control delay is the sum of the time spent by a driver from queuing to waiting for an acceptable gap in the traffic flow, while at the front of the queue (Humoody, 2020). An average control delay formula is provided by the Highway Capacity Manual of United Stated (Boundaries *et al.* 2010), the guideline only includes control delay as a measure of effectiveness for signalized and unsignalized intersections.

Conforming to Boundaries *et al.,* (2010), a roundabout is similar to the one used for the evaluation of the delay at unsignalized intersections since it assumes that roundabouts share the same basic control formulation with two-way and all-way intersections with STOP control in all directions. For adjusting to the effect of YIELD control, a factor of "+5" is included into the formula, this factor is related to the YIELD control performance of the approach on the subject.

The Hight Capacity Manual (HCM) present a procedure for the calculation of the average control delay for each approach of the roundabout individually, the average control delay (d) is a function of the lane capacity, and the degree of saturation is as follows:

$$d = \frac{3600}{C_{m.x}} + 900T \times \left[ \left( \frac{V_x}{C_{m.x}} - 1 \right) + \sqrt{ \left( \frac{V_x}{C_{m.x}} - 1 \right)^2 + \frac{\left( \frac{3600}{C_{m.x}} \right) \left( \frac{V_x}{C_{m.x}} \right)}{450\, T} } \right] + 5 \quad (42)$$

Where:

- $\mathbf{d}$: average control delay, expressed in sec/h
- $\mathbf{V_x}$: flow rate for movement x, expressed in veh/h
- $\mathbf{C_{m.x}}$: capacity of the movement x, expressed in vehicles/h, and
- $\mathbf{T}$: analysis period (T=0.25; 15 minutes is generally considered).

When the degree of saturation is more than roughly 0.9, the time of analysis period has a substantial effect on the average control delay. As a result, the suggested analysis time is 15 minutes (NCHRP, 2010 ; Boundaries *et al.*, 2010). Control delay rises exponentially as volumes approach capacity, implying that minor changes in volume have enormous implications on delay. Two effects must be considered when evaluating delay in near- or over-saturated conditions:

- <u>The first effect is that of residual queues:</u> Noaeen *et al.,* (2020) state that a residual queue is visible when an intersection is oversaturated because all vehicles are unable to reach the intersection, causing delays or even blocking the intersection.
  A roundabout entry that is close or at capacity might produce substantial residual waits that must be accounted for between subsequent periods. This effect is not taken into

consideration in the HCM approach. These parameters are taken into consideration in the delay equations proposed by Kimber and Hills (Humoody, 2020).

- The second is the metering effect of upstream supersaturated elements: When an upstream entry is overloaded, the conflicting volume in front of a downstream entry is less than the real demand. Therefore, the capacity of the downstream entry is greater than the capacity predicted based on the analysis of actual demand. Some empirical observations using video on-site estimation demonstrate that the roundabout stop delay can range from 12 to 26 sec, for an average delay in peak volume (Humoody, 2020).

Figure 44 illustartes how the control delay at an entry varies with entry capacity and conflicting flow, the greater the capacity of the roundabout, the shorter the vehicle delay.

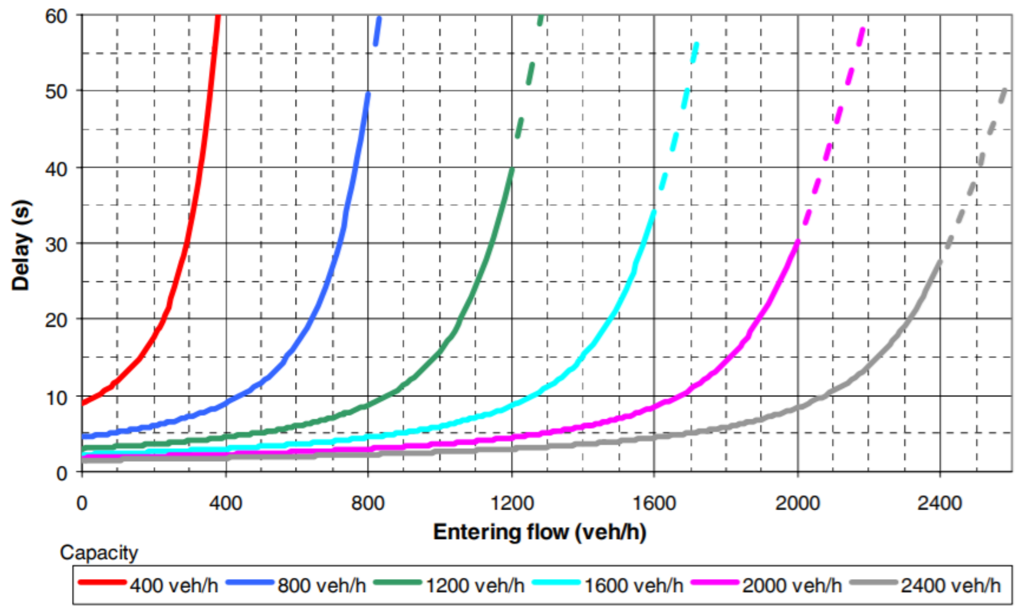



Figure 44 - Control Delay as a Function of Capacity and Entering Flow (KDOT, 2003).

- **Queuing delay**

Queuing delay is caused by time varying traffic volumes, delays and queue lengths are calculated using the theory of time dependent queue formation. The time spent in a line is referred to as queuing delay, this time does not include the significant acceleration and deceleration. This delay is equivalent to the control delay as described in the as described in the HCM 2010, less the geometric delay and the main stop/start delay (Humoody, 2020).

The equation developed by Kimber and Hollis in 1979 in the United Kingdom is based on "*The time-dependent queuing technique"*, the technique entails analyzing the probability distribution of different queue lengths as a function of time and based on these probabilities to calculate the average queue length, which is then used to compute the average queuing delay. Because this approach is computationally expensive, equations that provide a decent approximation to the average queues estimated from the probabilistic theory have been devised, The Kimber and Hollis method is described as follow (Tollazzi, 2015).

The methodology considers a short time interval, t, during which the demand flow, q, and capacity μ (where μ ≡ Qe), it may be assumed to be approximately constant. There are various scenarios based on the correlation between the ratio of flow and capacity, ρ, (ρ=q/μ), and, if ρ<1, on the relative values of Lo (the queue at the start of the time interval under consideration), and l (l=ρ/1-ρ), the equilibrium queue length.

If is $Fn$ a queuing function defined for $x$ (a time variable) by:

$$Fn\ (x) = \{((\mu \cdot x \cdot (1\ -\ \rho) +\ 1)^2\ + 4 \cdot \rho \cdot\ \mu \cdot x)^{\frac{1}{2}} - \{(\mu \cdot x \cdot (1 - \rho) + 1)\} \tag{43}$$

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

$For\ \rho \geq 1: Fn(t + t_0)$

*Where;*

$$t_0 = \frac{L_0(L_0 + 1)}{\mu \cdot (\rho \cdot (L_0 + 1) - L_0} \tag{44}$$

$For\ \rho < 1$:

$Option\ (A)$:

$$0 \leq L_0 < l : \qquad L(t) = Fn(t + t_0)$$

*Where;*

$$t_0 = \frac{L_0(L_0 + 1)}{L_0 \cdot (L_0 + 1) - L_0} \tag{45}$$

$Option\ (B)$:

$$L_0 = l : \qquad L(t) = l$$

$Option\ (C)$:

$$l < L_0 \leq 2l : \qquad L(t) = 2l - Fn(t + t_0) \tag{46}$$

*Where;*

$$t_0 = \frac{(2l - L_0) \cdot (2l - L_0 + 1)}{\mu \cdot (\rho \cdot (2l - L_0 + 1) - (2l - L_0))} \tag{47}$$

$Option\ (D)$:

$$L_0 > 2l : \qquad L(t) = L_0 + \left(\frac{\rho - L_0}{L_0 + 1}\right) \cdot \mu \cdot t \quad if \quad 0 \leq t \leq t_c \tag{48}$$

$$L(t) = 2l - Fn\,(t - t_c) \qquad if \qquad t > t_c \tag{49}$$

*Where:*

$$t_c = \frac{2l \; - L_0}{\mu \cdot \left(\frac{\rho \cdot L_0}{L_0 \; + \; 1}\right)} \tag{50}$$

Delay (D) per unit time may be computed as follows:

$$D = \frac{1}{t}\int_0^t L(t)\, dt \tag{51}$$

Tollazzi (2015) maintains that for giving the entry flow, the capacity (and hence the ratio of flow to capacity) for a time segment, and the queue length at the beginning of the segment, the queue length at the end of the segment is calculated. Time segments are treated sequentially by using the end queue for one segment as the start queue for the next. Thus, maximum delays can be determined. Generally, the queue at the beginning is assumed to be zero.

- **Geometric delay**

Vehicles experience delays due to the geometric characteristics of the roundabout; this delay occurs even in the absence of other vehicles since vehicles must reduce their speeds to reach the intersection, deviating from their normal trajectory, and then accelerate to the prior speed (Humoody, 2020).

This delay affects all vehicles at the intersection regardless of the time of day. Tollazzi (2015) also argues that a geometric delay is more perceptible during off-peak hours since the queuing delay is greater during peak hours. Thus, if a more complete evaluation of performance is needed, an estimate of geometric delay is appropriate, the estimation of a geometric delay is an important consideration when comparing roundabouts operation with different intersection types or even in network planning when is desired to know the effects of this delay on the route travel times and options.

The geometric delay at the roundabout depends on the proportion of vehicles that must stop at the entry line, the understanding of roundabout geometry and how it affects vehicle speed. Two factors that determine the proportion of stopped vehicles are the following (Tollazzi, 2015):

- If the traffic flow on the major road increases, the likelihood of getting stopped also increases. Hence, the likelihood of getting stopped is likewise proportional to the degree of saturation on the minor road.
- There will be a significant queue if the degree of saturation is high, therefore the likelihood of being stopped increases.

### 3.6.2 Degree of saturation

The degree of saturation is defined as the ratio of the demand at the roundabout entry to the capacity of the entry (KDOT, 2003). There is not a standards value for degree of saturation. However, for design purposes is recommended a maximum degree of saturation of 0.85, when the degree of saturation exceeds this range, it is more likely that the performance of the roundabout performance decreases significantly, especially over short time periods (NCHRP, 2010). Higher levels of saturation might cause an unstable operation in which high delays and lengthy queues may occur at the roundabout approach (KDOT, 2003).

The formula for computing the degree of saturation is as follows:

$$x_i = \frac{v_i}{c_i} \tag{52}$$

Where:

- $\boldsymbol{x_i}$: volume to capacity ratio at the entry of subject lane i
- $\boldsymbol{v_i}$: demand flow rate of the subject lane, i, expressed in veh/h
- $\boldsymbol{c_i}$: capacity of the subject lane, i, expressed in veh/h

### 3.6.3 Queue length

When evaluating the adequacy of the geometric design of the roundabout approaches, the queue length is an important factor (KDOT, 2003). In the literature, three types of queue length can be determined delay (Humoody, 2020):

- Average queue length: this queue length is the hourly average of two-minute maximum queues. Humoody (2020) states that the average queue length on an approach is equal to the vehicle-hours of delay per hour. When comparing roundabout performance with other intersection forms the average queue length is a useful parameter for measuring the effectiveness.

  Little's method can be used to compute the average queue length ($LQ$ vehicles), the formula for estimating this parameter is the following:

$$LQ = \frac{v \cdot d}{3600} \tag{53}$$

  Where:

  - $\boldsymbol{v}$ **:** the entering traffic flow, expressed in veh/h
  - ***d:*** the average delay, expressed in second, for each vehicle.

- 95th-percentile queue length: For design purposes it is determined the maximum resulting queue for a given approach.

$$Q_{95} = 900 \cdot T \left[ \left( \frac{V_x}{Cm_x} - 1 \right) + \sqrt{\left( 1 - \frac{V_x}{Cm_x} \right)^2 + \frac{\left( \frac{3600}{Cm_x} \right) \cdot \left( \frac{V_x}{Cm_x} \right)}{150 \cdot T}} \right] \left( \frac{Cm_x}{3600} \right) \tag{54}$$

Where:

- $\boldsymbol{Q_{95}}$: 5th percentile queue, expressed in veh
- $\boldsymbol{V_x}$: Flow rate for movement x, expressed veh/h
- $\boldsymbol{Cm_x}$: capacity of movement x, expressed in veh/h
- $\boldsymbol{T}$: analysis time period, h (0.25 for a 15-minutes period).

- Maximum queue length: the maximum queue length recorded over the entire simulation.

### 3.7 Simulation models

Traffic modeling is an effective tool to simulate and recreate accurately traffic as observed and measured on the street, which enables a wide range of traffic problems to be analyzed. Traffic modelling has been developed based on the experience of modelers integrating mathematical models into traffic systems and plays an important role in traffic engineering.

Simulation allows a better understanding of the nature of processes through the identification of the specific problems and the critical points of a system, before the implementation of projects to plan and manage traffic on a given road network.

A simulation can be classified into three main categories:

- **Deterministic or Stochastic:** A simulation is deterministic when all variables involved are deterministic. The stochastic simulation is a more complex representation of reality as it consists of two or more simulation variables that are presented as samples.
- **Static or dynamic:** A simulation is defined as static when time is an irrelevant variable, however, most of the application models consider time as an important factor, in these cases, it is characterized as dynamic.
- **Discrete or continuous:** In a discrete simulation, the simulation is made in intervals of time and on the assumption that variables are not altered during this time interval. The

passage of vehicles is a continuous simulation even though it is performed in short intervals due to the computational and numerical methods employed.

### 3.7.1 Classification of traffic simulation models

Simulation models are grouped in accordance with the level of aggregation and detail of their application scopes, traffic simulation models are classified into three categories: the microscopic, the macroscopic and the mesoscopic modelling, each category is explained below.

- **Macroscopic Models**

The macroscopic model has the highest level of aggregation and the lowest level of detail (Naukowe *et al.*, 2020), being considered the most appropriate for the design of control strategies since it describes traffic flows analytically and requires less execution time. Macroscopic models are called continuous flow modelling. for carrying out the analysis, the traffic flow is treated as a continuous stream that flows across the road. Nevertheless, traffic is a non-continuous flow that evolves simultaneously in space and time, so that its macroscopic variables will be meaningless if the average values are not used to characterize the traffic, thereby two different types of averages. temporal and spatial, are considered (Nor *et al.*, 2018). Hence, the measurement of traffic is done concerning the three characteristics including speed, flow and density, the behavior of these variables has been described by researchers to define the mathematical relationship of the variables (Rao *et al*., 2007). Thus, the mathematical theory used in macroscopic models is explained through Greenshields Model and the Greenberg Model (Nor *et al.*, 2018; Rao *et al.*, 2007).

- **Mesoscopic models**

Mesoscopic models have a high level of aggregation and a low level of detail (Naukowe *et al.*, 2020). These models are combination of microscopic and macroscopic modelling in which transport elements are analysed in small groups to simulate the dispersion of the platoon (Nor *et al.*, 2018).

There are two methods of mesoscopic modelling, the platoon dispersion and vehicle platoon behaviour. The platoon dispersion method occurs when a platoon moves downstream from an upstream intersection, in this case the distance between vehicles which may be due to differences in vehicle speeds, vehicle interactions such as lane changing and other interference. pedestrian parking and others. Whereas, the behaviour of the vehicle platoon consists of forecasting the arrival of the vehicle platoon over time, the total time of arrival in which it is considered a group of vehicles travelling a short headway and moving at the same speed (Nor *et al.*, 2018).

## 3.8 Microscopic models

Microscopic modelling is characterized by its low level of aggregation and a high level of detail, as they are a closer representation of reality (Naukowe *et al.*, 2020). When studying interactions of vehicles, more details lead to increased complexity so that they require more calibration effort in comparison to macroscopic and mesoscopic models. Microscopic models are based on the characteristics of different vehicle movements in the traffic flow, such as cars, buses, motorbikes, etc. Through these, data related to parameters such as flow, density, speed, travel time and delay, long queues, stops, pollution, fuel consumption and shock waves can be collected. In microscopic modelling, three main models are identified: car-following models, lane change models and individual driver gaps (Nor Azlan *et al.*, 2018).

### 3.8.1 Car following models

According to Nor *et al.*, (2018), in the car following theory the relationship between the preceding and the following vehicle is described as each vehicle is always decelerating or accelerating as a response to its surrounding stimulus, these models can be described as a situation like a platoon of cars unable to change lanes.

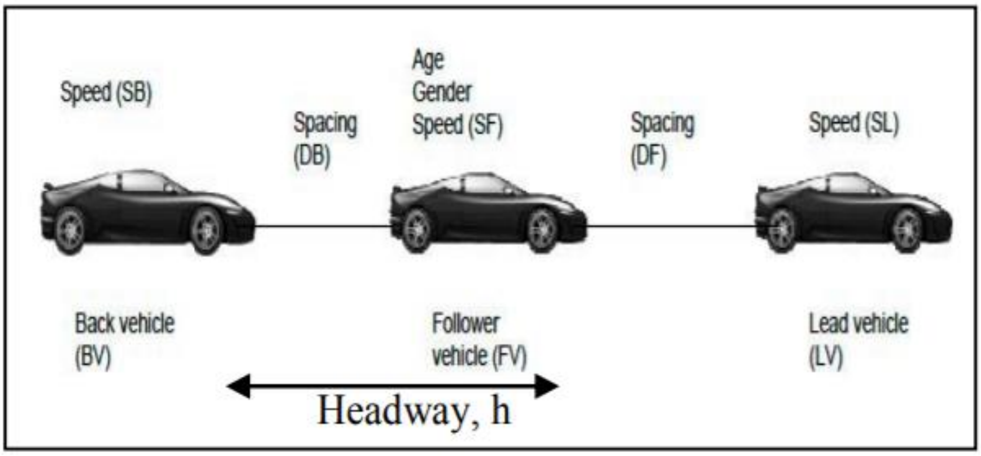



Figure 45 – Car following (Nor Azlan *et al.,* 2018).

Typically, the following car algorithm determined the identifies the interaction of vehicles with the lead vehicle and their distribution in a traffic flow, sometimes as a function of spacing, speed and acceleration at some times, since the closer the following vehicle is to the lead vehicle, more sensitive is the reaction of the following vehicle to the lead vehicle (Sullivan *et al.*, 2004; Nor Azlan *et al*., 2018). This sensitivity increases also with the speed because if the leading vehicle is moving at a slower speed, the following vehicles will slow down. This results in car platooning and traffic congestion.

Microscopic modelling is focused on the movement of individual vehicles and their relative time and space. The temporal separation is defined as the difference in time between vehicles successively intersecting at the point. Above (Figure 45), the time headway (h) is the horizontal distance between two vehicles. The separation (s) is the distance measured from the rear bumper of the following vehicle to the rear bumper of the preceding vehicles that pass-through a given point within a given time interval (Nor *et al.,* 2018).

### 3.8.2 Lane changing models

According to (Sullivan *et al.*, 2004), micro-simulation models use various algorithms and models of driver behavior to simulate the movement of individual vehicles in a network. Lane changing algorithms control vehicles to merge, mix and make lane changes within the traffic stream.

Lane changes are complicated manoeuvres that involve driver behavior, vehicle performance and inbound conditions behavior of drivers, vehicle performance and conditions within the traffic stream (Arjona, 2013).

The lane change model has been proposed by Gipps, the model estimates the driver's lane change behavior within the given time through a decision process. Two categories of lane change have been established. The first, which is known as a Mandatory Lane Change (MLC), occurs when a driver changes lanes to a given lane. For example, a driver changes to the right-hand lane when he/she wants to make a right turn at the next intersection. The second category is the Discretionary Lane Change (DLC), where the driver moves to the next lane to avoid following trucks and to increase speed (Nor *et al.,* 2018).

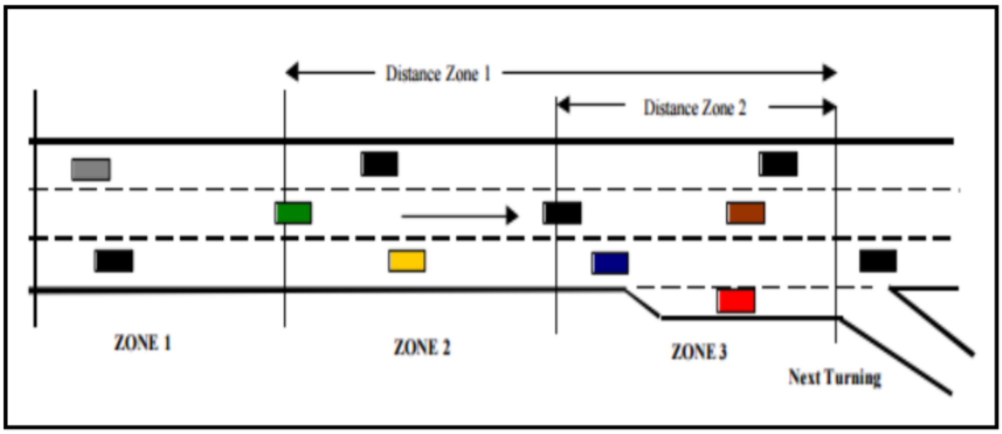



Figure 46 - Lane Changing (Nor *et al.*, 2018)

To analyze vehicle behavior, three zones are identified on the road *(*Nor *et al.,* 2018), as is illustrated in figure 46*:*

- **Zone 1** corresponds to the furthest distance from the next turn to measure vehicle lane change and driver speed, the speed and distance of the leading vehicles, and the speed and distance of the next leading vehicles in the other lane.
- In **zone 2** or the intermediate zone, vehicles look for a gap to turn into the turning lane, which affects the decision to change lanes.
- **Zone 3** is the shortest distance to the next turning point. In this case, the vehicle has started to change lanes in the merging lane to the next turn, so, it is necessary to reduce speed to provide ample space for the vehicle to make the manoeuvre and change lanes.

### 3.8.3 Gap acceptance models

The gap acceptance models were used to determine the size of the gap that will be accepted or rejected by a driver intending to merge or cross the intersection. Gap acceptance algorithms control how simulated vehicles turn into or through conflicting traffic (Arjona, 2013; Sullivan *et al.*, 2004). The parameters of the gap acceptance model are acceleration rate, desired speed, and speed acceptance. However, the acceleration rates, which is the ability of the vehicle to accelerate with the required safety gap, the maximum yield time to determine when the driver becomes intolerant if he/she cannot identify the gap, and the sight distance at the intersection is the most important.

As shown in figure 47, the gap is defined as the temporary distance between the leading and trailing vehicles in the target lane, while the passing gap is the space to the lead vehicle in the target lane. The delay gap is the gap to the delaying vehicle in the target lane. The critical gap is defined as the number of accepted gaps shorter than the number of rejected gaps, taking into account that the driver needs time to clear the intersection and decide.

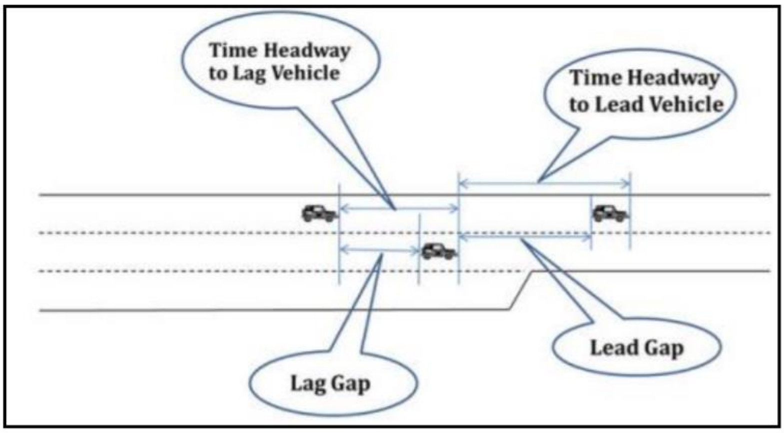



Figure 47 – Gap acceptance (Nor Azlan *et al.,* 2018)

### 3.8.4 Traffic simulation software

A variety of simulation software packages are available for modeling transportation networks. Various of these can model roundabouts, and frequently changing features. These models show individual vehicles and are therefore sensitive to the factors at that level: car-following behavior, carriageway behavior, lane-changing behavior, and traffic decisions making at intersections.

Since 1990, the most used simulation methods most employed are based on research and practice in the USA, the U.K, and Germany. When using these software packages, special attention should be given to ensure that the simulation model is applied correctly (Rivas et al., 2020):

- <u>Calibration of local conductor performance:</u> The calibration of stochastic models is more difficult than that of deterministic models because some calibration factors, such as those related to the driver aggressiveness, are often applied globally to all network elements and not just to roundabouts. In other cases, the model-specific coding is adjusted to reflect local driver behavior, including points of anticipation of clear acceptance and locations for discretionary and mandatory lane changes.
- <u>Volume pattern verification:</u> For network models with dynamic traffic assignment, the traffic volume on a given link may not match what was measured or projected and models with dynamic traffic.

### 3.8.5 Microsimulation softwares

Microscopic models model traffic at the level of each vehicle, its interaction with other vehicles (based on vehicle manoeuvres), its interaction with other vehicles (based on the manoeuvring of vehicles within the traffic flow), and its interaction with the infrastructure (by manoeuvres within the traffic flow), and their interaction with the infrastructure (by manoeuvres within the traffic flow) (Rivas *et al.*, 2020). Although they often demand substantial inputs and run time for their application, they can replicate the traffic in detail (Casta *et al.,* 2007).

In this type of conventional systems, it is of interest to know all the smallest details of a normally small amount of a vehicle and therefore its driver. The details of a normally reduced number of elements and factors surrounding the simulated phenomenon and achieve efficient and detailed solutions. Normally, this simulation is governed by a series of behavioral rules that determine how a vehicle accelerates and decelerates (model based on the theory of vehicle tracking) and how it changes lanes (lane change model); even describing how and when it changes its route (Rivas *et al.*, 2020).

### 3.8.6 Vissim

VISSIM is an acronym for the German words "Verkehr in Städten – Simulation. which is loosely translated into English as "traffic in towns – simulation" (TRC, 2006). This software is the most commonly microscopic simulation software used for roundabout capacity analysis, the program was developed in Karlsruh, Germany by the company PTV VISSION (Planung Transport Verkehr AG).

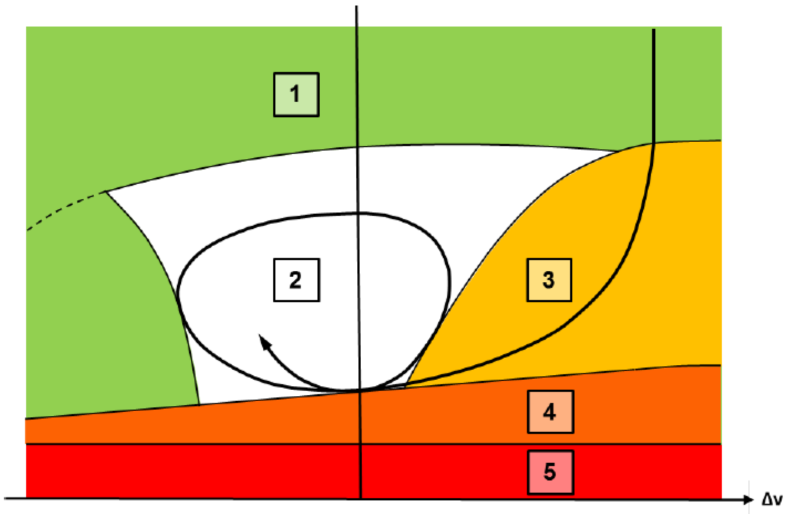
The Vissim program is a comprehensive tool used in the development of microscopic simulation modelling, it can render a 3D visualisation (Naukowe *at el.,* 2020). The microscopic modelling approach is based on individual vehicle behaviour and aims to describe in a precise form the dynamics of traffic. Thus, the simulated traffic network includes physical and psychological elements as well as characteristics of the drivers that interact with road elements. These elements are modelled from traffic rules, algorithms, and behavioural models, and are analysed in detail using the so-called psycho-physical driver behaviour model, which was developed by Wiedemann (Tettamanti *et al.,* 2015). Because of the microsimulation for the suggested case study in chapter 4, the specification is based on the key features for the consideration of urban area (Urban motorized in Vissim).

