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TRABAJO FIN DE MÁSTER

SIMULATION-BASED OPTIMIZATION OF TRAFFIC ON A ROUNDABOUT. THEORETICAL STUDY AND SIMULATION OF THE CURRENT SITUATION OF VIESQUES ROUNDABOUT (GIJÓN, SPAIN).

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RESUMEN

1. Introducción

En este Trabajo Fin de Máster (TFM de aquí en adelante) se lleva a cabo un extenso estudio de la literatura acerca de las rotondas en el cual se tratan los siguientes puntos: las rotondas modernas, seguridad de las rotondas, capacidad, análisis del comportamiento de una rotonda y control de rotondas mediante semáforos.

A continuación, se dedica un capítulo al tema de la simulación del tráfico, el cual se centra en los distintos tipos de modelos de simulación del tráfico existentes (simulación macroscópica, mesoscópica y microscópica) y en el software escogido para la simulación llevada a cabo en este TFM.

Además, este TFM incluye una parte práctica que consiste en la simulación de la rotonda de Viesques (Gijón, Asturias) mediante el software PTV Vissim. La rotonda objeto de estudio (cuya localización exacta es 43°31'29.0"N 5°38'21.6"W) se muestra en la figura 1.2 así como la nomenclatura empleada para denotar los distintos ramales de la misma. El proceso del modelado de la rotonda en dicho software es explicado en detalle y, posteriormente, se analizan los resultados obtenidos (tales como colas, tiempos de espera y capacidad de la rotonda).

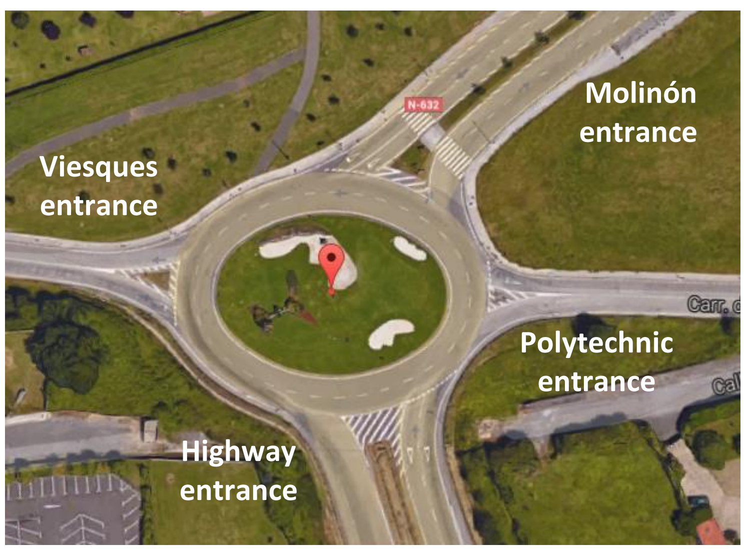


Figura 1.2. Rotonda objeto de estudio y nomenclatura

Cabe destacar que el presente TFM se complementa con el TFM titulado *“Simulation-based optimization of traffic on a roundabout. Improvements implementation by simulation to optimize Viesques roundabout capacity (Gijón, Spain)”.*

2. Estudio de la literatura

2. 1. Las rotondas modernas

Las rotondas tienen su origen más primario en el concepto de *tráfico giratorio*, el cual se puede decir que fue creado de forma paralela por William Phelps Eno (quién diseñó Columbus Circle en 1903 en Nueva York) y Eugene Henard (arquitecto de París y creador de intersecciones circulares como Place Charles de Gaulle en 1907). Dicho concepto consiste en un flujo de vehículos que circulan en una única dirección alrededor de un islote central.

En la década de 1930 nació en Estados Unidos un particular tipo de intersección circular llamado *rotary*, en la cual los vehículos entraban de forma tangencial y la prioridad la tenían los vehículos entrantes. Esto permitía altas velocidades de entrada y de salida que a menudo bloqueaban el tráfico en el interior de la rotonda. Tales problemas de bloqueo y los cada vez más frecuentes accidentes provocaron el declive de este tipo de intersección en Estados Unidos.

Las intersecciones giratorias siguieron proliferando, sobre todo en Inglaterra, pero las reglas que daban la prioridad a los vehículos no eran consistentes en los distintos países. Es en 1966 cuando en Inglaterra, con el propósito de solucionar los problemas de bloqueo y reducir el número de accidentes, se establece como obligatoria una regla por la cual todo vehículo entrante a una intersección giratoria debe ceder el paso a los vehículos que circulan por el interior de la misma, quienes tienen prioridad. En la década de 1970 las rotondas comienzan a hacerse populares en Francia, pero se tiene que esperar hasta 1984 para que esa misma regla sea obligatoria en todas las rotondas francesas.

Actualmente, las rotondas son consideradas como una herramienta muy útil para controlar el tráfico y su uso está normalizado en muchos países europeos.

Las rotondas modernas se caracterizan por tres requisitos fundamentales:

- Los vehículos que entran a la rotonda deben ceder el paso obligatoriamente a los vehículos que circulan por el interior de la misma.
- Los vehículos entran en la rotonda con un ángulo de 90 grados y son las isletas deflectoras las que guían la trayectoria de los vehículos hacía la derecha, reduciendo la velocidad de entrada de los mismos (a mayor deflexión menor velocidad de entrada).
- Los carriles de entrada a la rotonda sufren un abocinamiento, de tal forma que se aumenta la capacidad de la misma y se permite que los vehículos entren a una velocidad similar a la de los vehículos circulantes.

Por último, la Figura 2.8 muestra los elementos geométricos de las rotondas modernas, los cuales se explican en detalle en el presente TFM.

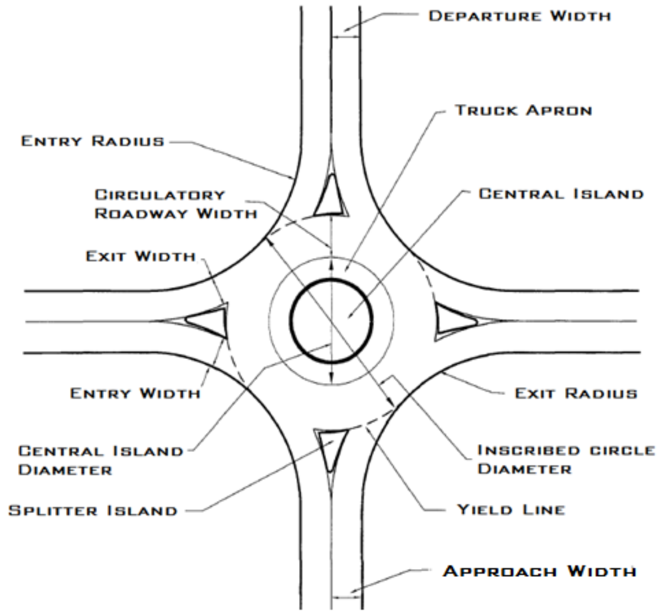


Figura 2.8. Elementos de una rotonda moderna

2. 2. Seguridad de las rotondas

Una de las ventajas de las rotondas y la principal razón de su proliferación en las últimas décadas es la seguridad de las mismas en comparación con otro tipo de intersecciones convencionales. Desde el nacimiento de las mismas se han llevado a cabo numerosos estudios que ponen de manifiesto su seguridad. En el presente TFM se incluye una recopilación exhaustiva de los estudios de seguridad más relevantes llevados a cabo. Dichos estudios tienen un denominador común: muestran la reducción de accidentes producida en rotondas en comparación con otras intersecciones.

Las razones por las que las rotondas son más seguras que otro tipo de intersecciones (tales como intersecciones en T o en cruz) son fundamentalmente tres:

- El tráfico en una única dirección alrededor de un islote central es una de las razones de la seguridad de las rotondas. Esto es debido a que el cruce, la fusión o la divergencia de la trayectoria de dos vehículos crea un potencial punto de conflicto, el cual está asociado al riesgo de accidente. Por tanto, la circulación de un único sentido reduce el número de puntos de conflictos y, asimismo, el riesgo de accidentes.
- Las bajas velocidades en las rotondas (causadas por la deflexión a la entrada que sufren los vehículos) son la principal causa de la seguridad de las mismas. Estas bajas velocidades permiten que el conductor tenga más tiempo para reaccionar ante potenciales conflictos, beneficia a los vehículos lentos homogeneizando velocidades y reduce la probabilidad de accidentes y su severidad.

- Otro elemento que hace de las rotondas una intersección segura es la facilidad de operación de las mismas, debido a la circulación en una única dirección y el hecho de ceder el paso a la entrada.

Por otra parte, se ha demostrado que uno de los factores que más positivamente afecta a la seguridad de las rotondas es el comportamiento de los conductores. Y por ello es importante el reconocimiento de las rotondas por parte de los usuarios y la visibilidad en las mismas. Es necesario que los vehículos que se aproximan a una rotonda la reconozcan con suficiente antelación tanto durante el día como durante la noche para que así los conductores puedan ajustar su velocidad. Además, se debe garantizar una visibilidad suficiente en la misma de tal forma que los vehículos entrantes tengan visibilidad a la izquierda y que los vehículos circulantes tengan visibilidad al frente y a la derecha, pero ninguno de ellos debe centrar su atención a través del islote central (de ahí que muchos islotes centrales estén decorados con jardines, fuentes o esculturas para así impedir la visibilidad a través de los mismos).

Finalmente cabe destacar que la velocidad afecta a la seguridad en las rotondas ya que a medida que la velocidad aumenta la distancia total de frenado también aumenta y el campo de visión del conductor se ve reducido (efecto túnel).

2. 3. Capacidad

En el caso de las rotondas no tiene sentido hablar de *capacidad global de una rotonda*. Esto es debido a que no hay una correspondencia unívoca entre la geometría de una rotonda y la capacidad global de la misma ya que la distribución del tráfico y el comportamiento de los conductores, además de la geometría de la rotonda, juegan un papel fundamental en la capacidad.

Tal concepto es sustituido por el de *capacidad de entrada de una rotonda*, el cual puede ser definido como la tasa máxima a la que puede razonablemente esperarse que los vehículos entren a la rotonda desde una entrada durante un periodo de tiempo determinado bajo condiciones geométricas y de tráfico fijas.

Hay muchos factores y de naturaleza muy diferente que afectan a la capacidad de entrada de una rotonda. Por tanto, es común que cada país o autor considere distintos factores con distinta importancia en sus guías para el cálculo de la capacidad. Aun así, la mayoría parece coincidir en la influencia en la capacidad de dos elementos básicos:

- El tráfico que circula alrededor del islote central, debido a que los vehículos que desean entrar en la rotonda deben esperar a que haya un hueco disponible en el tráfico circulante para unirse al mismo.

- Los elementos geométricos de la rotonda. En 1970 R.M. Kimber llevó a cabo un estudio empírico en colaboración con el Transport and Road Research Laboratory en el cual estudió la influencia de 35 elementos geométricos de una rotonda en su capacidad, de los cuales se demostró que sólo tenían influencia 6: el ancho de entrada, el ancho medio del ramal, la longitud efectiva de abocinamiento, el ángulo de entrada, el diámetro del círculo inscrito y el radio de entrada.

Por otra parte, se ha demostrado que la presencia de pasos de zebra localizados en las entradas de una rotonda disminuye la capacidad de entrada de la misma debido a que los vehículos entrantes están obligados a ceder el paso a los peatones y pierden huecos disponibles en el tráfico circulante (huecos que de otra forma podrían haber sido aprovechados). Además, ciertos factores ambientales (como la falta de visibilidad debida a la niebla o el pavimento mojado, entre otros) y la presencia de vehículos pesados reducen la capacidad de entrada de una rotonda.

En cuanto al cálculo de la capacidad, pueden distinguirse dos tipos de métodos de cálculo:

- Los métodos empíricos, que consideran que hay una dependencia entre el comportamiento del conductor y las características geométricas de una rotonda. Estos métodos se basan en regresiones y requieren una gran cantidad de datos de rotondas congestionadas, los cuales se obtienen mediante observaciones en campo o estudios en laboratorio. Destacan el método francés, el método inglés, el método alemán y el método suizo.
- Los métodos probabilísticos o también llamados métodos basados en la Teoría de Aceptación del Hueco. Dichos métodos sólo consideran la interacción entre vehículos de tal forma que un vehículo que desea entrar en una rotonda lo hará cuando haya un hueco disponible mayor que un determinado hueco crítico. Destacan el método australiano y los métodos alemanes.

Por último no puede dejar de mencionarse los software existentes en el mercado que permiten calcular la capacidad de entrada de una rotonda, de los cuales los más representativos han sido recopilados en el presente TFM junto con su origen, alcance, método de cálculo en el que se basan y resultados que permiten obtener.

2. 4. Análisis del comportamiento de una rotonda

Típicamente se emplean tres parámetros fundamentales para caracterizar, evaluar y/o estimar la calidad del nivel de servicio ofrecido por una rotonda (bajo condiciones geométricas y de tráfico fijas). Dichos parámetros son los siguientes:

- Grado de saturación: es el ratio entre la demanda de tráfico en una entrada y la capacidad de dicha entrada. Este parámetro informa de la idoneidad de un diseño para gestionar de forma efectiva un volumen de tráfico dado. Cuando el grado de saturación excede un cierto valor el comportamiento de la rotonda comienza a deteriorarse de tal forma que se forman colas y los tiempos de espera empiezan a incrementarse de forma exponencial. A falta de un estándar internacional algunos autores afirman que el grado de saturación de una rotonda no puede exceder un valor de 0,85-0,90 si se desea asegurar un funcionamiento satisfactorio.
- Tiempo de espera: es uno de los parámetros fundamentales para evaluar el funcionamiento de una rotonda y es el que determina el nivel de servicio que ofrece la misma.
- Longitud de cola: este parámetro se suele emplear para comparar la idoneidad de una rotonda frente a otro tipo de intersección y para predecir la interacción de una rotonda y sus alrededores. Cabe destacar que algunos autores miden la longitud de cola en unidades de longitud mientras que otros lo hacen en número de vehículos.

2. 5. Control de rotondas mediante semáforos

El comportamiento de una rotonda depende de la interacción entre el diseño geométrico de la misma, el comportamiento de los conductores y las condiciones de tráfico. Dicho comportamiento será más exitoso cuando la distribución de orígenes y destinos esté nivelada, resultando en flujos de tráfico compensados en todas las entradas y en las distintas partes del anillo central. Cuando los flujos de entrada están descompensados el comportamiento de la rotonda es aceptable siempre y cuando el volumen de demanda sea bajo. El problema surge cuando la demanda de tráfico aumenta. Estos flujos de entrada descompensados causan colas, tiempos de espera elevados e incluso la saturación de la rotonda.

Este problema de la congestión debido a flujos de entrada descompensados parece ser una consecuencia directa de la principal desventaja de las rotondas: las rotondas causan la pérdida de prioridad de todos los ramales que tienen acceso a la misma y, por lo tanto, la pérdida de jerarquía de los mismos. Por esa razón, las rotondas no son capaces de dar prioridad a los flujos de tráfico con la demanda más alta. Por ello, las rotondas con flujos de entrada descompensados necesitan un tratamiento especial con el fin de mejorar su nivel de servicio.

Una de las formas de controlar este tipo de rotondas es mediante semáforos. En los últimos años varios estudios muestran que el rendimiento de rotondas congestionadas se puede mejorar mediante el control semafórico de las mismas. Tales semáforos pueden colocarse tanto en las entradas como en el propio anillo circular, de tal forma que eviten que el tráfico anular tenga siempre la prioridad.

Otra forma de controlar este particular tipo de rotondas es mediante los llamados *semáforos dosificadores*, que básicamente consisten en un semáforo que regula el flujo de vehículos de acuerdo a las condiciones de tráfico de la rotonda ya que está comunicado con un sensor que detecta la longitud de las colas en las entradas de la misma. Los semáforos dosificadores permiten crear huecos en el flujo anular de tal forma que reducen las colas y los tiempos de espera en rotondas con flujos de entrada descompensados. Típicamente este sistema de control es una solución que funciona a tiempo parcial, es decir, únicamente en las horas pico cuando los volúmenes de tráfico son elevados.

3. Simulación del tráfico

3. 1. Tipos de modelos de simulación del tráfico

Básicamente, hay tres tipos de modelos de simulación del tráfico en función del nivel de detalle considerado y del foco de interés:

- Simulación macroscópica. El enfoque macroscópico modela el flujo de tráfico como si de un fluido compresible se tratase. Con este fin, estos modelos generalmente se basan en la teoría del flujo de tráfico continuo, que describe la evolución espacio-tiempo de los flujos macroscópicos caracterizándolos con variables tales como el volumen, la velocidad y la densidad. Los modelos macroscópicos representan el tránsito de vehículos desde un punto de vista agregado, de modo que todos los vehículos del mismo grupo siguen el mismo patrón de comportamiento.
  La macrosimulación se caracteriza por un bajo nivel de detalle en el que se consideran pocos parámetros (comparado con la microsimulación). Debido a este bajo nivel de detalle su complejidad computacional es también baja, lo que hace que este tipo de simulación sea adecuada para analizar extensas áreas geográficas. Típicamente la macrosimulación se emplea cuando se desea analizar un fenómeno de tráfico con una gran cantidad de elementos cuyas características y dimensiones son significativamente menores que el área del fenómeno.
- Simulación mesoscópica. Los modelos mesoscópicos combinan características de la macrosimulación y de la microsimulación y se sitúan en un punto intermedio entre ambas. Lo mismo sucede con su complejidad computacional. Este tipo de simulación es adecuada para analizar un fenómeno a un nivel intermedio entre la escala macro y micro. El objetivo de la mesosimulación no es centrarse en una situación de tráfico o en un sistema concreto si no que analiza un grupo de ciertos vehículos de tal forma que se pueda ubicar a dicho grupo de vehículos en todo momento desde que entran en la red hasta que la abandonan.

- Simulación microscópica. California Department of Transportation define la microsimulación como “el modelado dinámico y estocástico del movimiento de vehículos individuales dentro de un sistema de transporte”. En dicha definición destacan dos términos claves: dinámico y estocástico. La microsimulación es dinámica porque la matriz de orígenes y destinos cambia con el tiempo, lo que significa que las acciones e interacciones de cada vehículo individual se modelan durante la propia simulación a medida que los vehículos viajan por la red. Además, la microsimulación es estocástica porque los mismos inputs de vehículos producen diferentes outputs debido a que se emplean semillas de números aleatorios. Dicha semilla es el punto de partida para generar una secuencia única de números aleatorios. Esta secuencia es la encargada de simular de forma realista un rango de comportamientos de los conductores. Esto hace que cada réplica y sus resultados sean únicos. Por tanto, para encontrar valores medios es necesario simular varias réplicas con distintas semillas aleatorias.

  La característica clave de la microsimulación de tráfico es el modelado de cada vehículo como una entidad independiente. En cada intervalo de tiempo el movimiento de cada vehículo se modela de acuerdo con las características físicas de dicho vehículo, la interacción del vehículo con la infraestructura y la interacción del vehículo con otros vehículos.

  En cuanto a la escala de aplicación, los modelos de microsimulación generalmente no están diseñados para la simulación de redes de gran tamaño debido al nivel de detalle requerido (es necesario definir una gran cantidad de parámetros) y, por lo tanto, a su complejidad computacional.

3. 2. Software escogido: PTV Vissim

Al decidir qué tipo de modelo de simulación de tráfico debe ser empleado para simular la rotonda objeto de estudio (rotonda de Viesques) la autora del presente TFM opina que la microsimulación de tráfico es la mejor opción por varias razones:

- El objeto de simulación es básicamente la rotonda y su entorno, lo que hace que la red de simulación sea un área relativamente pequeña.
- Aunque el área de simulación es pequeña, se necesita un alto grado de detalle con el fin de simular aspectos tales como la geometría exacta de la rotonda, las señales de velocidad, los pasos de cebra y la presencia de peatones y ciclistas, entre otros.
- El modelado de cada vehículo como una entidad independiente, que es la principal característica de la microsimulación, hace que la simulación de la rotonda sea muy realista.

Por estas razones, la autora comenzó a buscar softwares de microsimulación y encontró el software PTV Vissim particularmente adecuado ya que, además de las razones anteriormente mencionadas, el software tiene un módulo llamado VisVAP para el control lógico del tráfico.

PTV Vissim no es un software libre ya que su uso requiere una licencia. La compañía PTV Group ofrece licencias gratuitas para estudiantes que deseen investigar usando dicho software. Por lo tanto, la autora de este TFM se puso en contacto con PTV Group para adquirir una licencia de investigadores y PTV Group aceptó su propuesta. Como consecuencia, PTV Vissim es el software utilizado para el desarrollo de la simulación del presente TFM.

4. Simulación experimental: rotonda de Viesques

En este capítulo se describe de forma detallada el proceso de construcción del modelo de la rotonda objeto de estudio en el software PTV VisSim. El propósito principal del modelo de la rotonda es simular la situación del tráfico en la misma de la forma más realista posible. Para lograr esto, es necesario definir una extensa variedad de parámetros y variables en el modelo.

Al final del presente capítulo se definen distintos parámetros de la simulación así como los volúmenes de los inputs de tráfico.

5. Gestión de resultados

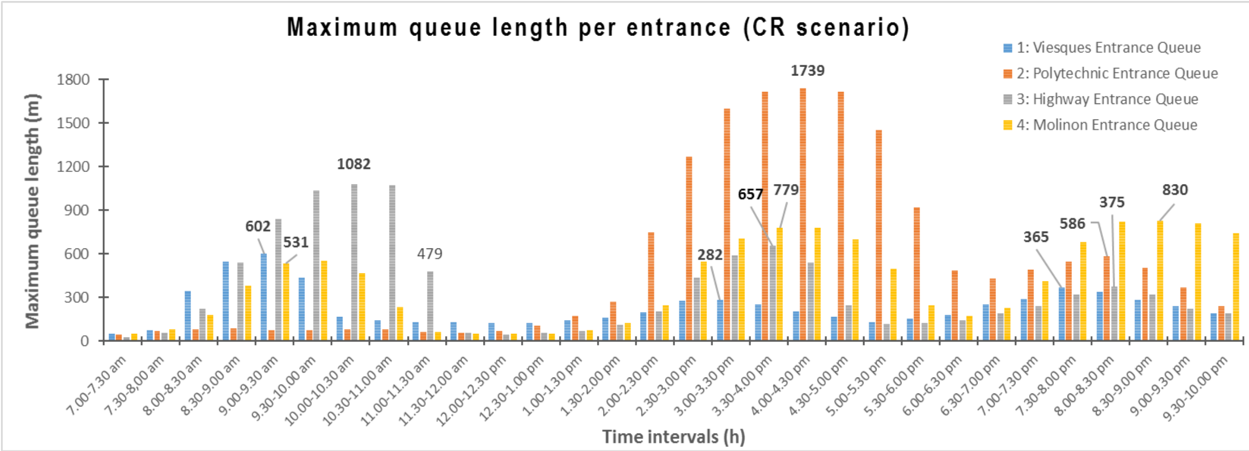
En este capítulo se analiza qué se mide durante la simulación y cómo obtener los resultados.

Además, dicho capítulo aborda el tema de la validación de resultados de tal forma que se calcula el número de réplicas que deben ser simuladas para garantizar un determinado nivel de confianza y margen de error.

6. Análisis de resultados

Este capítulo analiza los resultados obtenidos tras haber modelado la rotonda de Viesques en PTV Vissim y haber simulado las condiciones de tráfico en la misma.

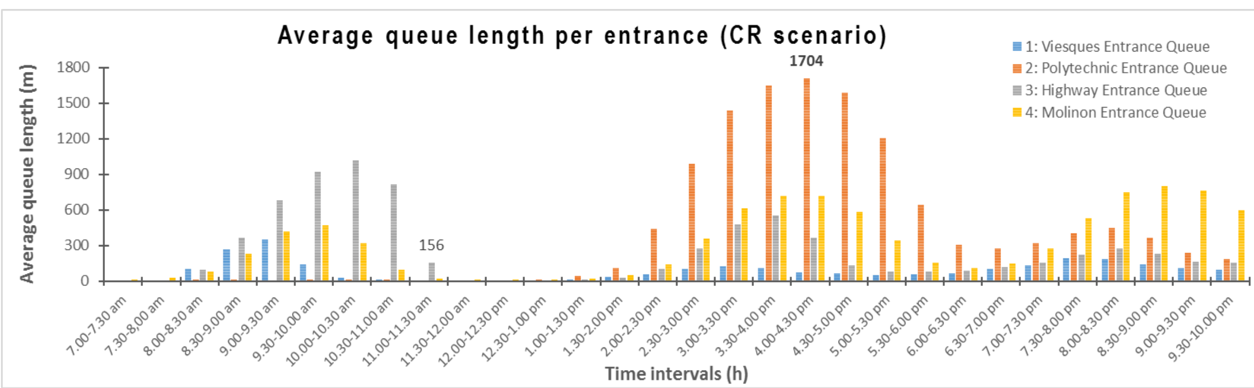
La gráfica 6.1 muestra la longitud de cola máxima por entrada. Como es de esperar, las colas más largas se forman durante las horas punta (entre las 8:00-9:30 am, 2:00-3:30 pm y 6:30-9:00 pm). Sin embargo, a veces las colas más largas se forman después del final de la hora punta. Esto es debido al efecto acumulativo de todas las llegadas de vehículos durante la hora pico.