- **Model thresholds**

The calibration of the model entails a significant portion of a study of the features of driver behavior, which means that a detailed examination and comprehension of the characteristics of the drivers traveling through the intersection will be required.

Human behaviour has a natural distribution, these parameters are related to the ability to perceive and estimate, in safety distances, in speed desires and in the acceptance of maximum accelerations or decelerations, which are characteristic of aggressive driving. Some of these parameters also depend on the vehicles such as: maximum speed and maximum acceleration and deceleration. This is a natural phenomenon that can be represented by normal distributions, even though there are no normal distributions although there is no exact knowledge about these distributions (PTV, 2021). Thus, therefore different parameters will be used randomly within the model to calculate the threshold values and driving functions.

According to (Casta *et al.*, 2007) a set of criteria and desirable distances describe driver perception and reaction; the vehicle is influenced because the driver perceives a vehicle in front with a lower speed than him/her, the vehicle starts the following process or even when the vehicle is in an emergency.




**Legend**

| | |
|---|---|
| Axes: **d**: Distance, **Δv**: Change in speed | **3**: Approaching state |
| **1**: "Unregulated behavior" state | **4**: Braking state |
| **2**: Following state | **5**: Collision state |

Figure 48 – Car following model Wiedemann 1974 (PTV, 2021)

For car-following scenarios, Vissim employs Wiedemann's 1974 psycho-physical model of driving behaviour. The essential idea behind this model is that when the driver of a faster-moving vehicle reaches his perception threshold towards a slower moving vehicle, it begins to decelerate. Because the faster going-driver cannot precisely detect the speed of the slower vehicles, the speed will drop below that vehicle until begins to accelerate gradually again after reaching another perceptual threshold. As a result, an iterative process of acceleration and deceleration occurs (PTV, 2021).

The behavior of drivers is reflected in different variables which can be speeds, safety distances, gaps, reaction times and even depend on the physical characteristics of the vehicles and the type of driver (old, young, female, etc).

In VISSIM, driver behaviour is modelled in four phases which are:

- Following
- Lane Change
- Lateral movement
- Traffic light control

Each of these phases is composed of different parameters which directly affect the interaction of the vehicles and can cause substantial differences in the simulation results, VISSIM assigns a driving behaviour.

For the simulation of the models, Vissim uses a random seed, the analyses within models employing simulations use random numbers generated from a single initial value (seed) within each analysis to minimize simulation errors in the simulation. In a model, it is observed with the change in driver behaviour (gentle, aggressive, etc.) and the types of participating vehicles (motorcycles, trucks, buses, etc.). Each time this so-called "seed" number is changed, these parameters will vary, generating different behavioural patterns and vehicular flow.

### 3.8.7 Application area

The following are some of the applications of VISSIM (PTV, 2021):

- It can be used to simulate signal-controlled intersections with stop, yield, stop, and time-controlled intersections, signals, stop signs, traffic-light controlled intersections with fixed time controllers or with traffic lights, etc.
- It can be used to evaluate and monitor the feasibility and impact of integrating mass transit systems into urban networks as in our case and to solve road problems.

- VISSIM allows to easily and quickly compare possible alternative solutions for the improvement of an intersection or a section of a specific road. It also allows the evaluation of public transportation by optimizing travel times and delays.
- Capacity analysis and testing of transit priority systems as well as analysis of traffic management systems such as alternative route control, traffic control, traffic control, access control and special lanes.
- Vulnerability analysis of large networks with the alternative routing option using dynamic allocation, routing using the dynamic assignment. Simulation of traffic-calmed areas including all relevant road users, Simulation and visualization of passenger flow in a multimodal transit center or 3D model.

# CHAPTER IV
# CASE STUDY

*"The reality about transportation is that it is future-oriented. If we are planning for what we have, we are behind the curve".*

**Anthony Foxx**

## 4.1 Study area

To analyze the models used to estimate the performance of roundabouts, an evaluation of a roundabout in Guimaraes, in the Azurem sector, was conducted. The roundabout in question is known as Circular da Quinta, and it is located near the campus of University of Minho campus.

This roundabout has become a key connection in the city, directing traffic flows from the streets Cónego Dr. Manuel Faria and Cap. Alfredo Guimaraes, and University Avenue.

Figure 49 – Roundabout under study (Google Earth, 2021)

### 4.1.1 Description of the streets

A description of the three roundabout approaches is made using the criteria required by the performance evaluation models. This characterization of each entry is based on roundabout designs (see appendix C-D), Google Earth, and field inspection.

**University Avenue**

The University of Minho campus is the main centre of the higher education in the area, the University Avenue has an extension of approximately 148 m, and there is a significant traffic flow of vehicles approaching the intersection from this avenue and pretending to access the campus must pass through the roundabout. The University Avenue links to the N101 highway, which is an important fast road in the area because links the city to the rest of the region.

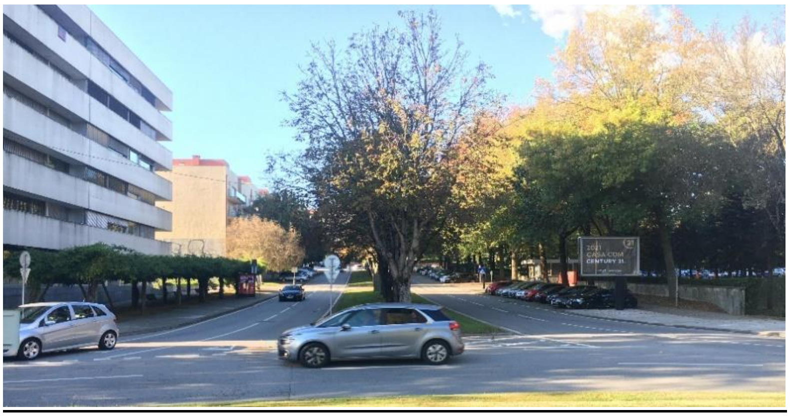


Figure 50 – University Avenue (Author)

**Cónego Dr. Manuel Faria Street**

Cónego Dr. Manuel Faria Street has an extension of about 300 m and constitutes an important road that leads to Teixeira de Pascoais Street, which is a transited road within a residential area. In the zone, there are gyms, cafés, schools, parking lots, and which also provides access to the city center, where are located the main shopping centers, the municipal stadium, the fire station, and the police station.

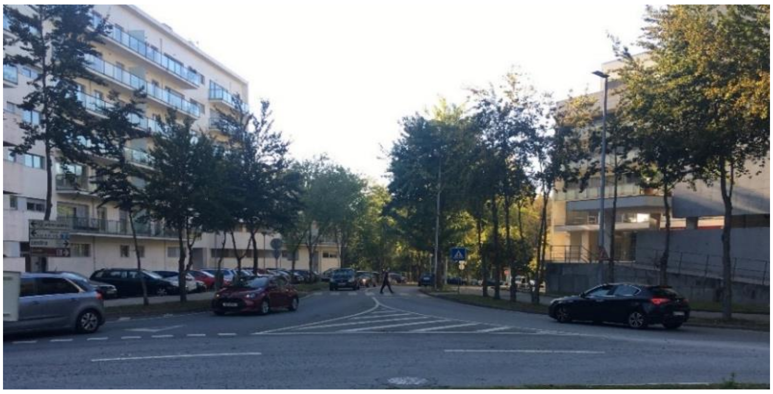


Figure 51 – Cónego Dr. Manuel Faria Street (Author)

**Cap Alfredo Guimaraes**

With an extension of about 287 meters and a reserved lane for parking, the street Cap Alfredo Guimaraes operates in one-way direction. In this street, vehicles are only allowed to enter the intersection, which means that the exiting traffic is non-existent. Cap Alfredo Street accommodates all the traffic flow coming down from the historical center of the city through the Gen. Humberto Delgado Street.

Figure 52 – Cap Alfredo Guimaraes Street (Author*)*

### 4.1.2 General description of the roundabout

The following is an analysis of the geometric characteristics and operation of the roundabout:

- The roundabout under consideration has three arms and a double circulating roadway. The University Avenue approach has two entry lanes and two exit lanes. While Cónego Dr. Manuel Faria Street is a one-lane approach with only one entry and exit lane. Although its geometry corresponds to a two-lane roadway, Cap. Alfredo Guimaraes approach operates as a one-lane road.

- All entries except for the Cap. Alfredo Guimaraes Street, have a relatively flat longitudinal slope and are perpendicular to each other. The lanes on Cónego Dr. Manuel Faria and Cap Alfredo Guimaraes Street have crosswalks on their approaches, at 26 m and 20 m from the yield line, respectively. The crosswalk on University Avenue is located 96 m from the yield line, this avenue is the only one that has a physical splitter island.

### 4.2 Methodology

The methodology to be followed in this case study is divided into three main parts:

- Firstly, the composition of the traffic of the intersection and the entry flows are calculated to obtain the matrixes origin-destination.

- Based on the geometric and traffic flow data of the intersection, the corresponding evaluation of the empirical and analytical models is carried out to estimate the capacities considering the key parameters of each methodology. In addition, the capacity evaluation, performance indicators of the intersections such as: delay, queuing, degree of saturation and queue length are also analyzed.

- Finally, the roundabout is modelled in VISSIM considering the geometric and traffic parameters of the intersection. The construction of the simulation model is based on the existing infrastructure and traffic characteristics and the calibration of the model is carried out considering the pre-setting to obtain a more realistic representation. Validation of the obtained data is made; this validation is made from the collection of independent data that allows a comparison of the state of the intersection.

### 4.3 Data collection and traffic flow analysis

This case study was conceptualized using traffic flow data received in 2017 since data collecting requires a substantial human resource. As seen in Figure 53, the traffic movements 2, 4, 5, 6, and 9 were excluded from the study because they were deemed negligible, as the traffic flows of these movements are extremely low.

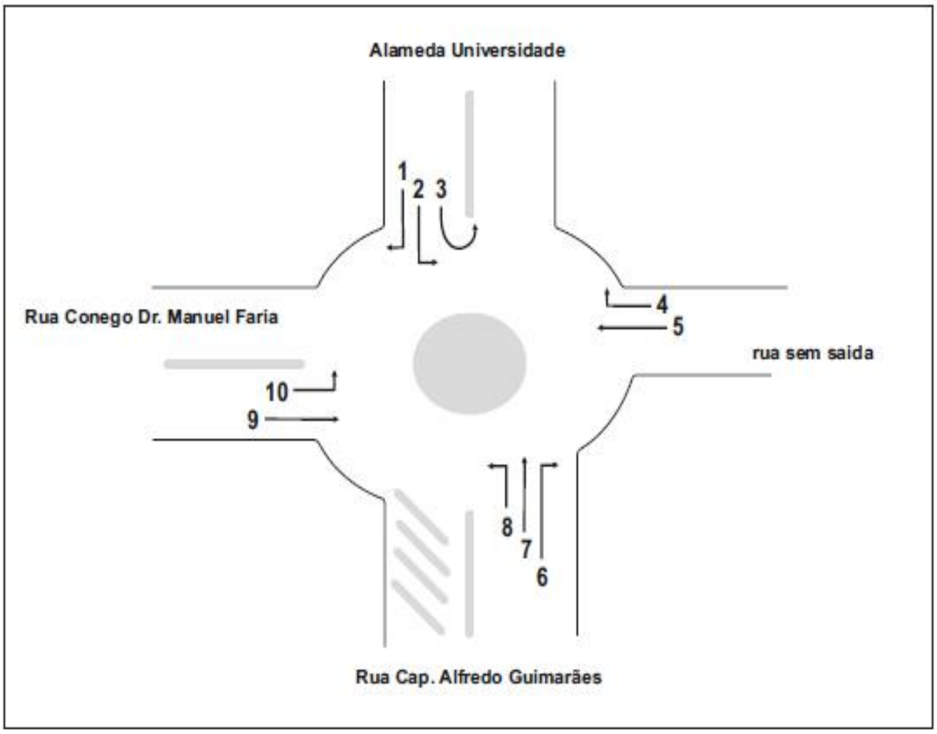



Figure 53 – Typical movements at the intersection (Municipality of Guimaraes, 2017).

The flow data used in this study was collected mainly during peak hours, divided into two periods, Morning, and afternoon; in the morning between 7:30 AM and 9:30 AM, and the afternoon between 4:30 PM and 6:30 PM, the traffic was counted in 15-minute intervals. As shown in figure 54, the 15 minutes with the highest flow recorded was in the morning between 8:45 and 9:00 a.m. and from 17:15 to 18:00.

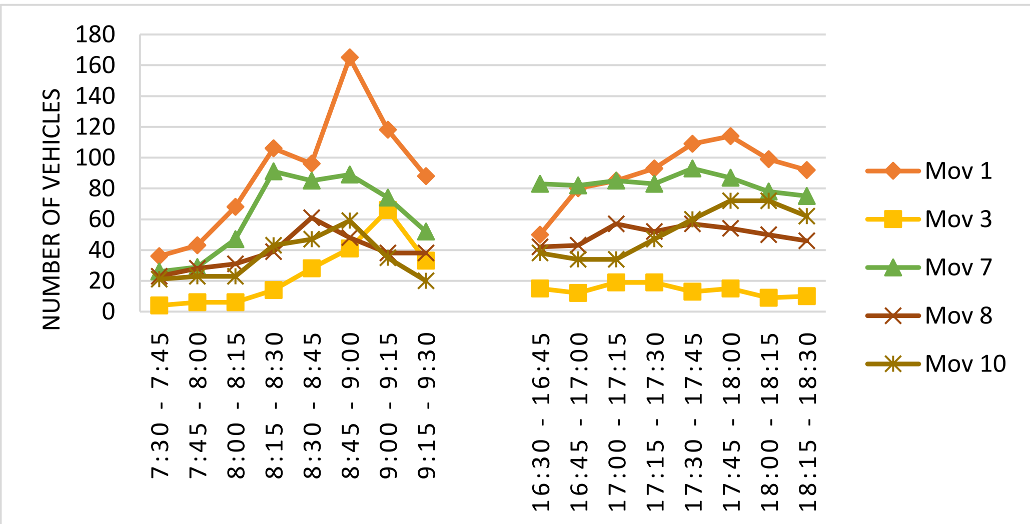



Figure 54 – Traffic volumes per movement during the analysis periods.

It should also be noted that the vast number of vehicles' exit the roundabout from Cónego Dr. Manuel Faria Street, as this street directs the circulating flow towards the city center. The great majority of the traffic flow exiting in this approach comes from University Avenue (movement 1, in Figure 53).

The composition of traffic was categorized into three main vehicles type: motorbikes (MC), lights (LIG), trucks (TRUCK), and buses (BUS). For the capacity estimation, all vehicles were converted to light vehicles, the factors used were 1.2 for two-wheelers, 1.5 for heavy vehicles and 1 for light vehicles.

As is illustrated in figure 55, the traffic composition was uniform during the analysis period at each of the entries of the roundabout. The number of cars (75%) represent the highest percentage of the vehicle composition circulating on the intersection, while the percentages of other types of vehicles are relatively low, buses 2%, and motorbikes and trucks in smaller proportions; 1% and 0.1% respectively.

Table 18 – Vehicle composition in the University Avenue

| **University Avenue** | | | | |
|---|---|---|---|---|
| MC | LIG | TRUCK | BUS | TOTAL |
| 16 | 1698 | 2 | 36 | 1752 |
| 1% | 97% | 0.1% | 2% | 100% |

Table 19 – Vehicle composition on the Cónego Dr Manuel Faria Street

| **Cónego Dr. Manuel Farias Street** | | | | |
|---|---|---|---|---|
| MC | LIG | TRUCK | BUS | TOTAL |
| 15 | 1808 | 2 | 41 | 1866 |
| 1% | 97% | 0.1% | 2% | 100% |

Table 20 – Vehicle composition on Cap Alfredo Guimaraes Street

| **Cap Alfredo Guimaraes Street** | | | | |
|---|---|---|---|---|
| MC | LIG | TRUCK | BUS | TOTAL |
| 5 | 672 | 1 | 12 | 690 |
| 1% | 97% | 0.1% | 2% | 100% |

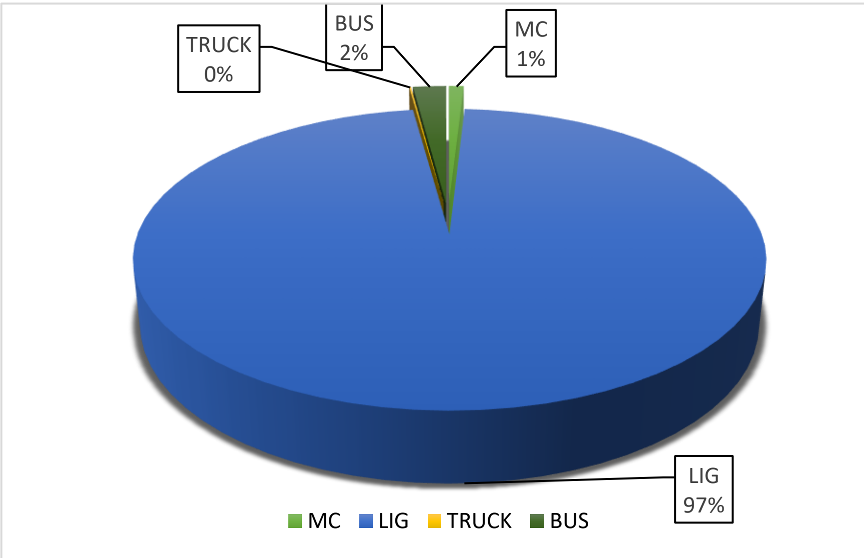



Figure 55 – Traffic composition by vehicle type

For the capacity evaluation, an analysis of the traffic flow data was carried out to determine the origin-destination matrix for the intersection at periods of the day (Morning and afternoon). After converting the total number of vehicles to equivalent vehicle units was determined the peak hour factors for both periods; in the morning between 8:15 and 9:15 a.m, and between 5:15 and 6:15 p.m.

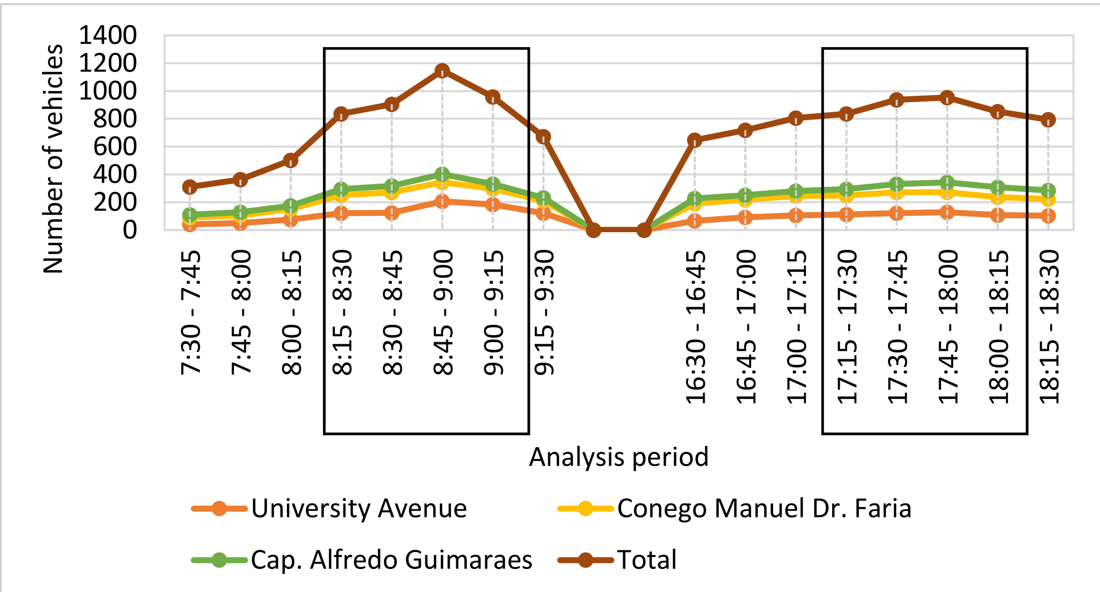



Figure 56 – Number of vehicles per analysis period

From the data collected of the entering and exiting traffic flow of each of the movements at the roundabout (Appendix F, two 2x3 origin-destination matrices are constructed, based on the selected peak hours. The matrix showing the behavior of the traffic flows in the morning is presented in table 21.

Table 21 – Matrix origin-destination (O/D), morning in pcu/h.

| O/D | University Avenue | Cónego Dr. Manuel Faria St |
|---|---|---|
| University Avenue | 259 | 665 |
| Cónego Dr. Manuel Faria St | 245 | 0 |
| Cap. Alfredo Guimaraes St | 364 | 238 |

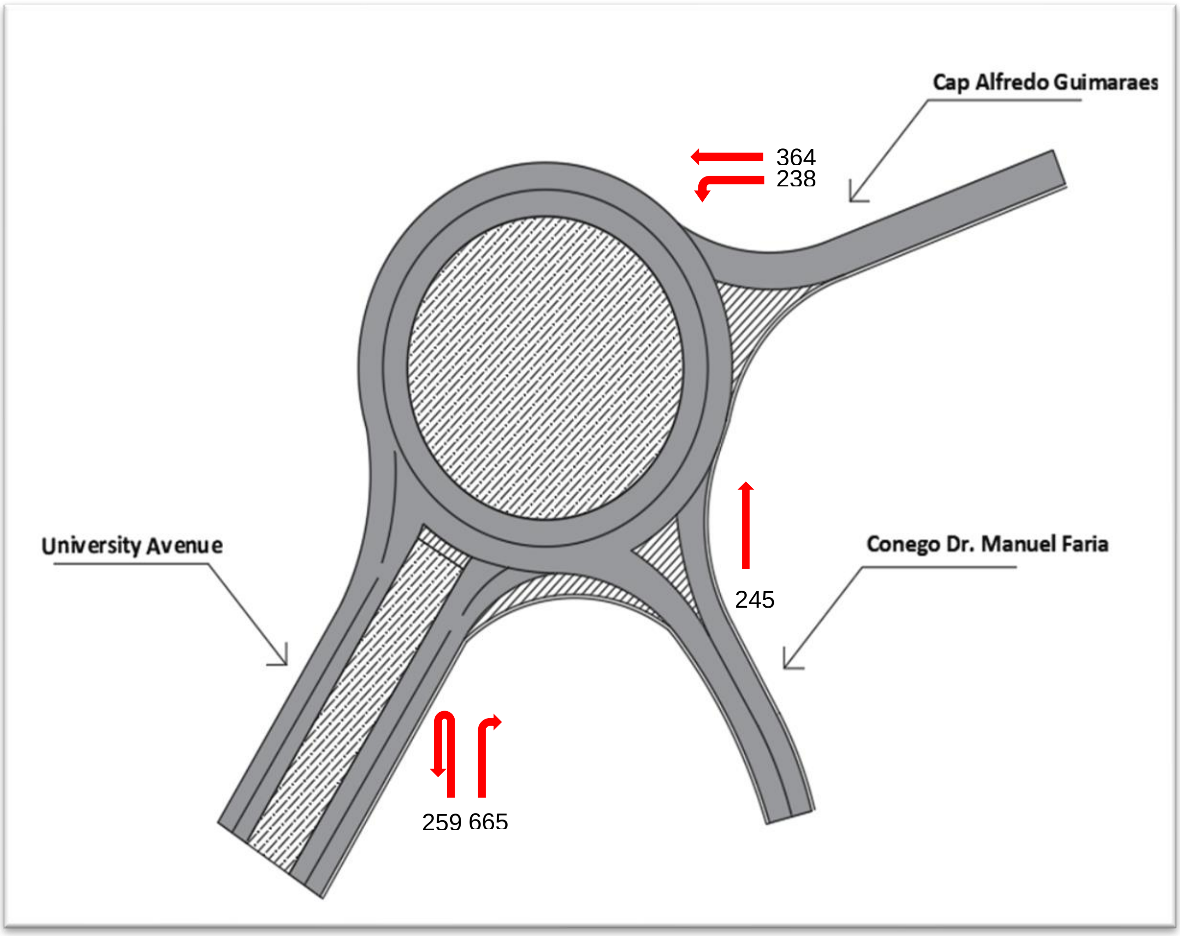



Figure 57 – Typical movements at the intersection (Morning).

Table 22 and Figure 58 represents the traffic for each of the movements at the intersection. As can be seen, in the afternoon, about 32% of the total traffic flow entering the intersection on University Avenue is directed towards Cónego Dr. Faria Street. The following is the matrix origin-destination and the diagram of traffic flow distribution of the roundabout in the afternoon.

Table 22 – Matrix origin-destination (O/D) Afternoon in pcu/h.

| O/D | University Avenue | Cónego Dr. Manuel Faria St |
|---|---|---|
| University Avenue | 79 | 459 |
| Cónego Dr. Manuel Faria St | 224 | 0 |
| Cap. Alfredo Guimaraes St | 372 | 293 |

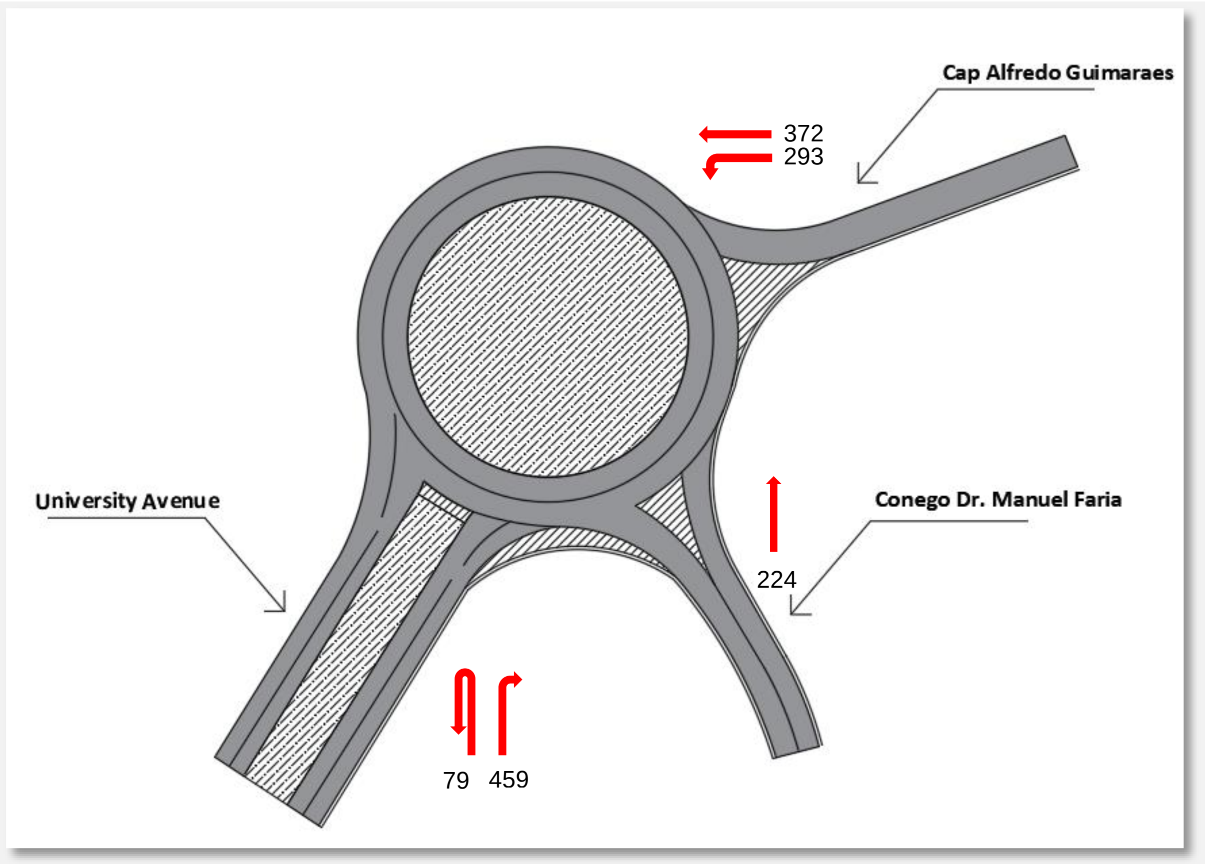



Figure 58 – Typical movements at the intersection (Afternoon)

The conflicting flows have been calculated for the two different periods of the day ($\boldsymbol{q_c^x}$), morning and afternoon. The results are shown in the following table:

Table 23 – Conflicting traffic flows

| Entry | Conflicting traffic flow (Morning) | Conflicting traffic flow (Afternoon) |
|---|---|---|
| University Avenue | $\mathbf{q_c^A} = q_{CB} = 238\ \text{pcu/h}$ | $293\ \text{pcu/h}$ |
| Cónego Dr. Manuel Faria St | $\mathbf{q_c^B} = q_{AA} = 259\ \text{pcu/h}$ | $79\ \text{pcu/h}$ |
| Cap. Alfredo St | $\mathbf{q_c^C} = q_{BA} + q_{AA} = 504\ \text{pcu/h}$ | $302\ \text{pcu/h}$ |

As table 23 shows that Cap. Alfredo Guimaraes Street has the largest conflicting traffic flow, especially in the morning hours, particularly in the morning hours, when compared to the other approaches. Regarding the other approaches where just one conflicting movement interferes with the entry, it was discovered that the conflicting flows on University Avenue are comparable in the morning and afternoon. The highest traffic flow for at the entry Cónego Dr. Manuel Faria was recorded in the morning, with levels substantially higher than those observed in the afternoon.

### 4.3.1 Geometric data collection

The geometric data have been obtained through Google Earth resources, this geographic information system allowed locating and visualizing the cartography of the study area. The data have been validated with the layouts provided by Alfredo Pimenta National Archive and the municipality of Guimaraes that contain the archives of Guimaraes.

It was not possible to obtain the detailed dimensioning of the geometry of the roundabout. However, it was obtained roundabout design plan of the project “Mini circular de Azurem” layout of the horizontal roundabout signalization in scale (1:1000) and the project “Rua Teixeira de Pascoais & Circular da Quintã” (See Appendix B and C).

The methodology used for collecting the geometric parameters of the roundabout consist of three parts:

- A satellite image of the study area was taken from Google Earth and scaled at AutoCAD and then, a sketch in AutoCAD following the geometric dimensions of the roundabout.
- Some data were no possible to visualize from Google Earth because of the green areas. Validation dimensions with the plans of the project Mini circular de Azurem and the project circular da Quinta (see appendix B and C).
- Some data that were no possible to visualize from Google Earth because of the green areas.

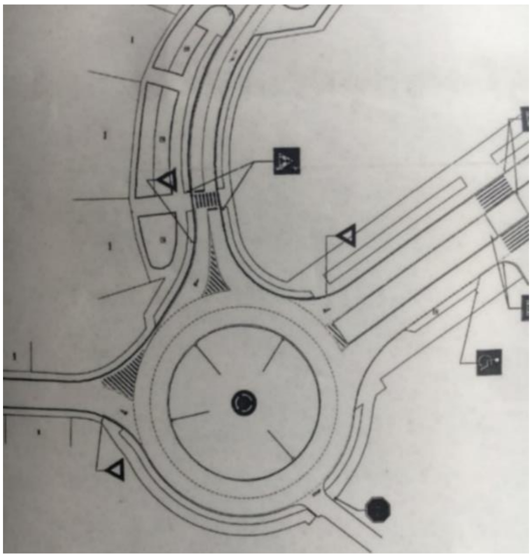


Figure 59 –Signalization project "minicircular a quinta", Scale: 1:1000 (Municipality of Guimaraes, 2021)


Figure 60 – Sketch of the roundabout

The basic geometric parameters information considered for the roundabout case study are summarized as follows:

Table 24 -Geometric parameter for the analysis of the models

| **Parameters** | | **Dimension (m)** | | |
|---|---|---|---|---|
| | | **Entry A** | **Entry B** | **Entry C** |
| Entry width | e. ENT | 7 | 6.5 | 5.2 |
| Approach width | v | 5.5 | 3.5 | 3.5 |
| Effective flare length | l' | infinity | 12 | infinity |
| Inscribed circle diameter | IDC | 57 | | |
| Entry radius | r | 22 | 30 | 22 |
| Entry angle | Ø | 53 | 28 | 24 |
| Circulatory roadway width | ANN | 9.5 | | |
| Splitter Island width | SEP | 9 | 8 | 0 |
| Number of conflicting lanes | - | 2 | | |
| Number of entry lanes | - | 2 | 1 | |
| **Entry A:** University Avenue<br>**Entry B:** Cónego Dr. Manuerl Faria<br>**Entry C:** Cap. Alfredo Guimaraes | | | | |

## 4.4 Capacity Evaluation

### 4.4.1 Empirical Models

The following are the results derived from the combination of geometry and traffic data for each of the empirical models. The study was performed during the two periods of the day, the capacity results are given in pcu/h.

- **TRL**

The results obtained for the two periods of the day are shown in figure 61. According to the geometric characteristics of the intersection and the traffic flow at the peak hours analyzed, except for the University Avenue, the intersection performs better in the afternoon, as experience high-capacity levels.

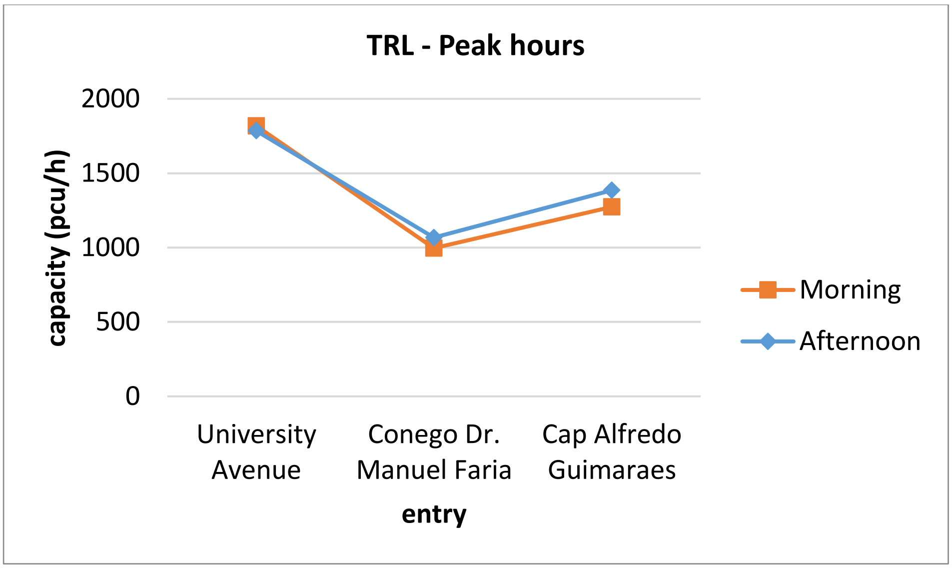



Figure 61 – Capacity in pcu/h for the TRL model– Peak hours

In the TRL model, the geometric characteristics of the entries have a strong influence on the capacity. Due to its geometric and conflicting flow features Cónego Dr. Manuel Faria Street performance the lowest capacity of all the approaches.

The entries on University Avenue and Cap Alfredo Guimaraes Street have an infinite effective flare length, which increases entry capacity by allowing for more vehicles. This parameter has the largest influence on the entry capacity, which explains why, despite having high conflicting traffic, Cap Alfredo Guimaraes performs better than entry Cónego Dr. Manuel Faria.

- **SETRA**

The capacity results from the SETRA model are displayed below. As seen in figure 62, the capacities of University Avenue, Cónego Dr. Manuel Faria, and Cap. Alfredo Guimaraes perform similarly in the afternoon.

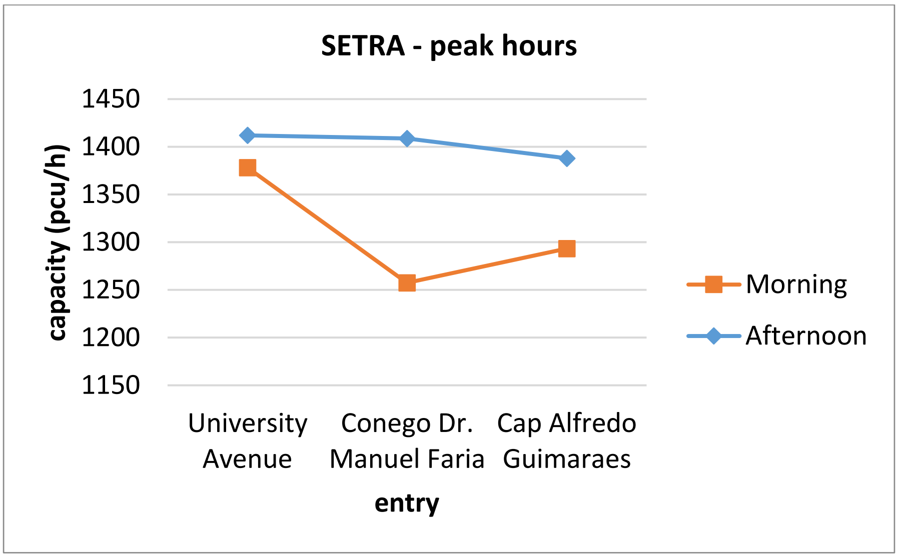



Figure 62 –Entry capacity in *pcu/h* for SETRA model– Peak hours

In the evaluation of the capacity through the SETRA model, it was evident the incidence of the impeding traffic at the entry capacity performance since the impeding traffic flow increases as capacity decreases. The geometric characteristics, and the conflicting and impeding traffic flows in the morning peak hour, decrease the capacity at the entry Cónego Dr. Manuel Faria, this entry has less capacity than the entry Cap Alfredo Guimaraes.

In general, the SETRA model considers specific geometric parameters for the capacity estimation, the approach width and the splitter island width have remarkably influence on capacity performance. For the characteristics of the entry on Cap. Alfredo Guimaraes Street, this parameter was set to zero, as it operates in one single direction and allows no exit of vehicles.

In the SETRA model, an increase in the splitter island width (SEP) up to 15 m, denotes a greater improvement in capacity, while dimensions above 15 m on this parameter has a little effect on capacity performance.

- **CERTU**

The entry capacity results obtained through CERTU model are quite similar for each of the entries during the two periods of the day, especially for the entry on University Avenue, for which the model estimates practically the same entry capacity for both periods. Figure 63 shows the result of the capacity estimation using this methodology.

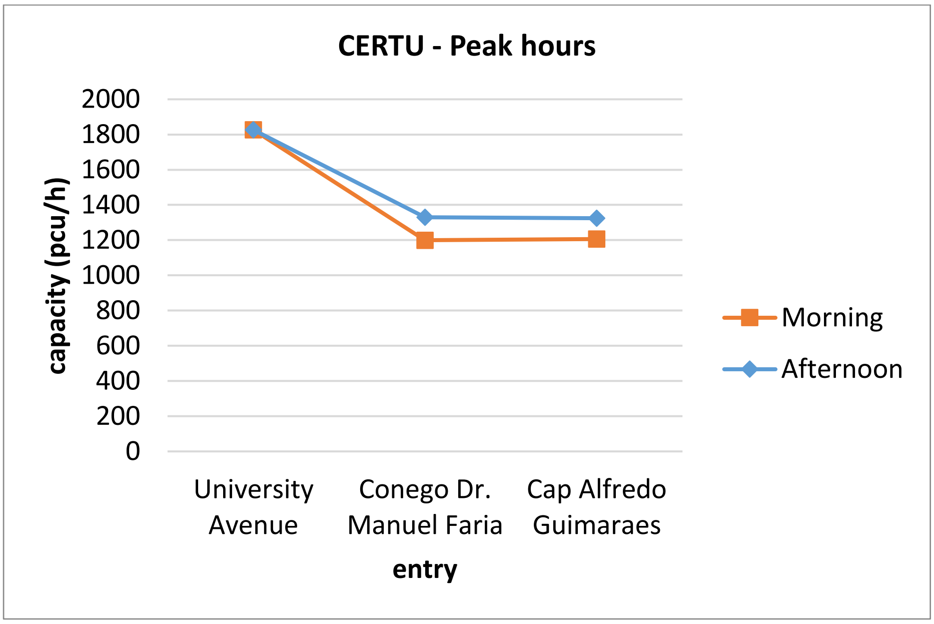



Figure 63 – Entry capacity in pcu/h for the CERTU model– Peak hours

The factors **γ** and **α** are based on geometric parameters of the roundabout were included in the capacity evaluation of the model, the values for these parameters are assigned from geometric parameters of the roundabout and the entries such as, the number of entry lanes, the circulatory roadway width, and the inscribed diameter circle (IDC), all these parameters are proportional to the capacity.

In this model, a larger conflicting flow denotes a lesser capacity, since it implies more conflicts at the evaluated entry. While a higher exiting traffic flow on the entry under consideration improves capacity since it permits more vehicles to exit the intersection.

- **FCTUC**

Even though the Portuguese model is an adaption of the TRL model, different coefficients were assigned to the TRL equations. Although the findings represent behavior that is similar to the basic model, they give substantially greater capacity data for the evaluation of the University Avenue. The capacity results obtained for the intersection peak hours from the Portuguese analysis are shown in figure 64.

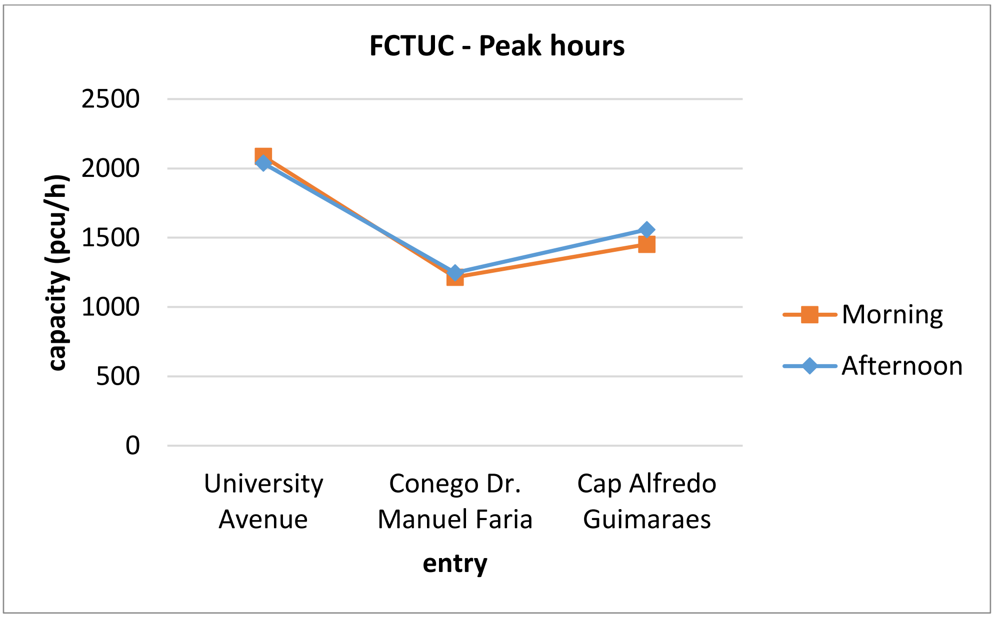



Figure 64 – Entry capacity in *pcu/h* for the FCTUC model– Peak hours

Despite using the same approach as the TRL model, the results of entry capacity are superior to those obtained through that model evaluation. One reason for the greater capacity estimate might be that a higher coefficient is given to the effective flare length, which is deemed infinite for the University Avenue and the Cap. Alfredo Guimaraes Street.

- **German empirical models**

<u>German exponential model</u>

The German exponential is not adapted to analyze roundabouts with only one entry lane and two conflicting lanes, as the German guideline provides no criteria for evaluating roundabout with these characteristics.

The data obtained by the means of the German exponential model is shown in figure 65. In concordance with the German exponential model, the roundabout has the highest capacity in the morning. However, the intersection has similar capacity performance for both periods of the day.

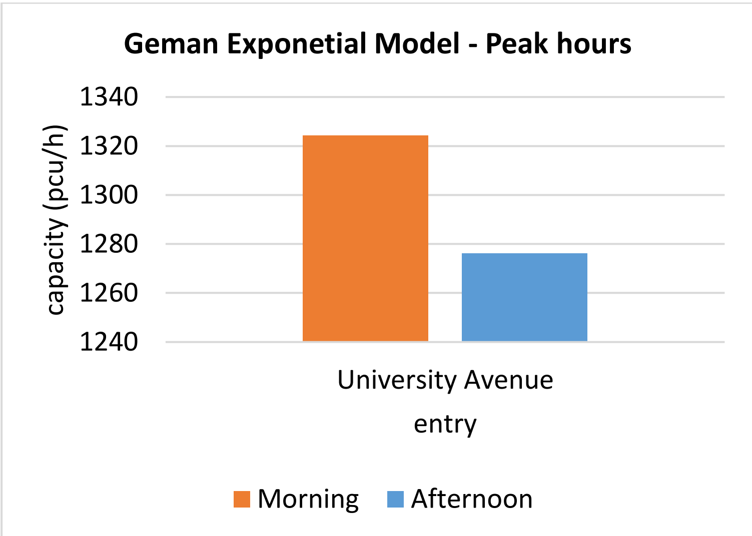



Figure 65 – Entry capacity in pcu/h for the German exponential model– Peak hours

For this model, the parameter with the greatest impact on the capacity performance is the conflicting traffic flow, it could be defined as inversely proportional to the entry capacity of the roundabout. The model also includes geometric parameters for the estimation of the capacity at the roundabout, particularly the number of entry lanes and in the circulating roadway.

German linear model

Different from the exponential model already analyze above, the German linear model allows to evaluation of the entry capacity for entries with two entry lanes and two circulating lanes. This method based on linear regressions is the most recent empirical methodology for the evaluation of capacity performance at roundabouts in Germany.

The coefficients or reduction factors for the analysis through this model are relatively smaller compared to ones used in the exponential model analysis. Therefore, the capacity estimation through this model is greater than in the exponential model as shown in figure 66.

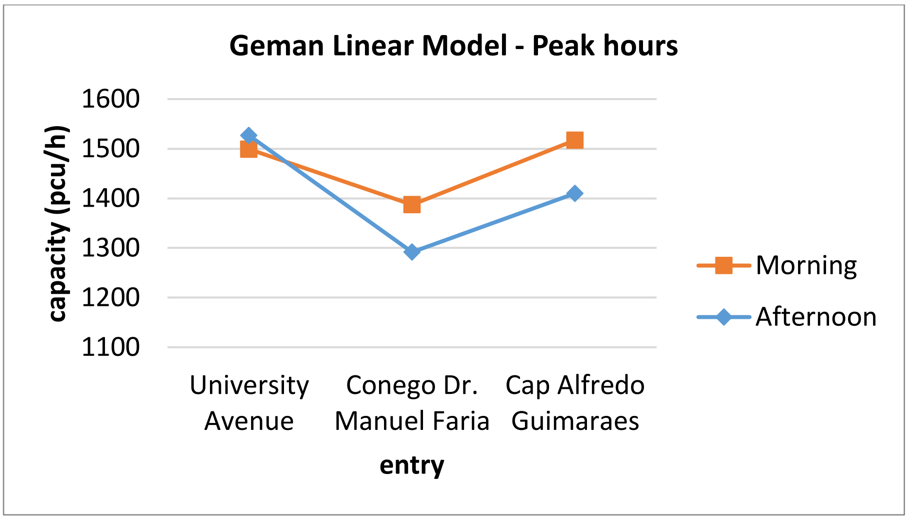



Figure 66 – Entry capacity in *pcu/h* for the German linear model– Peak hours

The model shows that, according to its characteristics, University Avenue has a slightly higher entry capacity in the afternoon peak hour.

- **Colombian model**

As can be seen in figure 67, the results obtained from the analysis of the Colombian model are the same for both periods of time because this model is based merely on geometric parameters that no change over the time.

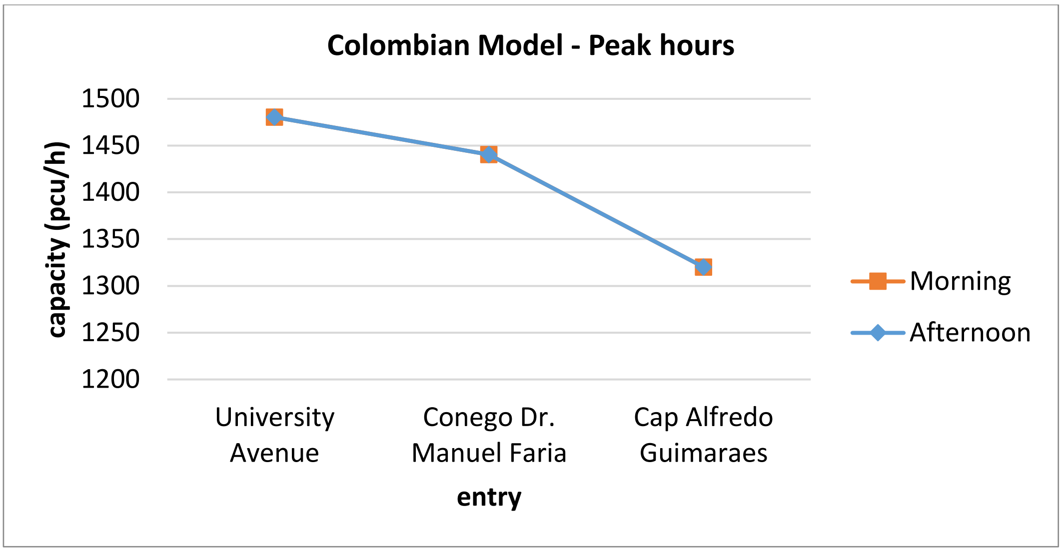



Figure 67 – Entry capacity in pcu/h for the Colombian model– Peak hours

When evaluating the performance of the capacity through this model, is found that the lowest capacity performance is for the Cap. Alfredo Guimaraes Street. In the equation provided by the INVIAS guideline, the entry width is the important parameter for the evaluation of entry capacity in which the model estimated an overage with this dimension and the circulatory roadway width. The geometric analysis of this model shows that University Avenue has the best performance, while entry Cap Alfredo Guimaraes has the lowest capacity results.

The length of the weaving section, which is the splitter island distance from the preceding entry to the nearest exit, is related to the distance between splitter island is inversely proportional to the capacity, a longer distance indicates that a driver who desires to exit the intersection as soon as possible will have to travel a long distance before unloading. Thus, a longer period at the intersection implies a larger likelihood of saturation, as result this model considers this parameter to be one of the most significant in the capacity evaluation.

- **Dutch Model**

The Dutch methodology for entry capacity evaluation does not enable the calculation of the capacity of the roundabout for multilane entries. As is illustrated in figure 68, the evaluation only was possible for the entries Cónego Dr. Manuel Faria and Cap Alfredo Guimaraes. The deferences found in capacity results maybe because for this model the conflicting traffic flow is the parameter that most affects the entry capacity.

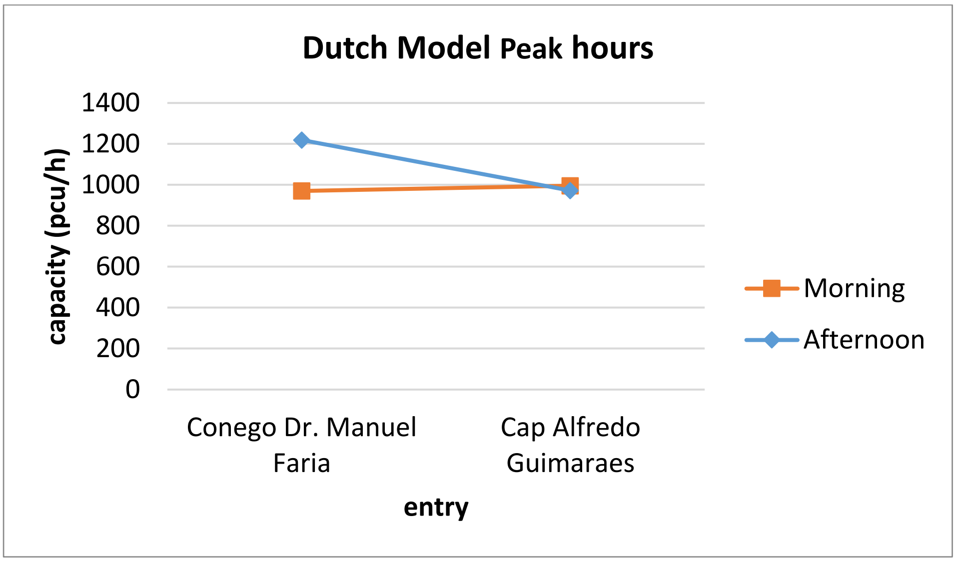



Figure 68 – Entry capacity in pcu/h for the Dutch model– Peak hours

There are two traffic flow parameters included in this model, conflicting flow, and exiting traffic. Therefore, a higher exiting traffic flow at the entry analyzed improves its capacity, as it allows vehicles to exit the intersection when there is high traffic at the intersection.

The model only allows to evaluate the capacity for single-lane roundabouts, the estimation was made entirely for the Cónego Dr. Manuel Faria and the Cap. Alfredo Guimaraes Street. The results indicate that in the morning the intersection has a better performance than in the afternoon since the traffic conflicting for this intersection are lower during this period.

The results obtained for all models in the two analysis periods are displayed in figures 69-70.