Gráfica 6.1. Longitud de cola máxima por entrada

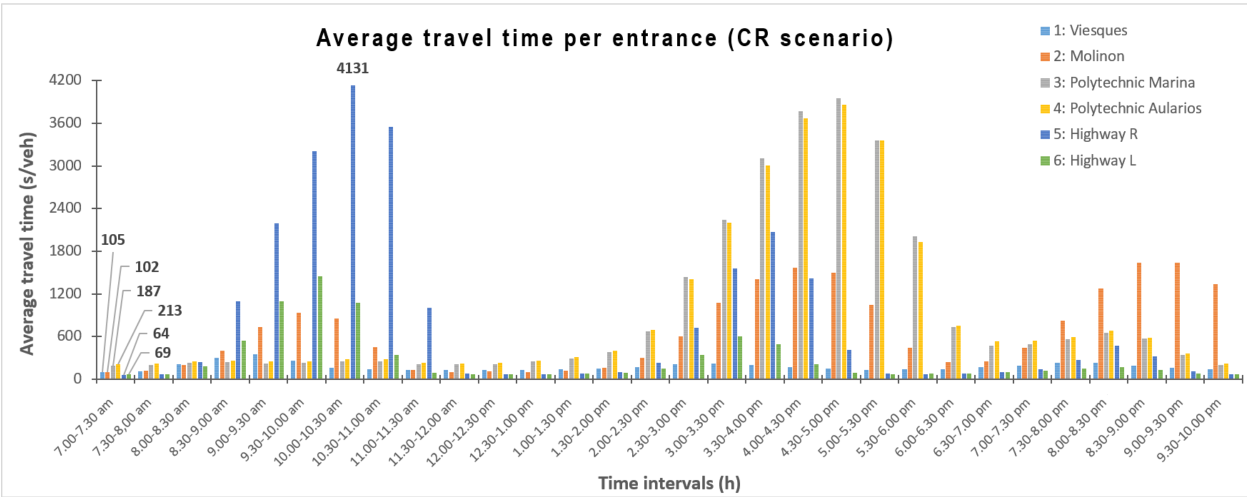
La gráfica 6.2 muestra la longitud de cola media por entrada. En algunos intervalos la longitud máxima de la cola es similar a la longitud media de la cola (por ejemplo, la longitud máxima de la cola en la entrada Polytechnic entre 4:00-4:30 pm es 1739 m, mientras que la longitud media es de 1704 m). Eso significa que los valores de la longitud de la cola no suceden durante un instante si no que se mantienen relativamente constante durante dicho intervalo de tiempo.

Por otro lado, hay valores máximos de longitud de la cola muy distintos a los valores medios (por ejemplo, la longitud máxima de la cola en la entrada Highway entre 11:00-11:30 am es de 479 m, mientras que el valor medio es de 156 m). Este hecho significa que ha habido un pico momentáneo de la cola pero más tarde la cola ha ido disminuyendo.



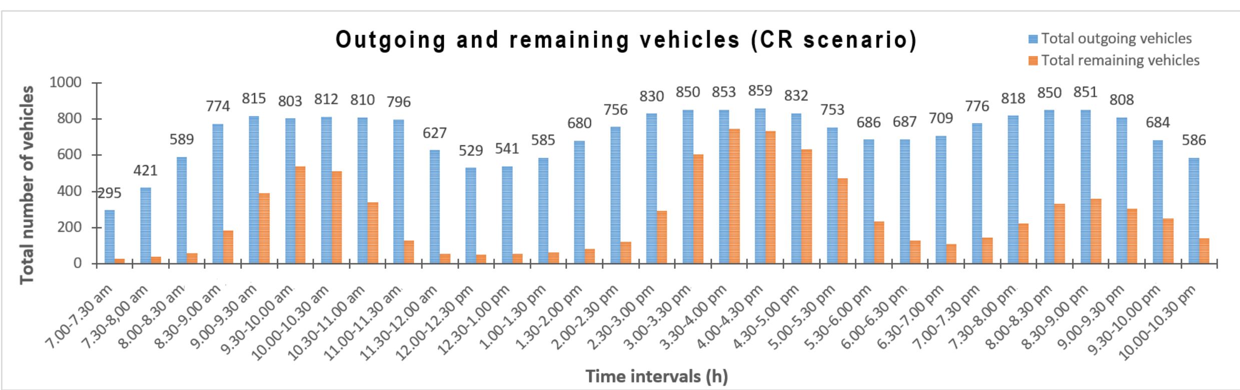
Gráfica 6.2. Longitud de cola media por entrada

La gráfica 6.3 muestra el tiempo medio de viaje de un vehículo desde el punto donde éste fue creado hasta la línea de ceda el paso de la rotonda. Al comienzo del día, entre las 7:00-7:30 am, el tiempo medio de viaje en cada entrada es muy bajo (105 s/veh en la entrada Viesques, 102 s/veh en la entrada Molinón, 187 s/veh en entrada Polytechnic Marina, 213 s/veh en la entrada Polytechnic Aularios, 64 s/veh en la entrada Highway R y 69 s/veh en la entrada Highway L). Estos tiempos medios de viaje son muy significativos ya que muestran las condiciones de tráfico óptimas. Por lo tanto, la autora de este TFM considera apropiado que estos resultados sean una referencia para comparar otros resultados con el fin de medir la eficiencia de la red. Por ejemplo, el tiempo medio de viaje en la entrada Highway R durante 10:00-10:30 am es de 4131 s/veh. Si este resultado se compara con el óptimo (64 s/veh) se podría decir que el tiempo medio de viaje es 65 veces el tiempo de viaje óptimo.



Gráfica 6.3. Tiempo medio de viaje por entrada

La gráfica 6.4 muestra el número total de vehículos que ya han llegado a su destino y que han dejado la red durante cada intervalo de tiempo (*total outgoing vehicles*) y el número total de vehículos que quedan en la red al final de cada intervalo de tiempo (*total remaining vehicles*). Es una gráfica muy interesante ya que muestra la capacidad máxima de la rotonda.



Gráfica 6.4. Outgoing y remaining vehicles

Calcular la capacidad de una rotonda no es una tarea sencilla, ya que depende de muchos factores. La simulación de tráfico es una muy buena herramienta para la estimación de la capacidad de una rotonda en condiciones específicas. En este caso, *total outgoing vehicles* indica la cantidad de vehículos que han utilizado la rotonda durante el intervalo de tiempo considerado. En la gráfica 6.4 se puede observar que el número de vehículos salientes tiende a estabilizarse en los momentos pico, cuando se excede la capacidad de la rotonda. Durante estos intervalos de tiempo la rotonda se satura y alcanza su máxima capacidad. La capacidad máxima de la rotonda es de aproximadamente 850 vehículos cada media hora, lo que significa alrededor de 1700 vehículos por hora.

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Department of Telecommunications and Information Processing
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Abstract

In this Master's Dissertation an analysis of the current situation of the Viesques roundabout (Gijón, Spain) was carried out and it was studied how it is possible to increase the capacity of the same.

In the current situation, this roundabout is a key point of the city and serves as a link between a major highway, the city center, the Polytechnic School of Engineering and the football stadium of the city. Because of the growing demand of traffic and the unbalanced entry flows (when the local football team plays), the roundabout capacity is exceeded and during the peak hours the roundabout is highly congested. At these hours massive queues are formed and users suffer long waiting times. To reduce the saturation of the roundabout it is common to find police presence regulating traffic manually.

The main goal of this thesis is to increase the capacity of the roundabout, so that queues and delays are reduced. To this end, the base situation of the roundabout is simulated and it is analysed how the capacity, queues and delays are affected by the following improvements:

- Changing the location of a zebra crossing located in one of the roundabout entrances.
- Adding an extra lane in two entrances and immediate exits in all the entrances.
- Implementing a control system using traffic lights actuated by sensors which detect vehicle queues.

Each of these scenarios are simulated by a traffic microsimulation software (PTV Vissim) so that it is possible to quantify the effects of such improvements. In addition, these scenarios are analyzed from an economic point of view and an approximate budget is calculated.

The thesis is structured as follows: the item under study and the aim of the study are introduced in Chapter 1. Chapter 2 includes a literature study of the topic. In Chapter 3 the issue of traffic simulation is addressed. Chapter 4 provides an explanation of the modeling of the current situation of the roundabout in the microsimulation software PTV Vissim. Chapter 5 discusses the types of results obtained in the simulations and the validation of these results. In Chapter 6 the results obtained in the simulation of the current situation of the roundabout (base scenario) are analyzed. Chapter 7 discusses the effects of changing the zebra crossing location. Chapter 8 examines flaring the entrances and adding immediate exits. Chapter 9 discusses the implementation of a traffic light control system actuated by detectors. In Chapter 10 a budget of each scenario is estimated. Finally, Chapter 11 presents the main conclusions of this thesis.

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1. Introduction

1.1. Item under study

The roundabout under study is situated in the suburbs of Gijón (see Figure 1.1), a small city which belongs to a region of the north of Spain called Asturias. More concretely, the specific location of this roundabout is 43°31'29.0"N 5°38'21.6"W. The surrounding area of the roundabout is composed by the Polytechnic Engineering School of Gijón, the A8 highway, the Molinón Stadium and a residential neighbourhood called Viesques.

Figure 1.1. Partial map of Gijón [Google Earth]

This roundabout connects the busy A8 highway with the centre of Gijón. In addition, this roundabout is the shortest access to the Polytechnic School of Engineering of Gijón from the A8 highway. Taking into account that there is a high amount of people in the region of Asturias who commutes every day to Gijón (to work or to study), the traffic flow of this roundabout is **highly congested during the rush hours**. The traffic in such roundabout is not regulated by traffic lights and police agents usually have to regulate it personally in the peak hours.

Figure 1.2 shows the roundabout with the nomenclature used hereinafter for each of the entries/exits.

Figure 1.2. Nomenclature of the roundabout [Google Earth]

Each of the entrances to the roundabout are characterised in more detail below.

- **Polytechnic School entrance**

Figure 1.3 shows the perspective of a driver who wants to enter the roundabout from the Polytechnic School of Engineering of Gijón. As it can be seen, the road has only one lane.

Figure 1.3. Polytechnic School entrance [Google Earth]

In the morning this entrance has a very low flow of vehicles. However, around 2:00 pm (when the morning shift classes conclude at the Polytechnic Engineering School) traffic density in this lane increases very sharply. The fact that teachers and students leave the university to have lunch causes a queue of around 500 meters of length on average (see Figure 1.4). The same happens around 7:00 pm when the afternoon shift concludes.

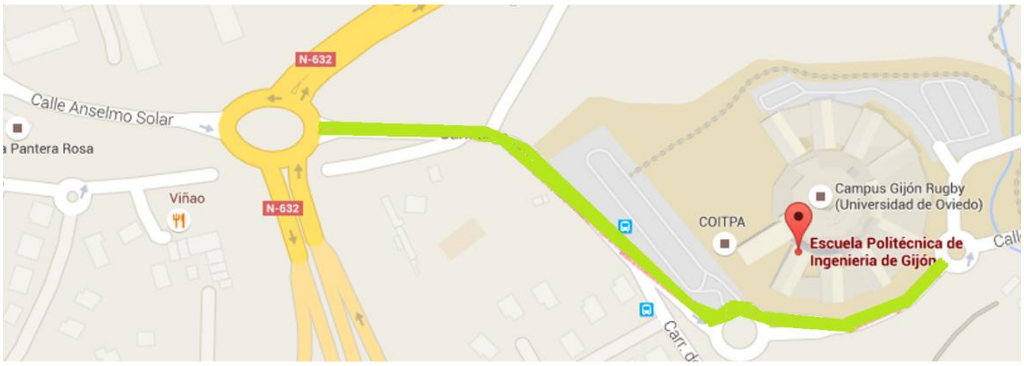


Figure 1.4. Queue formed at Polytechnic School entrance [Google Earth]

- **A8 Highway entrance**

Figure 1.5 shows the perspective of a driver who wants to enter the roundabout from A8 Highway. As it can be seen, the road has two lanes.

Figure 1.5. A8 Highway entrance [Google Earth]

A8 highway, also known as Cantabrian Highway, is extended along the north of Spain parallel to the Cantabrian Sea connecting cities such as Avilés, Oviedo and Santander. Due to the importance of this highway it is usually very busy. Furthermore, this roundabout is the fastest connexion to A8 highway for those who live in the east area of Gijón.

Although this entrance is always very busy, there is a peak of traffic during the morning (around 8:00-9:30 am) due to the high amount of people who commute to Gijón for working or studying. In addition, traffic usually increases slightly around 2:30 pm. Figure 1.6 illustrates the queue which is generated every morning.

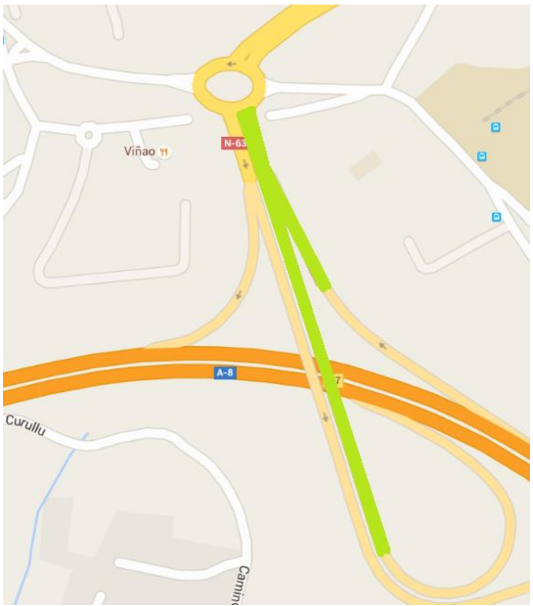


Figure 1.6. Queue formed at A8 highway entrance [Google Earth]

- **Molinón Stadium entrance**

Figure 1.7 shows the entrance from Molinón Stadium, where the local football team plays. As it can be seen, the road has two lanes. During the morning (around 8:00-9:30 am) there is a large amount of vehicles (which want to take the A8 Highway) forming a queue. It happens the same in the afternoon (around 2:30 pm) and in the evening (around 7:30 pm). Also, when there is a football match this entrance is usually full of vehicles trying to leave the stadium or the city.

Figure 1.7. Molinón Stadium entrance [Google Earth]

A queue of around 150 meters of length on average can be formed, as can be seen in Figure 1.8.

Figure 1.8. Queue formed at Molinón Stadium entrance [Google Earth]

- **Viesques entrance**

Figure 1.9 shows the entrance from the neighbourhood of Viesques to the roundabout. As it can be seen, the road only has one lane. Around 8:30 am this entrance is full of vehicles that are going to the university or the A8 Highway. In addition, a slight increase of the traffic is observed around 2:30 pm and also in the evening (between 6:30 pm and 9:00 pm).

Figure 1.9. Viesques entrance [Google Earth]

Figure 1.10 shows the most common queue formed in the rush hours which can be of around 500 meters long on average.

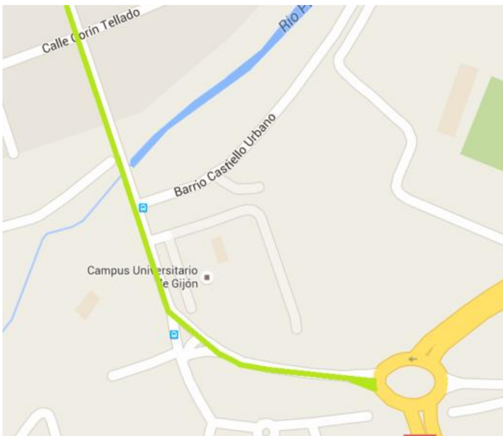


Figure 1.10. Queue formed at Viesques entrance [Google Earth]

1.2. Aim of the study

The aim of this study is to increase the capacity of the roundabout (which reduces the congestion of the same), so that queues and delays are reduced. For that end, the current situation of the roundabout is simulated by PTV Vissim software and it is studied how the capacity, queues and delays change in the following scenarios:

- When changing the location of a zebra crossing located in one of the roundabout entrances (explained in detail in Chapter *7. Analysis of location change of the zebra crossing*).
- When adding an extra lane in two entrances and immediate exits in all the entrances (explained in detail in Chapter *8. Analysis of immediate exits and flared entries*).
- When implementing a control system using traffic lights actuated by sensors which detect vehicle queues (explained in detail in Chapter *9. Analysis of traffic lights control actuated by vehicles*).

2. Literature study

2.1. Modern roundabouts

2.1.1. History of modern roundabouts

This chapter is mainly based on [1], [2], [3] and [4].

A review of the history of roundabouts cannot begin without highlighting the work of Todd, who has studied the origin and evolution of the roundabouts in *A History of Roundabouts in The United States and France* [5] and *A History of Roundabouts in Britain* [6].

As a starting point it should be noted that roundabouts could not have existed without the concept of *gyratory traffic*. Although there has always been a debate about who was the first, it could be said that William Phelps Eno and Eugene Henard created the concept of gyratory traffic concurrently. That concept consisted of a flow of vehicles in one direction around a central island.

William Phelps Eno designed Columbus Circle (see Figure 2.2) in New York City in 1903. On the other hand, Eugene Henard, the architect of Paris, proposed circular junctions with gyratory traffic for several intersections in Paris. It was in 1907 when Place Charles de Gaulle (formerly known as Place de l'Etoile, see Figure 2.1) became the first gyratory traffic junction of France. Both urban planners agreed in the idea of one-way circulation around the circular junction. However, their designs differed in the size of the central island.

Figure 2.1. Place Charles de Gaulle in 1921 [7]

Figure 2.2. Columbus Circle in 1907 [8]

During the decade of 1930 a type of circular intersection denominated *rotary* or *traffic circle* was designed in the United States (see Figure 2.3). Rotaries were designed under the principle of weaving movement. In a rotary the vehicles entered in a tangent instead of a 90 degree angle. Wide splitter islands were used to separate the different entering and exiting lanes. Entering vehicles had the priority, which allowed high speeds entrances and exits but also tend to block circulating traffic. It was usual that a road crossed the central island of the rotary splitting it in two. Also, pedestrians usually crossed the central island on foot, which was unsafe for them and also for drivers.

The priority of entering vehicles (called yield-to-right rule) produced locking problems in rotaries. That problem and the rising experiences of crashing caused the decline of rotaries in the decade of 1950 in the United States.

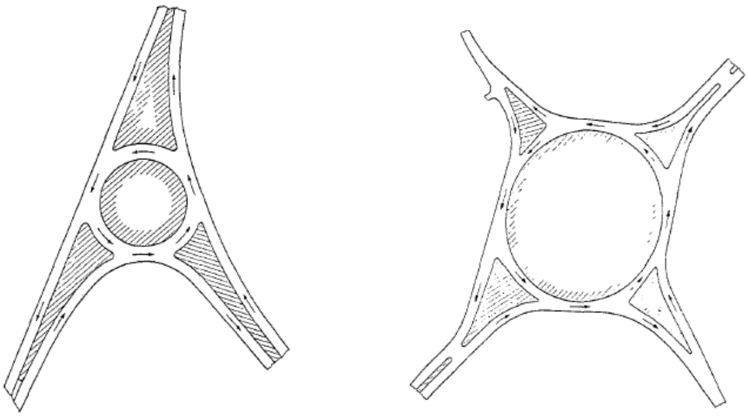


Figure 2.3. Examples of rotaries [9]

It was in 1926 in England when the term *roundabout* was born in order to substitute the word *gyratory*, but we had to wait until the 1960’s for that the first modern roundabout was designed. Up to that time the rules for priority were non consistent around the world mainly due to the fact that the traffic was relatively low and priority was not a key aspect. Priority varied from north-south or south-north priority in New York, priority to the first vehicle which arrived to the junction in some states of the United States or priority of entering vehicles to the junction.

During the 1950's the traffic began to increase in Britain and congestion was a usual problem in circular intersections. With the purpose of solving such problem the Road Research Laboratory (nowadays a private company called TRL Limited or Transport Research Laboratory) tested experimentally how the behaviour of roundabouts changed if circulating vehicles had the priority. It was found that this *yield-at-entry* rule (also called *give-way* or *off-side priority*) reduced delay times by 40%, increased capacity by 10% and reduced vehicle collisions by 40% [6].

In November 1966, in order to solve the locking problem and vehicles collisions in circular intersections, this yield-at-entry rule was established as a mandatory traffic rule in all circular intersections of Great Britain. This rule required entering vehicles in all circular junctions to yield to circulating vehicles.

Such yield-at-entry mandatory rule was an inflection point in the design of modern roundabouts. From that time the design of roundabouts as long circles in which merging and weaving were key aspects was left behind. Roundabouts began to be conceived and designed as small central islands with one-way traffic in which entering drivers had to look for a gap in the flow of circulating vehicles. This reduction of the central island, the approach of yield line to centre of the island and the widening of entering lanes increased capacity of roundabouts by between 10% and 50%.

In the decade of 1970 roundabouts began to be built in France and Australia. It was in 1984 when the yield-at-entry rule become mandatory on national highways throughout France, a country in which priority had always been given to the entering vehicles in a roundabout.

Arguably, modern roundabouts are relatively young in the United States due to the fact that the first two modern roundabouts were built in March 1990 in Summerlin (Nevada). In October 1997 a total of 38 modern roundabouts had been built in the United States. The use of roundabouts in the United States has been slowly proliferating over time to reach about 4800 roundabouts in December 2015 [10].

In Germany roundabouts have been built from 1930 until the 1960's when, for unknown causes, they fell from grace. In the late 1980 roundabouts barely existed in Germany. However, it was in that decade when, based on the British studies, German traffic planners started to experiment with roundabouts in urban and rural areas. The results of these experiments showed that, although large roundabouts used in the UK were not suitable for German roads due to their design features, these elements had a positive impact on safety, traffic flow and aesthetic reasons. Nowadays, roundabouts are considered as a useful tool for traffic control in Germany.

2.1.2. Main features of modern roundabouts

This chapter is mainly based on [2], [3] and [11].

In 1984 the Design Standard DTp 16/84 was published in Great Britain. Such standard exposed the requirement of deflection for entering vehicles (entry path curvature) and the concept of a new smaller roundabout. The concept of *modern roundabouts* (expression used in the United States to differentiate them from the old rotaries) was finally settled in 1984 so that modern roundabouts are characterized by three key features:

1. **Yield at entry**
   In modern roundabouts it is an indispensable requirement that entering vehicles are obligated to yield to traffic that circulate around the central island. Therefore, circulating vehicles have the priority and entering vehicles have to look for a gap in the circulating flow.
   Entering vehicles are controlled by YIELD signals at the entering lanes, which allows to maintain fluidity and high capacity.

Figure 2.4. YIELD signal in an entering lane of a roundabout [Google Images]

2. **Deflection at entry**
   Unlike the former rotaries, tangential entries are not allowed in roundabouts. Vehicles enter the roundabout at a 90 degree angle so that splitters islands and the central island deflect them to the right. The speed of the vehicles on the roundabout depends on how straight its trajectory is. The straighter it is, the more speed can be achieved. Therefore, the deflection at entry causes low speed entries.

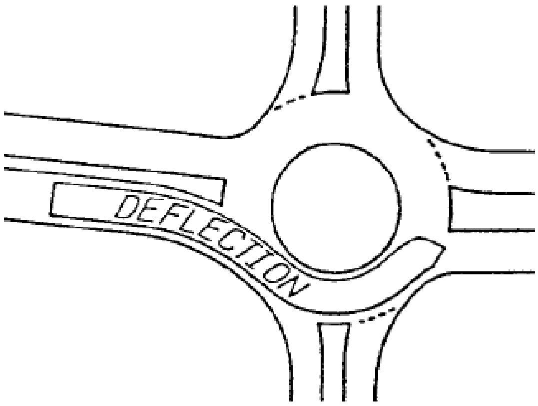


Figure 2.5. Deflection at entry [3]

The speed of the vehicles is controlled by the angle of curvature of the splitter islands and by the location and diameter of the central island so that the speed is always lower than a maximum value (typically 50 km/h).

3. **Flare at entry**

   Roundabouts have flared entering lanes in order to allow the entry of more vehicles, which increases capacity. Furthermore, flared entering lanes allow incoming vehicles to join the circulating traffic with similar speeds.

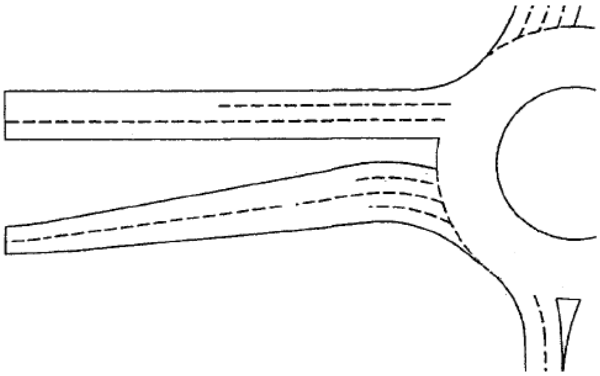


Figure 2.6. Flare at entry [3]

In addition, it should be pointed out that there are other features that distinguish modern roundabouts from old rotaries:

- Parking is not allowed in the central island nor at the entrances and exits of the roundabouts.
- Only one-way circulation around the central island is allowed.
- Pedestrian activity is the central island is not allowed. Pedestrians crossing are located back from the yield line and the splitter island is cut so that they can cross it (see Figure 2.7).

Figure 2.7. Splitter island cut by a pedestrian crossing [Google Images]

2.1.3. Elements and geometrical dimensions of modern roundabouts

This chapter is mainly based on [2], [3] and [12].