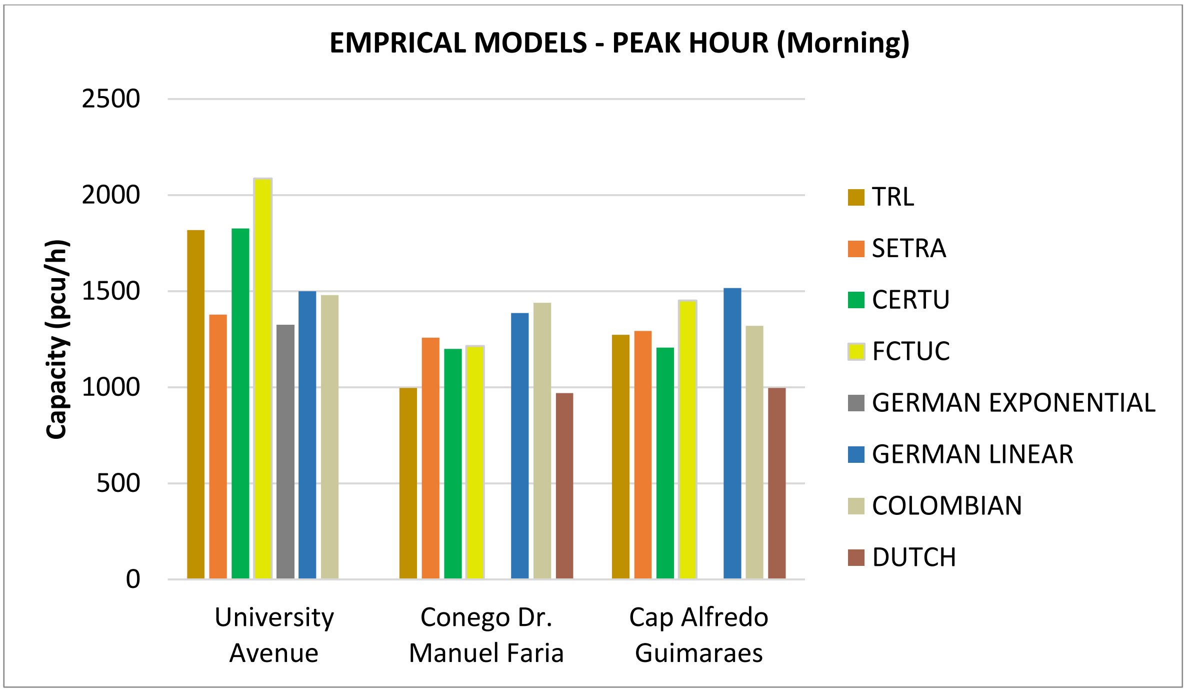



Figure 69 – Capacity evaluation for empirical models (Morning)

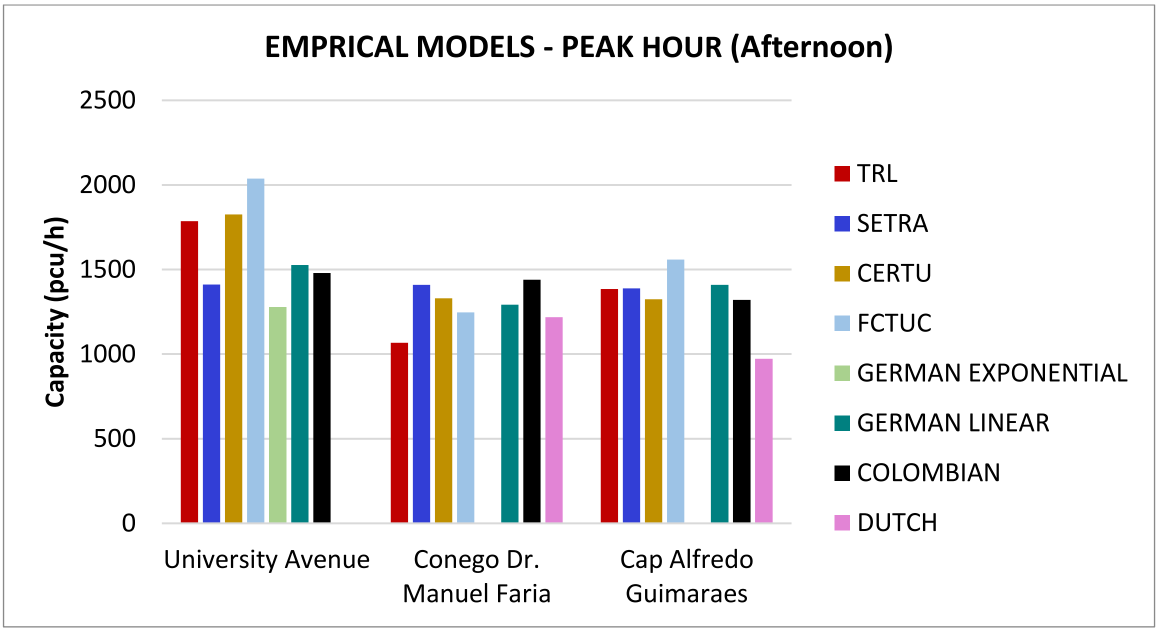



Figure 70 – Capacity evaluation for empirical models (Afternoon)

As a result of its geometric characteristics and traffic flow of the roundabout, University Avenue shows the greatest entry capacity for the peak hours. The lowest capacities are more evident at the entries Cónego Dr. Faria and Cap Alfredo Guimaraes, as models give a higher value to some parameters. Thus, the entry capacity performance results from highest to lowest are University Avenue, followed by the entry on Cap. Alfredo Guimaraes Street, and lastly the entry Cónego Dr. Manuel Faria.

The most significant variation in the results is experienced by the University Avenue, especially using the FCTUC model. It is also possible to state than the Dutch model tend to underestimate the entry capacity; it is reflected in both periods, morning, and afternoon.

### 4.4.2 Analytical models

Analytical models were evaluated using the critical gap and follow-up time data of each where the method was developed and then compared to the critical gap and follow-up time of Portugal, for the two peak hours. The critical gap and follow-up time for Portugal has been extracted from research conducted by data were extracted from research conducted by (Bastos, 2012), about the estimation of the critical gap and follow-up time of Portuguese roundabouts. Figure 71 shows the results obtained from the investigation.

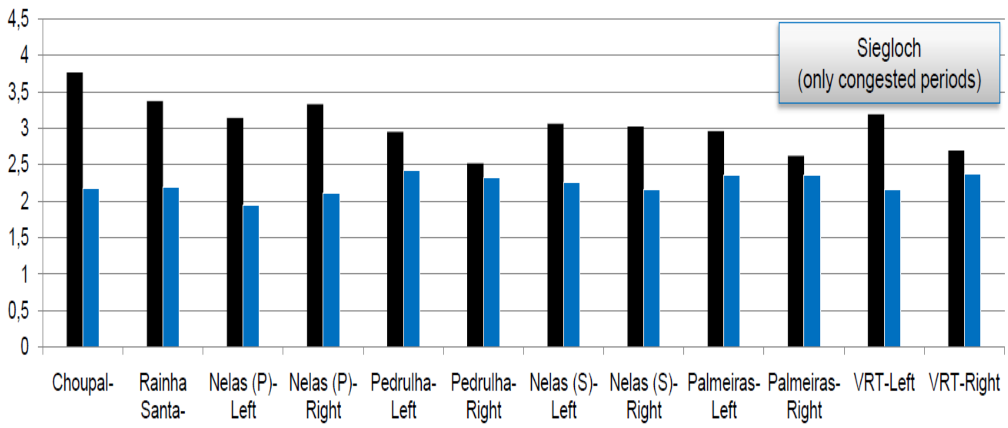



Figure 71 – Critical gap and follow-up time for Portuguese roundabouts (Bastos, 2012)

From figure 71 can be inferred that the range for critical gap [2.6 – 3.8 s] and follow up times variation is reasonably uniform [2.0 - 2.5 s]. To set out a comparison of the capacity results and evaluate the capacity model considering the Portugal realities has been considered a critical gap and follow-up time of 2.5 s and 3.8 s respectively.

The analysis is divided into two parts:

- comparison of the models based on the realities of the origin country, followed by the Portuguese conditions, and
- an examination of the models during the two peak hour times.

The following are the results obtained from the capacity evaluation:

- **HCM 2010**

The results of the capacity evaluation for each of the entries using the HCM 2010 are shown below. The factor of critical gaps and follow up time in Portugal are lower than the ones suggested in the HCM 2010 methodology. As can be seen in Figures 72-73, the capacity for Portugal conditions is higher than in the USA, since driver behaviors indicators and traffic conditions are different in both countries.

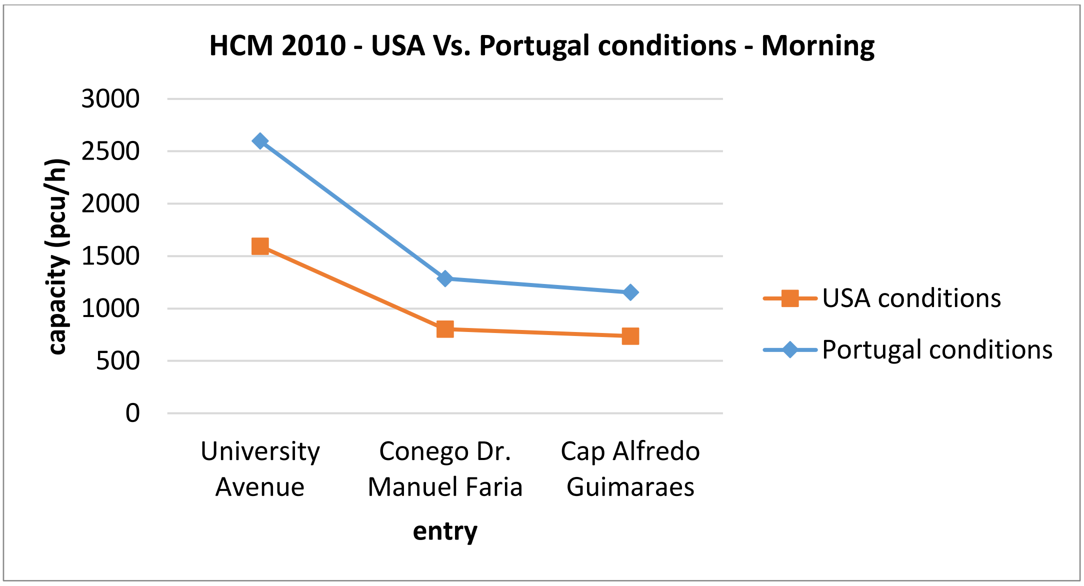



Figure 72 – HCM 2010 under USA and Portugal conditions (Morning)

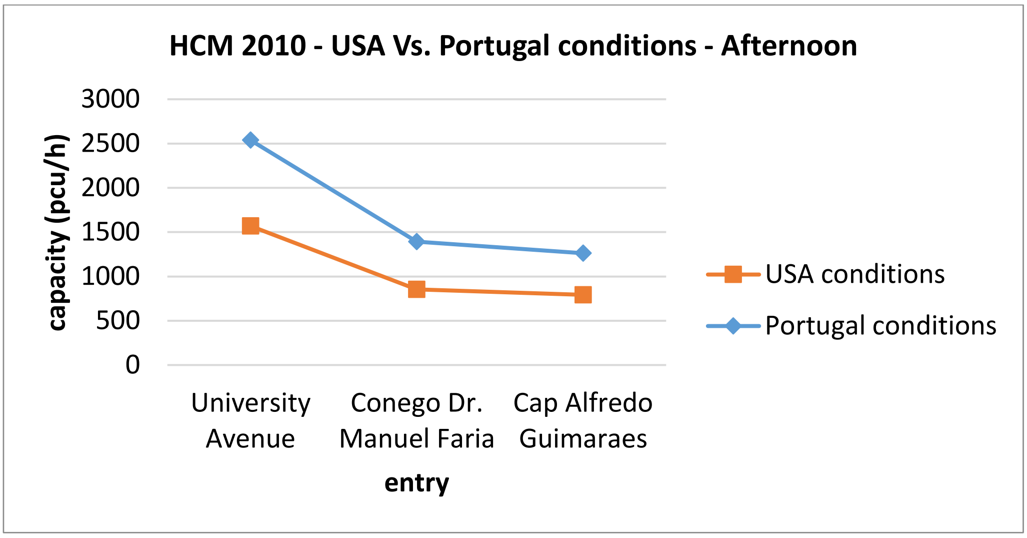



Figure 73 – HCM 2010 under USA and Portugal conditions (Afternoon)

- **Australian model**

The German model consider parameters such us the critical gaps and the follow-up times to describe the driving behavior at the intersection. Thus, the capacity performance of the roundabout through Portugal and Australian conditions are very similar, especially in the morning peak hour. This similarity on the results is due to both countries suggested a critical gap of 2.5 seconds. However, the follow-up time in Australia is shorter than in Portugal, because of that the entries in question a higher capacity has when evaluated using the Australian conditions.

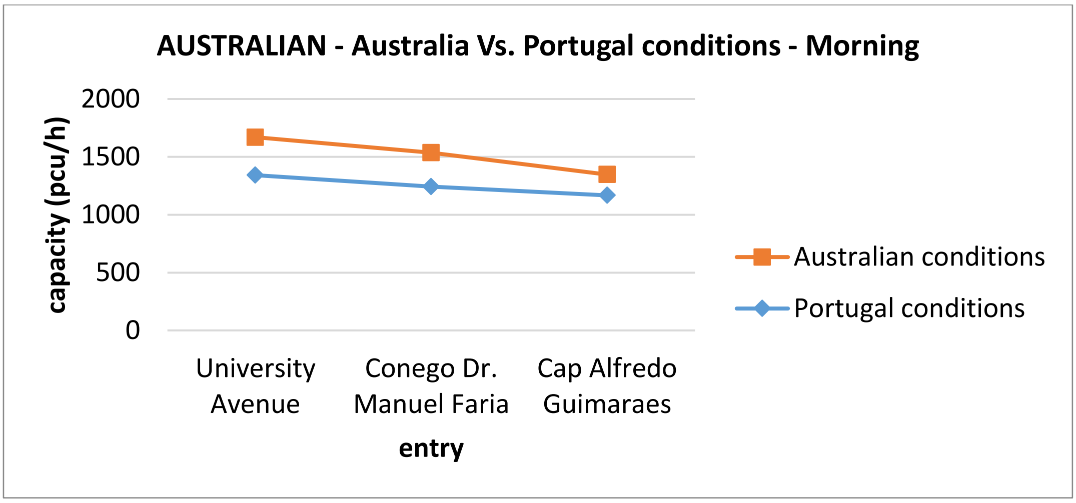



Figure 74 – Australian model under Australian and Portugal conditions (Morning)

In the afternoon, the model has a better capacity performance, as in this peak hour the traffic values for the intersection are lower than in the morning, in which University Avenue and Cap. Alfredo Guimaraes Street have a better entry performance on Portuguese conditions.

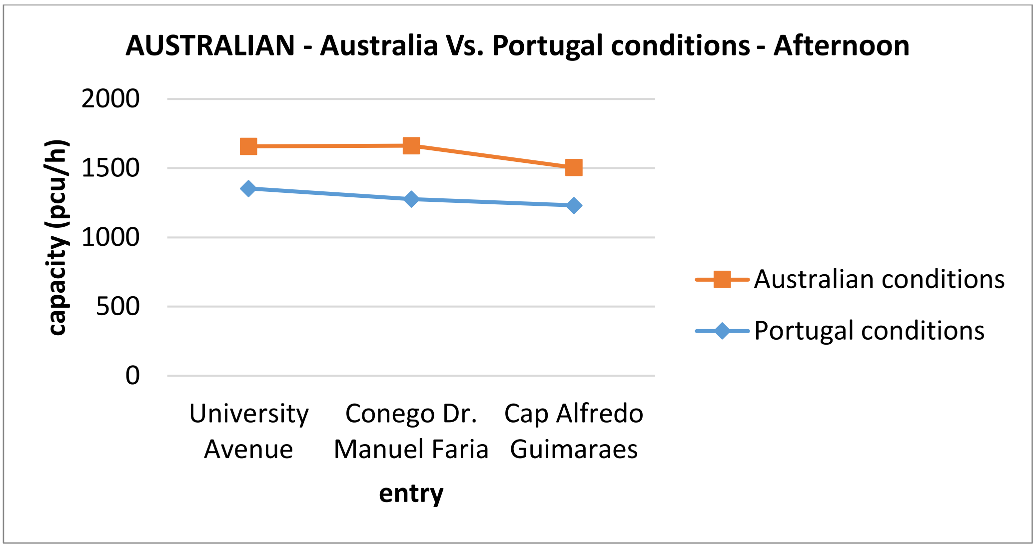



Figure 75 – Australian model under Australian and Portugal conditions (Afternoon)

- **German analytical model**

As is displayed in figures 76 - 77, the German capacity results under Portuguese conditions in the morning only reach similar capacities results in the morning. When it tested the equation with the Portuguese driver's behaviour of the single-entry lanes (Cónego Dr. Manuel Faria and Cap. Alfredo Guimaraes) are obtained a similar entry performance, while for the University Avenue, the capacity results show an important difference.

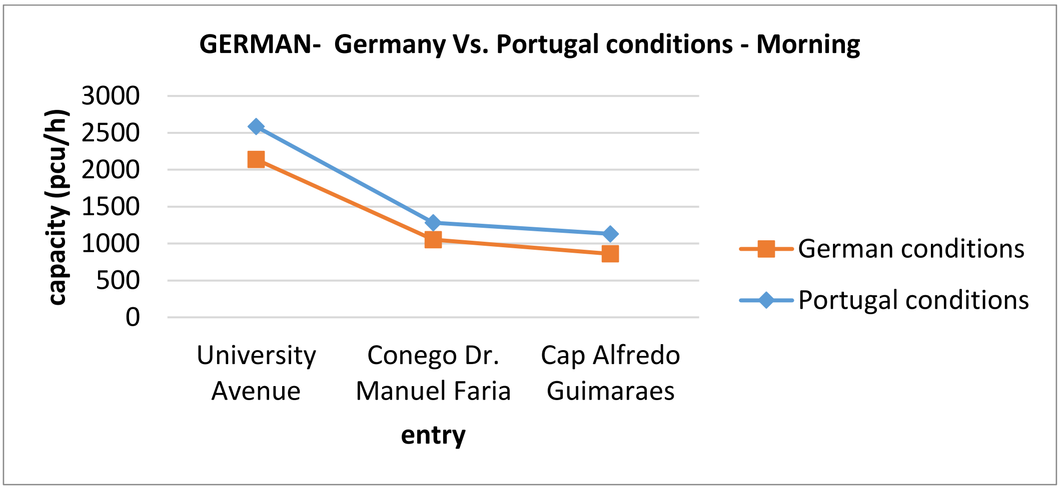



Figure 76 – German analytical model under German and Portugal conditions (Morning)

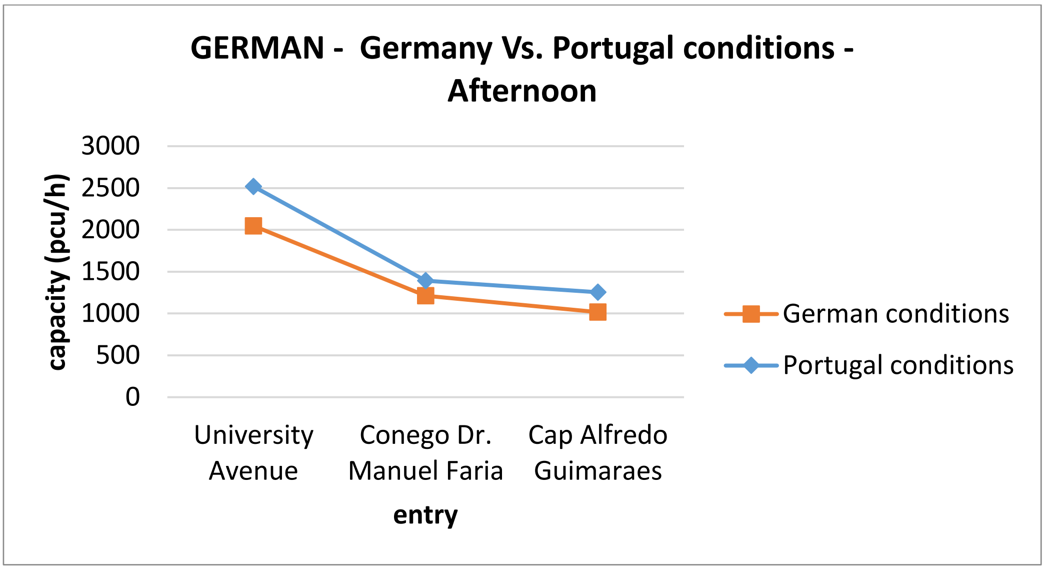



Figure 77 – German analytical model under German and Portugal conditions (Afternoon)

The results obtained for the three analytical models evaluated under Portugal conditions can be seen in figure 78-79, the findings are very similar for the analysis of the two single entry lanes, while the Australian approach achieves completely different results for University Avenue. Therefore, there is a remarkable difference in the entry capacity when analizing this approach.

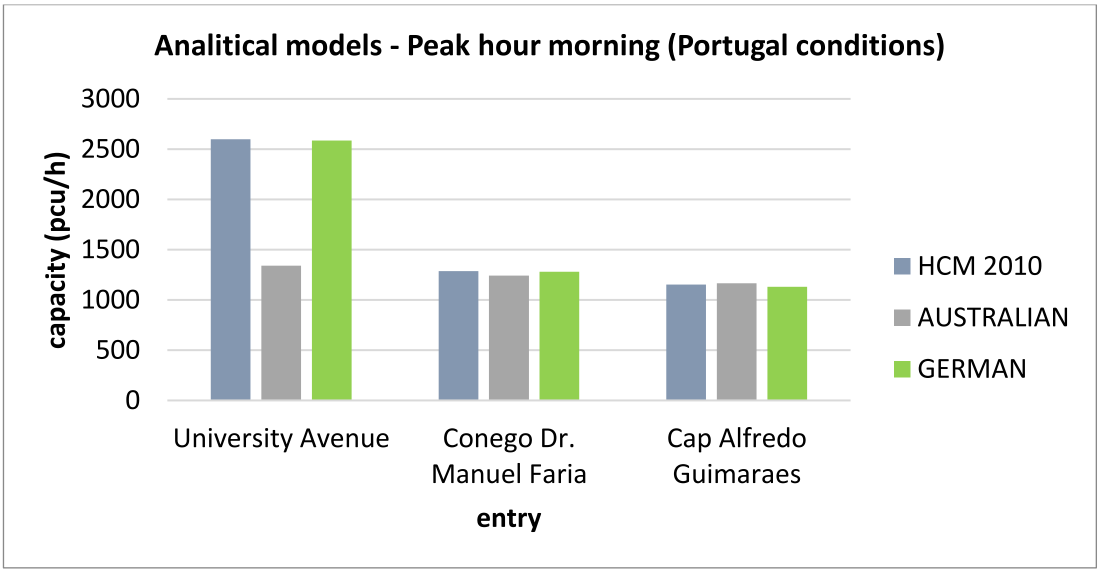



Figure 78 – German analytical model under German and Portugal conditions (Morning)

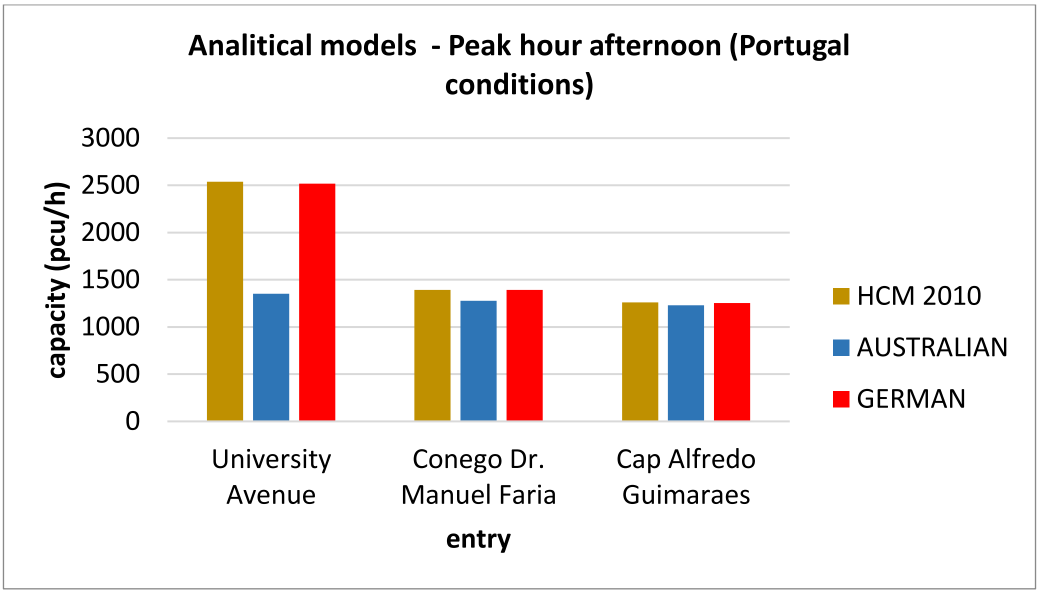



Figure 79 – Analytical models under Portugal conditions (Afternoon)

### 4.4.3 Degree of saturation

The following are the results of the degree of saturation for each of the entries of the roundabout.

The evaluation was performed for two periods of the day and for each model analyzed, as is displayed in figure 80. In the morning, most of the model shows that University Avenue has the greatest degree of saturation of the three entries analyzed. From a field inspection was proved that this avenue tends to achieve large traffic volume during the morning peak hour. During this time the traffic flow increased considerably since people travel to their workplace and other proportion must access the roundabout before entering the University campus.

The theory suggests that to consider that an entry operates under saturating conditions the degree of saturation be higher than 0.85 or 85%. In this case, although for the great majority of the models the entry on University Avenue reaches a significant level of degree of saturation, cannot be said that it is operating under such a condition, as this value varies between 0.35 and 0.75.

The degree of saturation differs from model to model due to the capacity resulting from each model.

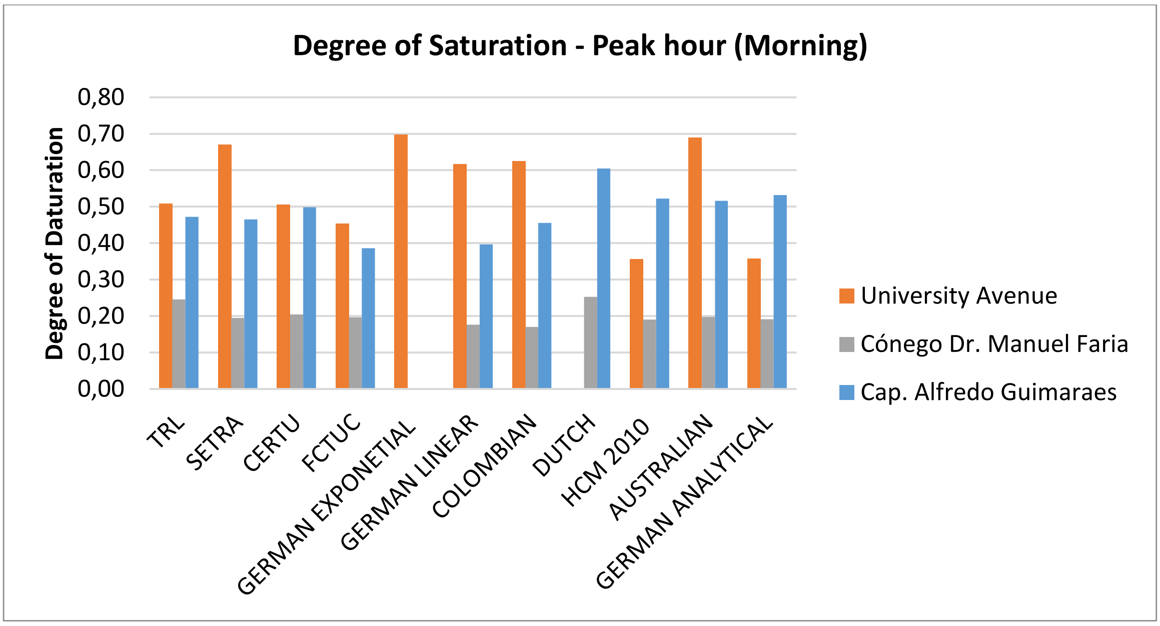



Figure 80 – Degrees of saturation (Morning)

In the afternoon, the cap Alfredo performance had the highest degree of saturation, except for the German exponential model that was not analysed. As is shown in figure 81, the greatest degree of saturation was obtained through the Dutch model. The analytical provide very similar degrees of saturation.

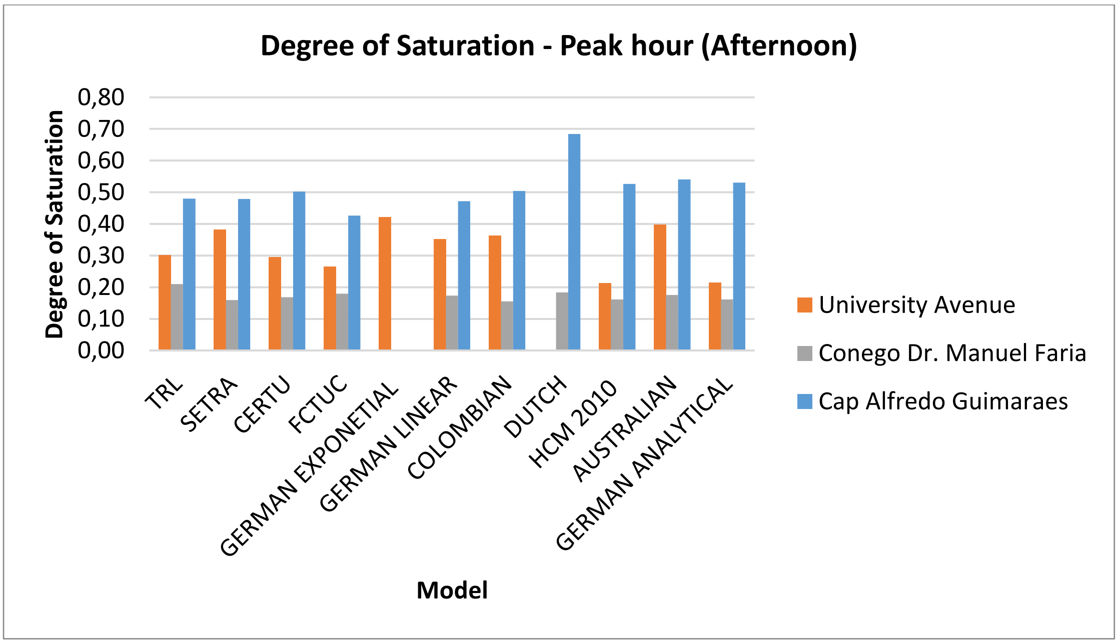



Figure 81 – Degrees of saturation (Afternoon)

## 4.5 Average Delay

Two types of delay were evaluated, the following are the results obtained for the average delay for each of the entries. Average delays have been estimated for each of the entries through equation 42. Based on the results obtained for the delay analysis, University Avenue experiences important variations on average delay, particularly in the morning. The average delay ranges from 8 to 12.73 seconds. In the afternoon, the entry performance is better compared to the morning, as drivers have a maximum delay of approximately 10 sec (figure 82).