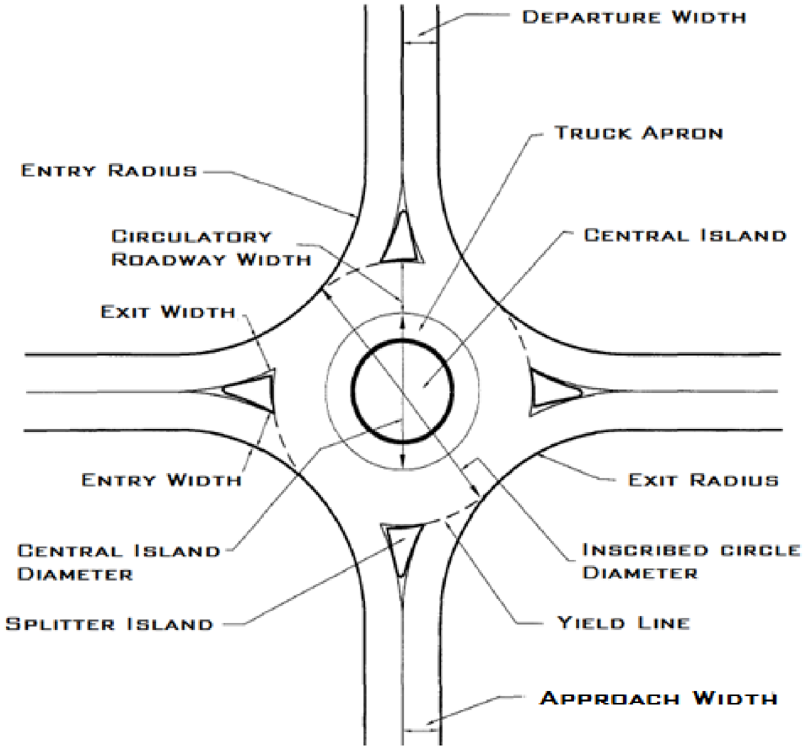


Figure 2.8. Elements of a modern roundabout [3]

The main elements and geometrical dimensions of a modern roundabout are:

- **Central island**

  The central island is the circular element in the center of a roundabout around which traffic circulates. Therefore, the central island is the area not destined to vehicle movement which is contained within the circulatory roadway.

  Although it is generally recommended that the central island is circular in shape, oval or elliptical shapes are not rejected. For elliptical central islands the eccentricity should be moderate. It is preferable that the eccentricity is between 0.75 and 1. In this way the deflection at the entrance is not very sharp and the speed of entering traffic is lower.

  In most cases the size of the central island is characterized by the **central island diameter.**

  The central island has several functions:

  - It is an obstacle to the roadways which are approaching the intersection. Therefore, the central island induces a reduction in speed and a change of course to get around.

- Due to its location and size it forces that vehicles change their trajectory. This added to the one-way circulation around the central island serve to prevent conflict points by the intersection of the roads.
- It is a tool for the perception of the intersection and adaptation to the environment.

- **Truck apron**
  It is an external ring which the central island sometimes has for vehicles which are too large, such as trucks, and do not have enough space to perform the maneuver of turn. The truck apron is usually built with different materials than the circulatory roadway so that users differentiate it from the circulatory roadway. The differences are not only in appearance but also in sound and comfort of movement.
- **Splitter island**
  A splitter island is an elevated or painted area located on an approach which separates entering from exiting lanes. Typically it has a triangular shape.
  As mentioned above, the splitter islands have several functions:
  - They indicate the proximity of a roundabout.
  - They are designed to generate and to control the deflection experienced by incoming vehicles and, therefore, to control the speed of entering traffic.
  - They also provide a space for pedestrians to cross the road in two stages.
  - They create a separation between the entering and exiting lanes. This makes that vehicles which are waiting to enter the roundabout have an adequate visibility and can predict in advance a safe entry to the circulating traffic.
- **Yield line**
  The yield line is a broken line painted in the pavement which is located in the entering lanes above the inscribed circle diameter. When vehicles reach this line they are obligated to yield to traffic that circulates around the central island. Vehicles can only cross the line when they find a gap in the circulating traffic.
- **Inscribed circle diameter**
  It is the basic parameter to characterize the size of a roundabout. It is the diameter of the circle that can be inscribed within the external line of the circulatory road.
- **Approach width**
  It is the width of the approaching roadway measured at a distance far enough from the central island so that there is no change in the width of the lane.
- **Departure width**
  It is the width of the departing roadway measured at a distance far enough from the central island so that there is no change in the width of the lane.
- **Entry width**
  It is the width measured perpendicularly from the right edge of the entering lane to the intersection point of the left edge and the inscribed circle.

- **Exit width**
  It is the width measured perpendicularly from the right edge of the exiting lane to the intersection point of the left edge and the inscribed circle.
- **Entry radius**
  It is the minimum radius of curvature of the right-side curb at the entry.
- **Exit radius**
  It is the minimum radius of curvature of the right-side curb at the exit.

2.2. Safety of roundabouts

2.2.1. Safety studies

This chapter is mainly based on [3], [12], [13], [14], [15], [16], [17] and [18].

One of the biggest advantages of roundabouts and largely the reason for their proliferation in recent decades is the safety of roundabouts over other types of conventional intersections. Since the introduction of modern roundabouts numerous studies which brought to light such safety have been conducted. A collection of the most relevant studies is shown below.

- One of the most prominent safety studies in Great Britain was carried out in 1984 by G. Maycock and R.D. Hall [13]. The aim of *"Accidents at four-arm roundabouts"* was to study accidents in 4-arm roundabouts to find the influence of the traffic flow and design elements of the roundabout.
  As a consequence of the study the following findings can be highlighted:
  - The average rate of accidents was 3.31 accidents which involved personal injuries per year (16% of them were classified as fatal or serious). The motorcyclists were involved in 30-40% of accidents, while cyclists only in 16% of them.
  - A linear model that relates the frequency of collision of each arm of the roundabout with the traffic flow and geometry of the roundabout was developed.
  - The developed model showed that to increase safety in roundabouts with highly flared entering lanes the greatest possible deflection at entry was required.

- In 1986, two years after the yield-at-entry rule became mandatory in France, the Centre D'Etudes Techniques de L'Equipement de l'Ouest studied 83 French roundabouts [14].
  The main conclusions of the study "*Evolution de la Securite Sur Les Carrefours Giratoires"* are listed below:
  - The transformation of a conventional intercession into a modern roundabout has a positive effect on safety.

- The establishment of the yield-at-entry as mandatory rule in 1984 in France improved safety in roundabouts.
- Roundabouts with small diameters are safer (have fewer accidents) than those of large diameters or oval shapes.

- R. T. Tudge presented in 1990 at the 15th edition of the Australian Road Research Board Conference a study on the safety of roundabouts in New South Wales, Australia. The study *"Accidents at roundabouts in New South Wales"* [15] analysed 230 intersections before and after being converted in roundabouts and 60 controlled sited (which were not roundabouts).

  While at intersections converted in roundabouts a significant decrease in crash experiences is observed, controlled sites suffered an increase in accidents during the same time period. More specifically, the results of the study showed that in intersections converted to roundabouts:
  - There had been a reduction in general accidents at roundabouts by 50%.
  - Fatal accidents had decreased by 63%.
  - Accidents with injuries had decreased by 45%.
  - Accidents with only damage had decreased by 40%.

- In 1991 F. Alphand, U. Noelle and B. Guichet published the article *"Roundabouts and road safety: state of the art in France"* in which a study of 522 roundabouts of Western France conducted in 1988 was described [16].

  As a result of the study it is known that there were 78 accidents in 1988 in these roundabouts (5 of them categorized as fatal and 26 with serious injuries). These accidents are analyzed and related with features of roundabouts such as location, shape, number of arms and traffic volume among others. Furthermore, the study makes a comparison between these accidents and accidents in 1238 intersections controlled by traffic lights. The study concludes that in the intersections controlled by traffic light occur twice as many accidents as in roundabouts.

- In 1994 C. Schoon and J. van Minnen published the article *"The safety of roundabouts in The Netherlands "* [17]. In this article the authors describe a study conducted in late 1992 in the Netherlands. The objective of the study was to analyse safety at roundabouts (emphasizing cyclists and motorcycles).

  The study analysed 181 intersections that had been converted into modern roundabouts. The intersections were studied an average of 5.3 years when they were a conventional intersection and an average of 2.0 years of being a modern roundabout. The results of this study showed a reduction of 51% in the number of accidents and 72% in the severity of them. However, the severity of the damage on bicycles and motorcycles decreased by 44%.

- In 1996 W. Brilon conducted a study on safety in the German roundabouts [18]. The study analyzed accidents in 34 locations before and after being converted into roundabouts (most of them were single-lane roundabout with inscribed circle diameter of about 30 m).
  The results of this study were:
  - The total number of accidents fell by 40%.
  - The more severe accidents were, the more reduction of accidents occurred.
  - The accident costs were reduced by 36% in urban areas. Instead, the cost saving was much higher in the roundabouts not located in urban areas: there was a reduction of 84%.
  - The number of pedestrian accidents was reduced from 8 to 2.
  - Regarding safety for cyclists, changes depend on whether the bicycle circulates around the circulating roadway or not. To bicycles that circulated mixed with the traffic on the roundabout, in the pedestrian path or in paths built outside the circulating roadway no significant changes were observed. However, in roundabouts where bicycles circulate in an external lane of the circulatory roadway accidents were increased from 1 to 8 accidents.

2.2.2. Causes for improved safety

This chapter is mainly based on [2], [3] and [12].

Although in the previous chapter the cited studies have shown the increased levels of safety at roundabouts compared to other types of conventional intersections, the purpose of this chapter is to delve into the reasons for this improvement of safety. These reasons are cited and analyzed below.

- Traffic flow in one direction around the central island is one of the reasons for the safety of roundabouts. This is because the crossing, merger or divergence of the trajectory of two traffic elements (such as vehicles, bicycles or pedestrians) creates a potential conflict point. Therefore, one-way circulation causes **the reduction of the number of conflict points** at roundabouts comparing to conventional intersections.
  A conflict point is associated with a risk of accident, so that the number of accidents in an intersection is related with the number of conflict points thereof. For this reason, the decrease in conflict points results in increased safety.
  Figure 2.9 shows a comparison of the conflict points in a conventional 3-arm intersection (T intersection) and in a 3-arm roundabout. As it can be seen, conflicts points decrease from 9 in a T intersection to 6 in a roundabout.

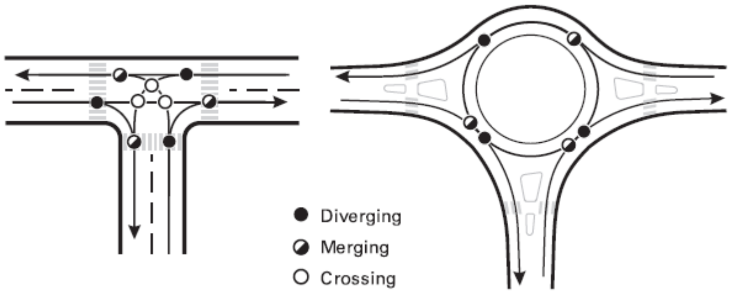


Figure 2.9. Conflict points in a 3-arm intersection and in a 3-arm roundabout [2]

In Figure 2.10 it is shown that while on a 4-arm intersection (also known as X or cross intersection) the number of conflict points is 32, in a 4-arm roundabout the number of points of conflict is only 8.

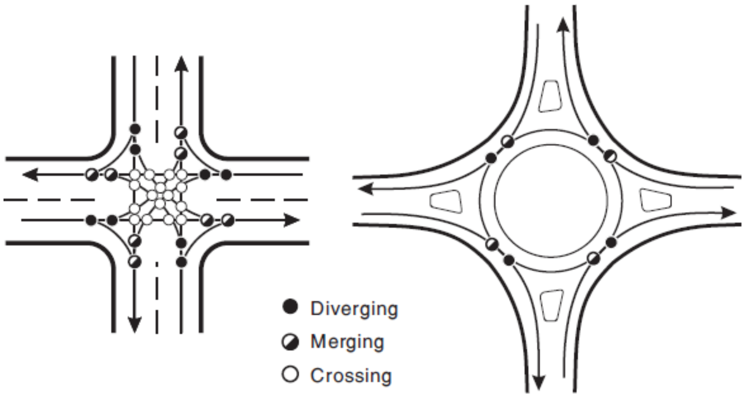


Figure 2.10. Conflict points in a 4-arm intersection and in a 4-arm roundabout [2]

Figures 2.9 and 2.10 illustrate the reduction of conflict points at one-lane roundabouts. Something similar happens in two-lane roundabouts. Two-lane roundabouts show a reduction of conflict points compared to the corresponding conventional intersections. However, new conflict points are introduced in this type of roundabouts due to fundamentally two reasons:

1. <u>Users employ the wrong lane.</u>
   For example, this happens when a vehicle is circulating on the inside lane and uses the outside lane to exit the roundabout, when it should use the inside lane (see vehicle B in figure 2.11). It also happens when a vehicle is changing lanes in the circulatory roadway (see vehicle D in Figure 2.11)

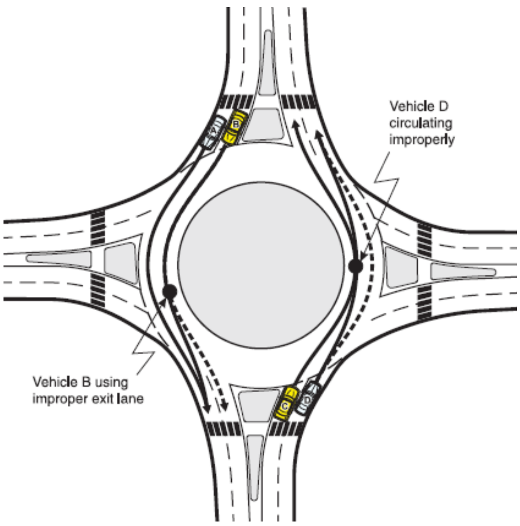


Figure 2.11. Use of the wrong lane [2]

2. Users make an improper turn.
For example, this happens when a vehicle in the inside lane wants to leave at the first exit of the roundabout (see vehicle B in Figure 2.12), since it is assumed that vehicles wishing to take the first exit should circulate on the outside lane. Also, it happens when a vehicle in the outside lane does not come out at the first or second exit (see vehicle D Figure 2.12).

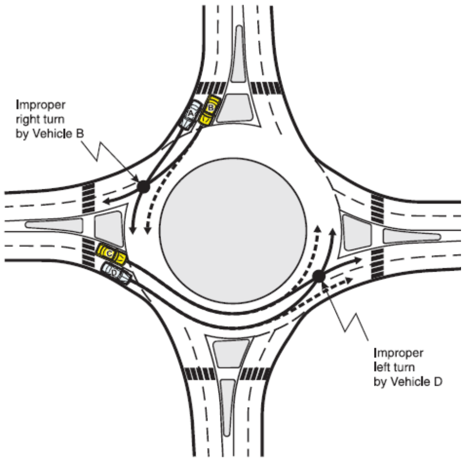


Figure 2.12. Improper turns [2]

In conclusion, roundabouts have a reduced number of conflict points, and thus greater security, compared to conventional intersections. However, it is necessary to distinguish between the roundabouts of a single-lane approach and multi-lane approach, since the former have fewer conflict points.

- **Low speeds** in roundabouts, mainly due to the deflection at entry, is the primary reason of safety of roundabouts. Such low speed is responsible for:
  - Drivers have more time to react to potential conflicts.
  - It benefits slower vehicles as it tends to homogenize speeds by reducing the difference in relative speeds of the different vehicles.
  - Reduces the probability of accidents and their severity.

- Another key elements of the safety of roundabouts is **the understanding and ease of operation by users**. This is achieved by circulating in one direction, the yield-at-entry and the aforementioned reduction of conflict points.

2.2.3. Recognition and visibility

This chapter is mainly based on [12], [19] and [20].

It has been demonstrated that of all the factors related to the design of roundabouts those which most positively affect in safety are associated with the driver's behaviour. Recognition and visibility decisively affect the behaviour of the driver and therefore the safety levels of roundabouts. Such elements affect safety from different perspectives.

On the one hand, it is necessary that **vehicles approaching the roundabout recognize it** in advance at a sufficient distance. No matter that the presence of the roundabout is announced by warning signals in sufficient time, to achieve the maximum levels of safety it is necessary that drivers visually recognize the roundabout as this reinforces their attitudes and decisions. The roundabout should be recognizable and visible both during the day and during the night (when the lighting plays an important role), so that drivers can adjust their speed.

On the other hand, it is necessary that **vehicles in the roundabout have sufficient visibility** so that they sense the presence and speed of other vehicles and thus manoeuvre accordingly. Firstly, it is important that entering vehicles have visibility to the left (vehicles to which must yield). Secondly, it is important that vehicles circulating around the central island have visibility to the front and to the right (vehicles that reduce their speed or vehicles that wish to enter the roundabout). In neither of these two situations they should focus their attention through the central island. For this reason and to promote concentration of drivers, to avoid distractions and to increase security, the central islands must contain elements of great size that work as a barrier to vision, such as large trees, shrubs or artistic elements. Moreover, these large ornamental elements help perception of the roundabout discussed above.

Figure 2.13 illustrates that a set of trees is an effective visual barrier on the central island.

Figure 2.13. Roundabout in Oviedo (Asturias, Spain) [Google Images]

In Europe it is typical that the central islands of roundabouts become a place of honor to place statues or other important monuments or elements. For instance, in the city of Oviedo (Asturias, Spain) is very common that in the central islands of the main roundabouts there are fountains (see Figure 2.14).

Figure 2.14. Roundabouts with fountains in Oviedo [made from images from Google Images]

2.2.4. Influence of the speed

This chapter is mainly based on [2], [12], [20] and [21].

According to most of the authors, such as W. Brilon and M. Vandehey [4], low speeds at roundabouts has proven to be the main cause of increased security in the same over conventional intersections. This is logical for several reasons:

- The higher the vehicle speed is, the greater distance is required to stop completely. This is because the total distance required for a vehicle to stop is the sum of the braking distance and the perception-reaction distance and both distances are proportional to the vehicle speed, as it can be seen in equation 1.

  $$D_{total\ stopping} = D_{braking} + D_{p-r} = \frac{v^2}{2\mu g} + v \cdot t_{p-r} \qquad \text{(eq. 1)}$$

  Where:
  - $D_{total\ stopping}$: total distance required for a vehicle to stop.
  - $D_{braking}$: distance required for a vehicle to brake.
  - D_{p-r}: distance required for a vehicle to perceive that it is necessary to stop and react.
  - v: speed of the vehicle.
  - μ: coefficient of friction between the vehicle wheel and the pavement.
  - t_{p-r}: percepction-reaction time.

- It has been shown that the visual field of a driver decreases as speed increases. This happens because the driver focus the attention on the vehicle's trajectory and the rotation of the eye is very limited in this situation.

Worldwide studies have exposed that increasing the curvature of the vehicle trajectory causes the reduction of the difference in relative speeds of incoming and circulating vehicles. This often leads to a reduction in the accident rate. However, it should be highlighted that in multi-lane roundabouts not because the curvature is greater (thus the speed is lower) this will ensure a lower accident rate. This is becaus0e in this type of roundabouts larger curvature produces greater friction between contiguous traffic streams. This can result in more collisions between vehicles of different lanes. Therefore, this enables to conclude that for each particular roundabout there is an optimal design speed which minimize accidents.

In *Safety analysis of roundabout designs based on geometric and speed characteristics* [21] S. Kim and J. Choi made a brief study of the literature of the safety studies conducted (the majority mentioned in chapter 2.2.1) and they summarized this literature commenting that drivers drive at different speeds in each segment of roundabouts and these fluctuations on speed can be associated with different rates of accidents. They also claim that the study of literature carried out shows that the number of accidents increases with increasing velocity differences in the distinct segments of the roundabout.

2.3. Capacity

2.3.1. Concept of roundabout capacity

This chapter is mainly based on [1], [2], [22] and [23].

The Highway Capacity Manual [22] defines the vehicle capacity of a facility as "the maximum number of vehicles that can pass a given point during a specified period under prevailing roadway, traffic and control conditions". However, and unlike conventional intersections, it makes no sense to talk about the *global capacity of a roundabout*. This concept cannot be applied to the roundabouts because there is no univocal correspondence between the geometry of a roundabout and the global capacity of it. This is because that the distribution of traffic and also drivers' behaviour, in addition to the geometric features, play an important role in the global capacity of roundabouts. This can be demonstrated with a simple example. Given the same roundabout two opposite situations could be analyzed:

- All users decide to leave the roundabout at the first exit. In this situation, the movements of all vehicles would be reduced to make a single turn right and the circulatory roadway would be barely occupied.
- All users decide to leave the roundabouts at the last exit. In this situation, the traffic would be more complex and the circulatory roadway would be more occupied.

Clearly the global capacity of the roundabout would be higher in the first situation. This example highlights the limited usefulness of the concept of overall capacity of a roundabout, since this depends on many factors and the same roundabout can have different global capacities for different traffic distributions.

The performance of old rotaries was explained by *weaving theory* until in 1966 the yield-at-entry rule became mandatory in Great Britain. This theory assimilated the circulatory roadway to a succession of weaving sections. Based on the weaving theory, Wardrop's Formula was used to calculate the capacity of roundabouts. In 1973 the Transport and Road Research Laboratory demonstrated that the circulatory roadway of a roundabout does not behave as a succession of waving sections. This discovery caused the abandonment of the concept of global capacity of a roundabout.

From that moment the circulatory roadway was considered as the addition of T intersections in which entering traffic analyses circulating traffic and inserted into the same when it is an acceptable gap. Therefore, the calculation of the capacity of a roundabout is focused on calculating the capacity of each of the T intersections in which the roundabout can be decomposed.

To model this new concept it is part of the base that in each of the T intersections in which the roundabout is decomposed there are two interrelated traffic variables: the approaching flow and the circulating flow (see Figure 2.15).

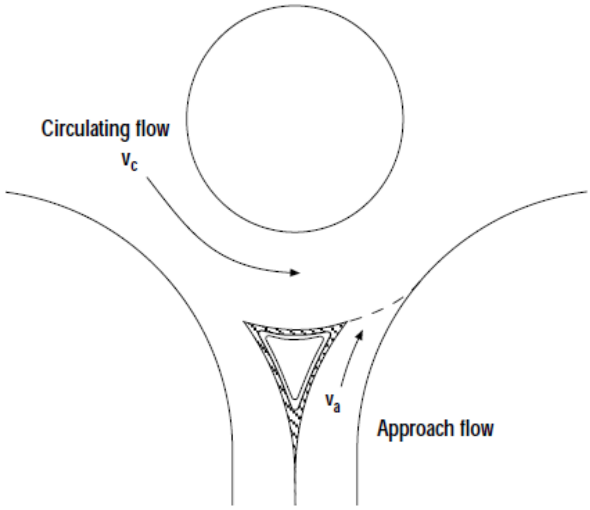


Figure 2.15. Roundabout decomposition in T intersections [22]

The ratio of the two variables is inverse due to when the circulating traffic increases the size of the gaps is reduced and thus the rate at which the approaching traffic enters the roundabout is lower. Instead, when the circulating traffic decreases the size of the gaps is increased so that more vehicles can enter the roundabout.

All the foregoing leads to the conclusion that the concept of global capacity of an intersection is not applicable and not useful in the case of roundabouts. This concept is replaced by the **entry capacity of a roundabout**. The entry capacity can be defined as "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" [2]. Each entry in a roundabout is assimilated to a T intersection and for calculating its capacity two basic elements must be considered:

- The characteristics of the circulating traffic on the roundabout that conflicts with the entry.
- The geometric characteristics of the entry of the roundabout.

2.3.2. Influential parameters in the entry capacity of a roundabout

This chapter is mainly based on [2], [12], [22], [24], [25] and [26].

Drivers only enter the roundabout when there is an available large enough gap in the circulating traffic. For that reason, and as discussed above, the **characteristics of the circulating traffic** is a totally decisive element in entry capacity.

In addition, **the geometrical features of the roundabout** affect the entry capacity of a roundabout. At the end of the decade of 1970 R.M. Kimber conducted an empirical research in collaboration with the Transport and Road Research Laboratory [24]. Kimber, who was then the Chief of Junction Design Section at TRRL, led an investigation about the traffic capacity of roundabouts whose field measurements can be divided into two main groups:

- The effects on the capacity of 35 geometric elements of roundabouts were evaluated in the test track of TRRL.
- Data about the saturation capacity of 86 public roundabouts were taken.

This extensive empirical research led to the conclusion that of the 35 studied geometric elements only 6 of them presented a significant effect on the capacity:

- Entry width
- Approach half width
- Effective flare length
- Entry angle
- Inscribed circle diameter
- Entry radius

According the results of the research, the entry width, the approach half width, the effective flare length and the entry angle have a higher effect on capacity. However, the two last geometric elements have a minor influence on the capacity.

- The <u>inscribed circle diameter</u> has a relatively small effect in capacity of roundabouts with inscribed diameters smaller than 50 m.
- For values of 20 m or more the <u>entry radius</u> has a small effect on capacity.

The **number of lanes**, and by extension the width of entry and the circulatory roadway, has a determinant effect on capacity. According to the Transportation Research Board [22], experience across several countries has shown that the capacity can be improved by increasing the number of lanes (both in the approach and in the way circulatory). Wider circulatory roadways allow increasing the rate at which the approaching vehicles enter to circulating traffic. This fact is highlighted in Figure 2.16, which shows the expected capacity for a roundabout with a single-lane in the circulatory roadway and for a double-lane, according to the analysis of capacity conducted by the US Department of Transportation in [2]. This figure illustrates that given a circulatory flow the maximum entry flow is greater in a double-lane roundabout (with diameter of 55 m) than in a single-lane roundabout (with diameter of 40 m).

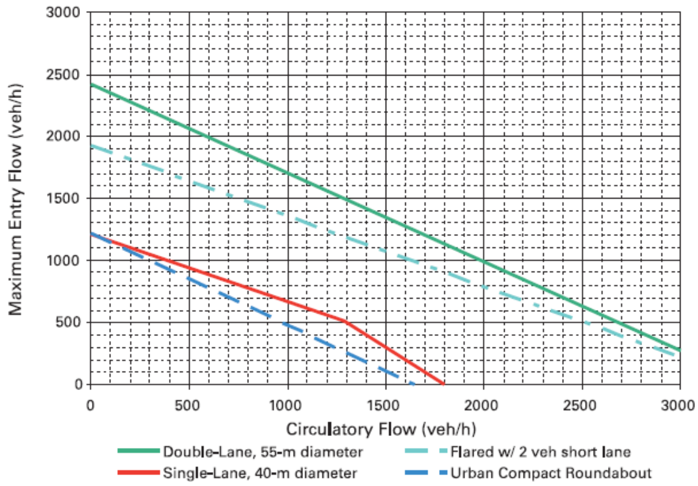


Figure 2.16. Capacity comparison of single-lane and double-lane roundabouts [2]

Another element that affects the entry capacity of a roundabout are pedestrians. The **presence of pedestrians** crossing the roadway at the entrances of the roundabout can reduce entry capacity, especially if they have the priority because there is a zebra crossing. In this situation the flow of pedestrians represents an obstacle to entering traffic, so that vehicles miss available gaps in the circulating traffic that otherwise they might have used to enter the circulatory roadway.