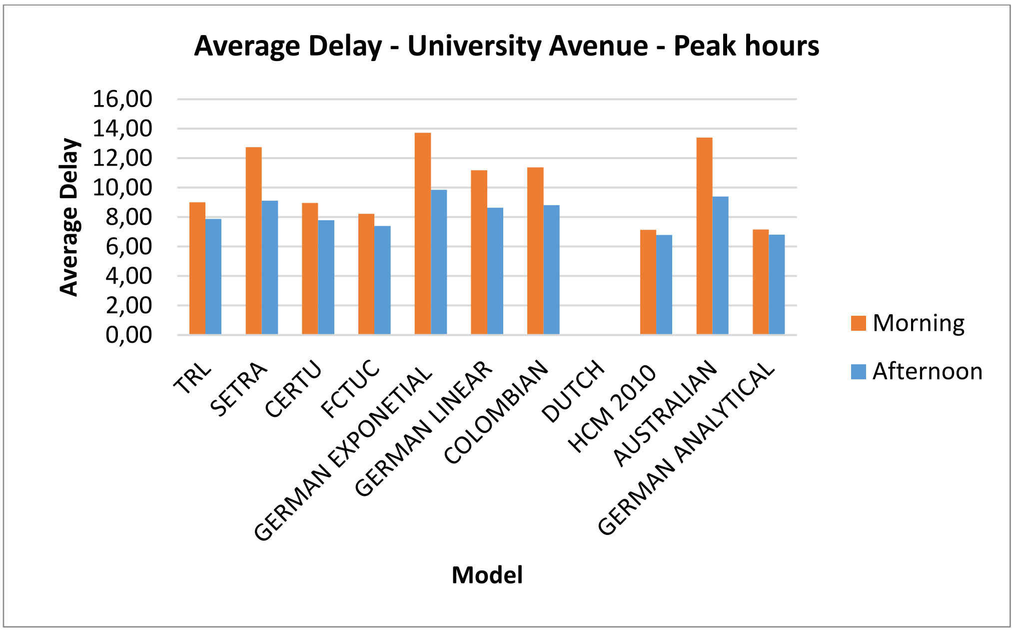



Figure 82 – Average delays for entry University Avenue

Cónego Dr. Manuel Faria Street shows no important variations on delay. During peak hours, the average delay varies from 8 sec to 10 sec. The performance of the delays in this entry can be seen in graph 83.

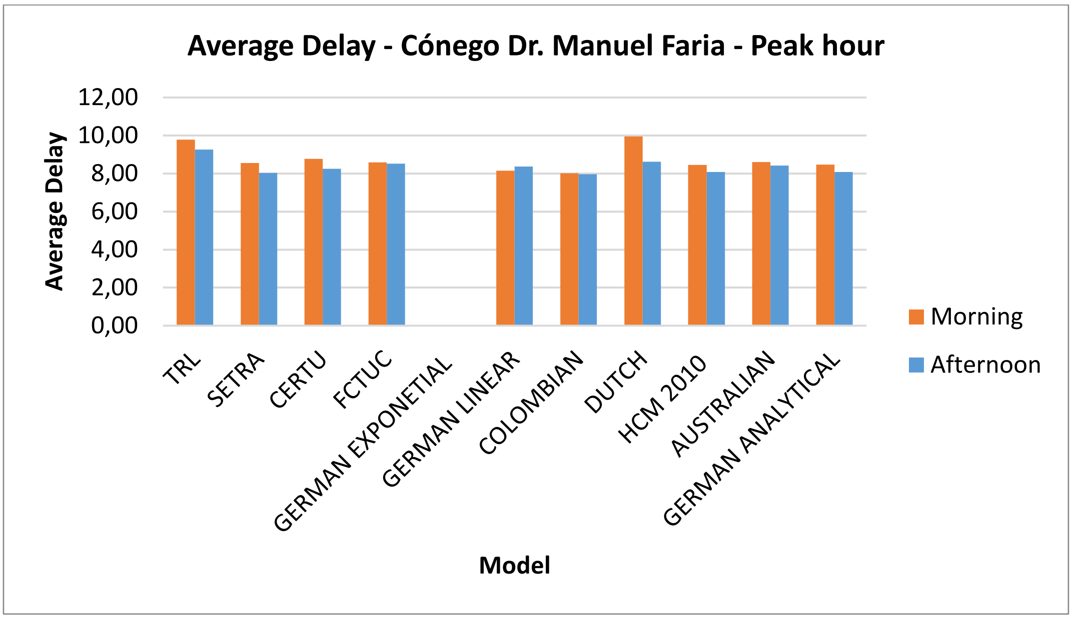



Figure 83 – Average delays for entry Cónego Dr. Manuel Faria

The performance of the capacity on Cap. Alfredo Guimaraes Street no have important changes during the periods of analysis. Nonetheless, in the morning the average delays on this entry are superior to the other entries. The largest delays come from the Dutch model.

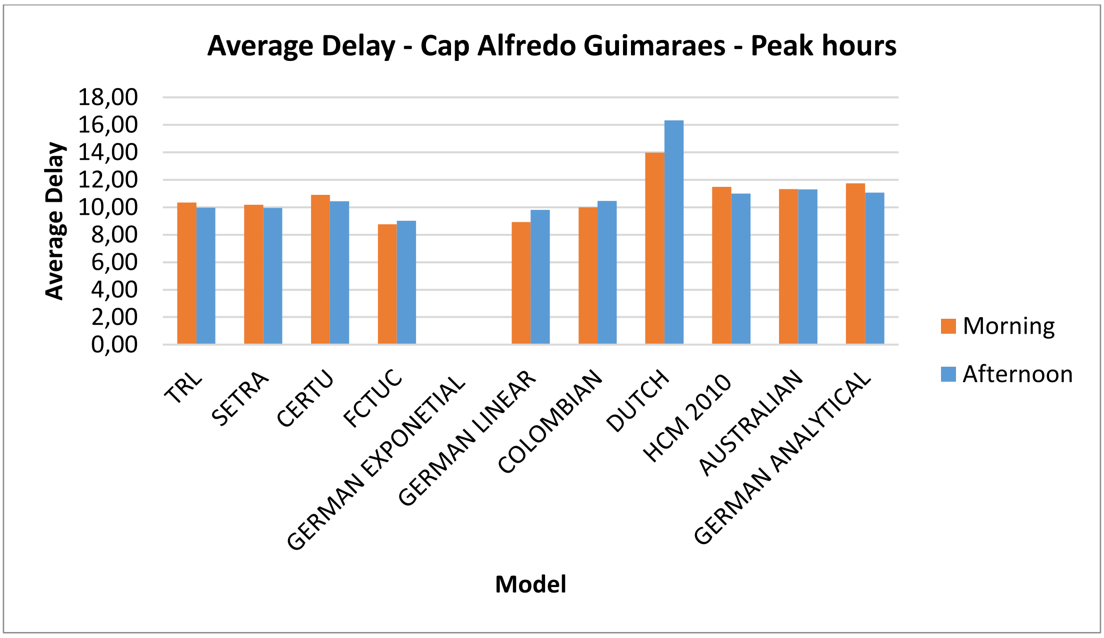



Figure 84 – Average delays for entry Cap Alfredo Guimaraes

## 4.6 Average queue length

The results obtained for the average queue length for each of the models analyzed are shown in figure 85. In the morning, University Avenue has the longest average queue lengths, followed by the Cap. Alfredo Guimaraes Street, which is due to the higher traffic volume at this entry. On Cónego Dr. Manuel Faria the delays are insignificant as the roundabout reach a queue length of fewer than 2 meters for all the models evaluated.

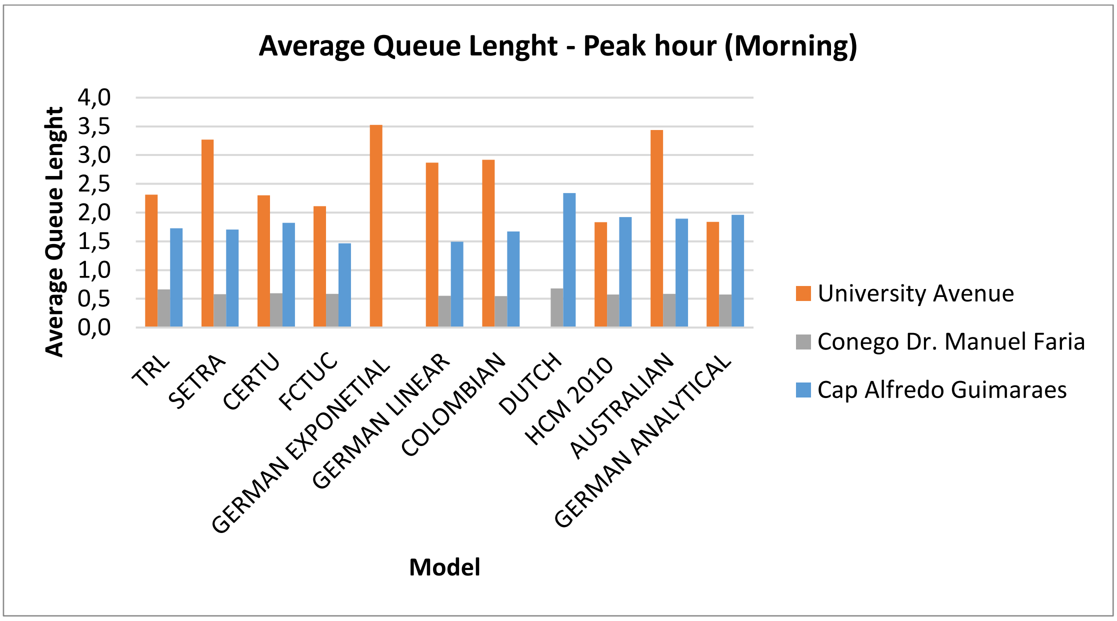



Figure 85 – Average queue length – Peak hour (Morning)

In the afternoon, the average queue lengths are shorter than in the morning in all the roundabout approaches. According to the models tested, the average queue lengths are larger in Cap. Alfredo Guimaraes Street (Figure 86).

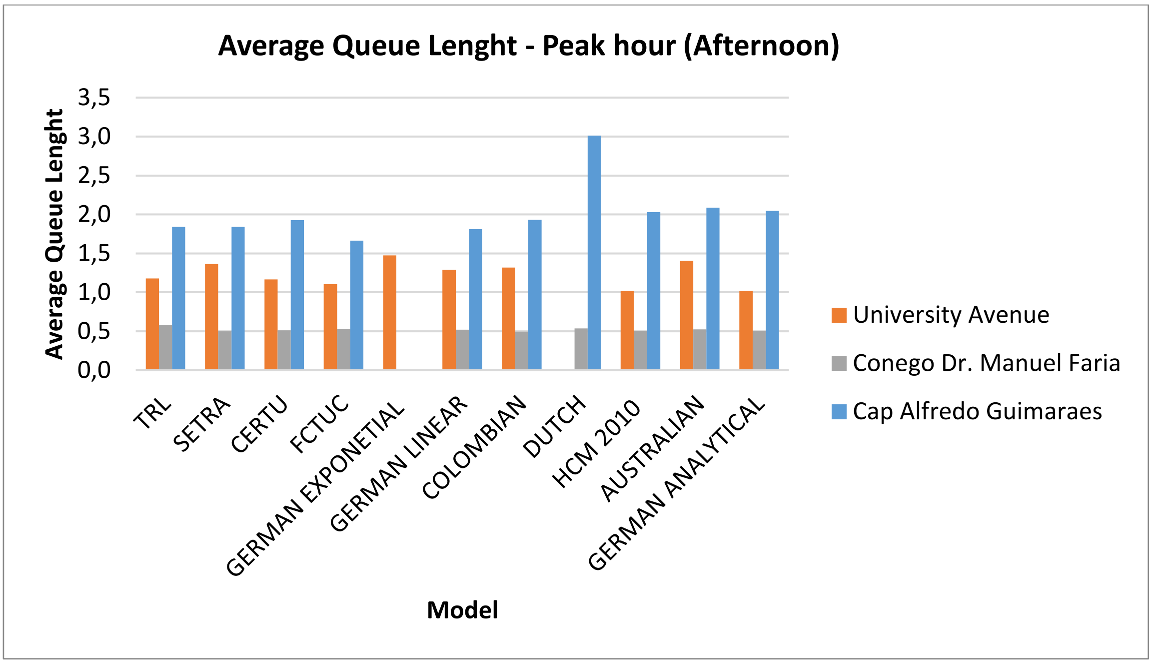



Figure 86 – Average queue length – Peak hour (Afternoon)

### 4.7 95th percentile queue length

Additionally, to the average delay and following the equation in HCM 2010, the 95th percentile queue length.

The results obtained can be seen in Figures 87 – 88. During the morning, 95% of the queues that form on University Avenue has a length of a minimum of 2 vehicles. On Cap. Alfredo Guimaraes Street, the queue lengths are shorter, the models provide uniforms queue results.

In the afternoon, the major traffic volume flow comes from the entry on the Cap. Alfredo Guimaraes. The evaluation reflects the increase of the queue length at the intersection with the variation of this parameter. Models tend to vary the length of the queue as the conflicting traffic flow increase as which implies that vehicles must stop to find a proper gap to enter the roundabout.

Considering the circulation traffic flow on the intersection, the results obtained from the calculation of the queue length for the roundabout in the afternoon shows short queues for this peak hour.

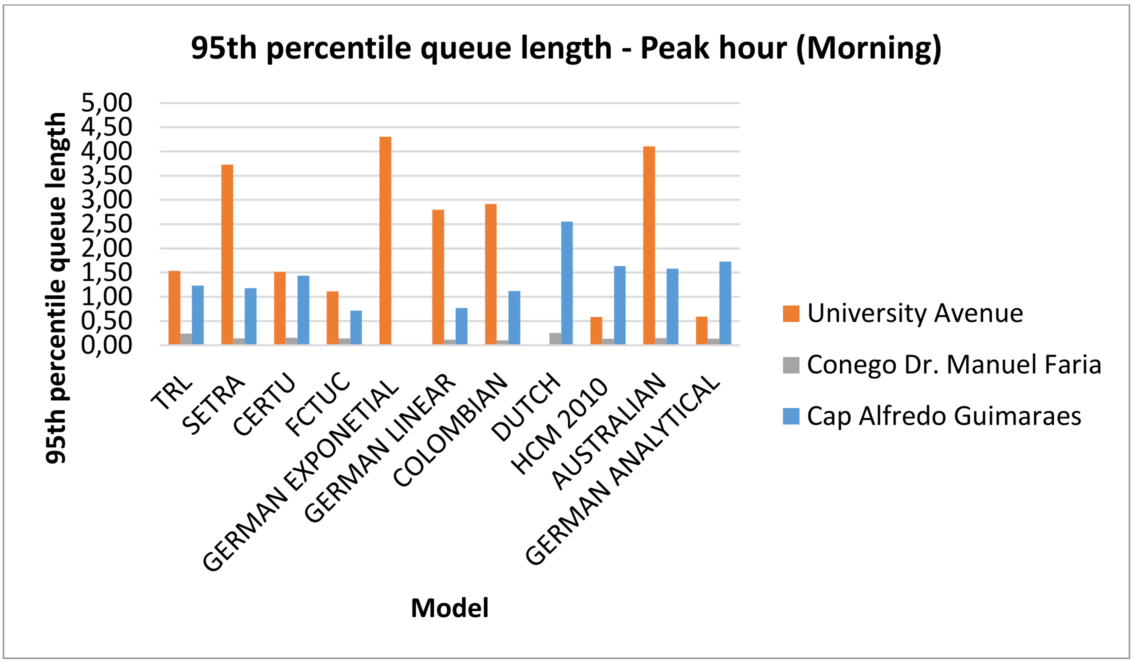



Figure 87 – 95th percentile queue length (Morning)

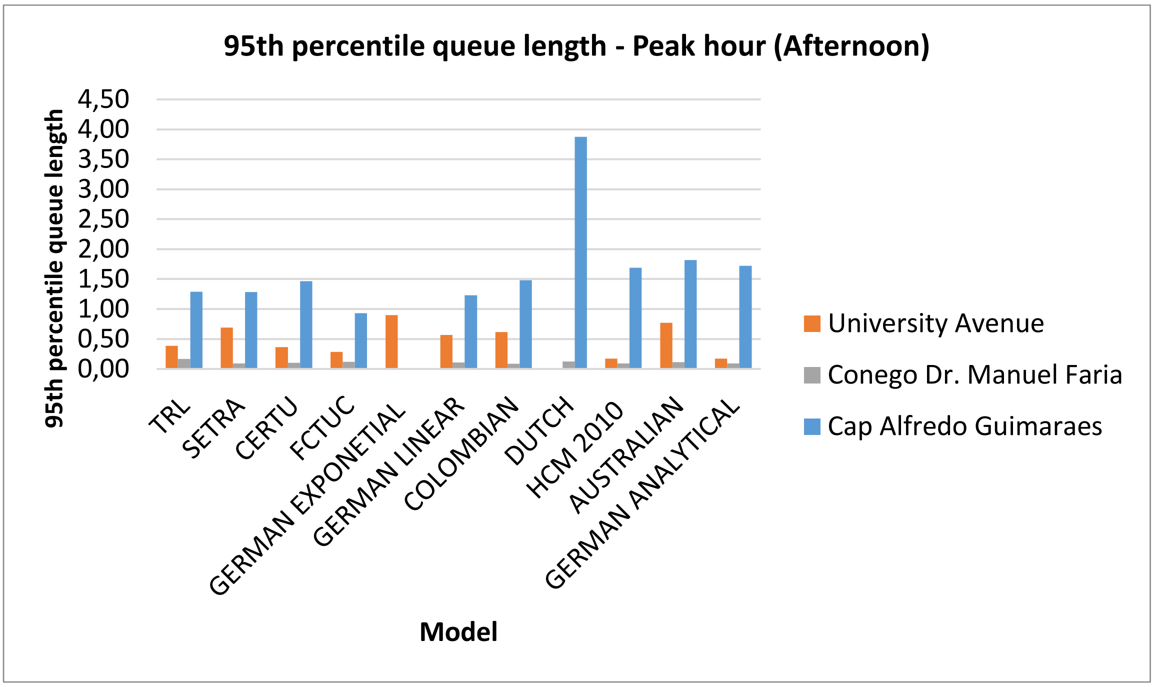



Figure 88 – 95th percentile queue length (Afternoon)

## 4.8 Modelling Methodology

The following stages were taken into consideration to appropriately construct the model that portrays the reality of the road intersection: Intersection roadway network, conflict areas, priority rule, vehicles input, and vehicles routes.

- <u>Roundabout road network</u>

To define approaches to the roundabout and create the roadway network, it was necessary to insert and scale a background image of the roundabout. The image was taken from Google Earth and exported to Vissim. Next, the links and connectors were assigned by taking into account the direction in which the approaches operate, the number and the lane width as well as the Inscribed Diameter circle dimension.

Figure 89 displays the traffic network of the roundabout, which is constructed on roadways and entries for the arms on University Avenue and Cónego Dr. Faria Street.

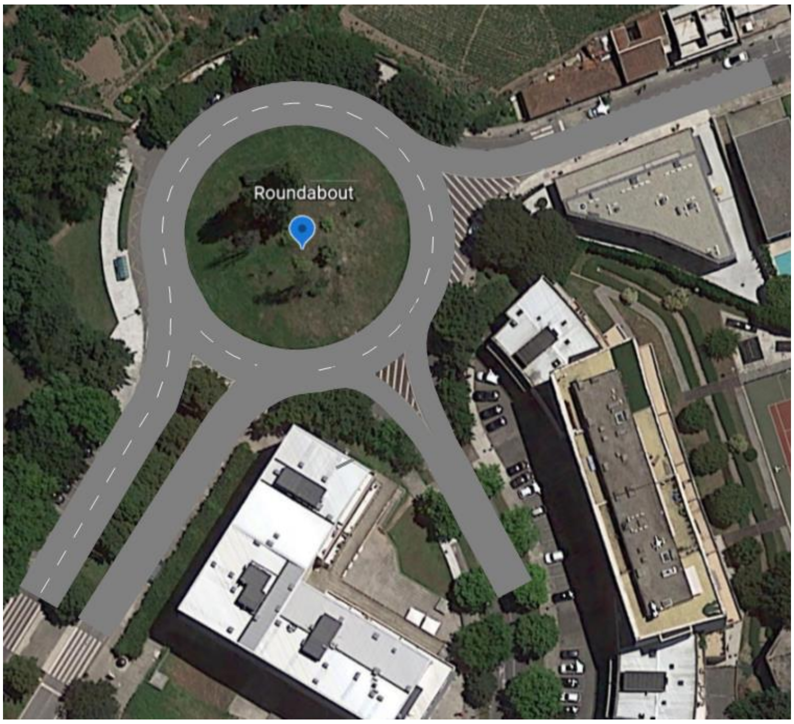



Figure 89 – road network of the roundabout in VISSIM

- Conflict points

As is shown in Figure 90 following the rule of priority in the ring of traffic, were established for each roundabout entry, vehicles circulating in the intersection have priority in the lanes and merging and diverging conflict points, were established for each roundabout entry.

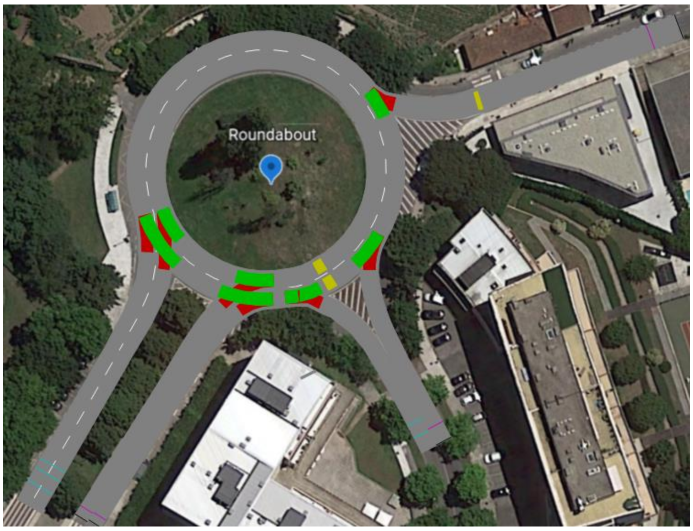



Figure 90 – Priority on the circulating lanes

- Conflict priority at the yield line

In this section, it was attributing the priority rule at the yield line. For each of the entries and lanes on the circulatory roadway of the roundabout. Figure 91 shows how these parameters were included in the roundabout entries.


Figure 91 – Priority at the yield lane

- <u>Vehicles routes</u>

Considering the movements that drivers can performance at the roundabout, were defined the routes for the 3 approaches of the roundabout. As can be seen from figure 91 to figure 96, five routes were insert in the program. The type and proportion of vehicles that were assigned for these routes indicated.


Figure 92 – University Av. to Cónego St

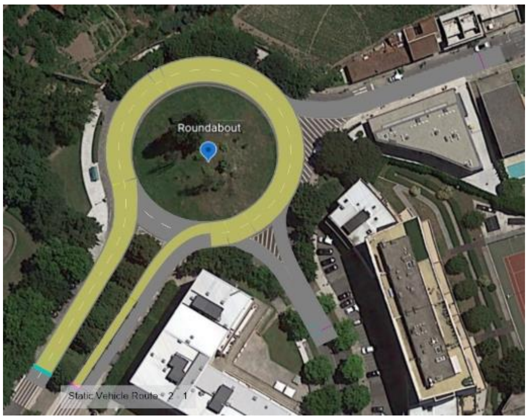



Figure 93 – University Av. to University Av.

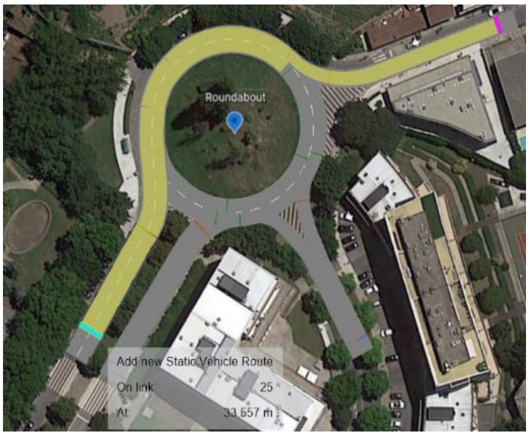



Figure 94 – Cap. Alfredo to University Av.

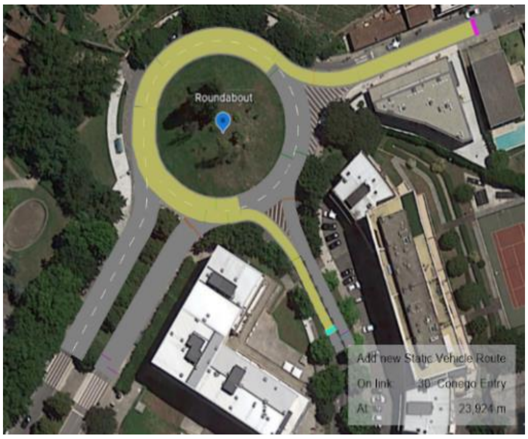



Figure 95 – Cap. Alfredo to Cónego St.

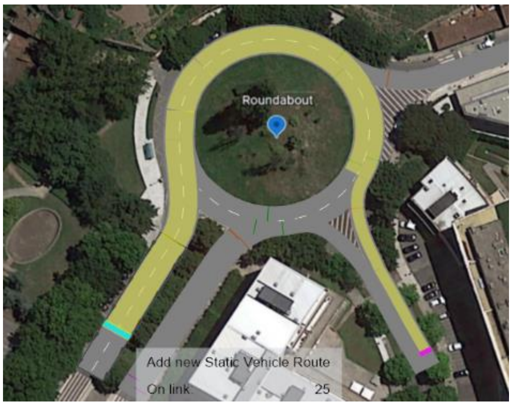



Figure 96 – Cónego to University Av.

### 4.8.1 Model calibration

To obtain a similar behaviour than in the reality, the calibration was made under the consideration of a roundabout in an urban area. Thus, it was selected the urban motorized option and the slow lane rule behaviour was activated to make the model more realistic, this option indicates to vehicles to reduce their speed when approaching obstacles.

The following are the parameters calibrated related to driving behavior for the roundabout in each section.

**Driving behaviour parameters for the following:** The parameters considered in this section for the analysis are explained as follow.

- <u>The minimum and maximum look ahead distance:</u> it was considered 0 and 20 m respectively, as it is the range for urban areas. This parameter indicates that vehicles can observe to react to other vehicles and interacting objects along its trajectory, thus, vehicles will interact with those located at a distance between 0 to 20 m to the front and lateral part.
- <u>The number of interaction objects:</u> The parameter recommended for predefined driving behaviour in urban areas was 4 vehicles, which means that vehicles can forecast the movements of four vehicles that circulate around them in this manner.
- <u>Standstill distance for static obstacles:</u> As the roundabout has a yield line at the entries, the calibration of the model was made considering two types 0.5 and 1 meter. This parameter has a significant impact on the results obtained for delays of the intersection.