Few studies on the effect of pedestrians on roundabout entry capacity have been conducted. The most important are the following two:

- In 1982 M. Marlow and G. Maycock [25] studied the effect that zebra crossings have in decreasing entry capacity of junctions. These authors developed an analytical model based on queueing theory to evaluate these effects and exemplified the application of the method in a roundabout.

- In 1993 W. Brilon, B. Stuwe and O. Drews [26] analyzed roundabouts which had crosswalks (pedestrians had the right-of-way) and developed a capacity reduction coefficient by an empirical analysis based on data from different German roundabouts. This coefficient is a function of the circulating flow rate and the volume of crossing pedestrians.
  Based on the obtained results (see Figure 2.17 and Figure 2.18) this capacity reduction coefficient M has more relevance as increasing the volume pedestrian (for the same flow circulating) and has less relevance with increasing circulating flow (for the same volume pedestrian).

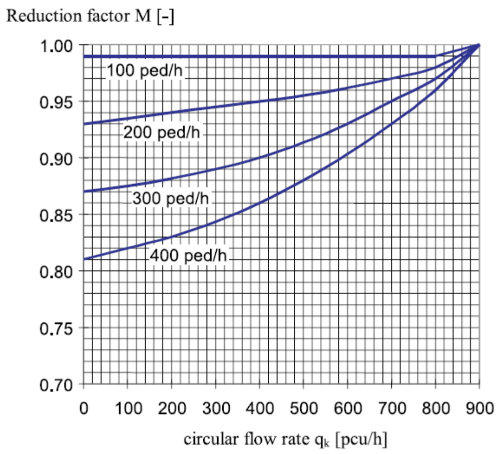


Figure 2.17. Capacity reduction factor M for a single-lane roundabout [2]

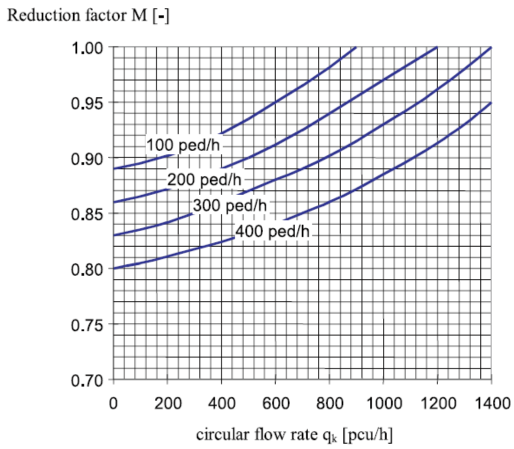


Figure 2.18. Capacity reduction factor M for a double-lane roundabout [2]

Therefore, pedestrian traffic is a variable that cannot be disregarded. The presence of pedestrians crossing crosswalks in a roundabout reduces the entry capacity because incoming vehicles must yield to pedestrians and miss gaps in the circulating traffic or either because vehicles wishing to leave the roundabout must give the right of way to pedestrians stopping before an exit and occupying the circulatory roadway.

Another set of factors that influence the entry capacity of roundabouts are those that could be included in a category called **psychosocial factors**: recklessness or prudence, courtesy to pedestrians and other vehicles, use of the intervals between vehicles, experience in roundabouts, among others.

Other factors that also affect the entry capacity are **environmental factors** such as the lack of visibility at night or due to fog or wet pavement due to rain, among others.

One factor that should be taken into account is the **presence of heavy vehicles**. Such a presence may reduce the entry capacity of a roundabout mainly because this type of vehicles need a larger gap to enter the circulating traffic and it takes a longer time for such incorporation.

In conclusion, there are many factors and with a very different nature that affect the entry capacity of a roundabout. Therefore, it is common that each country or author considers different factors and with a different importance in their guidelines to determine entry capacity of a roundabout.

2.3.3. Calculation methods

This chapter is mainly based on [3], [12], [23] and [27].

First of all it should be noted that the purpose of this chapter is not to present an exhaustive review of all existing capacity calculation methods. This would be a task that could be the only object of study of a Master's Dissertation because of the variety of methods, the various parameters that each method considers and the different methodologies and formulations. For that reason and to avoid an extremely long and deep development, this chapter presents a general overview of the most representative existing methods.

Calculation methods of capacity aim to obtain an expression which links traffic on the circulatory roadway and the maximum traffic that can reasonably be expected to enter the roundabout from an approach (under constant conditions and for a given time period).

Over the years different countries have been developing their own methods for calculating entry capacity of roundabouts. Basically, these methods can be classified into two groups taking into account their methodology and operative base:

- On one side there are the **empirical methods**, which consider that there is a dependence between the driver behaviour and the geometric features of the roundabout [27]. The empirical methods are based on regression analysis and they require a high amount of data of congested roundabouts. To collect these data these methods have their starting point in the observation and study of multiple roundabouts (either in field observations or in laboratory tests).
  Table 2.1 shows a brief description (in which the formulation of such methods is omitted) of the most representative empirical methods. In addition, and although they have not been included in Table 2.1, the Spanish method, the Portuguese method and the Danish method are other relevant empirical methods.

<table>
<tr><th></th><th>Origin</th><th>General description</th><th>Capacity is a function of…</th></tr>
<tr><td rowspan="2">French method</td><td>The French method was originally developed in 1987 by SETRA, the French national agency for the design of rural highways [28].</td><td>The formula developed by SETRA relates linearly the entry capacity of a <u>roundabout with a radius of 15 m or more</u> with the impeding flow (traffic circulating around the central island to the left of an entry hindering the entry of vehicles in such entry).</td><td rowspan="2">▪ Impeding flow
▪ Exit width
▪ Entry width
▪ Splitter island width
▪ Circulatory roadway width
▪ Inscribed circle diameter</td></tr>
<tr><td>In 1988 CETUR (a government organization responsible for the development of urban transport guidelines) developed a formula for capacity of urban roundabouts [29].</td><td>The formula developed by CETUR relates linearly the entry capacity of a <u>roundabouts with a radius smaller than 15 m</u> with the impeding flow.</td></tr>
</table>

	Origin	General description	Capacity is a function of...
British method	It was developed by the Transport Research Laboratory based on Kimber's equations [24].	It is probably the most refined method of existing ones, due to the long experience and empirical research of Great Britain. The British method results in a formula that relates linearly the entry capacity with the circulating flow, depending this relationship on the geometry of the roundabout. This expression has been obtained from a huge number of measurements of capacity at congested roundabouts.	▪ Circulating flow ▪ Approach width ▪ Average effective flare length ▪ Entry angle ▪ Entry radius ▪ Inscribed circle diameter ▪ Entry width
German method	The method was developed by the federal government.	Originally an exponential approximation was used to relate the entry capacity with the circulating flow. Between 1993 and 1966 more measurements of capacity were conducted. This led to a revision of the method and the exponential relation was replaced by a formula that relates linearly the aforementioned variables.	▪ Circulating flow ▪ Number of entering lanes ▪ Number of circulating lanes
Swiss method	The method was published in The Swiss Roundabout Guide in 1991, elaborated by the Institute of Transportation of the Federal Polytechnic School of Lausanne [30].	The method proposes a linear relationship between the entry capacity and the impeding flow.	▪ Number of circulating lanes ▪ Number of entering lanes ▪ Distance among conflict points

Table 2.1. Empirical capacity calculation methods (Own elaboration based on [3] and [31])

- On the other side there are the **methods based on the Gap Acceptance theory** (also called **probabilistic methods)**, which only consider interactions between vehicles [27]. This theory considers that vehicles circulate in the circulatory roadway leaving a certain distance (gap) between them. Vehicles wishing to enter the roundabout will enter when an available gap exceeds a certain threshold value called *critical gap*. In addition to the critical gap parameter these methods typically consider the *follow-up time* (which it is the time that a vehicle placed second in the yield line takes to stand on such line in a position to start the incorporation maneuver). Therefore, these methods calculate capacity by knowing the critical gap, the follow-up time and the distribution of the gaps in the annular traffic.

It should be highlighted that the theoretical basis of probabilistic methods are found in the work of W. Siegloch [32], J. Harders [33], J.C. Tanner [34] and M. McDonald and D.J. Armitage [35], which resulted in four formulas that respond to different combinations of the probability distributions of the gaps in the annular traffic of the roundabout.

Two of the most remarkable probabilistic methods are briefly descripted below. Also, probabilistic methods have been developed in Sweden an in the US, among others.

- **Australian method**

  In Australia a method based on the Gap Acceptance theory is used. That method is included in *Guide to Traffic Engineering Practice-Part 6: Roundabouts* [36] and has its origin in the *Special Report No. 45* of Australian Road Research Board [37] published in 1989.

  For roundabouts with multiple-lanes entries this method define two types of streams: the *dominant stream* (which is the one with the highest entering flow) and the *sub-dominant stream*.

  The Australian method calculates the entry capacity based on three parameters:

  1. The follow-up time, which is calculated for each lane and in turn depends on the inscribed circle diameter, the number of circulating lanes, the number of entering lanes and the traffic circulating.
  2. The critical gap, which in turn depends on the follow-up time, the number of circulating lanes and the average entry lane width.
  3. The number of useful gaps.

- **Methods in Germany**

  The German approach of calculating the entry capacity of a roundabout based on Gap Acceptance theory has its origin in the work of W. Siegloch and J. Harders. Over the years a probabilistic method has been developed and corrected.

  In 1997 N. Wu published *A Universal Formula for Calculating Capacity at Roundabouts* [38], which method is currently used by German standards. Such method calculates the capacity as an exponential equation dependent on the impeding flow, the number of lanes in the circulatory roadway, the number of entry lanes, the critical gap, the follow-up time and the minimal time gap on the circulatory roadway.

2.3.4. Software for calculating capacity

This chapter is mainly based on [2] and [3].

Over the years software to calculate the entry capacity of a roundabout and others parameters related to it (such as delays and queues) have been developed. Table 2.2 shows a compilation of the most representative software.

Software	Origin	Scope	Calculation method	Analysis of…
SIDRA	It was developed in Australia in 1984.	All roundabout configurations and other types of intersections	Australian probabilistic method	▪ Critical gaps ▪ Follow-up times ▪ Capacity ▪ Delays ▪ Queues ▪ Fuel ▪ Environmental measurements
KREISEL	It was developed in 1996 by W. Brilon and his research team at the Ruhr Universit (Germany).	All roundabout configurations	▪ German probabilistic methods ▪ British empirical method ▪ French empirical methods (CETUR and SETRA) ▪ Swiss empirical method ▪ Troutbeck method	▪ Capacity
ARCADY	It was originally developed by Transport Research Laboratory in 1981.	All roundabout configurations	British empirical method	▪ Capacities ▪ Queues ▪ Delays ▪ Crash frequencies (function of geometry) ▪ Prediction of the variability of queues and delays
RODEL	It was developed by R.B. Crown in 1987.	All roundabout configurations	British empirical method	▪ Capacity ▪ Delays ▪ It has been conceived to experiment with the geometric parameters in the design process of a roundabout.

Software	Origin	Scope	Calculation method	Analysis of…
GIRABASE	It was developed by CETE OUEST (a regional technical study organization in Nantes, France). It is accepted by CERTU and SETRA (urban and interurban French design institutes, respectively).	All roundabout configurations	French empirical method	▪ Capacity ▪ Delay ▪ Queues

Table 2.2. Software for calculating capacity (Own elaboration based on [2] and [3]).

2.4. Performance analysis

This chapter is mainly based on [2], [22] and [39].

According with the U.S. Department of Transportation in *Roundabouts: An Informational Guide (FHWA-RD-00-067)* [2] there are three fundamental parameters typically used to characterize, evaluate and/or estimate the quality of service offered by a roundabout (under given traffic conditions and a fixed geometry). These three measures are:

- **Degree of saturation** (also called volume-to-capacity ratio)
  The degree of saturation is "the ratio of the demand at the roundabout entry to the capacity of the entry" [2]. The degree of saturation assesses the suitability of a roundabout design to efficiently manage a given traffic volume. When the degree of saturation exceeds a certain threshold value the operation of the roundabout begins to deteriorate so that queues are formed and delays start to increase exponentially.
  Roundabouts: An Informational Guide (NCHRP Report 672) [39] says that despite the absence of an international standard for the critical value of the degree of saturation, the international experience shows that the degree of saturation of a roundabout cannot exceed a value of 0.85 to 0.90 if satisfactory operation wants to be ensured.

- **Delay**
  The delay is the fundamental parameter to evaluate the performance of a roundabout. Two types of delays can be distinguished:
  - <u>Control delay</u>
    It can be defined as "the time that a driver spends decelerating to a queue, queuing, waiting for an acceptable gap in the circulating flow while at the front of the queue, and accelerating out of the queue" [39]. Control delay for a given entering lane depends on the entry capacity of such lane and the degree of saturation of the same.
    The Highway Capacity Manual [22] uses the control delay to define the level of service. The **level of service** is a way to try to quantify the quality of service, which represents "how well a transportation facility or service operates from a traveler's perspective" [22].

Table 2.3 shows the different levels of service for unsignalised intersections established in the Highway Capacity Manual [22]. This table shows that each level of service is denominated by a letter that goes from A (representing the highest level of service) to F (minimum level of service).

	Degree of saturation	
Control delay (s/vehicle)	≤1.0	>1.0
0-10	**A**	**F**
>10-15	**B**	**F**
>15-25	**C**	**F**
>25-35	**D**	**F**
>35-50	**E**	**F**
>50	**F**	**F**

Table 2.3. Level of service for unsignalised intersections [22]

- <u>Geometric delay</u>
  It is the delay caused by the mere existence of the roundabout. The geometric delay can be defined as "the additional time that a single vehicle with no conflicting flows spends slowing down to the negotiation speed, proceeding through the intersection and accelerating back to normal operating speed" [2].

- **Queue length**
  The length of the queues (which in this Master's Dissertation are measured in meters) formed at the entrances of a roundabout is a measure that assesses the suitability of the design on the approaches of the roundabout.
  The queues length also serves to compare the suitability of roundabouts regarding other types of intersections and to predict the interaction between the roundabout and the environment (intersections or nearby roads).

2.5. Controlling roundabouts by traffic lights

2.5.1. The problem of unbalanced entry flows

This chapter is mainly based on [40] and [41].

The performance of a roundabout depends on the interaction between the geometrical design of the roundabout, drivers' behaviour and traffic conditions. Such performance is greatest when the distribution of origins and destinations is evenly shared out, resulting in balanced traffic flows at all entrances and in the different parts of the circulatory roadway.

When entry traffic flows are unbalanced, the performance of the roundabout is acceptable as long as the overall volume of demand is low. The problem starts to arise with increased traffic, even with intermediate levels of demand. This could happens when there is an approach with high traffic in which vehicles enter the circulatory roadway and, because they have the priority, prevent vehicles from any other approach to enter the roundabout. Such unbalanced entry flows causes long queues, long delays and the saturation of the roundabout.

This problem of congestion due to unbalanced flows seems to be a direct consequence of the main disadvantage of roundabouts: roundabouts cause the loss of priority of all approaches that access to it and therefore the loss of road hierarchy. For that reason, roundabouts are not able to prioritize flows with higher traffic demand.

In conclusion, roundabouts with unbalanced entry flows need a special treatment in order to improve their performance and level of service.

2.5.2. Control by traffic lights

This chapter is mainly based on [42].

According to the Department of Transport [42], in recent years several studies have brought to light that the performance of some congested roundabouts can be improved by controlling them with traffic lights. Traffic lights, which can be placed both in the approaches and in the circulatory roadway, regulate traffic and prevent that annular traffic has always the priority. Therefore, control by traffic lights allows the fluidity of traffic, resulting in balanced traffic flows and improved capacity and levels of service.

In 1997 County Surveyors' Society [43] studied the reasons for the signalisation of 161 roundabouts. Later, in 2006, the Department of Transport [42] did the same with 239 roundabouts. Both surveys found out that the main reason for introducing traffic lights control was queue control and increasing capacity, as it can be seen in table 2.4.

Reasons for signalisation	Survey in 1997	Survey in 2006
Queue control	70 %	80 %
Increased capacity	67 %	70 %
Accident reduction	30 %	72 %
UTC linkage	27 %	15 %
Pedestrians/cyclists	–	38 %
Other	24 %	– %

Table 2.4. Reasons for signalisation roundabouts in Great Britain [42]

Since the beginning of the 1990's Great Britain is betting on roundabouts control by using traffic lights. The Department of Transport [42] of such country cites the following reasons for introducing this type of control:

- **Capacity**
  As it has been discussed above, traffic lights can improve the capacity of roundabouts with unbalanced entry flows by improving traffic fluidity.

- **Delay**
  Delay on a roundabout entry is caused by the lack of entry capacity. Although control by traffic lights can reduce delays in some entries but also increase it in others, such control can reduce overall delays at the roundabout when this is highly congested.
- **Safety**
  In chapter *2.2.1. Safety studies* many studies that show the safety of roundabouts over other types of intersections have been mentioned. For roundabouts in which, for various reasons, the accident rate is relatively high (greater than or equal to 5 personal injury accidents per year) recent studies show that traffic lights control can reduce this rate.
- **Pedestrians and cyclists**
  Traffic light control reduces the risk of accidents for two-wheeled vehicles (especially bicycles) and allows the implementation of zebra crossings controlled for pedestrians and bicycles.

2.5.3. Control by metering lights

This chapter is mainly based on [44], [45] and [46].

A **metering light** (also called *metering signal*, *ramp meter* or *ramp signal*) is "a device, usually a basic traffic light or a two-section signal (red and green only, no yellow) light together with a signal controller that regulates the flow of traffic entering freeways according to current traffic conditions" [47]. The scope of the metering lights is not only limited to highways (freeways) but they can be used in roundabouts. In fact, their use in roundabouts with unbalanced entry flows or roundabouts with high volumes of demand is common.

In the field of roundabouts controlled by metering lights the work of R. Akçelik should be highlighted, more specifically: *Roundabout Metering Signals: Capacity, Performance and Timing* [44], *Analysis of Roundabout Metering Signals* [45] and *An investigation of the performance of roundabouts with metering signals* [46], among others.

Metering lights allow to create gaps in annular traffic so that they help to reduce queues and delays in roundabouts with unbalanced entry flows. Usually this type of control is a part-time solution that is activated only during peak hours when the traffic volumes are high.

The basic scheme of control by metering lights can be seen in Figure 2.19. Akçelik uses the following terminology for the different entries:

- **Metered approach**
  It is the entry that causes problems for a downstream approach (the controlling approach). This entry is regulated by a traffic light, which will be operating only at peak hours. This traffic light can be red (in that case the vehicles in that entry are obligated to stop) or can be green, yellow or switched off (in that case the roundabout operates normally by self-managing traffic flows).

- **Controlling approach**
  It is the entry that suffers long queues and delays because it has a high demand at peak hours and the traffic from the metered approach disrupts its access to the circulatory roadway. In this approach queue detectors are used so that this entry benefits from traffic dosage.

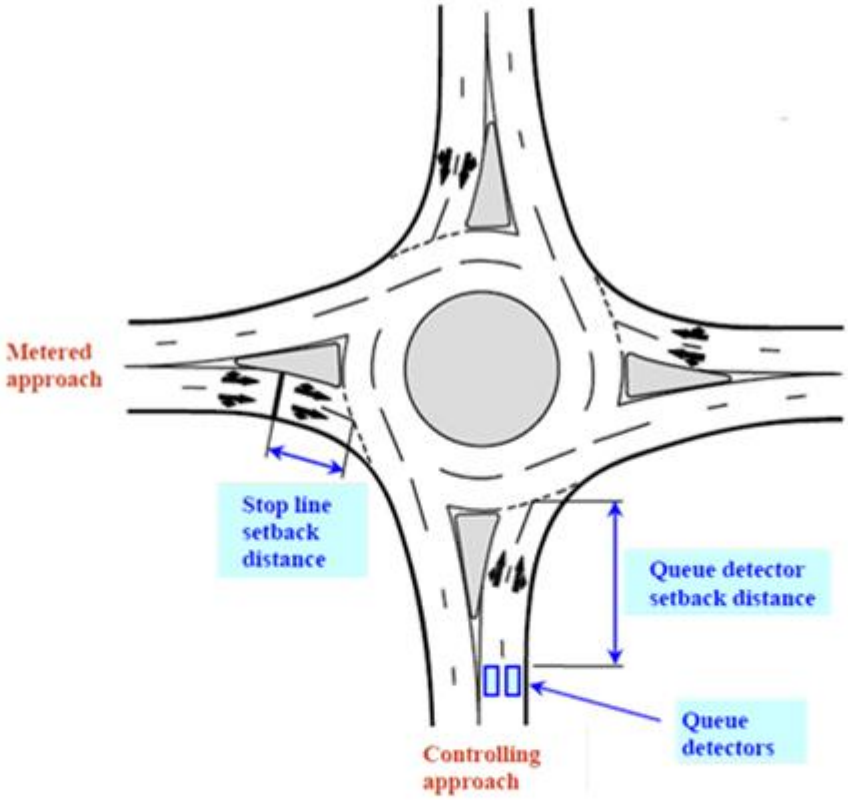


Figure 2.19. Metering lights in a roundabout ([40] partially modified)

The process of operation is simple. When queue detectors are activated (which means that the queue of vehicles reaches at least the position of the queue detectors) the traffic light of the metered approach turns red (according to a pre-establish cycle). This creates gaps in the circulatory roadway and makes that the traffic from the controlling approach can enter the roundabout fluently. When the red light cycle ends the roundabout operates normally again.

It should be noted that the explained above corresponds to the basic scheme, which can and should be modified to suit the characteristics of each roundabout. The basic scheme consists of the installation of a traffic light in only one of the entries of the roundabout (the metering approach) to regulate the entering traffic to the circulatory roadway and therefore reduce delays in other access (the controlling approach). Traffic lights and queue detectors can be installed in more than one entry or even in all entries in the roundabout if it would be necessary to increase capacity.

3. Traffic simulation

In this chapter the issue of traffic simulation is discussed. More specifically, the first part of this chapter analyses the different models of traffic simulation while the second part of this chapter focuses on the software used to perform the simulation of this Master's Dissertation.

3.1. Types of traffic simulation models

Basically, there are three types of traffic simulation models depending on the level of detail considered and the focus of interest: microscopic, mesoscopic and macroscopic models.

3.1.1. Microscopic simulation

This chapter is mainly based on [48], [49] and [50].

The California Department of Transportation defines traffic microsimulation as "the dynamic and stochastic modelling of individual vehicle movements within a system of transportation facilities" [48].

In this definition there are two key terms: dynamic and stochastic. Microsimulation is **dynamic** because the origin-destination matrix is dynamic and changes over time. This means that the actions and interactions of individual vehicles are modeled during the simulation time (typically in time steps smaller than 1 second) as they travel through the network. In addition, microsimulation is **stochastic** because the system performance depends on random variations. This is because the same inputs of vehicles may produce different outputs because random number seeds are used. Each seed is the starting point for generating a unique sequence of random numbers. Such sequence can realistically simulate a range of different drivers' behaviour. This makes that each run and its outputs are unique. Therefore, to find the average conditions it is necessary to simulate several runs with different random seeds.

The key feature of traffic microsimulation is the **modelling of each vehicle as a separate entity**. At each time step, the movement of each individual vehicle is modeled according to:

- <u>The physical features of such vehicle.</u> This makes it necessary to define the different existing types of vehicles and their relevant physical characteristics (mainly the length).

- <u>The vehicle interaction with the infrastructure.</u> Therefore, it is necessary to define in detail all the network traffic parameters (such us road geometry, signposting and zebra crossings, among others).

- <u>The vehicle interaction with other vehicles in the network.</u> Normally, interactions between vehicles are based on vehicle following algorithms, gap acceptance algorithms and lane changing algorithms.

As for the scale of application, microsimulation models are not generally designed for simulating large networks due to the level of detail required (a high amount of parameters have to be defined) and therefore its computational complexity. Typically, these models are used to analyze specific complex traffic problems such as “signalised roundabouts, bus priority, urban traffic control, ramp metering, traffic calming, road works design, car park location and control, pedestrian and cyclist interaction, traffic impact, incident management and traffic emissions” [49].