**Driving behaviour parameters for car-following model:** As the analysis was made according to the model Wiedemann 74, the following parameters were adjusted at the intersection:

- <u>Average standstill distance</u>: This parameter has a higher influence on the length of the waiting lines in the model, the validation of the model was made from the length of the waiting lines. Thereby, it was convenient to choose this parameter as that belong it can define the behaviour of vehicles at the entries of the roundabout. The Average standstill for 0.5. 0.8 and 0.1 m.

The other parameters were set with default values of Vissim for urban areas (Urban Motorized).

**Driving behaviour parameters for lane change:** For describing the lane change behaviour of the drivers at the roundabout, the model was adjusted from under the following considerations:

- Maximum deceleration: For the simulation this parameter was the default value of Vissim for the Wiedermann 74 model, considering that lane changes do not occur at high speeds in urban environments. Thus, this parameter does not affect simulation results.
- Waiting time before diffusion: For this parameter, it was assigned the default value of 60 seconds. For the simulation, there were eliminated vehicles that longer than this time to change lanes.
- Minimum clearance: Based on the new priority rules, the minimum clearance consider for the intersection was 5.0 m, as a value of default for Vissim.
- Minimum rear correction of lateral position: For the calibration, it was considered a default value of 3 km/h.

**Driving behaviour parameters for lateral movements:** The parameters used to calibrate the driving behaviour for the lateral movements of the vehicles are defined as follow.

- Desired position at the free flow: It is activated the option of the middle of the lane, this option was selected assuming that vehicles move involuntarily in the centre of the lane.
- Observe adjacent lanes: This option was activated to indicate vehicles move with more caution and consider the lateral distance to adjacent vehicles.
- Diamond queuing: This option has been enabled to obtain a more accurate measure of waiting queues.

*Note: The data used to calibrate this case study in Vissim is made from an analysis of the traffic network.*

### 4.8.2 Model validation

A total of 4 models were made, varying the average stopping distance and the stopping distance for static obstacles, and the clearance distance at the entries. It should be noted that, for each model, it was necessary to perform 5 simulations and to use the average results, since there may be changes in the results between each simulation. For each model, a warm-up time of 900 seconds was considered, after which the data were collected.

The validation of the models was made from field inspection and by using photo interpretation tools such as Google map, which collects a series of data on traffic conditions at the intersection, in addition to the validation of traffic flow parameters and travel times at the roundabout.

- **Traffic flow validation**

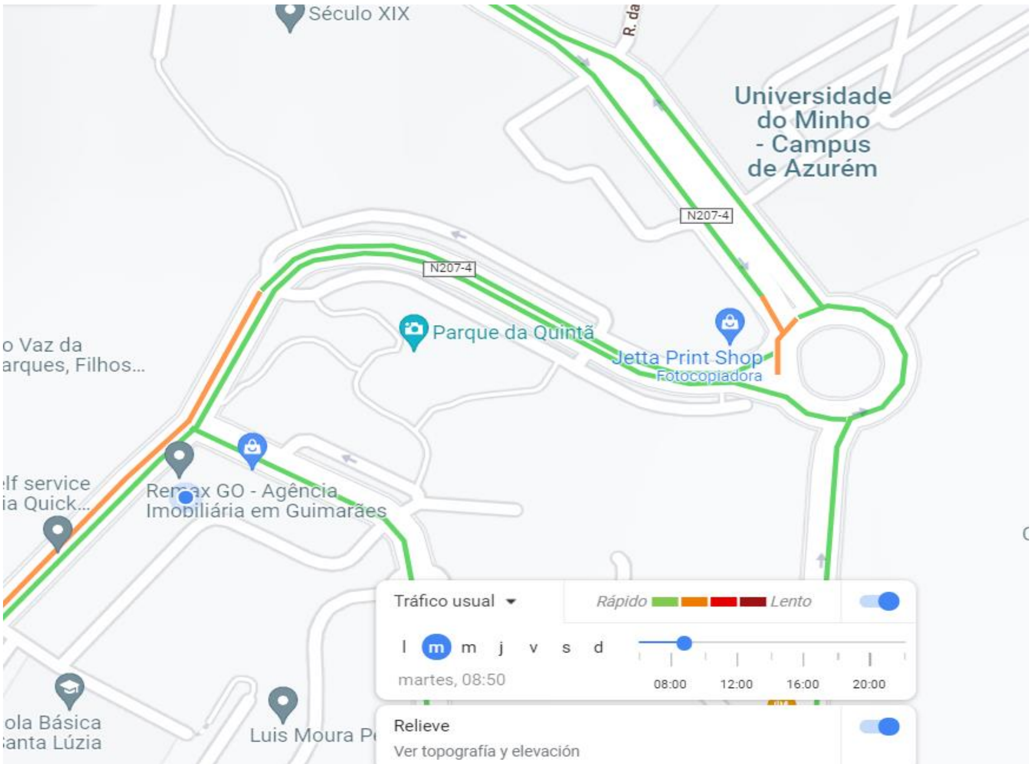



Figure 97 – Traffic at the roundabout – Morning (Google Maps, 2021)


Figure 98 – Traffic at the roundabout - Afternoon (Google Maps, 2021)

The validation of the model based on vehicular flows, the GEH statistic, named after Geoffrey E. Havers, who devised it in the 1970s while working as a transport planner in London, England, would be used to validate the model based on vehicular flows. Although the equation has the same mathematical structure as a chi-square test, it is not a real statistical test. Nonetheless, it is an empirical formula that is effective for some traffic analysis tasks (WD,2021).

- A GEH of less than 5 is considered a satisfactory match between modeled and observed hourly volumes in traffic modelling studies.
- GEH values in the range of 5 to 10 indicate that the data requires further investigation and correction.
- GEH values greater than 10 indicates that there is a problem with the travel demand model or the data, which could be as simple as a data entry error or as complex as a serious model calibration problem.

The equation for verifying traffic flows is shown as follows:

$$GEH = \sqrt{\frac{2(M-C)^2}{M+C}} \tag{55}$$

*M* is the hourly traffic volume obtained from the simulation and *C* is the actual hourly traffic count.

As the calibration was performed for the morning peak period, the model was validated in the afternoon peak period. The validation of the model using vehicle flows is summarized in the table below, contrasting the findings obtained with field measurements and those acquired with micro-simulation at afternoon peak hours.

Table 25- Traffic flow verification - Afternoon

| **Afternoon** | | | |
|---|---|---|---|
| **Entry** | **Traffic flow in Situ (pcu/h)** | **Entery flow in Vissim (pcu/h)** | **GEH** |
| University Avenue | 538 | 634 | 3,97 |
| Cónego Dr. Manuel Faria | 224 | 301 | 4,75 |
| Cap. Alfredo Guimaraes | 293 | 351 | 3,23 |
| | | 5 > | 3,98 |

According to the results of the traffic flow validation, it is concluded that the simulation fits the flow data collected in the field. As a result, the analysis is carried out using the model calibrated.

- **Travel times**

  The validation travel times were collected on October 13, 2021, between 8:15 AM and 9:15 AM. The travel times were taken on the circulatory roadway, as shown in figure 99. Approximately 35 meters was measured in Google Earth and during this time were recorded the travel time of 60 vehicles (See appendix D).

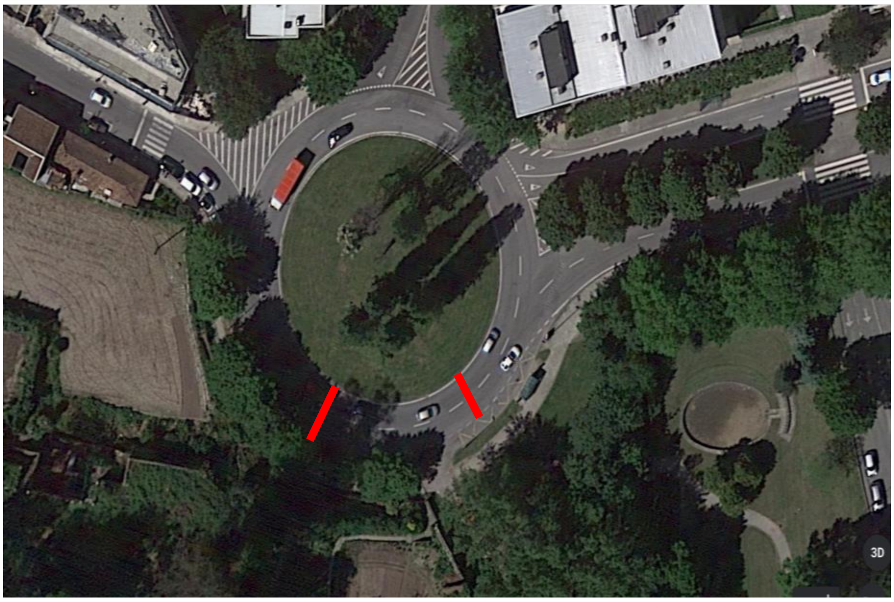



Figure 99 – data collection area (Google Maps, 2021)

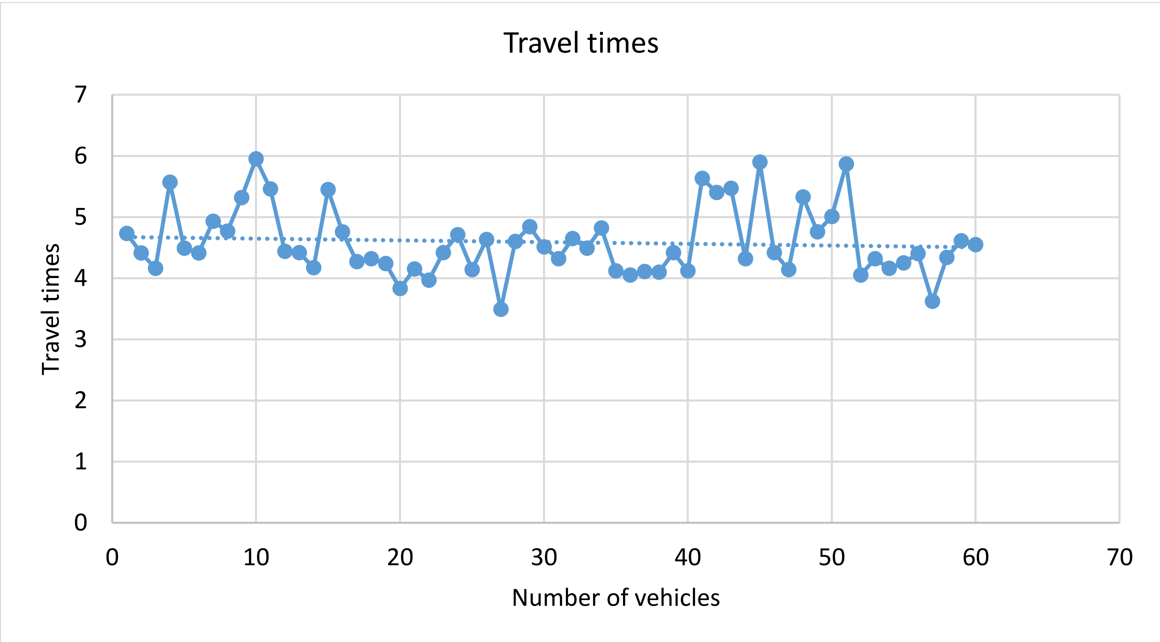



Figure 100 –Travel times

### 4.9 Capacity

Roundabout capacity is defined as the maximum flow that can be accommodated at the entry of a roundabout. The entry capacity of a roundabout is determined by two factors: the traffic flow at the roundabout that interferes with the entering flow, and the geometric characteristics of the roundabout.

Vissim does not directly compute the capacity of the roundabout. However, some strategies can be utilized to obtain the roundabout capacity. For the estimation of the capacity, the roundabout was oversaturated by increasing entry traffic volumes, which reducing the competitive traffic to which that entry was subjected. The number of vehicles that could complete the required movement in one hour was the entry capacity.

The following analysis was made only for the peak hour morning, due to its characteristics of the traffic. The results obtained are the following.

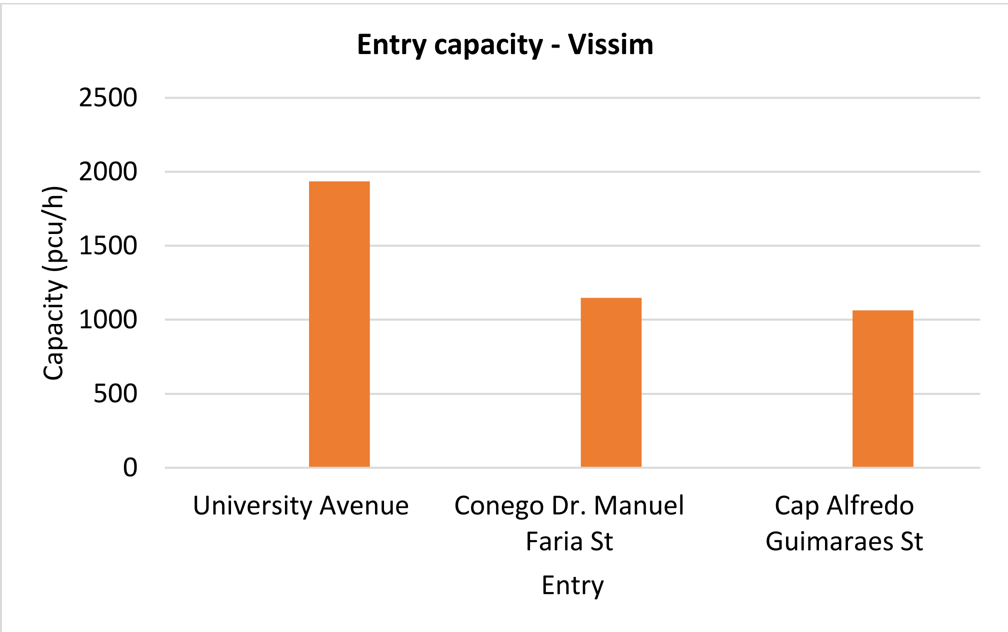



Figure 101 –Capacity VISSIM– Peak hour (Morning)

The capacity results demonstrate that the conflicting traffic a meaningful effect on entry capacity performance. Thus, due to its geometric characteristics and traffic volumes, University

Avenue experienced the highest capacity of all the entries, followed by the approach on Cónego Dr. Manuel Faria, which has a high conflicting traffic flow since it receives the conflicts with the entries University Avenue to Cap. Alfredo Guimaraes.

Entry Cap. Alfredo Guimaraes has the smallest entry capacity of all de entries, it is because despite its geometric characteristics, receive a large proportion of vehicles that attempt to enter the roundabout. The proportion of vehicles that can access the intersection through this entry is dependent on traffic flow from the other approaches of the intersection, particularly the traffic flow coming from University Avenue and Cónego Dr. Manuel Faria, towards the University Avenue exit.

### 4.10 Average delay

The results of the average delay for each entry show that University Avenue and Cap. Alfredo Guimaraes have the longest average delays because during the morning peak hour these entries receive an important traffic flow and remarkable traffic conflicting. Consequently, vehicles must wait for a longer amount of time to access the roundabout.

The performance of the delay on the simulation are like those obtained from evaluation through empirical and analytical models for the morning peak hours, the results of the average delay estimation for the intersection are shown in figure 102.

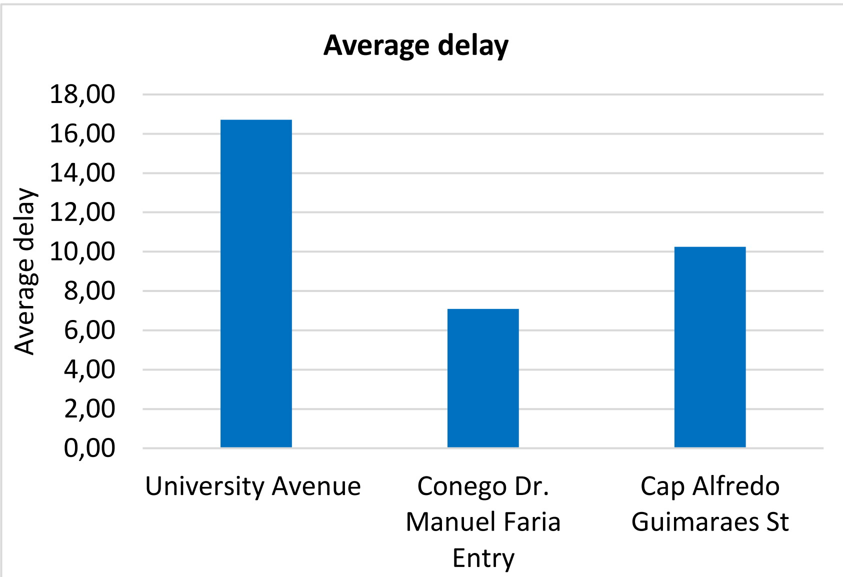



Figure 102 – Average delay VISSIM– Peak hour (Morning)

### 4.11 Average queue length

Figure 103 shows the simulation results for the average queue length of the entries for the period under consideration, University Avenue and Cap Alfredo Guimaraes Street have the longest queue length, while entry Cónego Dr. Manuel Faria record the shortest queue length of all the entries analyzed.

This performance indicator is determined by the traffic flow conditions of the roundabout since queues will occur on the approaches if there is a higher traffic volume per entry as well as a significant conflicting flow.

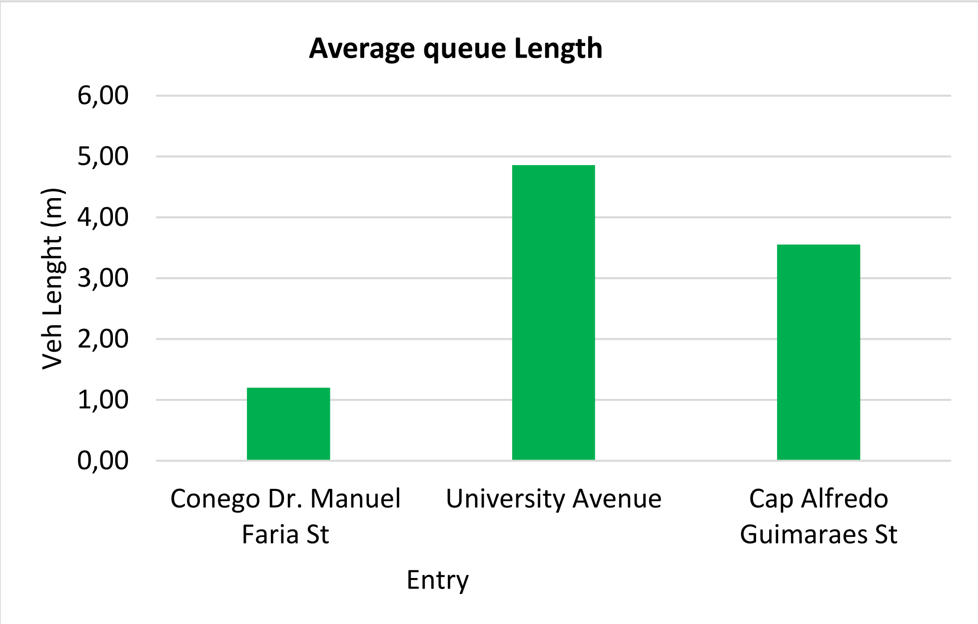



Figure 103 – Average queue length VISSIM– Peak hour (Morning)

### 4.12 Model comparison

The following is a comparison of the methodologies applied in the analysis of the performance. The study results are related to the capacity estimation and the performance indicators of the roundabout in the morning peak hour.

#### 4.12.1 Capacity

The entry capacity for each of the methodology are shown in table 26. The empirical and analytical models both reflect the highest capacity at the University Avenue entry. For the others two entries, empirical models differ from analytical models, as the capacity estimation through these shows that by its characteristics Cónego Dr. Manuel Faria can accommodate a greater number of vehicles than the entry Cap Alfredo Guimaraes.

Analytical models include some parameters such as, critical gap and follow-up time to describe the behavior of the drivers, they are that most closely resemble the data gathered in the micro simulation of the roundabout.

Table 26- Capacity evaluation

| Peak hour morning (pcu/h) | | | |
|---|---|---|---|
| **MODEL/ENTRY** | **University Avenue** | **Cónego Dr. Manuel Faria** | **Cap. Alfredo Guimaraes** |
| TRL | 1818 | 996 | 1274 |
| SETRA | 1378 | 1257 | 1293 |
| CERTU | 1826 | 1200 | 1207 |
| FCTUC | 2037 | 1247 | 1559 |
| GERMAN EXP | 1324 | - | - |
| GERMAN LINEAR | 1499 | 1387 | 1517 |
| COLOMBIAN | 1480 | 1440 | 1320 |
| DUTCH | - | 970 | 996 |
| HCM 2010 | 2599 | 1286 | 1153 |
| AUSTRALIAN | 1342 | 1242 | 1167 |
| GERMAN | 2586 | 1280 | 1132 |
| VISSIM | 1935 | 1147 | 1063 |

#### 4.12.2 Degree of saturation

Degrees of saturation have been computed; the results are shown in table 27. According to the models evaluated, University Avenue has the greatest degree of saturation of all the entries. The analytical model analyses reveal that the Australian model and the German analytical model provide low saturation results for this entry.

During this period, Cónego Dr. Manuel Faria Street has the lowest saturation of the entries, with empirical models estimating ranging from 0.18 to 0.25 the degree of saturation. For the

CERTU and FCTUC models, this indicator is 0.20 in this entry. The analytical models show a consistent result of 0.19.

The empirical models indicate that the relationship between traffic volumes and capacity for Cap Alfredo Guimaraes Street is between 0.39 and 0.60. While the saturation degree findings for the analytical models remain between 0.52 and 0.53.

Table 27- Degree of saturation for morning peak hour

| Degree of Saturation - Peak Hour Morning | | | |
|---|---|---|---|
| **Model** | **University Avenue** | **Cónego Dr. Manuel Faria** | **Cap. Alfredo Guimaraes** |
| TRL | 0.51 | 0.25 | 0.47 |
| SETRA | 0.67 | 0.19 | 0.47 |
| CERTU | 0.51 | 0.20 | 0.50 |
| FCTUC | 0.45 | 0.20 | 0.39 |
| GERMAN EXPONETIAL | 0.70 | - | - |
| GERMAN LINEAR | 0.62 | 0.18 | 0.40 |
| COLOMBIAN | 0.62 | 0.17 | 0.46 |
| DUTCH | - | 0.25 | 0.60 |
| HCM 2010 | 0.36 | 0.19 | 0.52 |
| AUSTRALIAN | 0.69 | 0.20 | 0.52 |
| GERMAN ANALYTICAL | 0.36 | 0.19 | 0.53 |
| VISSIM | 0.63 | 0.42 | 0.53 |

### 4.12.3 Performance indicators

Tables 28 - 30 summarize the results of the performance indicators of each of the entries of the roundabout based on models studied and for the micro-simulation models. More saturation indicates greater delays in the entry, resulting in longer queue lengths.

Taking into consideration the characteristics of the traffic, delays of up to 16.72 seconds were obtained for University Avenue. The analytical models, excluding the Australian model, exhibit shorter delays for the entry, consequently, it has the largest average queue length of this period. The German linear model and Colombian methodology are extremely close in the results.

Table 28- Performance indicators for University Avenue

| **Model** | **Performance indicators – Peak Hour Morning** | | | | |
|---|---|---|---|---|---|
| | **ENTRY - University Avenue** | | | | |
| | Vx | Cmx | Average Delay | Average Queue Lenght | 95th percentile queue length |
| TRL | 924 | 1818 | 9.01 | 2.3 | 1.53 |
| SETRA | | 1378 | 12.75 | 3.3 | 3.73 |
| CERTU | | 1826 | 8.97 | 2.3 | 1.51 |
| FCTUC | | 2037 | 7.74 | 2.0 | 0.86 |
| GERMAN EXPONETIAL | | 1324 | 13.73 | 3.5 | 4.30 |
| GERMAN LINEAR | | 1499 | 11.18 | 2.9 | 2.79 |
| COLOMBIAN | | 1480 | 11.38 | 2.9 | 2.91 |
| DUTCH | | - | - | - | - |
| HCM 2010 | | 2599 | 7.15 | 1.8 | 0.58 |
| AUSTRALIAN | | 1298 | 14.29 | 3.7 | 4.63 |
| GERMAN ANALYTICAL | | 2586 | 7.16 | 1.8 | 0.59 |
| VISSIM | - | | 16.72 | 4.8 | - |

Table 29- Performance indicators for Cónego Dr. Manuel Faria

| **Model** | **Performance indicators- Peak Hour Morning** | | | | |
|---|---|---|---|---|---|
| | **ENTRY - Cónego Dr. Manuel Faria** | | | | |
| | Vx | Cmx | Average Delay | Average Queue Lenght | 95th percentile queue length |
| TRL | 245 | 996 | 9.79 | 0.7 | 0.24 |
| SETRA | | 1257 | 8.55 | 0.6 | 0.14 |
| CERTU | | 1200 | 8.77 | 0.6 | 0.16 |
| FCTUC | | 1247 | 8.59 | 0.6 | 0.14 |
| GERMAN EXPONETIAL | | - | - | - | - |
| GERMAN LINEAR | | 1387 | 8.15 | 0.6 | 0.11 |
| COLOMBIAN | | 1440 | 8.01 | 0.5 | 0.10 |
| DUTCH | | 970 | 9.96 | 0.7 | 0.25 |
| HCM 2010 | | 1286 | 8.46 | 0.6 | 0.13 |
| AUSTRALIAN | | 1286 | 8.46 | 0.6 | 0.13 |
| GERMAN ANALYTICAL | | 1280 | 8.48 | 0.6 | 0.14 |
| VISSIM | | | 7.08 | 0.5 | - |

Table 30- Performance indicators for Cap. Alfredo Guimaraes

| **Model** | **Performance indicators- Peak Hour Morning** | | | | |
|---|---|---|---|---|---|
| | **ENTRY - Cap Alfredo Guimaraes** | | | | |
| | Vx | Cmx | Average Delay | Average Queue Lenght | 95th percentile queue length |
| TRL | 602 | 1274 | 10.33 | 1.7 | 1.23 |
| SETRA | | 1293 | 10.18 | 1.7 | 1.18 |
| CERTU | | 1207 | 10.91 | 1.8 | 1.43 |
| FCTUC | | 1705 | 8.26 | 1.4 | 0.57 |
| GERMAN EXPONETIAL | | - | - | - | - |
| GERMAN LINEAR | | 1517 | 8.92 | 1.5 | 0.77 |
| COLOMBIAN | | 1320 | 9.99 | 1.7 | 1.12 |
| DUTCH | | 996 | 13.98 | 2.3 | 2.55 |
| HCM 2010 | | 1153 | 11.48 | 1.9 | 1.64 |
| AUSTRALIAN | | 1153 | 11.48 | 1.9 | 1.64 |
| GERMAN ANALYTICAL | | 1132 | 11.74 | 2.0 | 1.73 |
| VISSIM | | | 10.24 | 1.5 | - |

The results of the traffic performance at Cónego Dr. Manuel Faria and Cap Alfredo Guimaraes in of each model studied are similar. The micro-simulation results indicate that the average delays are lower than those estimated for the same entries through the other methodologies. The average queue lengths have small variation for models, except for Dutch model, the results of average queue lengths are shorter than 2.0.

# CHAPTER V

# CONCLUSIONS

*"Not every innovation in transportation is going to come from government or even a large enterprise. There are smart people out there with tools and skills to come up with great ideas".*

**Anthony Foxx**

### 5.1 General conclusions

Roundabouts have become an essential element of the transportation network of cities all over the world. The advantages that roundabouts offer to the road network constitute an excellent solution to solve the great majority of intersection problems, as their implementation minimizes the number of conflict points at junctions and control driver speed by the imposition of geometric features.

Capacity is a key indicator to know under which the roundabouts operate and to evaluate the roundabout performance, which should be designed to operate at no more than 85% of its estimated capacity, from this value, there is a significant variation of performance indicators by the increase of traffic congestion.