Table 3.1 show a list of the most important existing microsimulation software:

Software	Organization	Country
AIMSUN 2	Universitat Politècnica de Catalunya, Barcelona	Spain
ANATOLL	ISIS and Centre d’Etudes Techniques de l’Equipement	France
AUTOBAHN	Benz Consult - GmbH	Germany
CASIMIR	Institut National de Recherche sur les Transports et la Sécurité	France
CORSIM	Federal Highway Administration	USA
DRACULA	Institute for Transport Studies, University of Leeds	UK
FLEXSYT II	Ministry of Transport	Netherlands
FREEVU	University of Waterloo, Department of Civil Engineering	Canada
FRESIM	Federal Highway Administration	USA
HUTSIM	Helsinki University of Technology	Finland
INTEGRAT ION	Queen’s University, Transportation Research Group	Canada
MELROSE	Mitsubishi Electric Corporation	Japan
MICROSIM	Centre of parallel computing (ZPR), University of Cologne	Germany
MICSTRAN	National Research Institute of Police Science	Japan
MITSIM	Massachusetts Institute of Technology	USA
MIXIC	Netherlands Organisation for Applied Scientific Research - TNO	Netherlands
NEMIS	Mizar Automazione, Turin	Italy
PADSIM	Nottingham Trent University - NTU	UK
PARAMICS	The Edinburgh Parallel Computing Centre and Quadstone Ltd	UK
PHAROS	Institute for simulation and training	USA
PLANSIM-T	Centre of parallel computing (ZPR), University of Cologne	Germany
SHIVA	Robotics Institute - CMU	USA
SIGSIM	University of Newcastle	UK
SIMDAC	ONERA - Centre d'Etudes et de Recherche de Toulouse	France
SIMNET	Technical University Berlin	Germany
SISTM	Transport Research Laboratory, Crowthorne	UK
SITRA-B+	ONERA - Centre d'Etudes et de Recherche de Toulouse	France
SITRAS	University of New South Wales, School of Civil Engineering	Australia
TRANSIMS	Los Alamos National Laboratory	USA
THOREAU	The MITRE Corporation	USA
TRAF-NETSIM	Federal Highway Administration	USA
VISSIM	PTV System Software and Consulting GMBH	Germany

Table 3.1. Some microsimulation software [50]

It should be noted that due to the random nature of microsimulation models, they usually require a huge number of replicas to achieve results with a certain level of confidence. This leads to extremely high simulation times to validate a model, which is the main disadvantage of microsimulation.

3.1.2. Macroscopic simulation

This chapter is mainly based on [51], [52], [53], [54] and [55].

Macroscopic approach model the traffic flow as if it were a compressible fluid. To this end, these models are usually based on traffic flow continuum theory, which describes the space-time evolution of macroscopic flows characterizing them with variables such as volume, speed and density.

Unlike the microscopic models, macroscopic models represents the traffic of vehicles from an aggregate point of view, so that all the vehicles of the same group follow the same pattern of behaviour and the traffic is usually represented in terms of "total flows per time period and averaged travel time per time period" [51]. In macroscopic simulation the origin-destination matrix is static.

Macroscopic simulation is characterized by a low level of detail. Therefore, a low amount of parameters (compared with microsimulation) are considered. Due to the low level of detail of macroscopic models regarding microscopic models its computational complexity is also much lower. This results in low simulation times and makes that this type of simulation is suitable for analyzing large geographic areas.

Macrosimulation can be used when we want to analyse a phenomenon with a high amount of elements whose dimensions and descriptive factors are significantly smaller than the area of the phenomenon. These models are suitable when the behaviour of the whole system and the overall trend are more relevant than a detailed analysis of each of the vehicles.

Some of the macroscopic software for traffic simulation that can be found on the market are:

- CUBE
- EMME/2
- FREFLO
- KRONOS
- METACOR
- METANET
- OmniTRANS
- OREMS
- SATURN
- TransCAD
- TRANSYT-7F

- VISUM

3.1.3. Mesoscopic simulation

This chapter is mainly based on [54] and [55].

Mesoscopic models combine features of macro and micro models. According to J. Barceló [54] two different approaches in mesoscopic traffic simulation can be distinguished:

- On the one hand there are mesoscopic models in which traffic flow is modelled as *packages* of vehicles that move dynamically in the network, so that vehicles are not modeled individually.
- On the other hand, some mesoscopic models model traffic flow using simplified dynamics of individual vehicles.

The level of detail considered by this type of models is in an intermediate point between the micro and macro approach, and so is its computational complexity and therefore the simulation time.

Mesoscopic models are suitable for analyzing a phenomenon at an intermediate level between the macro and micro scale. These models do not focus on a traffic situation or in a defined system but they emphasizes the analysis of a group of certain vehicles. The goal of mesoscopic models is to know the reactions of a group of certain vehicles and to pinpoint their location since they enter the network until they leave it.

Some of the mesoscopic software for traffic simulation are listed below:

- CONTRAM
- DYNASMART
- DYNAMIT
- DTASQ
- MEZZO

3.2. PTV Vissim

This chapter is focused on PTV Vissim, the software used to perform the simulation of this Master's Dissertation.

3.2.1. Motivation for choosing PTV Vissim

When deciding which type of traffic simulation model should be used to simulate the roundabout under study, the authors of this thesis thought that using a traffic microsimulation software was the best option for several reasons:

- The object of simulation is basically the roundabout and its surroundings, which makes the simulation network a relatively small area.
- Although the simulation area is small, a high degree of detail is needed in order to simulate aspects such as the exact geometry of the roundabout, speed signaling, zebra crossings and the presence of pedestrians and cyclists, among others.
- The modeling of each vehicle as an independent entity, which is the main feature of microsimulation, makes that the simulation of the roundabout is highly realistic.

For these reasons, the authors began looking for microsimulation software and found PTV Vissim software particularly suitable because, in addition to the aforementioned reasons, the software has a module called VisVAP for logic traffic control.

PTV Vissim is not a free software but its use requires a license. The company PTV Group offers licenses for students wishing to research using such software. Therefore, the authors of this thesis contacted PVT Group to acquire a license for researchers and PTV accepted their proposal. As a consequence, Vissim is the software used to develop the simulation of this Master's Thesis.

3.2.2. About PTV Vissim

This chapter is mainly based on [56], [57] and [58].

PTV Group claim that "PTV Vissim software is the leading microscopic simulation software for modeling multimodal transport operations being used worldwide by the public sector, consulting firms and universities" [56].

Talking about the main outlines of the history of the software, Vissim was originally developed by the University of Karlsruhe (Germany) during the 1970's. In the year 1994, the company PTV started to commercialize the product. Nowadays, PTV and Innovative Transportation Concepts Inc. are working together in order to maintain, develop and distribute the software.

Focusing on technical aspects of the program, "VISSIM is a microscopic, time step and behaviour-based simulation model developed to model urban traffic and public transport operations and flows of pedestrians” [58]. The software can analyse traffic transport operations (related with vehicles and pedestrians) under conditions such as lane configuration, vehicle behaviour, speed limitations, vehicle composition, traffic signal and traffic lights among others.

Table 3.2 shows some of the application areas of PTV Vissim.

Comparison of junction geometry – Model various junction geometries. – Simulate the traffic for multiple node variations. – Account for the interdependency of different modes of transport (motorized, rail, cyclists, pedestrians). – Analyse numerous planning variants regarding level of service, delays or queue length. – Graphical depiction of traffic flows.
Traffic development planning – Model and analyse the impact of urban development plans. – Have the software support you in setting up and coordinating construction sites. – Benefit from the simulation of pedestrians inside and outside buildings. – Simulate parking search, the size of parking lots, and their impact on parking behaviour.
Capacity analysis – Realistically model traffic flows at complex intersection systems. – Account for and graphically depict the impact of throngs of arriving traffic, interlacing traffic flows between intersections, and irregular intergreen times.
Traffic control systems – Investigate and visualize traffic on a microscopic level. – Analyse simulations regarding numerous traffic parameters (for example speed, queue length, travel time, delays). – Examine the impact of traffic-actuated control. – Develop actions to speed up the traffic flow.
Signal systems operations and re-timing studies – Simulate travel demand scenarios for signalized intersections. – Analyse traffic-actuated control with efficient data input, even for complex algorithms. – Create and simulate construction and signal plans for traffic calming before starting implementation. – Vissim provides numerous test functions that allow you to check the impact of signal controls.
Public transit simulation – Model all details for bus, tram, subway, light rail transit, and commuter rail operations. – Analyze transit specific operational improvements, by using built-in industry standard signal priority. – Simulate and compare several approaches, showing different courses for special public transport lanes and different stop locations (during preliminary draft phase). – Test and optimize switchable, traffic-actuated signal controls with public transport priority (during implementation planning).

Table 3.2. Application areas of PTV Vissim [56]

3.2.3. The traffic simulation model

This chapter is mainly based on [56], [58] and [59].

The accuracy of a traffic microsimulation model, as the roundabout network under study, is strongly dependent on the quality of how the vehicles are modelled through the network. Unlike simpler simulation models which employ constant speeds and deterministic logic for tracking vehicles, VISSIM uses the psycho-physical driver behaviour model developed by Wiedemann in 1974.

Figure 3.1 shows a graphical representation of the Wiedemann model. The vertical axis shows the distance between the front and the rear distance of two consecutive vehicles and the horizontal axis shows the difference of velocity between both.

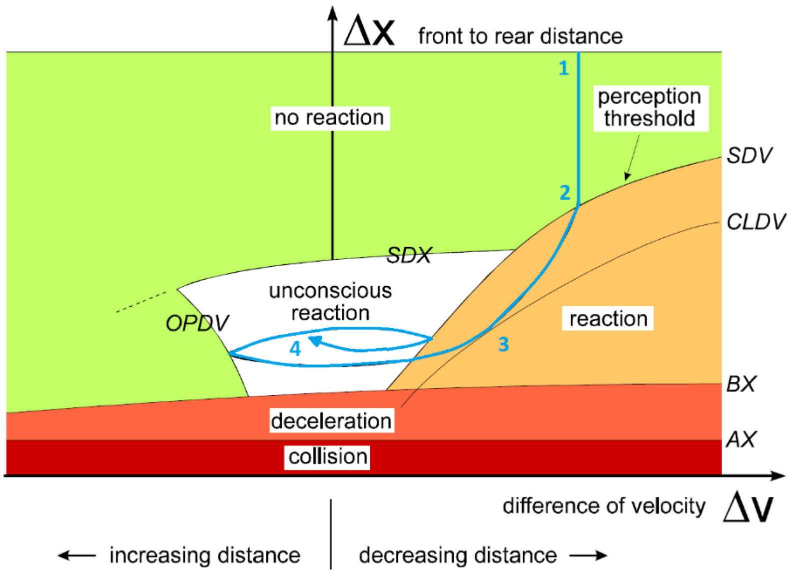


Figure 3.1 Car following logic [58]

Wiedemann model is based on the idea that a driver can be in one of the four driving modes explained below:

- **No reaction (free driving)**
  In this mode the vehicle is not influenced by preceding vehicles, so the driver reaches and maintains his desired speed.
- **Reaction (approaching)**
  In this mode the vehicle starts to be influenced by the preceding vehicle (perception threshold). Therefore, it has to adapt its speed to the speed of the preceding vehicle. As the vehicle is approaching to the preceding vehicle, it applies deceleration so that the speed differential of the two vehicles is zero at the moment that the rear vehicle has reached its desired safety distance.
- **Unconscious reaction (following)**
  This is the mode in which a vehicle follows the preceding car without any conscious acceleration or deceleration. The vehicle simply follows the preceding car trying to maintain the safety distance.
- **Deceleration or collision (braking)**
  This mode happens when the vehicle has to apply moderate or high deceleration rates because the safety distance is under the desired value. For instance, this happens when a preceding vehicle changes abruptly of lane or his speed.

For each mode, the acceleration is described as a result of the following parameters: speed, speed difference, distance to the preceding vehicle, the individual characteristics of the driver and the vehicle characteristics. The vehicle switches from one mode to another as soon as it reaches a certain threshold that can be expressed as a function of speed difference and distance difference.

Looking at the blue arrow in Figure 3.1 the behaviour of a vehicle which is approaching to a slower vehicle (because ΔX decreases) can be seen. Firstly, the vehicle is in the blue straight line between (1) and (2), maintaining a "no reaction" mode (i.e. free driving). Once the SDV perception threshold (2) has been reached the vehicle enters in the reaction area, where it has to reduce its speed. Later it approaches to another threshold (3) (called CLDV) where the vehicle has to reduce the speed even more to enter in the unconscious reaction area. Then the vehicle continues in the unconscious reaction mode as long as it remains between OPDV, SDX and SDV thresholds.

In short, the idea on which this model is based is that a driver starts to decelerate when it reaches his individual perception threshold because he is approaching to a slower vehicle. Since the driver does not know exactly the speed of the slow vehicle, he decreases his speed until he starts to slightly accelerate again after reaching another perception threshold. This leads to an iterative process of deceleration and acceleration.

Moreover, Vissim not only allows vehicles to react to preceding vehicles in a lane, but vehicles on adjacent lanes are also taken into account. Basically, vehicles in PTV Vissim change lane because of one of these two reasons:

- Necessary lane change, in order to fulfil the routing decisions.
- Free lane change, because of better traffic conditions (more room or higher speed, for instance).

When a driver wants to change lane the first step is to find an appropriate gap in the destination flow. In PTV Vissim the gap size depends on the speed of the vehicle wishing to change lanes and also on the speed of the vehicle in the other lane which is approaching to the vehicle wishing to change to that lane.

4. Experimental simulation

In this chapter the process of building the roundabout model in PTV Vissim is described. For the development of this model “Vissim 5.30-05 User Manual” [58] has been used as a support tool.

The main purpose of the model is to simulate the traffic situation in the roundabout as realistically as possible. To achieve this, it is necessary to define a set of parameters and variables in the model.

4.1. Units

For the development of the model and the interpretation of the results metric units are chosen, as it can be seen in Figure 4.1.

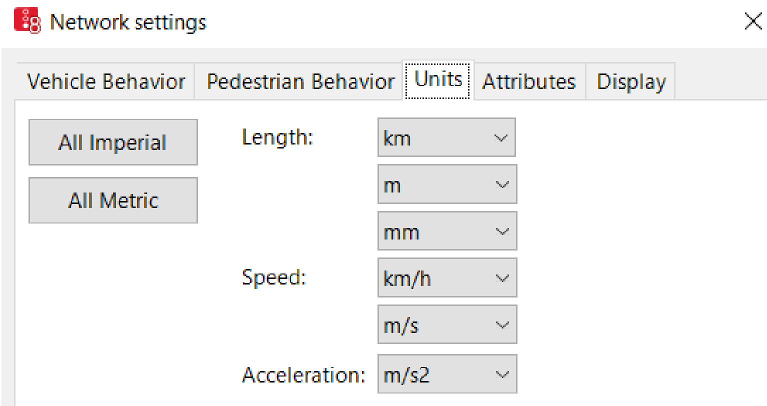


Figure 4.1. Units tab in PTV Vissim

4.2. Using a map as a background

Building a realistic model requires maintaining the geometrical fidelity between the model and reality. This is crucial for the simulation success and also for the correct definition of the geometrical elements of the traffic network.

To model the network of the roundabout and its environment exactly it is useful to use a scaled map as a background. In this case a Google Earth map of the area (which can be seen in Figure 4.2) has been used.

In order to achieve that the background of the model had the real dimensions it is only necessary to adjust the software scale to Google Earth map scale (see Figure 4.3).

Figure 4.2. Background image used [Google Earth]

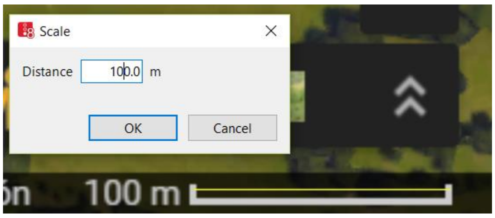


Figure 4.3. Adjusting scales

Once the adjustment of scales is finished the mapping of the network can be started. The network elements maintain the same scale as the image used as a background, so that elements such as the roundabout or roads can be traced exactly.

4.3. Links (roads)

The first step to develop the network traffic in PTV Vissim is to trace the links (i.e. the roads). Each entering and exiting road of the roundabout is represented by one link regardless of the number of lanes that the road had. The different links that constitute the network are listed below.

4.3.1. Viesques links

As it can be seen in Figure 4.4, Viesques entrance has been created as a unique link (yellow line) using the background as a guide. Each yellow point is just where the link (road) changes of curvature.

Figure 4.4. Viesques entrance link

Figure 4.5. Viesques exit link

Figure 4.5 shows Viesques exit link. The only difference between Viesques entrance and Viesques exit is the direction of the traffic. In the entrance vehicles circulate towards the roundabout while in the exit vehicles get away from the roundabout.

Once the link has been drawn it is necessary to edit its properties (see Figure 4.6).

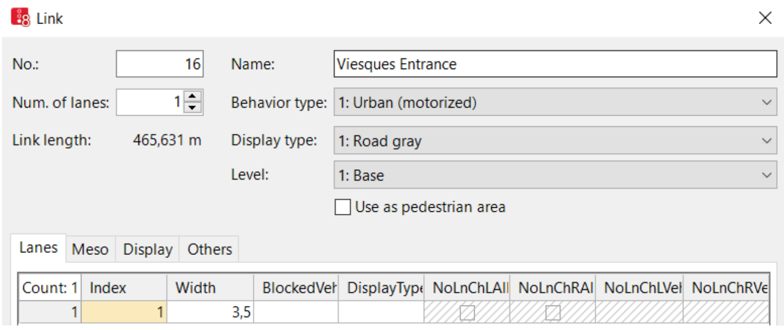


Figure 4.6. Viesques link properties

The properties of the link are listed below:

- **No**: Unique identifier of the link. It can only be edited when the link is created.
- **Name**: Label of the link.
- **Num. of Lanes**: Number of lanes per link direction. (Viesques entrance has only one lane and the same happens with Viesques exist.)
- **Link length**: It shows the length according to the graphic drawing done with the mouse. This value is not subject to changes because it is calculated directly by the software.
- **Behaviour Type**: Selection of the link behaviour type that controls driving behaviour.
- **Display Type**: Selection the link display type that controls texture characteristics.

The most important parameters related to lanes are shown below:

- **Lane Width**: This parameter defines the width of each lane of the link. (In this model the width of each lane is 3.5 m.)
- **Blocked Vehicle Classes:** One or more lanes of the link can be closed to any vehicle class. This fact affects the behaviour of vehicles that are not allowed in the lane as it follows:
  - Changing to that lane (even if it is a consequence of a routing decision) is not allowed.
  - Vehicles are not allowed to enter the lane from a vehicle input unless the vehicle has not allowed access to all lanes of the road (link). In that case the vehicle will enter to that lane but will not change lanes and will try to leave that lane as soon as possible if there is an adjacent lane in which its circulation is allowed.
  - However, vehicles are allowed to move to the lane if coming from a connector.

In Viesques entrance and exit there are no constraints. However, in A8 highway entrance and exits bikes are not allowed to circulate.

4.3.2. Molinón Stadium links

In Figure 4.7 the link which represents the Molinón Stadium entrance can be seen. Figure 4.8 shows the Molinón Stadium exit link.

Figure 4.7. Molinón Stadium entrance link

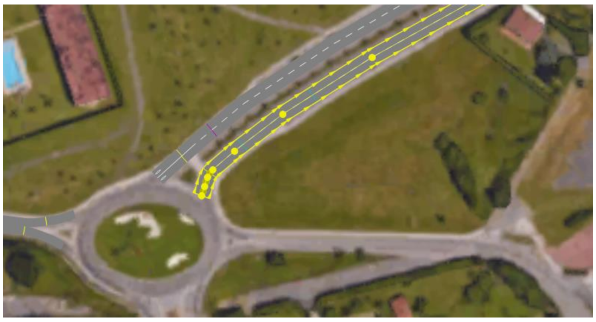


Figure 4.8. Molinón Stadium exit link

Both links have two lanes (see “No. of Lanes: 2” in Figure 4.9) and all kind of vehicles are allowed.

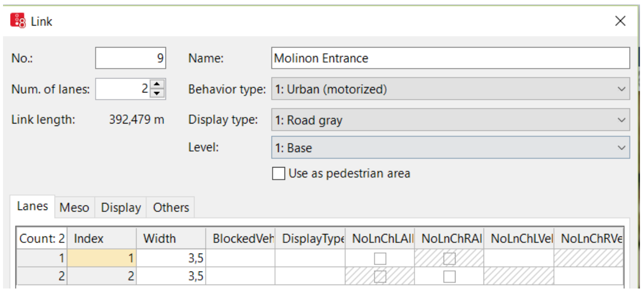


Figure 4.9. Molinón Stadium link properties

4.3.3. Polytechnic School links

Polytechnic School entrance and exit (which can be seen in Figure 4.10) links have been created analogous to Viesques links.

Figure 4.10. Polytechnic School links

4.3.4. Highway links

Two different entrance links can be seen in Figure 4.11. Firstly, there is a link which communicates A8 highway east direction with the roundabout. This link is basically a two-lane road which makes a curve of almost 270 degrees to guide traffic to the roundabout. Secondly, there is a link which connects A8 highway west direction with the roundabout. This link is a one-lane road.

Figure 4.12 also shows two different exit links. Firstly, there is a link which communicates the exit of the roundabout with A8 highway east direction. This is a two-lane road which finishes joining the highway. The second link connects the first link with A8 highway west direction. It is a one-lane road.

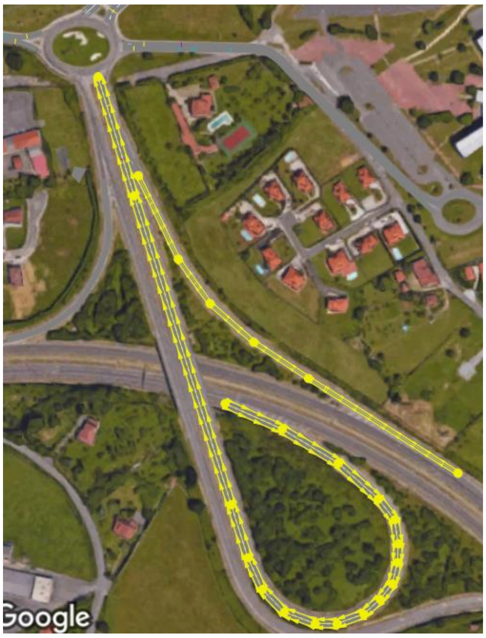


Figure 4.11. Highway entrance links

Figure 4.12. Highway exit links

It should be noted that in neither of the four aforementioned links (i.e. on the highway) is permitted the circulation of bikes. Figure 4.13 shows the implementation of this prohibition.

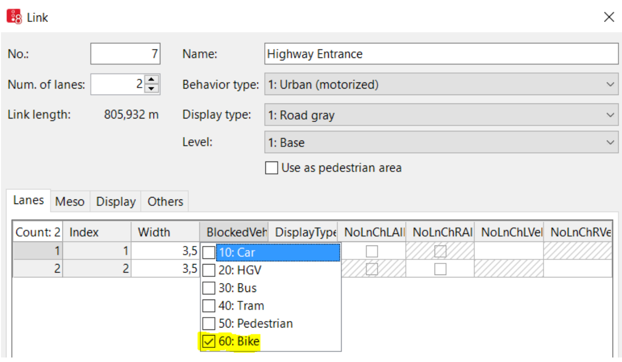


Figure 4.13. Implementation of bikes prohibition in the Highway

4.3.5. Roundabout link

In order to create the link of the circulatory roadway of the roundabout a spline has been employed, as it can be seen in Figure 4.14. Such spline has intermediate points to match the curvature with the background as much as possible.

Figure 4.14. Roundabout link

4.4. Simulating Highway bridge

In A8 Hihgway link there is a flyover pass that has to be simulated. In order to build it, the link which is over the ground has to be placed at the bridge height (7 meters). Figure 4.15 illustrates the flyover structure on the right and the parameters that have to be configured (Z-offset start and Z-offset end 7 meters both) on the left. That means that the link is going to be 7 meters above ground level.

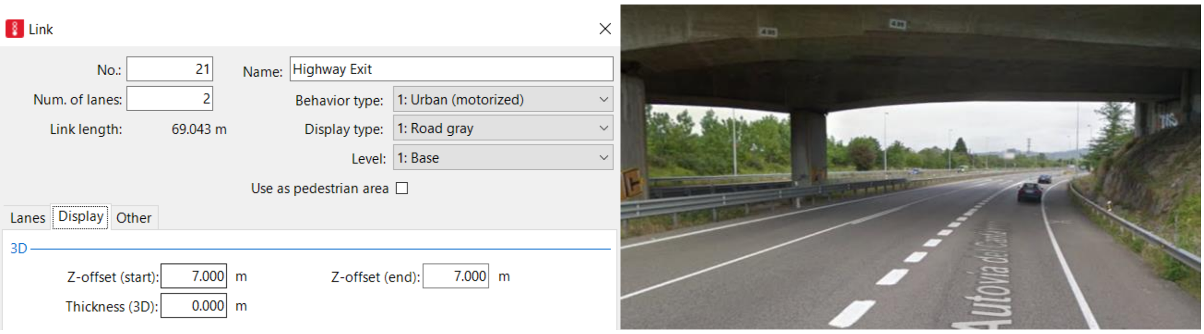


Figure 4.15. Bridge in A8 Highway

4.5. Connectors

With the aim of creating a road network, links (i.e. roads) need to be connected with other links. To connect two links it is not enough to place one link on top of another but a connector needs to be created.

The following attributes can be defined for a connector:

- **Name**: Label which defines the connector.

- **Behaviour type**: Selection of the connector behaviour type that controls driving behaviour.
- **Display type**: Selection the connector display type that controls texture characteristics.
- **From link / To link**: This is the most important parameter in connectors. It defines the assignment of lane(s) of the connector with the lanes of both the start and the destination link. Lane 1 represents the rightmost lane. Multiple lanes can be selected.

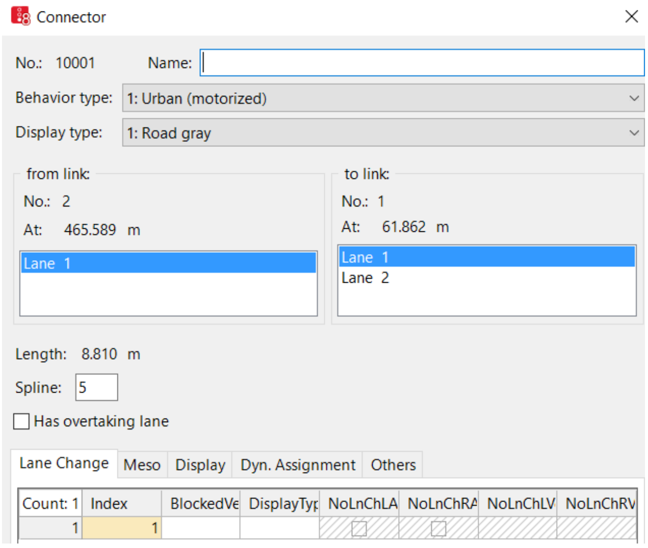


Figure 4.16. Connector parameters

Figure 4.17 shows the main connectors of the roundabout area.

Figure 4.17. Main connectors

Table 4.1 shows all the links and connectors of the model. Also, the number of lanes per link or connector and its length are specified.

Name	Nº lanes	Length [m]
Roundabout Road	2	159,773
Viesques Entrance	1	465,631
Viesques Exit	1	465,407
Polytechnic Exit	1	312,588
Polytechnic Entrance	1	578,783
Polytechnic Way	1	136,834
Highway Entrance	2	805,932
Highway Exit	2	791,13
Molinón Entrance	2	392,479
Molinón Exit	2	420,4
Highway Incorporation	1	372,936
Highway Incorporation 2	1	177,583
Molinón Entrance	2	21,824
Roundabout junction	2	13,202
Viesques Entrance Connector	1	8,81
Viesques Exit Connector	1	8,157
Polytechnic Exit connector	1	10,681
Polytechnic Entrance connector	1	8,933
Polytechnic connector	1	4,131
Highway Entrance connector	2	12,991
Highway Exit connector	2	9,149
Molinón Entrance connector	2	10,837
Molinón Exit Connector	2	12,25
Highway connector	1	45,825
Highway connector	1	35,566
Molinón Entrance	2	1,001

Table 4.1. Links and connectors

4.6. Automobile traffic

4.6.1. Vehicle composition

The vehicle composition represents the mix of vehicle types and it has to be defined prior to the input flow definition. Basically, a vehicle composition consists of a list of one or more vehicle types. To each of these types, a flow percentage and a speed distribution are assigned. In addition, pedestrian flow can be defined as a vehicle composition, but preferably it should be defined as a pedestrian composition.

In a vehicle composition the following parameters can be defined:

- **Vehicle type**: Defines for which vehicle type (car, bus, HGV or bike, among others) is defined the following data.
- **Desired speed**: Definition of the speed distribution that characterizes the circulation of such vehicle type.

- **Relative flow**: It is the relative percentage (proportion) of such vehicle type.

In the simulation model under study two types of vehicle compositions have been defined:

1. **City traffic flow.** This flow has been thought for urban roads (such as Viesques entrance/exit, Polytechnic School entrance/exit and Molinón Stadium entrance/exit). As it can be seen in Figure 4.18, this traffic flow is composed of vehicles such as cars, HGV, buses and bikes. Moreover, the speeds have been defined depending of the type of vehicle. Also, three variety of cars have been created in order to simulate the difference between slow and fast drivers. In relation to relative flow, there is a 90% of cars, 3% of trucks, 4% of buses and 3% of bikes.

Cou	No	Name
1	1	City traffic flow
2	2	Highway traffic flow

Count: 6	DesSpeedDistr	VehType	RelFlow
1	30: 30 km/h	100: Car	0,350
2	40: 40 km/h	100: Car	0,200
3	50: 50 km/h	100: Car	0,350
4	40: 40 km/h	200: HGV	0,030
5	30: 30 km/h	300: Bus	0,040
6	12: 12 km/h	600: Bike	0,030

Figure 4.18. City traffic flow

2. **Highway traffic flow.** This flow has been thought for A8 Highway. Figure 4.19 shows that this traffic flow is composed of the same vehicles that the city traffic flow except of bikes, due to its circulation is not allowed in highways. Moreover, the speeds have been defined considering the speeds allowed in this kind of roads. In relation to relative flow, the cars form a 95%, trucks 2.5% and bus 2.5% of the traffic flow.

Cou	No	Name
1	1	City traffic flow
2	2	Highway traffic flow

Count: 3	DesSpeedDistr	VehType	RelFlow
1	120: 120 km/h	100: Car	0,950
2	100: 100 km/h	200: HGV	0,025
3	100: 100 km/h	300: Bus	0,025

Figure 4.19. Highway traffic flow

4.6.2. Vehicle inputs: traffic volumes

Vehicle inputs have to be defined only at the starting point of network entering lanes. Traffic volumes are defined for each link and each time interval in *vehicles per hour* (even if the time intervals are different from one hour). Vehicles enter the link according to a Poisson distribution within a time interval. If the defined traffic volume exceeds the link capacity the vehicles stay 'stacked' outside the network until there is available space. Moreover, variable time traffic volumes can be defined in PTV Vissim. This is very useful to simulate rush hours.

In a vehicle input the following parameters can be defined (see Figure 4.20):

- **Name**: Optional name of the vehicle input.
- **Link**: It refers to the link where the vehicle input is placed.
- **Volume**: Traffic volume in vehicles per hour.
- **Vehicle Composition**: It represents the mix of vehicle types and speed distribution.

Cou	No	Name	Link	Volume(0)	VehComp(0)
1	1	Highway A8 LD	7: Highway Entrance	800,0	2: Flow Highway traffic

Figure 4.20 Inputs parameters

Table 4.2 shows all the vehicle inputs created in the simulation model.

No	Name	Link	Volume(0)	VehComp(0)
1	Highway A8 LD	7: Highway Entrance	0,0	2: Highway traffic flow
2	Polytechnic 1	5: Polytechnic Entrance	300,0	1: City traffic flow
3	Polytechnic 2	6: Polytechnic Way	500,0	1: City traffic flow
4	Molinon	9: Molinon Entrance	1100,0	1: City traffic flow
6	Viesques	2: Viesques Entrance	1100,0	1: City traffic flow
7	Highway A8 RD	11: Highway Incorporation	300,0	2: Highway traffic flow

Table 4.2. Inputs created for the simulation

4.7. Routing decisions

In the same way that happens in reality, vehicles circulate in the roundabout knowing what exit they are going to take. For this reason it is needed that every vehicle knows its route. For this purpose, PTV Vissim allows routing decisions per vehicle.

PTV Vissim understands that a *route* is "a fixed sequence of links and connectors from the routing decision point to at least one destination point" [58]. Each routing decision point can have several destinations. Moreover, there is the possibility of restricting (for example, the routing decision affects only vehicles of a certain class).

There are different types of routing decisions. However, static decision is chosen for this simulation model. This means that vehicles' route depends on a static percentage for each destination.

Figure 4.21 shows the static routing decision in A8 Highway entrance. From a start point (red) to any of the defined destinations (green), each routing decision point can have multiple destinations. The link sequence is shown as a yellow band from the start point to the defined destinations.

Figure 4.21. Routing decision example

After the definition of all destinations per routing decision it is necessary to define the route properties. The main property to be defined is the relative flow in each destination. As it can be seen in Figure 4.22, which represents the route properties of Highway entrance, the last column of the table shows the relative flow in each exit. That means that all vehicles which go across Highway entrance are going to be divided like this: 15% to Polytechnic exit, 50% to Molinón Stadium exit, 30 % to Viesques exit and 5% to Highway exit.

Count: 4	VehRoutDec	No	Name	DestLink	DestPos	RelFlow(0)
1	1	1		4: Polytechnic Exit	89,472	0,150
2	1	3		10: Molinon Exit	57,825	0,500
3	1	4		3: Viesques Exit	29,699	0,300
4	1	5		8: Highway Exit	25,581	0,050

Figure 4.22. Route properties of Highway entrance

The same procedure has been followed to model the rest of the decision routes.

4.8. Conflict areas

A conflict area can be defined as the place where two links/connectors overlap. For each conflict area, it is necessary to select which of the conflicting links has the right-of-way.

Each driver has to make a plan in order to decide how to cross a conflict area. For example, a vehicle wishing to enter the roundabout must observe the circulating traffic in the roundabout and decide in which available gap he wants to enter the roundabout. Then the driver accelerates or decelerates to enter the roundabout or stop. If there is no available gap the driver has two possibilities: brake or continue circulating trying to enter in another available gap.

Vehicles who have the priority have to react under the road conditions too. For instance, in case that the yielding vehicle could not complete the entrance to the roundabout because the driver estimated the situation too optimistically, the vehicles which circulates in the roundabout have to brake or even to stop to avoid the crash. Moreover, if a queue builds up from a signal downstream the conflict area, the vehicles in the main stream will try not to stop in the conflict area, preventing to block the crossing stream.

Taking into account that a conflict area is created where two links/connectors overlap, the conflicts areas of the roundabout (yellow section) are shown in Figure 4.23.

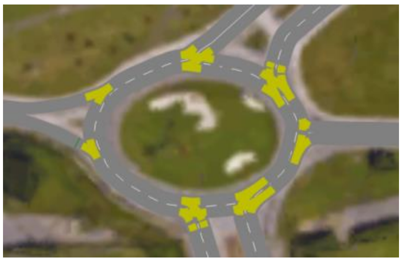


Figure 4.23. Conflict areas of the roundabout by default

By default, PTV Vissim draws yellow the conflict areas, which means that all movements yield. However, there are other ways to manage the conflict areas depending on the colours:

- Road in green: main road (right of way)
- Road in red: minor road (yield)

Figure 4.24 shows how the conflict areas of the roundabout have been managed. Vehicles in the circulatory roadway have the priority whereas vehicles in all the entrances have to yield.

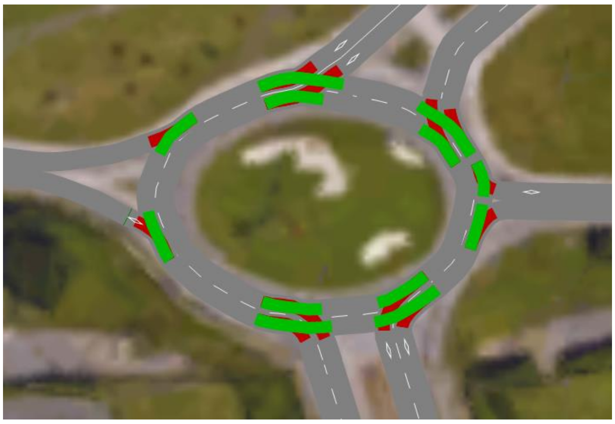


Figure 4.24. Conflict areas of the roundabout

There are several element such as the visibility and the gap to manoeuvre which are relevant for some conflict situations. This attributes affect the calculation of the plan for each vehicle approaching the conflict area.

4.8.1. Visibility

It is the maximum distance from where an approaching vehicle in the minor road can see vehicles on the other link so that the vehicle can plan the driving in advance. For example, Figure 4.25 shows an image of Polytechnic School entrance in which the yellow painted road area indicates the point in which a driver starts to see the vehicles in the circulatory roadway of the roundabout (because of the tree). On the other hand, the red area represents where the driver is not able to see what happens in the left side of the circulatory roadway.

Figure 4.25. Visibility of Polytechnic School entrance [Google Earth]

Figure 4.26 shows how to measure the visibility distance until the crossing point using Google Earth. In this case, the visibility distance is 33 m.

Figure 4.26. Visibility distance measurement in Polytechnic School entrance [Google Earth]

Figures 4.27, 4.28 and 4.29 show that there are no obstacles which make the visibility difficult in Highway, Viesques and Molinón entrances.

Figure 4.27. Highway entrance [Google Earth]

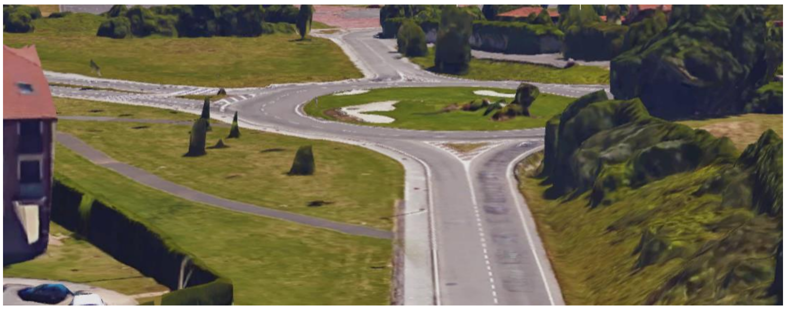


Figure 4.28. Viesques entrance [Google Earth]

Figure 4.29. Molinón Stadium entrance [Google Earth]

It makes no sense for a driver to plan how to enter the roundabout when he is far away from it because other vehicles can go into the roundabout during the time that the driver spends to reach the yield line. For that reason, a visibility of 50 meters has been considered in all the entrances except in Polytechnic entrance (33 m).

Figure 4.30 shows an image of the roundabout in which the colours mean:

- **Green**: Accurate visibility.
- **Orange**: Uncertain visibility.
- **Red**: No visibility.

Figure 4.30. Entrance visibility

4.8.2. Gap to manoeuvre

This chapter is partly based on [57].

At roundabouts drivers have to decide when it is safe to merge into the circulating traffic. Entering vehicles have to look for an available gap in the major stream in which vehicles have the priority. Figure 4.31 shows a graphical description of a gap, which is usually measured in seconds. If a driver enters the roundabout in a gap then the gap is *accepted*. In the opposite case, it is referred to as a r*ejected* gap. If a vehicle in the main stream has to reduce its speed or change lane the gap is categorised as a *forced* gap. Otherwise it is an *ideal* gap.

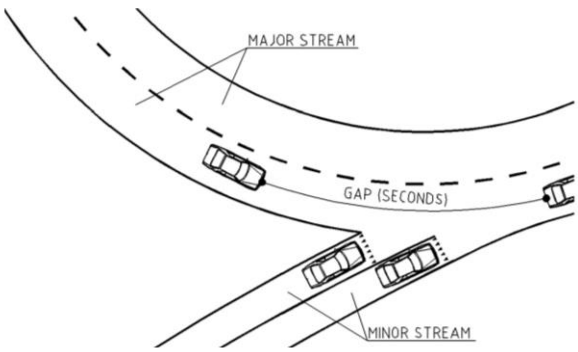


Figure 4.31. Graphical description of a gap [57]

When a driver is waiting for a gap he usually rejects some gaps. The sizes of the accepted gaps and the rejected gaps for different drivers do not provide information about the smallest gap they would accept or the biggest gap they would reject. Therefore, it is necessary to calculate a *critical gap*. In roundabouts, the critical gap is the smallest gap that a driver is willing to use to enter into the circulating traffic. All gaps less than the critical gap would be rejected and all gaps greater than or equal to the critical gap would be accepted.

To determine how long the critical gap should be last, this model has been based on the article "Calibrating of gap times for VISSIM software" [60]. This article analyzes which critical time gap is advisable to define in the software PTV Vissim. In order to estimate this critical time gap the authors studied several intersections of the Czech Republic from June to December 2010. This study consisted of videotaping these intersections and then analyzing them by using a computer. Figure 4.32 shows the obtained results.

Roundabouts one or two lanes on entrance / roundabout				
Prefers	Opposite direction	Measured time	Measured speed	VISSIM time
One lane entrance	One lane roundabout	3.9	26.2	3.3
Right entrance lane	Right roundabout lane	3.7	29.9	3.2
Left entrance lane	Right roundabout lane	3.6	29.9	3.1
	Left roundabout lane	3.8	31.5	3.3

Figure 4.32. Recommended critical time gaps (in seconds) for PTV Vissim [60]

It is generally known that there is a high variance in individual time gap values. While relatively small gaps are sufficient for some drivers to perform their manoeuvre, others need more amount of time to perform the movement. Unfortunately PTV Vissim does not allow for including these effects into the simulation. For this reason, this model only takes into account the average value of critical time gap.

In PTV Vissim two types of gaps can be defined (see Figure 4.33).

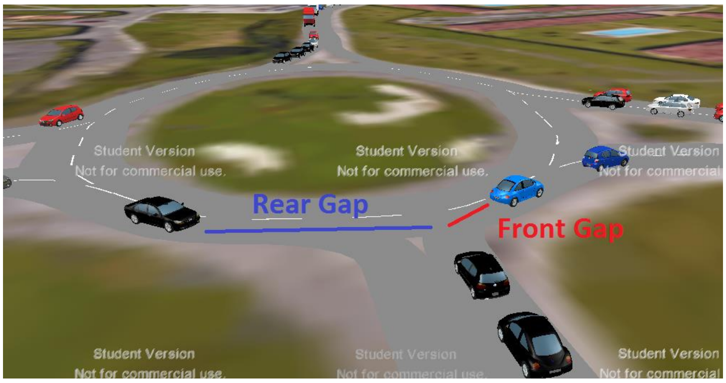


Figure 4.33. Rear gap and front gap in PTV Vissim

- **Front gap**.
  PTV defines the front gap as "the minimum gap in seconds between the rear end of a vehicle on the main road and the front end of a vehicle on the minor road" [58].

A driver in an entering lane wishing to enter the roundabout has to decide when it is the precise moment to start the movement. Once the driver sees that the vehicle on the circulatory roadway (main road) has already crossed the conflict area he starts the movement (if and when the rear gap was enough).

To determine the front gap the authors of this thesis propose to calculate it as the time in which the vehicle in the main road travels a distance equal to its length.

It is supposed that the average measured speed of vehicles on the roundabout is 30 km/h. Also, the length of a vehicle vary between 2 m (bike) and almost 19 m (the largest trucks) so an average size of 10 meters is considered.

$$\bar{t} = \frac{\bar{L}}{v} = \frac{10\ m}{30\ km/h} = 1.2\ s$$

Therefore, the front gap in this model has an average value of 1.2 seconds.

- **Rear gap.**
  The rear gap "is the minimum gap in seconds between the rear end of a vehicle on the minor road and the front end of a vehicle on the main road" [58]. Trying to be conservative and taking into account the Figure 4.32, a value slightly higher is set for the rear gap in this model: 4.5 seconds.

4.8.3. Other parameters for conflict areas

Other parameters which can be defined in PTV Vissim in conflict areas are listed below.

- **Safety distance factor.**
  It is a value that "multiplies the normal desired safety distance of a vehicle on the main road to determine the minimum headway that a vehicle from a minor road must provide at the moment when it is completely inside the merging conflict area" [58]. For each vehicle class, a separate factor can be set. In this model a safety distance factor of 4 has been considered.
- **Additional stop distance.**
  This parameter affects only the vehicles in the minor road of an intersection. It is the imaginary distance that the stop line is moved upstream of the conflict area. Therefore, "yielding vehicles stop further away from the conflict and also need to travel a longer distance until they pass the conflict area" [58]. In a roundabout yielding vehicles stop just at the beginning of the conflict area. For that reason, it is not necessary to add stop distance.
- **Observe adjacent lanes.**
  When this option is activated "the incoming vehicles on the minor road pay attention to the vehicles on the prioritized link which are going to change to the conflicting lane" [58]. This option must be activated in a roundabout due to safety reasons. A driver who is looking for a gap in the outside lane of the circulatory roadway not only has to

be focused in this lane but also in the inside lane. That is because drivers in the inside lane of the circulatory roadway have priority in relation to the entering drivers.

- **Anticipate routes**.
  According to PTV, “this factor describes the percentage of incoming vehicles on the minor road which consider the routes of those approaching vehicles on the main road that will turn at an upstream position and thus will not reach the conflict area” [58].
  Some of the drivers who are waiting in the entering lane for an available gap identify the cars which are going to take just the previous exit by seeing their turn signal. Although it is very difficult to determine what percentage of drivers do this, such anticipated behaviour could increase the capacity of the roundabout. Due to the lack of formal studies about such percentage only a 5% of the drivers are considered to have this skill. In this way the authors of this thesis are conservative and the capacity of the roundabout is not increased without having sufficient certainty.
- **Avoid blocking**.
  This factor “describes the percentage of vehicles on the main road which will not enter the crossing conflict area as long as they cannot expect to clear it immediately” [58].
  In PTV Vissim the prioritized vehicle will not enter the conflict area if:
  - The room downstream of the conflict area is less than the length of such plus 0.5 metres.
  - “The blocking vehicle is slower than 5 m/s and slower than 75% of its desired speed” [58].

  Unfortunately, not all the drivers behave in this way and avoid the block of the road. In the model under study it has been defined that only 40% of the drivers act to avoid blocking.

Table 4.3 shows all the parameters explained in this section related with conflict areas which have been defined in the model.

Link1	VisibLink1	Link2	VisibLink2	Status	FrontGapDef
1: Roundabout Road	50,0	10001: Viesques Entrance Connector	10,0	2 waits for 1	1,2
1: Roundabout Road	50,0	10002: Viesques Exit Connector	10,0	2 waits for 1	1,2
3: Viesques Exit	50,0	10002: Viesques Exit Connector	10,0	Passive	1,2
1: Roundabout Road	50,0	10003: Polytechnic Exit connector	10,0	2 waits for 1	1,2
1: Roundabout Road	50,0	10004: Polytechnic Entrance connecto	10,0	2 waits for 1	1,2
5: Polytechnic Entrance	33,0	10005: Polytechnic connector	10,0	2 waits for 1	1,2
1: Roundabout Road	50,0	10006: Highway Entrance connector	10,0	2 waits for 1	1,2
1: Roundabout Road	50,0	10007: Highway Exit connector	10,0	2 waits for 1	1,2
8: Highway Exit	50,0	10007: Highway Exit connector	10,0	Passive	1,2
1: Roundabout Road	50,0	10008: Molinon Entrance connector	10,0	2 waits for 1	1,2
1: Roundabout Road	50,0	10009: Molinon Exit Connector	10,0	2 waits for 1	1,2
10: Molinon Exit	50,0	10009: Molinon Exit Connector	10,0	Passive	1,2
7: Highway Entrance	50,0	10010: Highway connector	10,0	2 waits for 1	1,2
8: Highway Exit	50,0	10011: Highway connector	10,0	2 waits for 1	1,2
12: Highway Incorporation 2	50,0	10011: Highway connector	10,0	Passive	1,2
10000: Roundabout junction	50,0	10009: Molinon Exit Connector	10,0	Passive	1,2

Link1	SafDistFactDef	AddStopDist	ObsAdjLns	AnticipRout	AvoidBlock	RearGapDef
1: Roundabout Road	4,0	0,0	☑	5,0 %	40,0 %	4,5
1: Roundabout Road	4,0	0,0	☑	5,0 %	40,0 %	4,5
3: Viesques Exit	4,0	0,0	☑	5,0 %	40,0 %	4,5
1: Roundabout Road	4,0	0,0	☑	5,0 %	40,0 %	4,5
1: Roundabout Road	4,0	0,0	☑	5,0 %	40,0 %	4,5
5: Polytechnic Entrance	4,0	0,0	☑	5,0 %	40,0 %	4,5
1: Roundabout Road	4,0	0,0	☑	5,0 %	40,0 %	4,5
1: Roundabout Road	4,0	0,0	☑	5,0 %	40,0 %	4,5
8: Highway Exit	4,0	0,0	☑	5,0 %	40,0 %	4,5
1: Roundabout Road	4,0	0,0	☑	5,0 %	40,0 %	4,5
1: Roundabout Road	4,0	0,0	☑	5,0 %	40,0 %	4,5
10: Molinon Exit	4,0	0,0	☑	5,0 %	40,0 %	4,5
7: Highway Entrance	4,0	0,0	☑	5,0 %	40,0 %	4,5
8: Highway Exit	4,0	0,0	☑	5,0 %	40,0 %	4,5
12: Highway Incorporation 2	4,0	0,0	☑	5,0 %	40,0 %	4,5
10000: Roundabout junction	4,0	0,0	☑	5,0 %	40,0 %	4,5

Table 4.3. Parameters of conflict areas

4.9. Speed limit

A *desired speed decision* is defined in PTV Vissim at a location where a permanent speed change should become effective. Each vehicle gets a new speed once it crosses over the desired speed decision. Only then the vehicle reacts to the new speed (either by acceleration or deceleration according to the speed decision). The typical application of desired speed decisions is the simulation of a speed limit sign in reality.

As it can be seen in Figure 4.34, in Viesques entrance there is a speed limit sign which forbids that vehicles circulate faster than 40 km/h. Once vehicles cross over the sign drivers must obey this speed limit until the existence of other speed limit. In order to simulate this situation, it is necessary to create a desired speed limit sign in the model (see Figure 4.35).

Figure 4.34. Speed limit sign in Viesques entrance [Google Earth]

For each type of vehicle the desired speed needs to be defined. In Viesques entrance the speed decisions affects cars, trucks and bus due to bikes' speed is lower than 40 km/h.

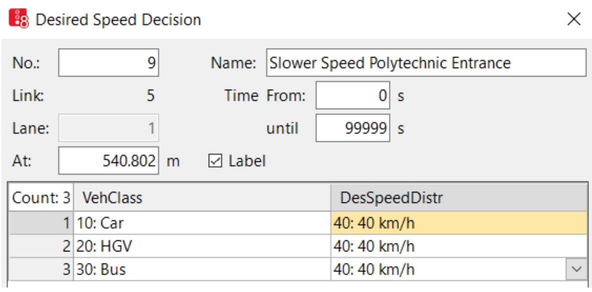
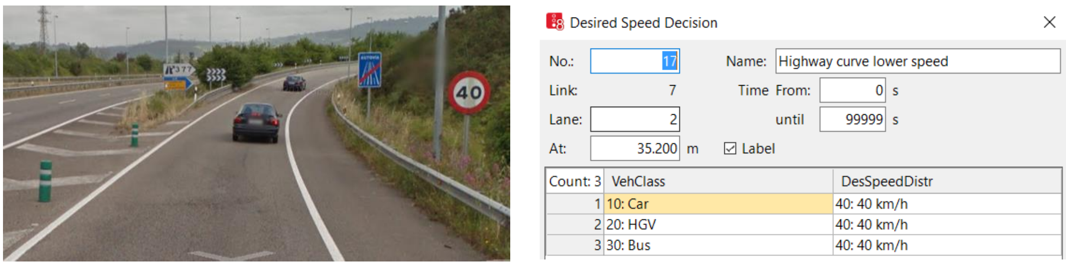


Figure 4.35. Desired speed decision

In order to simulate the vehicles behaviour as realistic as possible, the simulation model implements all the speed limit signs of the network. For example, Figure 4.36 shows all the speed limit signs of A8 Highway entrance.