Models for estimating the capacity at roundabouts were developed to solve the existing problems presented on the initial roundabout typologies, which worked with different yield rules, that were modified to the modern roundabout typology where the improvements based on new and current geometric characteristics were introduced originating new empirical and analytical models to estimate the capacity and performance of roundabouts.

The comparison of different methodologies for evaluating the performance of the roundabout is important for determining the operation of these intersections in the road network, since it helped to corroborate the methods used in different countries*, i.e.,* the values of capacity are similar amongst the different methods namely for undersaturation conditions. The approaches of the models are based on geometric factors and driving characteristics, and although each country has adjusted their methodologies to their own traffic characteristics, the results determine that besides that the difference, the models' approaches are similar.

Except for the Colombian model, both empirical and analytical models take conflicting traffic flow into account when calculating entry capacity. The French models, SETRA and CERTU, and the Dutch model consider the exit flows as important parameters in the roundabout performance. Thus, the capacity is calculated using an impeding traffic flow which describe the influence of the exiting flow on entry capacity.

Critical gaps and follow-up times are the basis for capacity analysis through analytical models. According to the logit models developed, the likelihood of accepting the gap decreases as the speed of the conflicting vehicle increases and increases with increasing distance from the conflicting vehicle.

Traffic microsimulation is a very helpful tool to describe in detail existing scenarios and vehicle behaviour on the road network. For the microsimulation was used PTV Vissim, the program allows characterizing the behaviour of drivers and their interaction with obstacles, and other events that drivers experience while driving on the road infrastructure.

For the microsimulation of the roundabout, it was possible to calibrate and validate the model from data and field inspection of roundabout study. Although the software does not provide capacity data, it was possible to estimate this parameter by using some strategies for the evaluation of the entry capacity.

### 5.1.1 Specific conclusions for the case study

The case study of the performance of the roundabout under consideration by the different models shows that the intersect roundabout operates in acceptable conditions. Although the results show that the intersection is not operating under saturated conditions, considering that an entry is deemed saturated when it has a degree of saturation of more than 85 %, it was verified in the field that the intersection reflects saturation during times of the day, this is because this exit has congestion at peak hours (slow traffic), which impacts the traffic in the roundabout.

The most significant capacity variations of the models occurred on University Avenue, and these differences are a consequence of the geometric characteristics and traffic conditions over the period studied. Models use conflicting traffic as an important parameter for calculating entry capacity, the results reveal that for most of them the higher is the conflicting flow, the lower the capacity of the entry.

In general terms, the empirical models analyzed in this study show comparable results for the entries Cónego Dr. Manuel Faria and Cap Alfredo Guimaraes. Therefore, entry Cap. Alfredo Guimaraes has better performance for the empirical models that takes into consideration specific geometric parameters in the performance evaluation.

The analytical models show that owing to traffic circumstances and driver behaviour conditions, the highest capacity correlate to the University Avenue entry, while the worst performance is observed on Cap. Alfredo Guimaraes. Moreover, even though the analytical models were adjusted and analyzed using the driving behaviour parameters of Portuguese drivers, it can be seen models have some important differences for University Avenue. Nonetheless, data obtained are close to those obtained through the analysis of the other models.

The performance indicators take capacity into consideration, empirical, analytical, and micro-simulation approaches for each entry are important parameters to have an overview of the operation of the roundabout. For this study, some parameters were collected in the field to analyse, calibrate, and evaluate the model using microsimulation.

### 5.1.2 Future works

For future works, the study could be extended to include new models, especially those used to estimate the capacity of other types of roundabouts, *i.e.,* the methodology for calculating the capacity of Dutch turbo roundabouts, since it does not provide a method for estimating the capacity of multi-roundabouts.

Although a good analysis of the analytical and microsimulation models with critical gaps and follow-up times of the Portuguese drivers was achieved, a study of the critical gaps and follow-up times of Guimaraes drivers could be carried out and included in the analysis to obtain more approximate data.

The roundabout is in an urban area that link with residential areas. Thus, it is recommended that future studies perform a microsimulation that considers the traffic flows, pedestrians, and the driver behaviour on the roads that intersect with the roundabout approaches to have a better understanding of the influence of the traffic flow at the exits on the roundabout performance. Although a correct calibration was developed, it was observed that the approaches that link with the roundabout have a considerable traffic flow and pedestrian interruptions.

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## 5.2 Appendix

### 5.2.1 Appendix A: Roundabout capacity model test

- Conflicting flows

$$\boldsymbol{Q_c^A} = Q_{BC} + Q_{BD} + Q_{CD} = 290 + 1200 + 130 = 1620\, pcu/h$$

$$\boldsymbol{Q_c^B} = Q_{CA} + Q_{CD} + Q_{DA} = 120 + 130 + 140 = 390\, pcu/h$$

$$\boldsymbol{Q_c^C} = Q_{DA} + Q_{DB} + Q_{AB} = 140 + 950 + 230 = 1320\, pcu/h$$

$$\boldsymbol{Q_c^D} = Q_{AB} + Q_{AC} + Q_{BC} = 230 + 180 + 290 = 700\, pcu/h$$

- Exiting flows

  - $\boldsymbol{Q_u^A} = Q_{BA} + Q_{CA} + Q_{DA} = 220 + 120 + 140 = 480\, pcu/h$
  - $\boldsymbol{Q_u^B} = Q_{AB} + Q_{CB} + Q_{DB} = 230 + 190 + 950 = 1370\, pcu/h$
  - $\boldsymbol{Q_u^C} = Q_{AC} + Q_{BC} + Q_{DC} = 180 + 290 + 250 = 720\, pcu/h$
  - $\boldsymbol{Q_u^D} = Q_{AD} + Q_{BD} + Q_{CD} = 175 + 1200 + 130 = 1505\, pcu/h$

**Empirical models**

The entry capacity results calculated through empirical models are as follows:

- **TRL**

1. Accumulation factor ($K$)

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

$$\mathrm{K} = 1 - 0.00347\ (60 - 30) - 0.978\ \times \left\{ \frac{1}{40} - 0.05 \right\} = 1.02$$

2. Sharpness of flare ($S$) expressed in (*m/m*).

$$S = 1.6 \times \frac{(e - v)}{l'} = 1.6 \times \frac{(6.5 - 7)}{\infty} = -0.008$$

3. Constant $X_2$ depending on $e.$ $v.$ and $S$

$$\mathrm{X}_2 = \frac{v + (e - v)}{1 + 2\mathrm{S}} = \frac{7 + (6.5 - 7)}{1 + 2(-0.008)} = 6.606$$

4. Maximum storage capacity $(F)$:

$$\mathrm{F} = 303 \times \mathrm{X}_2 = 303 \times (6.606) = 2001.618$$

5. Potential for accumulation $(t_p)$:

$$t_p = 1 + \frac{0.5}{1 + M} = 1 + \frac{0.5}{1 + 2.225} = 1.155$$

With:

$$\mathrm{M} = exp\left\{\left(\frac{IDC - 60}{10}\right)\right\} = exp\left\{\left(\frac{68 - 60}{10}\right)\right\} = 2.225$$

6. Correction factor $(f_c)$:

$$f_c = 0.21 \times t_p \times (1 + 0.2 \times X_2) = 0.21 \times 1.155 \times (1 + 0.2 \times 6.606) = 0.563$$

Checking if it is fulfilled that $\boldsymbol{f_c} \times \boldsymbol{Q_c} < \boldsymbol{F}$ for each of the entries:

- ✓ $f_c \times Q_c^A < F = 0.563 \times 310 < 2001.618 = 174.530 < 2001.618$
- ✓ $f_c \times Q_c^B < F = 0.563 \times 481 < 2001.618 = 270.803 < 2001.618$
- ✓ $f_c \times Q_c^C < F = 0.563 \times 494 < 2001.618 = 278.122 < 2001.618$

- ✓ $f_c \times Q_c^D < F = 0.563 \times 292 < 2001.618 = 164.396 < 2001.618$

Therefore, the capacity of each entry is determined as follows:

$$Q_e^A = K \times (F - f_c \times Q_c^A) = 1.02 \times (2001.618 - 174.530) = 1863.630\, pcu/h$$

$$Q_e^B = K \times (F - f_c \times Q_c^B) = 1.02 \times (2001.618 - 270.803) = 1765.431\, pcu/h$$

$$Q_e^C = K \times (F - f_c \times Q_c^C) = 1.02 \times (2001.618 - 278.122) = 1757.966\, pcu/h$$

$$Q_e^D = K \times (F - f_c \times Q_c^D) = 1.02 \times (2001.618 - 164.396) = 1870.906\, pcu/h$$

- **SETRA**

The factor $Q_u^*$ dependent on the exiting traffic flow and the splitter island width and is calculated as follow:

$$\mathbf{Q_{uA}^*} = Q_u^A \times \frac{15 - \mathrm{SEP}}{15} = 415 \times \frac{15 - 13}{15} = 55.33$$

$$\mathbf{Q_{uB}^*} = Q_u^B \times \frac{15 - \mathrm{SEP}}{15} = 485 \times \frac{15 - 13}{15} = 64.66$$

$$\mathbf{Q_{uC}^*} = Q_u^C \times \frac{15 - \mathrm{SEP}}{15} = 422 \times \frac{15 - 13}{15} = 56.26$$

$$\mathbf{Q_{uD}^*} = Q_u^D \times \frac{15 - \mathrm{SEP}}{15} = 340 \times \frac{15 - 13}{15} = 45.33$$

The impeding traffic $Q_g$ for each entry of the roundabout, the results are given below:

$$Q_g^A = \left(Q_c^A + \frac{2}{3} \times Q_{uA}^*\right) \times [1 - 0.085 \times (ANN - 8)]$$

$$Q_g^A = \left(310 + \frac{2}{3} \times 55.33\right) \times [1 - 0.085 \times (7 - 8)] = 376.372\, pcu/h$$

$$Q_g^B = \left(Q_c^B + \frac{2}{3} \times \ Q_{uB}^*\right) \times [\ 1 - 0.085 \times (ANN - 8)]$$

$$Q_g^B = \left(481 + \frac{2}{3} \times 64.66\right) \times [\ 1 - 0.085 \times (7 - 8)] = 568.656\ pcu/h$$

$$Q_g^C = \left(Q_c^C + \frac{2}{3} \times \ Q_{uC}^*\right) \times [\ 1 - 0.085 \times (ANN - 8)]$$

$$Q_g^C = \left(494 + \frac{2}{3} \times 56.26\ \right) \times [\ 1 - 0.085 \times (7 - 8)] = 576.685\ pcu/h$$

$$Q_g^D = \left(Q_c^D + \frac{2}{3} \times \ \mathrm{Q}_{\mathrm{uD}}^*\right) \times [\ 1 - 0.085 \times (\mathrm{ANN} - 8)]$$

$$Q_g^D = \left(\ 292 + \frac{2}{3} \times 45.33\right) \times [\ 1 - 0.085 \times (7 - 8)] = 349.609\ pcu/h$$

Finally, the capacity of each entry ($\boldsymbol{C_e}$) expressed in *pcu/h* through the SETRA model is calculated as follows:

Entry A:

$$C_e^A = \left(1330 - 0.7 \times \ Q_g^A\right) \times [\ 1 + 0.1 \times (ENT - 3.5)]$$

$$C_e^A = (1330 - 0.7 \times 376.372\ ) \times [1 + 0.1 \times (6.5 - 3.5)] = 1386.505\ pcu/h$$

Entry B:

$$C_e^B = \left(1330 - 0.7 \times \ Q_g^B\right) \times [\ 1 + 0.1 \times (ENT - 3.5)]$$

$$C_e^B = (1330 - 0.7 \times \ 568.656) \times [\ 1 + 0.1 \times (6.5\ - 3.5)] = 1211.523\ pcu/h$$

Entry C:

$$C_e^C = \left(1330 - 0.7 \times \ Q_g^B\right) \times [\ 1 + 0.1 \times (ENT - 3.5)]$$

$$C_e^C = (1330 - 0.7 \times \ 576.685\ ) \times [\ 1 + 0.1 \times (6.5\ - 3.5)] = 1204.217\ pcu/h$$

Entry D:

$$C_e^D = \left(1330 - 0.7 \times\ Q_g^D\right) \times [\,1 + 0.1 \times (\mathrm{ENT} - 3.5)]$$
$$C_e^D = (1330 - 0.7 \times\ 349.609) \times [\,1 + 0.1 \times (6.5\ - 3.5)] = 1410.856\,pcu/h$$

- **CERTU**

The factor α is calculated taking into consideration the dimensions of the geometric parameters ANN and IDC, from the table 9 is estimated a value equal to 0.7 for α . The calculation of the impeding traffic flow of each entry is given by:

$$Q_g^A =\ 0.7 \times Q_c^A + 0.2\ \times\ Q_u^A$$
$$Q_g^A =\ 0.7\ \times\ 310 + 0.2\ \times\ 415 = 300\,pcu/h$$

$$Q_g^B =\ 0.7 \times Q_c^B + 0.2\ \times\ Q_u^B$$
$$Q_g^B =\ 0.7\ \times 481 + 0.2\ \times\ 485 = 643.7\,pcu/h$$

$$Q_g^C =\ 0.7 \times Q_c^C + 0.2\ \times\ Q_u^C$$
$$Q_g^C = 0.7\ \times 494 + 0.2\ \times\ 422 = 430.2\,pcu/h$$

$$Q_g^D =\ 0.7 \times Q_c^D + 0.2\ \times\ Q_u^D$$
$$Q_g^D = 0.7\ \times 292 + 0.2\ \times\ 340 = 272.4\,pcu/h$$

For the entry capacity calculation is considered a $\gamma = 1.5$ as the scenario corresponds to a multilane roundabout. Thus, the entry capacity using the CERTU model expressed in *pcu/h* are given as follows:

Entry A

$$C_e^A = 1.5 \times (1500 - 0.83 \times Q_g^A)$$
$$C_e^A = 1.5 \times (1500 - 0.83 \times 300\,) = 1876.50\,pcu/h$$

Entry B

$$C_e^B = 1.5 \times (1500 - 0.83 \times Q_g^B)$$
$$C_e^B = 1.5 \times (1500 - 0.83 \times 643.7) = 1448.59\, pcu/h$$

Entry C

$$C_e^C = 1.5 \times (1500 - 0.83 \times Q_g^C)$$
$$C_e^C = 1.5 \times (1500 - 0.83 \times 430.2\ ) = 1714.04\, pcu/h$$

*Entry D*

$$C_e^D = 1.5 \times (1500 - 0.83 \times Q_g^D)$$
$$C_e^D = 1.5 \times (1500 - 0.83 \times 272.4) = 1910.86\, pcu/h$$

- **FCTUC**

For capacity estimation at each of the entries, the Portuguese model evaluates the same parameters as the TRL model. The results are shown as follows:

$$\mathrm{K} = 1 - 0.00163(\emptyset - 30) - 3.431\ \times \left\{\frac{1}{\mathrm{r}} - \ 0.05\right\}$$

$$\mathrm{K} = 1 - 0.00163\ (30 - 30) - 3.431 \times \left\{\frac{1}{40} - \ 0.05\right\} = 1.09$$

$$S = 1.6 \times \frac{(e - v)}{l'} = 1.6 \times \frac{(6.5 - 7)}{\infty} = -0.008$$

$$\mathrm{X}_2 = \frac{v + (e - v)}{1 + 2S} = \frac{7 + (6.5 - 7)}{1 + 2(-0.008)} = 6.606$$

$$\mathrm{F} = \ 335.47 \times \mathrm{X}_2 = 335.47 \times (6.606)\ =\ 2216.114$$

$$t_p = 1\ + \frac{0.983}{1 + M} = 1\ + \frac{0.983}{1 + 0.223}\ = 1.804$$

With:

$$\mathrm{M} = exp\left\{\left(\frac{IDC-60}{10}\right)\right\} = exp\left\{\left(\frac{45-60}{10}\right)\right\} = 0.223$$

$$f_c = 0.611 \times t_p \times (-0.457 + 0.2 \times X_2) = 0.611 \times 1.804 \times (-0.457 + 0.2 \times 6.606)$$
$$= 0.952$$

Checking if it is fulfilled that $\boldsymbol{f_c} \times \boldsymbol{Q_c} < \boldsymbol{F}$ for each of the entries:

- ✓ $f_c \times Q_c^A < F = 0.952 \times 310 < 2216.114 = 295.12 < 2216.114$
- ✓ $f_c \times Q_c^B < F = 0.952 \times 481 < 2216.114 = 457.912 < 2216.114$
- ✓ $f_c \times Q_c^C < F = 0.952 \times 494 < 2216.114 = 470.288 < 2216.114$
- ✓ $f_c \times Q_c^D < F = 0.952 \times 292 < 2216.114 = 277.984 < 2216.114$

Therefore, the capacity of each entry is determined as follows:

$$Q_e^A = K \times (F - f_c \times Q_c^A) = 1.09 \times (2216.114 - 295.12) = 2093.883 \ pcu/h$$

$$Q_e^B = K \times (F - f_c \times Q_c^B) = 1.09 \times (2216.114 - 457.912) = 1916.440 \ pcu/h$$

$$Q_e^C = K \times (F - f_c \times Q_c^C) = 1.09 \times (2216.114 - 470.288) = 1902.950 \ pcu/h$$

$$Q_e^D = K \times (F - f_c \times Q_c^D) = 1.09 \times (2216.114 - 277.984) = 2112.562 \ pcu/h$$

- **German models**

From Table 10 the regression parameters A and B, 1553 and 6.69 respectively. Thus, the capacity is using the exponential German equation 26 for each of entry is computed as follows:

$$\mathrm{Q_e} = \mathrm{A} * \mathrm{e}^{\frac{-\mathrm{B}*\mathrm{Q_c}}{10000}}$$

Entry A

$$Q_e^A = A * e^{\frac{-B * Q_c^A}{10000}} = 1553 * e^{\frac{-6.69*310}{10000}} = 1262.127\ pcu/h$$

Entry B

$$Q_e^B = A * e^{\frac{-B * Q_c^B}{10000}} = 1553 * e^{\frac{-6.69*481}{10000}} = 1125.694\ pcu/h$$

Entry C

$$Q_e^C = A * e^{\frac{-B * Q_c^C}{10000}} = 1553 * e^{\frac{-6.69*494}{10000}} = 1115.946\ pcu/h$$

Entry D

$$Q_e^D = A * e^{\frac{-B * Q_c^D}{10000}} = 1553 * e^{\frac{-6.69*292}{10000}} = 1277.417\ pcu/h$$

For the calculation of entry capacity using the linear, the empirical parameters C and D are 1380 and 0.50 respectively, these two empirical parameters depend on the number of lanes and on the conflicting roadway.

$$Q_e = C + D * Q_c$$

Finally, the results of the capacity calculation for each entry expressed in pcu/h are given below.

Entry A

$$Q_e^A = C + D * Q_c^A = 1380 + 0.50 * 310 = 1535\ pcu/h$$

Entry B

$$Q_e^B = C + D * Q_c^B = 1380 + 0.50 * 481 = 1621\ pcu/h$$

Entry C

$$Q_e^C = C + D * Q_c^C = 1380 + 0.50 * 494 = 1627\ pcu/h$$

Entry D

$$Q_e^D = C + D * Q_c^D = 1380 + 0.50 * 292 = 1526\ pcu/h$$

- **Colombian**

Since the geometric features of the entries are the same, the capacity of all the entries is equal. Therefore, entry capacity for all the entries is computed as follows:

$$\boldsymbol{e} = \frac{7 + 6.5}{2} = 6.75\ m$$

$$\boldsymbol{Q_p} = \frac{160\ (7)\ \left(\frac{1 + 6.75}{7}\right)}{1 + \frac{7}{40}} = 1055.32\ pcu/h$$

**Analytical models**

- **HCM 2010**

The capacity analysis using the HCM is carried out for each of the entry lanes independently. The capacity is then calculated for each of the lanes (left and right); the analysis is made as follows:

The following values are obtained for parameters A and B for left entry lanes:

$$A = \frac{3600}{t_f} = \frac{3600}{4.29} = 839.161$$

$$B = \frac{t_c - \frac{t_f}{2}}{3600} = \frac{3.19 - \frac{4.29}{2}}{3600} = 0.00029$$

The capacity of left entry lane at each entry in *pcu/h* is obtained for each of the entries from equation 37:

$$C = A * e^{-B * v_c}$$

$$C_A = A * e^{-B*v_{cA}} = 839.161 * e^{-0.00029*169} = 799.052\, pcu/h$$

$$C_B = A * e^{-B*v_{cB}} = 839.161 * e^{-0.00029*53} = 826.361\, pcu/h$$

$$C_C = A * e^{-B*v_{cC}} = 839.161 * e^{-0.00029*256} = 779.118\, pcu/h$$

$$C_D = A * e^{-B*v_{cD}} = 839.161 * e^{-0.00029*347} = 758.826\, pcu/h$$

Parameters for calculating the capacity of right entry lanes $\boldsymbol{t_f} = 4.11$ and $\boldsymbol{t_c} = 3.19$:

$$A = \frac{3600}{t_f} = \frac{3600}{4.11} = 875.912$$

$$B = \frac{t_c - \frac{t_f}{2}}{3600} = \frac{3.19 - \frac{4.11}{2}}{3600} = 0.00315$$

The capacity of left entry lanes at each of the entries from the following formula:

$$C = A * e^{-B*v_c}$$

$$C_A = A * e^{-B*v_{cA}} = 875.912 * e^{-0.00315*153} = 540.944\, pcu/h$$

$$C_B = A * e^{-B*v_{cB}} = 875.912 * e^{-0.00315*191} = 479.918\, pcu/h$$

$$C_C = A * e^{-B*v_{cC}} = 875.912 * e^{-0.00315*216} = 443.574\, pcu/h$$

$$C_D = A * e^{-B*v_{cD}} = 875.912 * e^{-0.00315*277} = 366.030\, pcu/h$$

Therefore, the entry capacity for each entry are as follows:

$$C_A = 799.052 + 540.944 = 1339.996\, pcu/h$$

$$C_B = 826.361 + 479.918 = 1306.279\, pcu/h$$

$$C_C = 779.118 + 443.574 = 1222.692\, pcu/h$$

$$C_D = 758.826 + 366.030 = 1124.856\, pcu/h$$

- **Australian**

The entry capacity estimation through Australian models is calculated as follows:

$$Q_e = \frac{3600 \times q_c(1 - q_c \times \Delta) \times e^{-(q_c(T - \Delta))}}{1 - e^{(-q_c \times T_0)}}$$

$$Q_e^A = \frac{3600 \times 0.0861(1 - 0.0861 \times 1) \times e^{(-0.0861(2.5 - 1))}}{1 - e^{(-0.0861 \times 2.1)}} = 1505.094\, pcu/h$$

$$Q_e^B = \frac{3600 \times 0.1336(1 - 0.1336 \times 1) \times e^{(-0.1336(2.5 - 1))}}{1 - e^{(-0.1336 \times 2.1)}} = 1394.018\, pcu/h$$

$$Q_e^C = \frac{3600 \times 0.1372(1 - 0.1372 \times 1) \times e^{(-0.1372(2.5 - 1))}}{1 - e^{(-0.1372 \times 2.1)}} = 1385.730\, pcu/h$$

$$Q_e^D = \frac{3600 \times 0.081(1 - 0.081 \times 1) \times e^{(-0.081(2.5 - 1))}}{1 - e^{(-0.081 \times 2.1)}} = 1517.208\, pcu/h$$

- **German model**

The entry capacity through the German model is determined as follows:

$$G = 3600 \times \left(1 - \frac{t_{min} \times q_k}{n_k \times 3600}\right)^{n_k} \times \frac{n_z}{t_f} \times e^{-\frac{q_k}{3600} \times \left(t_g - \frac{t_f}{2} - t_{min}\right)}$$

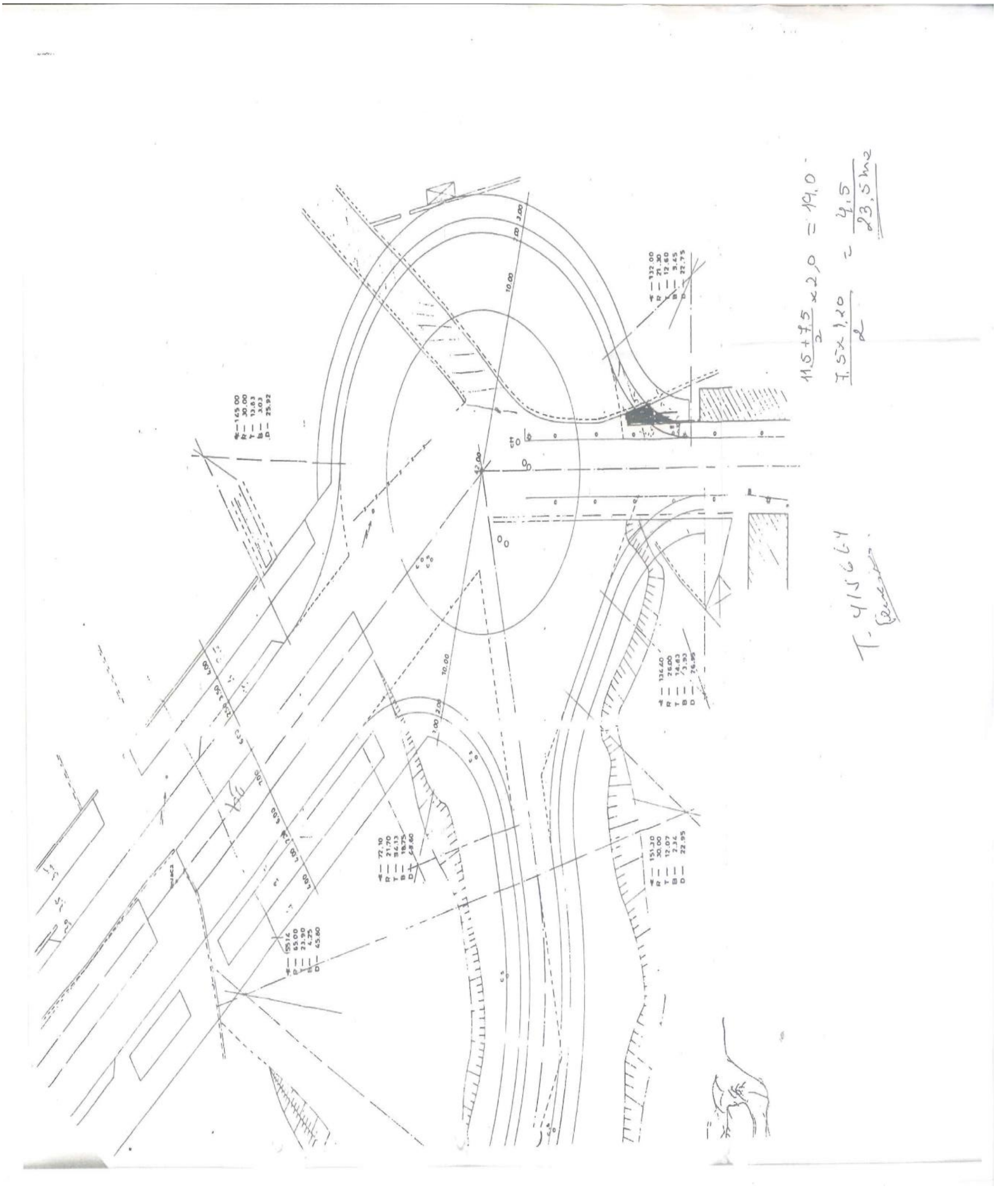
$$G_A = 3600 \times \left(1 - \frac{2.10 \times 310}{2 \times 3600}\right)^2 \times \frac{2}{2.88} \times e^{-\frac{310}{3600} \times \left(4.12 - \frac{2.88}{2} - 2.10\right)} = 1967.589\, pcu/h$$

$$G_B = 3600 \times \left(1 - \frac{2.10 \times 481}{2 \times 3600}\right)^2 \times \frac{2}{2.88} \times e^{-\frac{481}{3600} \times \left(4.12 - \frac{2.88}{2} - 2.10\right)} = 1709.964\ pcu/h$$

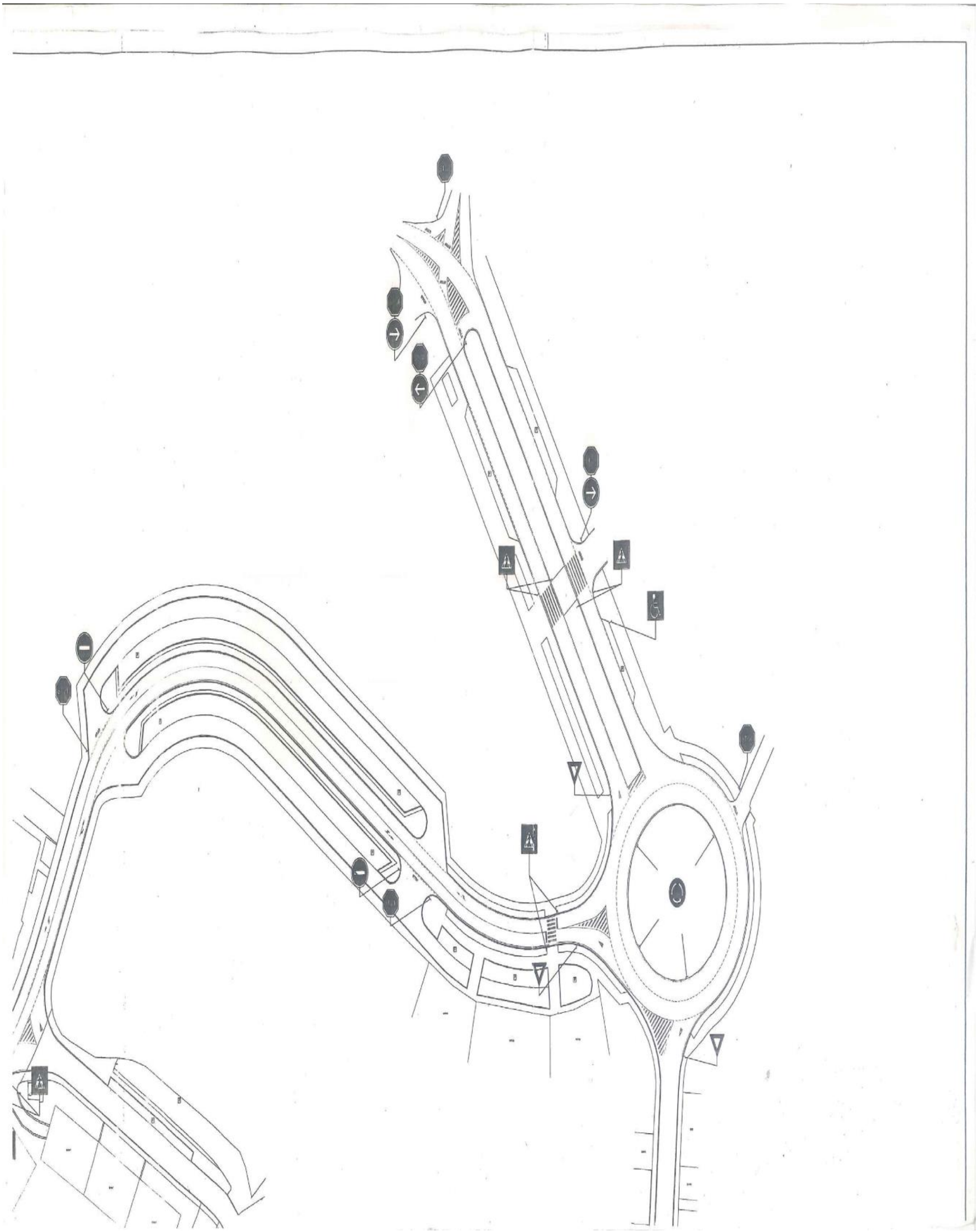
$$G_C = 3600 \times \left(1 - \frac{2.10 \times 494}{2 \times 3600}\right)^2 \times \frac{2}{2.88} \times e^{-\frac{494}{3600} \times \left(4.12 - \frac{2.88}{2} - 2.10\right)} = 1691.367\ pcu/h$$

$$G_D = 3600 \times \left(1 - \frac{2.10 \times 292}{2 \times 3600}\right)^2 \times \frac{2}{2.88} \times e^{-\frac{292}{3600} \times \left(4.12 - \frac{2.88}{2} - 2.10\right)} = 1996.148\ pcu/h$$

### 5.2.2 **Appendix B:** Layout of the roundabout



### 5.2.3 **Appendix C:** Layout of the roundabout signalization project “Minicircular a Quinta”



### 5.2.4 **Appendix D:** Travel times

| No. Vehicles | Travel time |
|---|---|
| 1 | 4,8 |
| 2 | 4,52 |
| 3 | 4,23 |
| 4 | 5,71 |
| 5 | 4,62 |
| 6 | 4,57 |
| 7 | 4,92 |
| 8 | 4,67 |
| 9 | 5,33 |
| 10 | 5,91 |
| 11 | 5,74 |
| 12 | 4,59 |
| 13 | 4,40 |
| 14 | 4,11 |
| 15 | 5,53 |
| 16 | 4,8 |
| 17 | 4,31 |
| 18 | 3,87 |
| 19 | 4,22 |
| 20 | 4,77 |
| 21 | 4,5 |
| 22 | 4,73 |
| 23 | 4,82 |
| 24 | 4,51 |
| 25 | 4,2 |
| 26 | 4,61 |
| 27 | 3,56 |
| 28 | 4,6 |
| 29 | 4,97 |
| 30 | 4,53 |
| 31 | 4,8 |
| 32 | 4,55 |
| 33 | 4,41 |
| 34 | 4,69 |
| 35 | 4,4 |
| 36 | 4,70 |
| 37 | 4,46 |
| 38 | 4,10 |
| 39 | 4,78 |
| 40 | 4,29 |
| 41 | 4,43 |
| 42 | 4,20 |
| 43 | 5,81 |
| 44 | 5,60 |
| 45 | 5,32 |
| 46 | 4,31 |
| 47 | 5,35 |
| 48 | 5,66 |
| 49 | 5,76 |
| 50 | 4,22 |
| 51 | 5,11 |
| 52 | 5,84 |
| 53 | 4,16 |
| 54 | 4,39 |
| 55 | 4,24 |
| 56 | 4,25 |
| 57 | 4,40 |
| 58 | 3,78 |
| 59 | 4,30 |
| 60 | 4,60 |

## 5.2.5 **Appendix E:** Entry capacity estimation

- TRL

**TRL**

**TRL - Morning**

| ENTRY | ϕ | r | IDC | e | l' | v | Qc | K | S | X2 | F | M | tp | fc | fc*Qc | fc*Qc < F | Capacity<br>Qe |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| University Avenue | 53 | 22 | | 7 | 100000 | 5,5 | 238 | 0,92 | 0,00 | 7,00 | 2121 | | | 0,65 | 155 | Ok | 1818 |
| Conego Dr. Manuel Faria | 28 | 30 | 57 | 6,5 | 12 | 3,5 | 259 | 1,02 | 0,40 | 3,61 | 1094 | 0,74 | 1,29 | 0,47 | 120 | Ok | 996 |
| Cap Alfredo Guimaraes | 24 | 22 | | 5 | 100000 | 5,2 | 504 | 1,03 | 0,00 | 5,00 | 1515 | | | 0,54 | 273 | Ok | 1274 |

**TRL - Afternoon**

| ENTRY | ϕ | r | IDC | e | l' | v | Qc | K | S | X2 | F | M | tp | fc | fc*Qc | fc*Qc < F | Capacity<br>Qe |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| University Avenue | 53 | 22 | | 7 | 100000 | 5,5 | 293 | 0,92 | 0,00 | 7,00 | 2121 | | | 0,65 | 190 | Ok | 1785 |
| Conego Dr. Manuel Faria | 28 | 30 | 57 | 6,5 | 12 | 3,5 | 79 | 1,02 | 0,41 | 3,56 | 1080 | 0,74 | 1,29 | 0,46 | 36 | Ok | 1068 |
| Cap Alfredo Guimaraes | 24 | 22 | | 5 | 100000 | 5,2 | 302 | 1,03 | 0,00 | 5,00 | 1515 | | | 0,54 | 163 | Ok | 1386 |

- SETRA

**SETRA**

**SETRA - Morning**

| ENTRY | Geometric parameters | | | | Traffic flow parameters | | | Capacity |
|---|---|---|---|---|---|---|---|---|
| | ENT | SEP | ANN | Qc | Qu | Qu* | Qg | Ce |
| University Avenue | 7 | 9 | | 238 | 868 | 347 | 442 | 1378 |
| Conego Dr. Manuel Faria | 6,5 | 8 | 9,5 | 259 | 903 | 421 | 518 | 1257 |
| Cap Alfredo Guimaraes | 5 | 0 | | 504 | 0 | 0 | 293 | 1293 |

**SETRA - Afternoon**

| ENTRY | Geometric parameters | | | Traffic flow parameters | | | | Capacity |
|---|---|---|---|---|---|---|---|---|
| | ENT | SEP | ANN | Qc | Qu | Qu* | Qg | Ce |
| University Avenue | 7 | 9 | | 293 | 674 | 270 | 406 | 1412 |
| Conego Dr. Manuel Faria | 6,5 | 8 | 9,5 | 79 | 752 | 351 | 352 | 1409 |
| Cap Alfredo Guimaraes | 5 | 0 | | 302 | 0 | 0 | 176 | 1388 |

- CERTU

| CERTU | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| **CETUR - Morning** | | | | | | | | | |
| **ENTRY** | **Geometric parameters** | | | | | **Traffic flow parameters** | | | **Capacity** |
| | **SEP** | **ANN** | **IDC** | **y** | **α** | **Qc** | **Qu** | **Qg** | **Qe** |
| University Avenue | 9 | | | 1,5 | | 238 | 868 | 340 | 1826 |
| Conego Dr. Manuel Faria | 8 | 9,5 | 57 | 1 | 0,7 | 259 | 903 | 362 | 1200 |
| Cap Alfredo Guimaraes | 0 | | | 1 | | 504 | 0 | 353 | 1207 |

| CERTU - Afternoon | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| **ENTRY** | **Geometric parameters** | | | | | **Traffic flow parameters** | | | **Capacity** |
| | **SEP** | **ANN** | **IDC** | **y** | **a** | **Qc** | **Qu** | **Qg** | **Qe** |
| University Avenue | 9 | 9,5 | 57 | 1,5 | | 293 | 674 | 340 | 1827 |
| Conego Dr. Manuel Faria | 8,2 | 9,5 | 57 | 1 | 0,7 | 79 | 752 | 205 | 1329 |
| Cap Alfredo Guimaraes | 0 | 9,5 | 57 | 1 | | 302 | 0 | 212 | 1324 |

- FCTUC

| FCTUC | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **FCTUC - Morning** | | | | | | | | | | | | | | | | |
| **ENTRY** | | | | | | | | | | | | | | | | **Capacity** |
| | **ϕ** | **r** | **IDC** | **e** | **l'** | **v** | **Qc** | **K** | **S** | **X2** | **F** | **M** | **tp** | **fc** | **fc*Qc** | **fc*Qc < F** | **Qe** |
| University Avenue | 53 | 22 | | 7 | 100000 | 5,5 | 238 | 0,98 | 0,00 | 7,00 | 2348 | | | 0,90 | 215 | Ok | 2087 |
| Conego Dr. Manuel Faria | 28 | 30 | 57 | 6,5 | 12 | 3,5 | 259 | 1,06 | 0,40 | 3,61 | 1211 | 0,74 | 1,56 | 0,25 | 66 | Ok | 1215 |
| Cap Alfredo Guimaraes | 24 | 22 | | 5 | 100000 | 5,2 | 504 | 1,03 | 0,00 | 5,00 | 1677 | | | 0,52 | 262 | Ok | 1452 |

| FCTUC - Afternoon | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **ENTRY** | | | | | | | | | | | | | | | | | **Capacity** |
| | **ϕ** | **r** | **IDC** | **e** | **l'** | **v** | **Qc** | **K** | **S** | **X2** | **F** | **M** | **tp** | **fc** | **fc*Qc** | **fc*Qc < F** | **Qe** |
| University Avenue | 53 | 22 | | 7 | 10000 | 5,5 | 293 | 0,98 | 0,00 | 7,00 | 2347 | | | 0,90 | 264 | Ok | 2037 |
| Conego Dr. Manuel Faria | 28 | 30 | 57 | 6,5 | 12 | 3,5 | 79 | 1,06 | 0,41 | 3,56 | 1195 | 0,74 | 1,56 | 0,24 | 19 | Ok | 1247 |
| Cap Alfredo Guimaraes | 24 | 22 | | 5 | 10000 | 5,2 | 302 | 1,03 | 0,00 | 5,00 | 1677 | | | 0,52 | 157 | Ok | 1559 |

- German analytical models

| EXPONENTIAL MODEL | | | | |
|---|---|---|---|---|

| German exponential model - Morning | | | | |
|---|---|---|---|---|
| ENTRY | | | | Capacity |
| | A | B | Qc | Qe |
| University Avenue | 1553 | 6,69 | 238 | 1324 |
| Conego Dr. Manuel Faria | *No allowed* | | | |
| Cap Alfredo Guimaraes | *No allowed* | | | |

| German exponetial model - Afternoon | | | | |
|---|---|---|---|---|
| ENTRY | | | | Capacity |
| | A | B | Qc | Qe |
| University Avenue | 1553 | 6,69 | 293 | 1276 |
| Conego Dr. Manuel Faria | *No allowed* | | | |
| Cap Alfredo Guimaraes | *No allowed* | | | |

| LINEAR MODEL | | | | |
|---|---|---|---|---|

| German linear model - Morning | | | | |
|---|---|---|---|---|
| ENTRY | | | | Capacity |
| | C | D | Qc | Qe |
| University Avenue | 1380 | 0,50 | 238 | 1499 |
| Conego Dr. Manuel Faria | 1250 | 0,53 | 259 | 1387 |
| Cap Alfredo Guimaraes | | | 504 | 1517 |

| German linear model - Afternoon | | | | |
|---|---|---|---|---|
| ENTRY | | | | Capacity |
| | C | D | Qc | Qe |
| University Avenue | 1380 | 0,50 | 293 | 1527 |
| Conego Dr. Manuel Faria | 1250 | 0,53 | 79 | 1292 |
| Cap Alfredo Guimaraes | | | 302 | 1410 |

- Colombian model

| Colombian Model - Morning | | | | | | |
|---|---|---|---|---|---|---|
| ENTRY | | | | | | Capacity |
| | W | L | e1 | e2 | e | Qp |
| University Avenue | 9,5 | 24 | 7 | 9,5 | 8,3 | 1480 |
| Conego Dr. Manuel Faria | | 33 | 6,5 | | 8,0 | 1440 |
| Cap Alfredo Guimaraes | | 55 | 5 | | 7,3 | 1320 |

| Colombian Model - Afternoon | | | | | | |
|---|---|---|---|---|---|---|
| ENTRY | | | | | | Capacity |
| | W | L | e1 | e2 | e | Qp |
| University Avenue | 9,5 | 24 | 7,0 | 9,5 | 8,3 | 1480 |
| Conego Dr. Manuel Faria | | 33 | 6,5 | 9,5 | 8,0 | 1440 |
| Cap Alfredo Guimaraes | | 55 | 5,0 | 9,5 | 7,3 | 1320 |

- Dutch

| Dutch Model - Morning | | | |
|---|---|---|---|
| ENTRY | | | Capacity |
| | B | Cexit | Aentry |
| University Avenue | *No allowed* | | |
| Conego Dr. Manuel Faria | 259 | 903 | 970 |
| Cap Alfredo Guimaraes | 504 | 0 | 996 |

| Dutch Model - Afternoon | | | |
|---|---|---|---|
| ENTRY | | | Capacity |
| | B | Cexit | Aentry |
| University Avenue | *No allowed* | | |
| Conego Dr. Manuel Faria | 79 | 674 | 1219 |
| Cap Alfredo Guimaraes | 302 | 752 | 972 |

- HCM2010

**HCM 2010**

**HCM 2010 - Morning (USA conditions)**

| ENTRY | | | | | left-hand turn | | right-hand turn | | | | Entry capacity |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | tf left turn | tf right turn | tc | Qc | A | B | A | B | Left-hand turn capacity | Right-hand turn capacity | Qe |
| University Avenue | 4,29 | 4,11 | 3,19 | 238 | 839 | 0,000290278 | 876 | 0,000315278 | 783 | 813 | 1596 |
| Conego Dr. Manuel Faria | - | | | 259 | - | | 876 | 0,000886111 | - | 804 | 804 |
| Cap Alfredo Guimaraes | | | | 504 | | | 876 | 0,000315278 | | 737 | 737 |

**HCM 2010 - Morning (Portugal conditions)**

| ENTRY | | | | | left-hand turn | | right-hand turn | | | | Entry capacity |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | tf left turn | tf right turn | tc | Qc | A | B | A | B | Left-hand turn capacity | Right-hand turn capacity | Qe |
| University Avenue | 2,5 | 2,50 | 2,80 | 238 | 1440 | 0,000431 | 1440 | 0,000430556 | 1300 | 1300 | 2599 |
| Conego Dr. Manuel Faria | - | | | 259 | - | | 1440 | 0,000777778 | - | 1286 | 1286 |
| Cap Alfredo Guimaraes | | | | 504 | | | 1440 | 0,000430556 | | 1153 | 1153 |

**HCM 2010 - Afternoon (USA conditions)**

| ENTRY | | | | | left-hand turn | | right-hand turn | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | tf left turn | tf right turn | tc | Qc | A | B | A | B | Left-hand turn capacity | Right-hand turn capacity | Entry capacity |
| University Avenue | 4,29 | 4,11 | 3,19 | 293 | 839,2 | 0,000290278 | 876 | 0,000315278 | 771 | 799 | 1569 |
| Conego Dr. Manuel Faria | - | | | 79 | - | | 876 | 0,000886111 | - | 854 | 854 |
| Cap Alfredo Guimaraes | | | | 302 | | | 876 | 0,000315278 | | 792 | 792 |

**HCM 2010 - Afternoon (Portugal conditions)**

| ENTRY | | | | | left-hand turn | | right-hand turn | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | tf left turn | tf right turn | tc | Qc | A | B | A | B | Left-hand turn capacity | Right-hand turn capacity | Entry capacity |
| University Avenue | 2,50 | 2,50 | 2,80 | 293 | 1440 | 0,000430556 | 1440 | 0,000430556 | 1269 | 1269 | 2538 |
| Conego Dr. Manuel Faria | - | | | 79 | - | | 1440 | 0,000777778 | - | 1392 | 1392 |
| Cap Alfredo Guimaraes | | | | 302 | | | 1440 | 0,000430556 | | 1262 | 1262 |

- Australian model

| AUSTRALIAN | | | | | |
|---|---|---|---|---|---|
| | | | | | |
| **AUSTRALIAN - Morning (Australia conditions)** | | | | | |
| **ENTRY** | | | | **** | **Capacity** |
| | **T** | **To** | **Δ** | **Qc** | **Qe** |
| University Avenue | 2,5 | 2,1 | 1,0 | 0,07 | 1670 |
| Conego Dr. Manuel Faria | | | 2,0 | 0,07 | 1537 |
| Cap Alfredo Guimaraes | | | | 0,14 | 1347 |
| | | | | | |
| **AUSTRALIAN - Morning (Portugal conditions)** | | | | | |
| **ENTRY** | **Geometric parameters** | | | **Traffic flow parameters** | **Capacity** |
| | **T** | **To** | **Δ** | **Qc** | **Qe** |
| University Avenue | 2,5 | 2,8 | 1,0 | 0,07 | 1342 |
| Conego Dr. Manuel Faria | | | 2,0 | 0,07 | 1242 |
| Cap Alfredo Guimaraes | | | | 0,14 | 1167 |
| | | | | | |
| **AUSTRALIAN - Afternoon (Australia conditions)** | | | | | |
| **ENTRY** | **Geometric parameters** | | | **Traffic flow parameters** | **Capacity** |
| | **T** | **To** | **Δ** | **Qc** | **Qe** |
| University Avenue | 2,5 | 2,1 | 1,0 | 0,08 | 1658 |
| Conego Dr. Manuel Faria | | | 2,0 | 0,02 | 1663 |
| Cap Alfredo Guimaraes | | | | 0,08 | 1505 |
| | | | | | |
| **AUSTRALIAN - Afternoon (Portugal conditions)** | | | | | |
| **ENTRY** | **Geometric parameters** | | | **Traffic flow parameters** | **Capacity** |
| | **T** | **To** | **Δ** | **Qc** | **Qe** |
| University Avenue | 2,5 | 2,8 | 1,0 | 0,08 | 1353 |
| Conego Dr. Manuel Faria | | | 2,0 | 0,02 | 1276 |
| Cap Alfredo Guimaraes | | | | 0,08 | 1231 |

- German model

| GERMAN - Morning (Germany conditions) | | | | | | | |
|---|---|---|---|---|---|---|---|
| ENTRY | | | | | | | Capacity |
| | tmin | nk | nz | tf | tg | qk | Qc |
| University Avenue | 2,1 | 2,0 | 2 | 2,8 | 4,1 | 238 | 2140 |
| Conego Dr. Manuel Faria | | | 1 | | | 259 | 1053 |
| Cap Alfredo Guimaraes | | | 1 | | | 504 | 860 |

| GERMAN - Morning (Portugal conditions) | | | | | | | |
|---|---|---|---|---|---|---|---|
| ENTRY | | | | | | | Capacity |
| | tmin | nk | nz | tf | tg | qk | Qc |
| University Avenue | 2,1 | 2,00 | 2 | 2,5 | 2,8 | 238 | 2586 |
| Conego Dr. Manuel Faria | | | 1 | | | 259 | 1280 |
| Cap Alfredo Guimaraes | | | 1 | | | 504 | 1132 |

| GERMAN - Afternoon (Germany conditions) | | | | | | | |
|---|---|---|---|---|---|---|---|
| ENTRY | | | | | | | Capacity |
| | tmin | nk | nz | tf | tg | qk | Qc |
| **University Avenue** | 2,1 | 2,00 | 2 | 2,8 | 4,1 | 293 | 2048 |
| **Conego Dr. Manuel Faria** | | | 1 | | | 79 | 1211 |
| **Cap Alfredo Guimaraes** | | | 1 | | | 302 | 1017 |

| GERMAN - Afternoon (Portugal conditions) | | | | | | | |
|---|---|---|---|---|---|---|---|
| ENTRY | | | | | | | Capacity |
| | tmin | nk | nz | tf | tg | qk | Qc |
| **University Avenue** | 2,1 | 2,00 | 2 | 2,5 | 2,8 | 293 | 2519 |
| **Conego Dr. Manuel Faria** | | | 1 | | | 79 | 1391 |
| **Cap Alfredo Guimaraes** | | | 1 | | | 302 | 1254 |

## 5.2.6 **Appendix F:** Traffic flows

| | Movement 1 | | | | Movement 3 | | | | Movement 7 | | | | Movement 8 | | | | Movement 9 | | | | Movement 10 | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Periodo | MC | LIG | PES | BUS | MC | LIG | PES | BUS | MC | LIG | PES | BUS | MC | LIG | PES | BUS | MC | LIG | PES | BUS | MC | LIG | PES | BUS |
| 7:30 - 7:45 | 0 | 35 | 0 | 1 | 0 | 4 | 0 | 0 | 1 | 24 | 0 | 1 | 0 | 22 | 0 | 1 | 0 | 0 | 0 | 0 | 1 | 19 | 0 | 1 |
| 7:45 - 8:00 | 1 | 40 | 0 | 2 | 0 | 6 | 0 | 0 | 1 | 27 | 0 | 1 | 0 | 27 | 0 | 1 | 0 | 0 | 0 | 0 | 0 | 23 | 0 | 0 |
| 8:00 - 8:15 | 1 | 63 | 1 | 3 | 0 | 5 | 0 | 1 | 0 | 46 | 0 | 1 | 1 | 28 | 0 | 2 | 0 | 0 | 0 | 0 | 0 | 20 | 0 | 3 |
| 8:15 - 8:30 | 1 | 103 | 0 | 2 | 0 | 13 | 0 | 1 | 1 | 89 | 0 | 1 | 1 | 37 | 0 | 1 | 0 | 1 | 0 | 0 | 1 | 42 | 0 | 0 |
| 8:30 - 8:45 | 3 | 90 | 0 | 3 | 0 | 27 | 0 | 1 | 0 | 84 | 0 | 1 | 0 | 59 | 0 | 2 | 0 | 0 | 0 | 0 | 0 | 47 | 0 | 0 |
| 8:45 - 9:00 | 2 | 158 | 1 | 4 | 0 | 40 | 0 | 1 | 0 | 87 | 0 | 2 | 1 | 47 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 57 | 0 | 1 |
| 9:00 - 9:15 | 0 | 117 | 0 | 1 | 0 | 66 | 0 | 0 | 0 | 73 | 0 | 1 | 0 | 37 | 0 | 1 | 0 | 0 | 0 | 0 | 0 | 34 | 0 | 1 |
| 9:15 - 9:30 | 1 | 86 | 0 | 1 | 0 | 33 | 0 | 0 | 0 | 52 | 0 | 0 | 0 | 35 | 0 | 3 | 0 | 0 | 0 | 0 | 0 | 20 | 0 | 0 |
| | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | | |
| 16:30 - 16:45 | 1 | 45 | 0 | 4 | 0 | 14 | 0 | 1 | 0 | 80 | 0 | 3 | 1 | 41 | 0 | 0 | 0 | 1 | 0 | 0 | 0 | 37 | 0 | 1 |
| 16:45 - 17:00 | 1 | 79 | 0 | 0 | 0 | 12 | 0 | 0 | 1 | 75 | 1 | 5 | 0 | 42 | 0 | 1 | 0 | 0 | 0 | 0 | 0 | 34 | 0 | 0 |
| 17:00 - 17:15 | 0 | 85 | 0 | 0 | 0 | 19 | 0 | 0 | 0 | 83 | 0 | 2 | 0 | 55 | 0 | 2 | 0 | 0 | 0 | 0 | 0 | 32 | 0 | 2 |
| 17:15 - 17:30 | 0 | 92 | 0 | 1 | 0 | 18 | 0 | 1 | 1 | 81 | 0 | 1 | 0 | 51 | 0 | 1 | 0 | 0 | 0 | 0 | 1 | 46 | 0 | 0 |
| 17:30 - 17:45 | 1 | 106 | 0 | 2 | 0 | 13 | 0 | 0 | 0 | 92 | 1 | 0 | 1 | 56 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 59 | 0 | 1 |
| 17:45 - 18:00 | 1 | 110 | 0 | 3 | 0 | 15 | 0 | 0 | 0 | 86 | 0 | 1 | 1 | 52 | 0 | 1 | 0 | 0 | 0 | 0 | 0 | 72 | 0 | 0 |
| 18:00 - 18:15 | 2 | 95 | 0 | 2 | 0 | 9 | 0 | 0 | 1 | 77 | 0 | 0 | 2 | 46 | 0 | 2 | 0 | 0 | 0 | 0 | 1 | 69 | 1 | 1 |
| 18:15 - 18:30 | 1 | 90 | 0 | 1 | 0 | 10 | 0 | 0 | 1 | 73 | 0 | 1 | 0 | 44 | 0 | 2 | 0 | 0 | 0 | 0 | 0 | 61 | 0 | 1 |
| *% Veh by mov* | 1% | 97% | 0% | 2% | 0% | 98% | 0% | 2% | 1% | 97% | 0% | 2% | 1% | 96% | 0% | 3% | 0% | 100% | 0% | 0% | 1% | 97% | 0% | 2% |
| Total veh | 4310 | | | | | | | | | | | | | | | | | | | | | | | |
| Morning veh | 1989 | | | | | | | | | | | | | | | | | | | | | | | |
| Evening Veh | 2321 | | | | | | | | | | | | | | | | | | | | | | | |