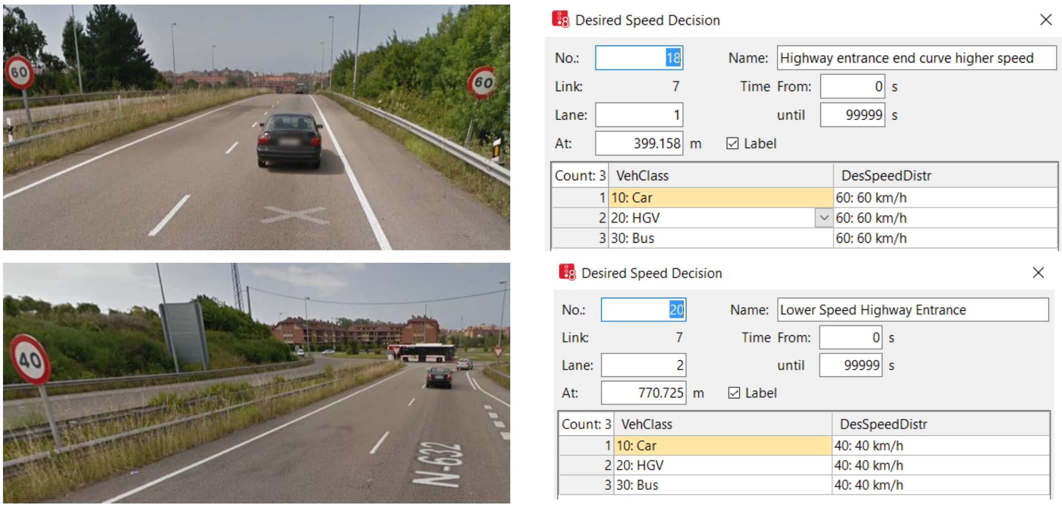


Figure 4.36. Speed limit signs and desired speed decision window of Highway entrance

4.10. Pedestrians

In PTV Vissim the movement of pedestrians is based on the physical study “Social force model for pedestrian dynamics” [61]. Indeed, one of the authors of the aforementioned study, Prof. Dr. Dirk Helbing from ETH Zurich, collaborates with PTV as scientific advisor. In this study it is proposed that the movement of pedestrians can be seen as if they were subject to *social forces*. According to such study, these forces have no direct origin in the personal environment of pedestrians but they are measures of internal motivations that drive people to carry out certain actions (in this case movements) [61].

Figure 4.37 shows why the simulation of pedestrian must be implemented in the model. The reason is that a zebra crossing, which affects the behaviour of the vehicles, is located in Molinón entrance an exit.

Figure 4.37. Zebra crossing in Molinón Stadium entrance in the reality (left) and in the model (right)

4.10.1. Pedestrian compositions

Different types of pedestrians (such as women, men, children and wheelchair users) can be grouped to form classes of pedestrians similar to vehicle classes. In the model under study a composition of pedestrians called “Roundabout Pedestrian Flow” has been created, which is shown in Table 4.4.

PedType	DesSpeedDistr	RelFlow
100: Man	1002: 2.88 km/h (0.	0,100
100: Man	1008: 4.75 km/h (1.	0,040
100: Man	1010: 5.51 km/h (1.	0,300
200: Woman	1002: 2.88 km/h (0.	0,100
200: Woman	1008: 4.75 km/h (1.	0,040
200: Woman	1010: 5.51 km/h (1.	0,300
250: Woman	1009: 4.93 km/h (1.	0,100
300: Wheelc	1001: 2.09 km/h (0.	0,020

Table 4.4. Pedestrian Composition

In Table 4.4 the first column shows the name of each type of pedestrian. The second column displays the speed of each type (note that one pedestrian type can have different speed values, as happens in reality). The last column is the relative flow of each pedestrian type.

4.10.2. Links and conflict areas

This section deals with links carrying pedestrian flows for modelling the interactions of vehicular and pedestrian traffic. There are some differences between vehicle and pedestrian links. For instance, pedestrian links are not defined by direction. Therefore, pedestrians can move in the link independently of the direction and it is not necessary to build two ways in the link. Figure 4.38 shows the pedestrian links created in the model.

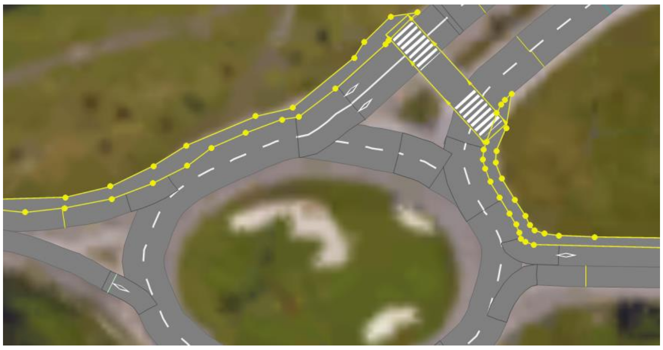


Figure 4.38. Pedestrian links

Conflict areas appear when interactions between vehicles and pedestrians take place, in the same way that conflicts areas appear when two vehicle links overlap. Therefore, for each conflict area it is necessary to select which of the conflicting links has the right of way. Moreover, interactions between vehicles and pedestrians can be handled by the means of signal control.

The zebra crossing under study does not have any type of traffic light so pedestrians are the ones who have right of way and vehicles have to yield them. In order to simulate this behaviour between pedestrian and vehicles it is necessary to set two aspects. Firstly, as Figure 4.39 shows, the main and the minor road are defined. The main road is the zebra crossing (in green) and the minor road is the vehicular road (in red). Secondly, as it can be seen in Table

4.5, the parameters of the conflict area are configured (in the same way as conflict area parameters were explained).

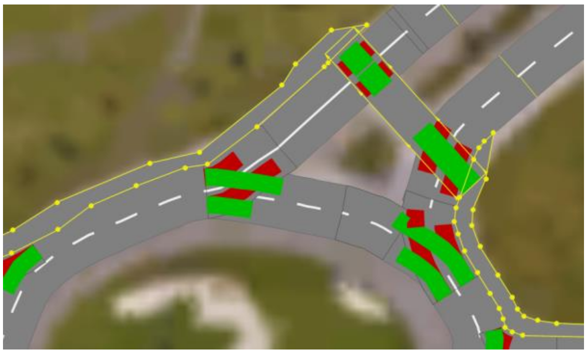


Figure 4.39. Conflict areas of the zebra crossing

Link1	VisibLink1	Link2	VisibLink2	Status	FrontGapDef	RearGapDef
10: Molinon Exit	50,0	15: Pedestrian zebra crossing	50,0	1 waits for 2	20,0	20,0
13: Molinon Entrance	50,0	15: Pedestrian zebra crossing	50,0	1 waits for 2	20,0	20,0
10: Molinon Exit	50,0	14: Pedestrian zebra crossing	50,0	1 waits for 2	20,0	20,0
13: Molinon Entrance	50,0	14: Pedestrian zebra crossing	50,0	1 waits for 2	20,0	20,0

Link1	Link2	SafDistFactDef	AddStopDist	ObsAdjLns	AnticipRout	AvoidBlock
10: Molinon Exit	15: Pedestrian zebra crossing	5,0	3,0	☑	0,0 %	100,0 %
13: Molinon Entrance	15: Pedestrian zebra crossing	5,0	3,0	☑	0,0 %	100,0 %
10: Molinon Exit	14: Pedestrian zebra crossing	5,0	3,0	☑	0,0 %	100,0 %
13: Molinon Entrance	14: Pedestrian zebra crossing	5,0	3,0	☑	0,0 %	100,0 %

Table 4.5. Parameters of pedestrian conflict areas

4.10.3. Inputs

For a selected walkable space or pedestrian links inputs of pedestrians can be defined and edited similar to vehicle inputs. For these inputs of pedestrians PTV Vissim creates pedestrians at random points in time according to pedestrian compositions and input volumes.

In order to create the pedestrians inputs it is required to define the areas where the inputs of pedestrian take place. Figure 4.40 illustrates a simulation capture where the two green squares are the selected area on which pedestrian inputs are placed.

Figure 4.40. Pedestrian inputs

For each pedestrian input (pedestrian Viesques input and pedestrian Polytechnic School input) several parameters can be defined (see Figure 4.41).

- **Input Name**: Optional name of the input of pedestrians. It refers to all data entered for the same area.

- **Volume**: It represents the number of pedestrians per hour. The amount of pedestrians generated in both inputs are 200 pedestrian/hour.
- **Composition section**: It represents the mix of pedestrian types and speed distribution. The composition “Roundabout Pedestrian Flow” previously defined is selected in both inputs.

Name	Volume(0)	PedComp(0)
Pedestrians Viesques Input	200,0	11: Roundabout Pedestrian Flow
Pedestrians Polytechnic School Input	200,0	11: Roundabout Pedestrian Flow

Figure 4.41. Pedestrian inputs parameters

4.10.4. Routes

Routing decisions for pedestrians can be defined similar to routing decisions for vehicles. A pedestrian route is a fixed sequence of areas that starts at the routing decision area (red dot) and ends at a destination area (green dot). Moreover, each routing decision point can have multiple destinations resembling a tree with multiple branches [58].

The pedestrian routing type used is the “Static Route”, which means that pedestrian routes from a start area (red dot) to one of the defined destinations (blue dot) are made using a static percentage for each destination. Figure 4.42 shows a red line from the red dot to the blue dot indicating the shortest route followed by pedestrians.

Figure 4.42. Pedestrians routing decision

However, it should be noted that this shortest route (red line) cannot be taken by pedestrian because there are not pedestrian links. Pedestrians can only walk on dedicated pedestrian area (like the yellow ones represented in Figure 4.42). Where these walkable spaces overlap pedestrians can pass from one space to the other. As it was discussed before, these areas have no direction and pedestrians follow the routing decisions taking into account the “Social Force Model For Pedestrian Dynamics” [61].

4.11. Simulation parameters

4.11.1. Time intervals

The simulation period starts at 6:30 am and finishes at 10:30 pm, making a total of 16 hours of simulation per day. The warm-up period is 30 minutes (between 6:30 am and 7:00 am). Overall, in the roundabout there are three peaks of traffic:

- At the beginning of the morning (8:00 am – 9:30 am).
- In the afternoon (2:00 pm – 3:30 pm).
- In the evening (6:30 pm – 9:00 pm).

The simulation model is split up in time intervals of 30 minutes (1800 seconds) in which the volume of vehicles in each entry of the roundabout changes.

The simulation is composed of a variable number of replicas (see Chapter *5.5. Determining sample size. Results validation*) of 16 hours each replica. Each replica is run using a different random seed in order to change the profile of the arriving traffic. Finally, the arithmetic mean of the results of this multiple simulation with different random seeds is calculated.

Table 4.6 shows the time intervals used by the simulation and how the traffic volume varies during the day. The last six columns represent the traffic volume (in vehicles per hour) of the different vehicle inputs of the roundabout. Red cells indicate the peak hours.

Interval Number	Interval Seconds	Time Interval	Traffic Volume Per Input [Vehicles/hour]					
			Polytechnic		Molinón	Viesques	Highway	
			Poly.1	Poly.2			High.L	High.D
1	0	6:30:00-7:00:00	40	40	160	120	200	70
2	1800	7:00:00-7:30:00	50	50	200	160	300	100
3	3600	7:30:00-8:00:00	60	60	350	200	400	140
4	5400	8:00:00-8:30:00	70	70	500	300	500	150
5	7200	8:30:00-9:00:00	75	75	600	400	600	140
6	9000	9:00:00-9:30:00	60	60	550	350	550	140
7	10800	9:30:00-10:00:00	60	60	450	350	500	130
8	12600	10:00:00-10:30:00	60	60	300	300	450	90
9	14400	10:30:00-11:00:00	60	60	200	300	450	90
10	16200	11:00:00-11:30:00	60	60	200	300	400	80
11	18000	11:30:00-12:00:00	60	60	200	300	350	80
12	19800	12:00:00-12:30:00	70	70	200	300	350	90
13	21600	12:30:00-13:00:00	100	100	200	300	400	90
14	23400	13:00:00-13:30:00	120	120	300	300	450	100
15	25200	13:30:00-14:00:00	140	140	400	300	500	100
16	27000	14:00:00-14:30:00	220	240	500	350	550	125
17	28800	14:30:00-15:00:00	260	270	600	400	650	130
18	30600	15:00:00-15:30:00	220	230	530	400	500	120

Interval Number	Interval Seconds	Time Interval	Traffic Volume Per Input [Vehicles/hour]					
			Polytechnic		Molinón	Viesques	Highway	
			Poly.1	Poly.2			High.L	High.D
19	32400	15:30:00-16:00:00	200	200	500	350	400	100
20	34200	16:00:00-16:30:00	160	160	400	300	350	90
21	36000	16:30:00-17:00:00	90	90	300	250	300	70
22	37800	17:00:00-17:30:00	60	60	250	200	250	60
23	39600	17:30:00-18:00:00	120	80	300	280	300	80
24	41400	18:00:00-18:30:00	150	120	350	300	350	110
25	43200	18:30:00-19:00:00	160	140	450	380	370	130
26	45000	19:00:00-19:30:00	170	145	550	400	380	140
27	46800	19:30:00-20:00:00	170	150	600	450	410	150
28	48600	20:00:00-20:30:00	140	150	600	400	400	130
29	50400	20:30:00-21:00:00	120	120	450	350	350	110
30	52200	21:00:00-21:30:00	80	80	400	320	250	80
31	54000	21:30:00-22:00:00	30	50	300	250	200	70
32	55800	22:00:00-22:30:00	20	20	200	200	180	60

Table 4.6. Time intervals and vehicle Inputs

4.11.2. Other simulation parameters

The parameters of the simulation must be configured once the model is finished. Figure 4.43 shows the simulation parameter window.

- **Comment:** Sentence which identifies the simulation replica.
- **Period:** This is the period of time which is simulated. In this model the simulation lasts 55800 seconds (31 intervals of 1800 seconds each, because the warm up period is not included).
- **Start Time**: It is the time that the clock shows at the beginning of the simulation.
- **Start Date:** This parameter "can be used to specify a date for signal controllers that have a date-dependent logic. This date is passed to the controller DLL" [58].
- **Simulation Resolution**: This is "the number of times the vehicle's position will be calculated within 1 simulated second (range 1 to 10)" [58]. The more movements per second, the

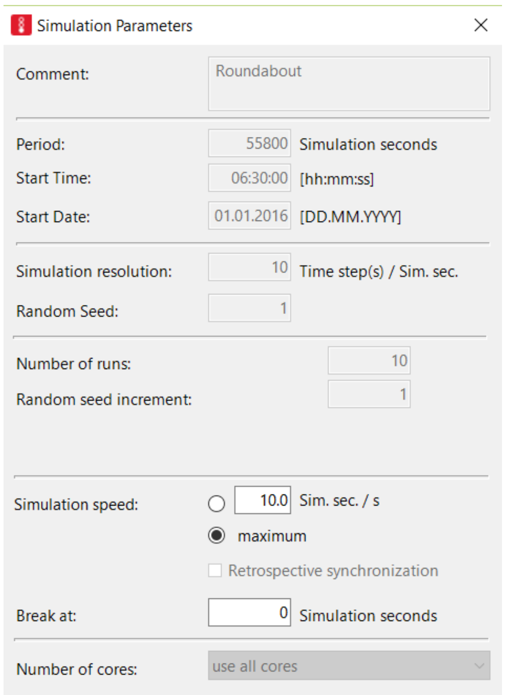


Figure 4.43. Simulation parameters

smoother vehicles move. PTV recommends 3 or more time steps per simulation second.

- **Random Seed:** This is the parameter which "initialises the random number generator" [58]. It should be highlighted that simulation replicas which have identical input files and identical random seeds generate identical results. For that reason, it is necessary to use different random seeds in order to change arriving traffic profile and, thus, to change results. Using random seeds allow to simulate a stochastic variation of inputs of vehicles. For obtaining meaningful results PTV recommends "to calculate the arithmetic mean based on the results of multiple simulation runs with different random seed settings" [58].
- **Number of runs:** Number of replicas.
- **Simulation Speed:** It is the number of seconds simulated which represents a real time second. This parameter is useful because basically it controls the simulation time so that "if *maximum* is selected, the simulation will run as fast as possible" [58]. It should be noted that the change of the value of this parameter does not affect the results.
- **Break at:** "After reaching the time entered here, VISSIM automatically switches into the Single Step mode. This option can be used to view traffic conditions at a certain time during the simulation without having to watch the simulation all the time before" [58].
- **Number of cores:** This parameter allows to select the number of processor cores that are used during simulation, which depends on the hardware of the computer used.

5. Management of results

In this chapter it is analysed what is measured during the simulation and how to obtain the results.

5.1. Travel time

In order to measure the time spent by a vehicle from a point of the network to another point *travel time sections* have been implemented in the model.

Each travel time section consists of a start and a destination cross section. The average travel time is determined as the time spent by a vehicle between crossing the start section and the destination section.

Travel time sections have been implemented in all the entrances of the roundabout to know the time spent by a vehicle from the beginning of an entry road (the vehicle's input point) to the yield line of that road at the roundabout. Figure 5.1 shows the start section (red line) and the destination section (green line) in Viesques entrance.

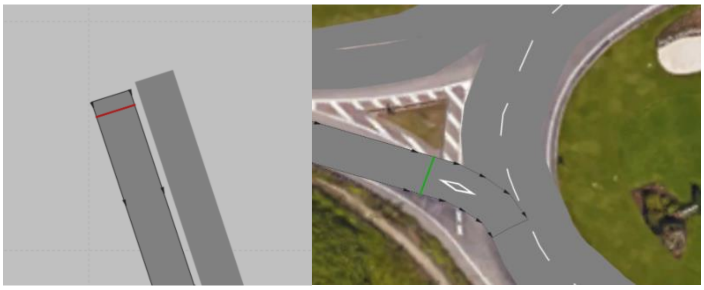


Figure 5.1. Travel time section in Viesques entrance

Polytechnic and Highway entrance are formed of two links each. Therefore, the travel times in these sections have been measured independently. As it can be seen in Table 5.1, Polytechnic entrance travel time sections are formed by Polytechnic Marina and Polytechnic Aularios, whereas Highway entrance travel times are formed by Highway R and Highway L.

Table 5.1 not only represents travel time sections but also the length of the travel section (in the last column). That data could be interesting because once the travel time and the distance are known, the average speed can be easily calculated.

Name	StartLink	StartPos	EndLink	EndPos	Dist
Viesques	2: Viesques Entrance	1.358	10001: Viesques Entrance Connector	19.772	838.41
Molinon	9: Molinon Entrance	1.867	13: Molinon Entrance	19.071	846.76
Polytechnic Marina	6: Polytechnic Way	0.724	10004: Polytechnic Entrance connecto	4.675	1688.65
Polytechnic Aularios	5: Polytechnic Entrance	2.009	10004: Polytechnic Entrance connecto	3.987	1309.53
Highway R	11: Highway Incorporation	2.775	16: Highway Entrance	159.999	1264.66
Highway L	7: Highway Entrance	1.542	16: Highway Entrance	159.280	1517.46

Table 5.1. Travel times sections in the entrances

Table 5.2 shows an example of the results obtained from the first 7200 seconds of the simulation. It should be noted that the results are divided in intervals of 1800 seconds due to the simulation being divided in that intervals too. The first column indicates the simulation replica. The second column represents the time interval in seconds. The third column shows the name of the entrance where the travel time is measured. The fourth column is the number of vehicles which have crossed this particular travel time section. Finally, the last column shows the average time (in seconds) spent by all the vehicles in this travel time section during each time interval.

SimRun	TimeInt	VehicleTravelTimeMeasurement	Vehs(All)	TravTm(All)
1	1800-3600	1: Viesques	89	102,03
1	1800-3600	2: Molinon	90	104,04
1	1800-3600	3: Polytechnic Marina	18	172,69
1	1800-3600	4: Polytechnic Aularios	19	153,62
1	1800-3600	5: Highway R	51	63,62
1	1800-3600	6: Highway L	123	135,13
1	3600-5400	1: Viesques	109	126,37
1	3600-5400	2: Molinon	175	127,75
1	3600-5400	3: Polytechnic Marina	36	174,64
1	3600-5400	4: Polytechnic Aularios	39	148,76
1	3600-5400	5: Highway R	66	68,67
1	3600-5400	6: Highway L	184	144,10
1	5400-7200	1: Viesques	221	290,79
1	5400-7200	2: Molinon	215	143,62
1	5400-7200	3: Polytechnic Marina	31	236,08
1	5400-7200	4: Polytechnic Aularios	40	146,37
1	5400-7200	5: Highway R	54	309,14
1	5400-7200	6: Highway L	185	365,98

Table 5.2. Sample of results measured by travel time sections

5.2. Queues

One of the most important results of the simulation of the roundabout under study is the length of the queues formed in the entrances. In order to obtain results related with queues, there is a *queue counter* feature in PTV Vissim that provides an output with the following results:

- Average queue length
- Maximum queue length
- Number of vehicles stopped within the queue

Figure 5.2 shows where the queue counters are located (red line). A queue counter works "counting from the location of the queue counter (on the link or connector) upstream to the final vehicle that is in queue condition. If the queue backs up onto multiple different approaches (as happens in Highway and Viesques entrances) the queue counter will record information for all of them and report the longest as the maximum queue length" [58].

Figure 5.2. Localization of queue counters

According to PTV, “the back of the queue is monitored until there is not a single vehicle left over on the approach that still meets the queue condition, though other vehicles between the initial start and the current end of the queue do not longer meet the queue condition” [58]. Although the first vehicles directly upstream of the queue counter were not in queue condition any more the queue is still monitored as long as there is a “queue remainder”.

Also, it should be highlighted that the queue length is an output that PTV Vissim gives in units of length (meters) not in number of cars.

In order to get the desired queue measurement, additional information is needed. As it can be seen in Figure 5.4, the following data must be defined:

- **Queue definition**: A vehicle is considered to be in queue condition if “its speed is less than the **Begin speed** but and has not exceed the **End speed** yet” [58]. This means that a vehicle which is circulating faster than the End speed and begins to decrease his speed (see Figure 5.3) is not considered to be in queue until his speed is under the Begin speed. From that moment, the vehicle is considered to be in queue until his speed does not exceed the End speed (yellow line in Figure 5.3). Once the vehicle speed has exceeded the End speed, the vehicle is not in queue anymore.

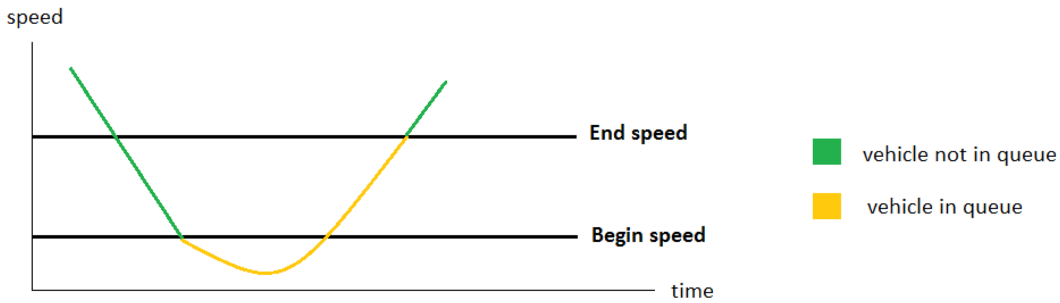


Figure 5.3. Queue condition

The authors of this thesis have defined the Begin speed as 5 km/h and the End speed as 10 km/h.

- **Max. Headway:** It is defined as the maximum distance between two consecutive vehicles so that the queue is not broken. A value of 20 m has been set.

- **Max. Length:** This parameter defines the maximum length of the queue (no matter if the current queue is longer). It “is helpful if longer queues are noticed in a network of subsequent junctions but the queues have to be evaluated for each junction separately” [58]. The maximum length considered in the simulation model is 5 km.

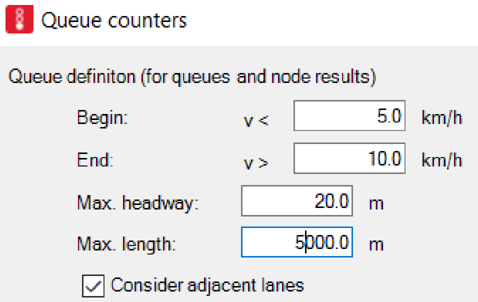


Figure 5.4. Queue counter parameters

The results obtained during the simulation have the structure shown in Table 5.3. The queue data (average queue length and Maximum queue length) is measured in meters for each entrance (Viesques, Polytechnic, Highway and Molinón) and for each time interval. For each queue counter the following information is shown:

- **Average queue length (Qlen)**: It is obtained from the current queue length measured in each time step. It is calculated as the arithmetical mean of that values for each time interval.
- **Maximum queue length (QLenMax)**: It is the maximum length of the queue in each time interval.
- **Number of stops (QStops)**: It is the sum of all the times that all the vehicles have to stop in the queue in that time interval.

SimRun	TimeInt	QueueCounter	QLen	QLenMax	QStops
2	1800-3600	1: Viesques Entrance Queue	2,20	77,71	23
2	1800-3600	2: Polytechnic Entrance Queu	4,57	75,86	26
2	1800-3600	3: Highway Entrance Queue	0,82	21,58	18
2	1800-3600	4: Molinon Entrance Queue	14,60	51,67	81
2	3600-5400	1: Viesques Entrance Queue	9,56	102,01	87
2	3600-5400	2: Polytechnic Entrance Queu	1,51	26,62	12
2	3600-5400	3: Highway Entrance Queue	4,53	61,92	83
2	3600-5400	4: Molinon Entrance Queue	23,74	86,88	172
2	5400-7200	1: Viesques Entrance Queue	105,70	361,88	629
2	5400-7200	2: Polytechnic Entrance Queu	9,95	63,20	64
2	5400-7200	3: Highway Entrance Queue	33,39	109,78	315
2	5400-7200	4: Molinon Entrance Queue	50,10	122,17	415
2	7200-9000	1: Viesques Entrance Queue	140,09	549,55	406
2	7200-9000	2: Polytechnic Entrance Queu	4,42	35,16	15
2	7200-9000	3: Highway Entrance Queue	138,67	226,62	654
2	7200-9000	4: Molinon Entrance Queue	136,18	190,20	579

Table 5.3. Queue counter results

5.3. Incoming vehicles

It is interesting to know the number of vehicles that have entered the network traffic. PTV Vissim shows data on incoming vehicles through a text output (see Figure 5.5) in which the following measures are found:

- **Time**: It is the instant of time when the vehicle enters the network (i.e. when it is created).
- **Link, Lane**: It specifies in which link and lane the vehicle has entered.
- **VehNr (Vehicle Number)**: It is a feature of each vehicle. This number is function of the time when a vehicle enters the network. The smaller is the number, the earlier the vehicle entered.
- **VehType (Type of Vehicle)**: It is a number which represents the type of vehicle. For instance:
    - 100: Car
    - 200: Truck
    - 300: Bus
    - 600: Bike

Time;	Link;	Lane;	VehNr;	VehType;
1.1;	2;	1;	1;	100;
5.5;	9;	1;	2;	100;
18.5;	2;	1;	3;	300;
21.9;	2;	1;	4;	100;
30.2;	11;	1;	5;	100;
36.8;	9;	1;	6;	100;
45.3;	5;	1;	7;	100;
47.7;	11;	1;	8;	100;
50.9;	5;	1;	9;	100;
54.5;	5;	1;	10;	300;
55.4;	2;	1;	11;	100;
55.9;	9;	1;	12;	100;
76.9;	7;	1;	13;	100;
81.2;	7;	2;	14;	100;
84.6;	9;	2;	15;	100;
86.9;	7;	1;	16;	100;
93.2;	7;	2;	17;	100;
95.6;	7;	1;	18;	100;
113.7;	2;	1;	19;	100;

Figure 5.5. Vehicle entered data text output

5.4. Vehicle network performance

Vehicle network performance evaluates several parameters that are aggregated for the whole simulation run and the whole network. This is useful to get a global perspective of how the traffic network is working. Table 5.4 show a sample of the results provided by this tool.

Vehicle network performance provides as output with the following results:

- **Average delay time** “DelayAvg(All)”: It shows the result of [Total Delay Time/Total vehicles in Network] during the time interval (in s/veh).
- **Average number of stops** “StopsAvg(All)”: It shows the result of [Total number of stops/Total vehicles in the network] during each time interval.
- **Average speed** “SpeedAvg(All)”: It shows the result of [Total Distance/Total Travel Time] (in Km/h).
- **Average stopped delay** “DelayStopAvg(All)”: It shows the result of [Total Stopped Delay/Active Vehicles in the network] during a time interval.
- **Distance** “DistTot(All)”: It is the total distance (in km) travelled by all the vehicles in the network during a time interval.
- **Travel time** “TravTmTot(All)”: It is the total travel time (in seconds) of all active vehicles in the network.
- **Total delay Time** “DelayTot(All)”: The delay time of a vehicle in one time step is “the part of the time step spent because the actual speed is lower than the desired speed. It is calculated by subtracting the quotient of the actual distance travelled in this time step and the desired speed from the length of the time step” [58]. It is measured in seconds.
- **Number of Stops** “StopsTot(All)”: It represents the total number of stops of all active vehicles in the network during a time interval. For PTV Vissim a stop occurs “if the speed of the vehicle was greater than zero at the end of the previous time step and is zero at the end of the current time step” [58].
- **Total Stopped Delay** “DelayStopTot(All)”: It is the total stopped time (in seconds) of all active vehicles during a time interval. A vehicle is considered to be stopped when the speed of such vehicle is zero.
- **Number of active vehicles** “VehAct(All)”: Total number of vehicles in the network at the end of the time interval.
- **Number of arrived vehicles** “VehArr(All)”: Total number of vehicles which have already reached their destination and left the network during the time interval.
- **Latent delay time** “DelayLatent”: “Total waiting time of vehicles which could not enter the network” [58].
- **Demand Latent**: It is the “number of vehicles which could not enter the network (from vehicle inputs)” [58].

SimRun	TimeInt	DelayAvg(All)	StopsAvg(All)	SpeedAvg(All)	DelayStopAvg(All)	DistTot(All)	TravTmTot(All)
3	0-1800	23,92	0,41	43,52	5,50	613,11	50717,00
3	1800-3600	19,66	0,56	45,75	7,39	768,94	60502,00
3	3600-5400	37,37	1,19	40,61	12,53	1197,23	106119,80
3	5400-7200	105,95	4,91	27,18	50,28	1579,67	209239,80
3	7200-9000	217,03	10,58	17,18	121,10	1731,35	362712,20
3	9000-10800	420,99	23,36	9,36	258,75	1640,43	630892,30
3	10800-12600	499,72	26,83	7,87	315,96	1550,52	708967,90
3	12600-14400	344,13	18,59	11,86	206,94	1481,05	449547,50
3	14400-16200	72,63	2,96	32,46	33,28	1255,93	139292,10
3	16200-18000	43,72	1,09	38,43	10,32	1122,47	105152,60
3	18000-19800	33,54	0,87	40,04	9,03	982,65	88341,40
3	19800-21600	42,90	0,96	37,98	9,92	1039,10	98482,40
3	21600-23400	50,56	1,32	36,43	12,04	1224,23	120962,30
3	23400-25200	61,38	1,96	34,65	19,56	1414,23	146917,20
3	25200-27000	87,04	3,10	29,83	42,66	1517,83	183150,10

SimRun	TimeInt	DelayTot(All)	StopsTot(All)	DelayStopTot(All)	VehAct(All)	VehArr(All)	DelayLatent	DemandLatent
3	0-1800	7724,79	134	1775,79	27	296	13,40	0,00
3	1800-3600	8491,61	240	3193,34	39	393	9,30	0,00
3	3600-5400	24404,26	774	8182,09	49	604	8,80	0,00
3	5400-7200	97472,52	4519	46255,11	138	782	120,20	0,00
3	7200-9000	239821,53	11696	133816,44	242	863	172,70	0,00
3	9000-10800	517817,54	28731	318262,43	439	791	191,00	0,00
3	10800-12600	601668,60	32305	380415,03	361	843	46,20	0,00
3	12600-14400	346196,59	18699	208184,36	160	846	66,70	0,00
3	14400-16200	53890,37	2198	24690,99	63	679	21,50	0,00
3	16200-18000	27628,21	687	6524,90	59	573	43,70	0,00
3	18000-19800	18982,18	494	5109,26	45	521	32,10	0,00
3	19800-21600	24536,28	551	5672,37	50	522	38,10	0,00
3	21600-23400	33317,74	873	7933,47	58	601	31,50	0,00
3	23400-25200	46649,32	1492	14865,47	100	660	27,40	0,00
3	25200-27000	73985,28	2634	36262,52	91	759	41,40	0,00

Table 5.4. Sample of Vehicle network performance

5.5. Determining sample size. Results validation

This chapter is partially based on [62].

In order to obtain significant results it is necessary to determine the number of replicas that have to be simulated. To obtain meaningful simulation results it is necessary to simulate several replicas of the simulation model varying the random seed of each. First of all, two variables have to be defined:

- A confidence level of 95%.
- An error margin of 5%.

That means that the "95% of the time one particular observation is correct within +/- 5%" [62]. Once these two parameters are established, the next step is to simulate a trial run. Finally, the number of replicas to fulfil with both requirements (confidence level and accuracy margin) can be calculated considering the following formula (eq. 5.1).

$$n = \left(\frac{t_{\alpha/2,m-1} \cdot s}{k \cdot \bar{x}}\right)^2 \quad \text{(eq. 5.1, [62])}$$

Where:

- n is the number of replicas.
- $\bar{x}$ is the arithmetic mean of the trial run.
- s is the standard deviation of the trial run.
- $t_{\alpha/2,m-1}$ is the student t distribution (where $\alpha/2$ is 0.025 -for a confidence level of 95%- and m is the size of the trial run).
- k is the error margin (0.05).

From now on, the calculation of the number of replicas needed (n) in the base scenario *Current roundabout* (see Chapter 6) is going to be explained as an example. It should be highlighted that the same procedure explained below has been followed with the 3 remaining scenarios (which are explained later in Chapter 7, Chapter 8 and Chapter 9 respectively).

For each scenario four types of results are obtained:

- Maximum queue length per entrance in each time interval.
- Average queue length per entrance in each time interval.
- Average travel time per entrance in each time interval.
- Total outgoing vehicles and total remaining vehicles in each time interval.

However, the results with more variability and, therefore more number of runs required, are the maximum and average queue length. For this reason, both results are the ones taken into account in order to calculate the number of replicas needed (n).

To calculate the number of replicas needed (n) the following procedure has to be made:

1. Table 5.5 shows the **raw data output of PTV Vissim** which are taken into account to calculate the number of runs required (n).
All the starting data are values of the "Average queue length" and "Maximum queue length" during each time interval (of 30 minutes each interval) and in each roundabout entrance (Viesques entrance, Polytechnic entrance, Highway entrance and Molinón entrance) during a trial of 20 runs ($m = 20$).

Queue results in CR scenario				
Trial run	**Time interval**	**Entrance**	**Average Queue Length [m]**	**Maximum Queue Length [m]**
1	7.00-7.30 am	1: Viesques Entrance Queue	1,6	52,5
1	7.00-7.30 am	2: Polytechnic Entrance Queue	0,6	19,3
1	7.00-7.30 am	3: Highway Entrance Queue	0,8	22,6
1	7.00-7.30 am	4: Molinon Entrance Queue	11,8	49,9
1	7.30-8,00 am	1: Viesques Entrance Queue	6,3	77,1
1	7.30-8,00 am	2: Polytechnic Entrance Queue	4,4	136,8
1	7.30-8,00 am	3: Highway Entrance Queue	4,4	54,2
1	7.30-8,00 am	4: Molinon Entrance Queue	27,1	69,0
1	8.00-8.30 am	1: Viesques Entrance Queue	172,1	419,9
1	8.00-8.30 am	2: Polytechnic Entrance Queue	6,4	59,9
1	8.00-8.30 am	3: Highway Entrance Queue	56,6	161,2
1	8.00-8.30 am	4: Molinon Entrance Queue	45,5	94,3
…	…	…	…	…
20	7.00-7.30 am	1: Viesques Entrance Queue	2,2	77,7
20	7.00-7.30 am	2: Polytechnic Entrance Queue	4,6	75,9
20	7.00-7.30 am	3: Highway Entrance Queue	0,8	21,6
20	7.00-7.30 am	4: Molinon Entrance Queue	14,6	51,7
20	7.30-8,00 am	1: Viesques Entrance Queue	9,6	102,0
20	7.30-8,00 am	2: Polytechnic Entrance Queue	1,5	26,6
20	7.30-8,00 am	3: Highway Entrance Queue	4,5	61,9
20	7.30-8,00 am	4: Molinon Entrance Queue	23,7	86,9
20	8.00-8.30 am	1: Viesques Entrance Queue	105,7	361,9
20	8.00-8.30 am	2: Polytechnic Entrance Queue	9,9	63,2
20	8.00-8.30 am	3: Highway Entrance Queue	33,4	109,8
20	8.00-8.30 am	4: Molinon Entrance Queue	50,1	122,2
…	…	…	…	…

Table 5.5. Starting data

2. The next step is to **calculate the arithmetic mean ($\overline{x}$)** for the "Average queue length" and the "Maximum queue length" in each time interval and in each entrance for all the trial runs considered ($m = 20\ runs$), which is shown in Table 5.6.

Queue results in CR scenario			
Time interval	**Entrance**	$\overline{x}_{Average\ Queue\ Length}$ **[m]**	$\overline{x}_{Max\ Queue\ Length}$ **[m]**
7.00-7.30 am	1: Viesques Entrance Queue	1,33	51,66
7.00-7.30 am	2: Polytechnic Entrance Queue	1,74	42,16
7.00-7.30 am	3: Highway Entrance Queue	0,99	27,10
7.00-7.30 am	4: Molinón Entrance Queue	12,67	48,92
7.30-8,00 am	1: Viesques Entrance Queue	4,22	75,24
7.30-8,00 am	2: Polytechnic Entrance Queue	4,80	64,81
7.30-8,00 am	3: Highway Entrance Queue	5,00	52,69
7.30-8,00 am	4: Molinón Entrance Queue	27,14	78,26
….	…..	….	….
9.00-9.30 pm	3: Highway Entrance Queue	167,31	223,58
9.00-9.30 pm	4: Molinón Entrance Queue	766,08	808,63
9.30-10.00 pm	1: Viesques Entrance Queue	97,47	191,78
9.30-10.00 pm	2: Polytechnic Entrance Queue	189,72	236,29
9.30-10.00 pm	3: Highway Entrance Queue	159,60	187,57
9.30-10.00 pm	4: Molinón Entrance Queue	596,03	739,21

Table 5.6. Arithmetic mean of Average and Maximum queue length

3. The next step is to **calculate the standard deviation (s)** for the same values, as it can be seen in table 5.7.

Queue results in CR scenario			
Time interval	**Entrance**	$s_{Average\ Queue\ Length}$ **[m]**	$s_{Max\ Queue\ Length}$ **[m]**
7.00-7.30 am	1: Viesques Entrance Queue	0,8	21,1
7.00-7.30 am	2: Polytechnic Entrance Queue	1,5	23,6
7.00-7.30 am	3: Highway Entrance Queue	0,6	11,1
7.00-7.30 am	4: Molinón Entrance Queue	2,0	7,3
7.30-8,00 am	1: Viesques Entrance Queue	2,4	26,1
7.30-8,00 am	2: Polytechnic Entrance Queue	3,5	26,7
7.30-8,00 am	3: Highway Entrance Queue	2,7	16,9
7.30-8,00 am	4: Molinón Entrance Queue	9,0	18,8
….	…..	….	….
9.00-9.30 pm	3: Highway Entrance Queue	464,4	450,4
9.00-9.30 pm	4: Molinón Entrance Queue	161,5	129,4
9.30-10.00 pm	1: Viesques Entrance Queue	253,7	224,3
9.30-10.00 pm	2: Polytechnic Entrance Queue	554,1	540,0
9.30-10.00 pm	3: Highway Entrance Queue	466,7	457,3
9.30-10.00 pm	4: Molinón Entrance Queue	236,2	192,6

Table 5.7. Standard deviation of Average and Maximum queue length

4. The final step is to **calculate the number of replicas (n)** for each time interval using equation 5.1, as it can be seen in table 5.8.

Queue results in CR scenario			
Time interval	**Entrance**	$n_{Average\ Queue\ Length}$ **[replicas]**	$n_{Max\ Queue\ Length}$ **[replicas]**
7.00-7.30 am	1: Viesques Entrance Queue	632	280
7.00-7.30 am	2: Polytechnic Entrance Queue	1226	525
7.00-7.30 am	3: Highway Entrance Queue	529	282
…..	….	…..	….
5.00-5.30 pm	3: Highway Entrance Queue	**28813**	**13682**
….	….	….	….
9.30-10.00 pm	2: Polytechnic Entrance Queue	14269	8737
9.30-10.00 pm	3: Highway Entrance Queue	14305	9941
9.30-10.00 pm	4: Molinón Entrance Queue	263	114
Maximum number of replicas		**28813**	**13682**

Table 5.8. Replicas needed for CR scenario (Confidence level 95%, Margin of error 5%)

It is important to note that results are divided in time intervals. Therefore, it is necessary to evaluate the number of replicas needed in each time interval and to take the maximum value of all of them (red numbers in Table 5.8) in order to accomplish with the requirements of confidence level and error margin. In the example given of Current Roundabout scenario the number of replicas required is $n = 28813\ replicas$.

It should be highlighted that the analysis of the results is a really challenging task due to the variability of the results between different replicas (which is a feature and one of the main disadvantages of microsimulation software). Little changes in vehicles inputs (i.e. random seed) can change completely the roundabout behaviour and therefore the results obtained in each replica. That explains the high values of standard deviation and the need of simulating many replicas in order to obtain meaningful results.

However, simulating 28813 replicas is not viable due to the fact that each replica takes approximately 1 hour of simulation. That would mean 28813 hours of simulation (1200 days). In order to decrease the number of runs needed, confidence level and margin error are going to be reduced to the following values:

- A confidence level of 80%.
- An error margin of 20%.

This reduction modify the number of replicas needed (n) as shown in table 5.9.

Queue results in CR scenario			
Time interval	Entrance	$n_{Average\ Queue\ Length}$ [replicas]	$n_{Max\ Queue\ Length}$ [replicas]
7.00-7.30 am	1: Viesques Entrance Queue	16	7
7.00-7.30 am	2: Polytechnic Entrance Queue	31	13
7.00-7.30 am	3: Highway Entrance Queue	14	7
…..	….	…..	….
5.00-5.30 pm	3: Highway Entrance Queue	**739**	**351**
….	….	….	….
9.30-10.00 pm	2: Polytechnic Entrance Queue	366	224
9.30-10.00 pm	3: Highway Entrance Queue	367	255
9.30-10.00 pm	4: Molinón Entrance Queue	7	3
Maximum number of replicas		**739**	**351**

Table 5.9. Replicas needed for CR scenario (Confidence level 80%, Margin of error 20%)

Taking into account that the replicas needed ($n = 739\ replicas$, which means 30 days of simulation) is still a high value, a balance between meaningful results and number of replicas has to be made. Moreover, there is no significant variability in the results as long as the number of replicas is increased over a value of 70-90 replicas (at least not enough variability to continue simulating more replicas). Finally, in Table 5.10 it can be seen:

- The number of replicas needed (n) for a confidence level of 80% and a margin of error of 20% for each scenario.
- The number of replicas made for each scenario.

		Confidence level	Margin error	n	Number of replicas made
Scenario	Current roundabout	80%	20%	739	**70**
	Location change of the zebra crossing	80%	20%	28069	**80**
	Immediate exits and flaring the entries	80%	20%	3765	**90**
	Traffic lights control actuated by vehicles	80%	20%	203	**80**

Table 5.10. Replicas needed and made in each scenario

To end this chapter, **the authors of this thesis would like to emphasize that they are fully aware of the number of replicas which should be made to obtain significant results**. However, for a mere matter of available time the authors have had to reach a compromise between time and accuracy.

6. Analysis of the current situation of the roundabout

6.1. Sample of simulation

In this section some samples of the simulation during different parts of the day are shown. It is interesting to observe how the traffic saturation of the roundabout grews as the rush hour is approaching. Figure 6.1 shows the traffic at 7:00 am, Figure 6.2 at 7:45 am, Figure 6.3 at 8:20 am and figure 6.4 at 9:00 am. It should be noted that this base scenario is called **CR** (Current Roundabout).

Figure 6.1. Traffic in the roundabout at 7:00 am

Figure 6.2. Traffic in the roundabout at 7:45 am

Figure 6.3. Traffic in the roundabout at 8:20 am

Figure 6.4. Traffic in the roundabout at 9:00 am

6.2. Simulation results

6.2.1. Queues

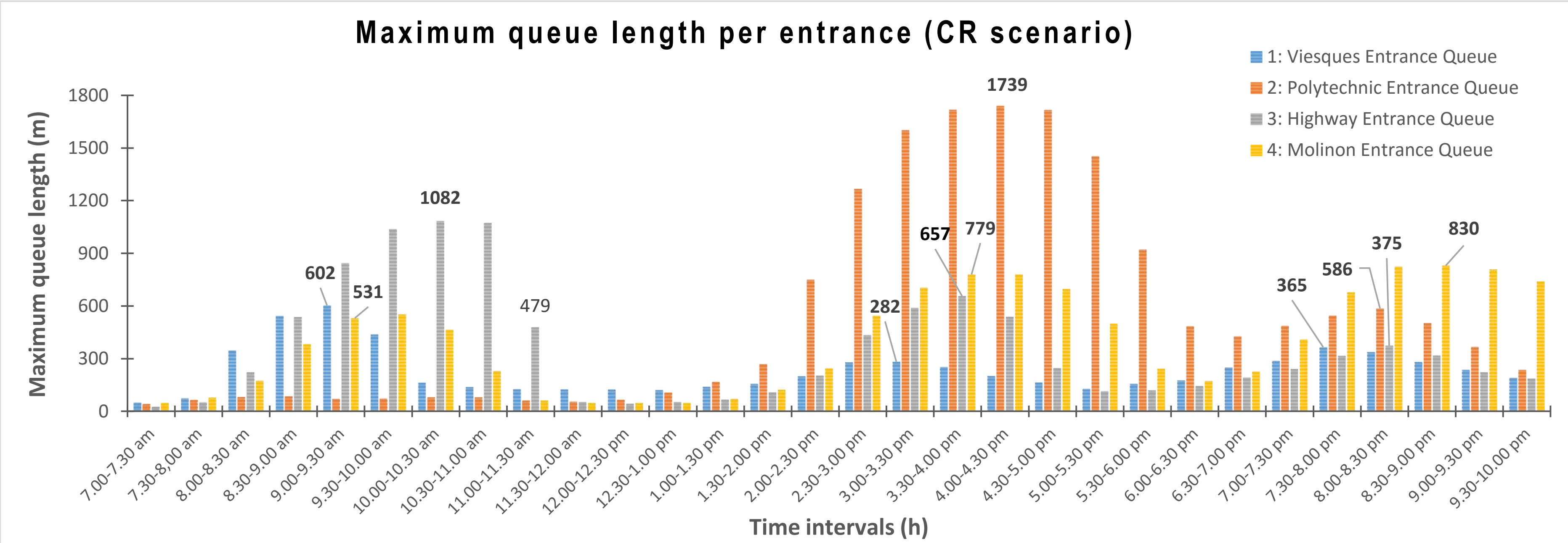


Chart 6.1. Maximum queue length (per entrance) vs. time intervals in CR scenario

Chart 6.1 shows the maximum queue length (in meters) in each entrance of the roundabout in each time interval of the simulation. As it is expected, larger queues are formed during the rush hours (between 8:00-9:30 am, 2:00-3:30 pm and 6:30-9:00 pm). However, the largest ones are sometimes formed sometime after the end of the rush hour. This is because of the accumulative effect of all the vehicle arrivals during the traffic peak hour.

During the morning peak of the day the largest queue takes place in Highway entrance (1082 m), following by Viesques entrance (602 m) and Molinón entrance (531 m). During the afternoon peak the largest queue is formed in Polytechnic entrance (1739 m), following by Molinón entrance (779 m), Highway entrance (657 m) and Viesques entrance (282 m). Finally, during the evening peak the largest queue takes place in Molinón entrance (830 m), following by Polytechnic entrance (586 m), Highway entrance (375 m) and Viesques entrance (365 m). It is important to take into account that all these values are maximum values.

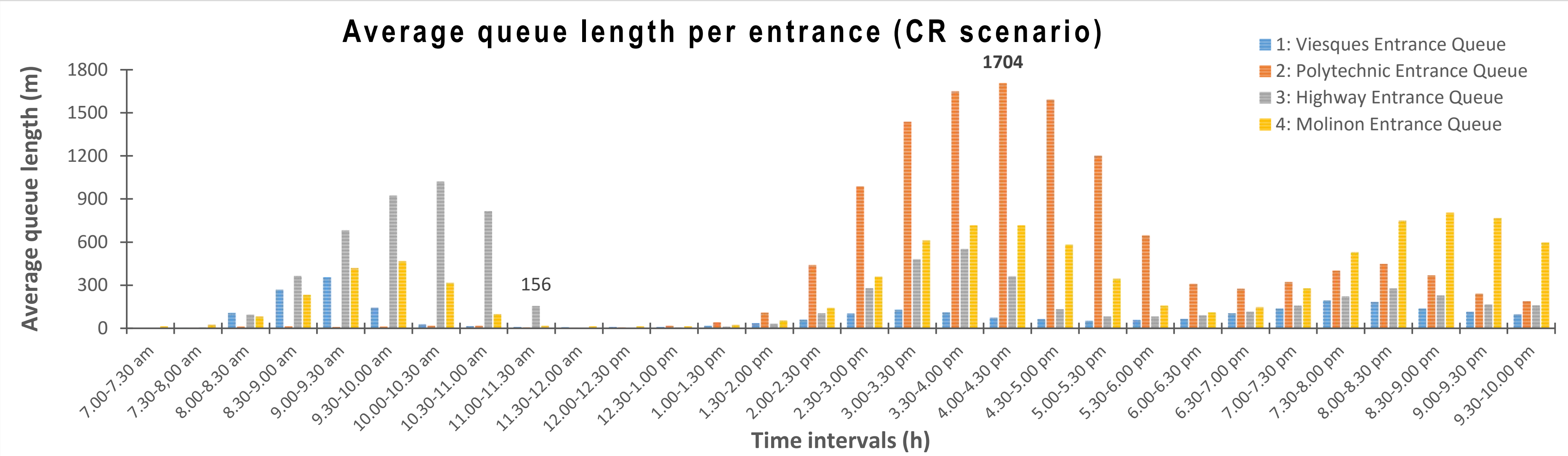


Chart 6.2. Average queue length (per entrance) vs. time intervals in CR scenario

Chart 6.2 shows the average queue length per entrance and per time interval. The maximum queue length is close to the average queue length in some intervals (for instance, maximum queue length in Polytechnic entrance between 4:00-4:30 pm is 1739 m whereas the average length is 1704 m). That means that the values of queue do not happen during an instant but they are remained relatively constant during the time interval. On the other hand, there are maximum queue length values very different form the average queue values (for instance, maximum queue length in Highway entrance between 11:00-11:30 am is 479 m whereas the average value is 156 m). That fact means that there has been a momentary peak of queue but later the queue has been decreasing.

6.2.2. Travel time

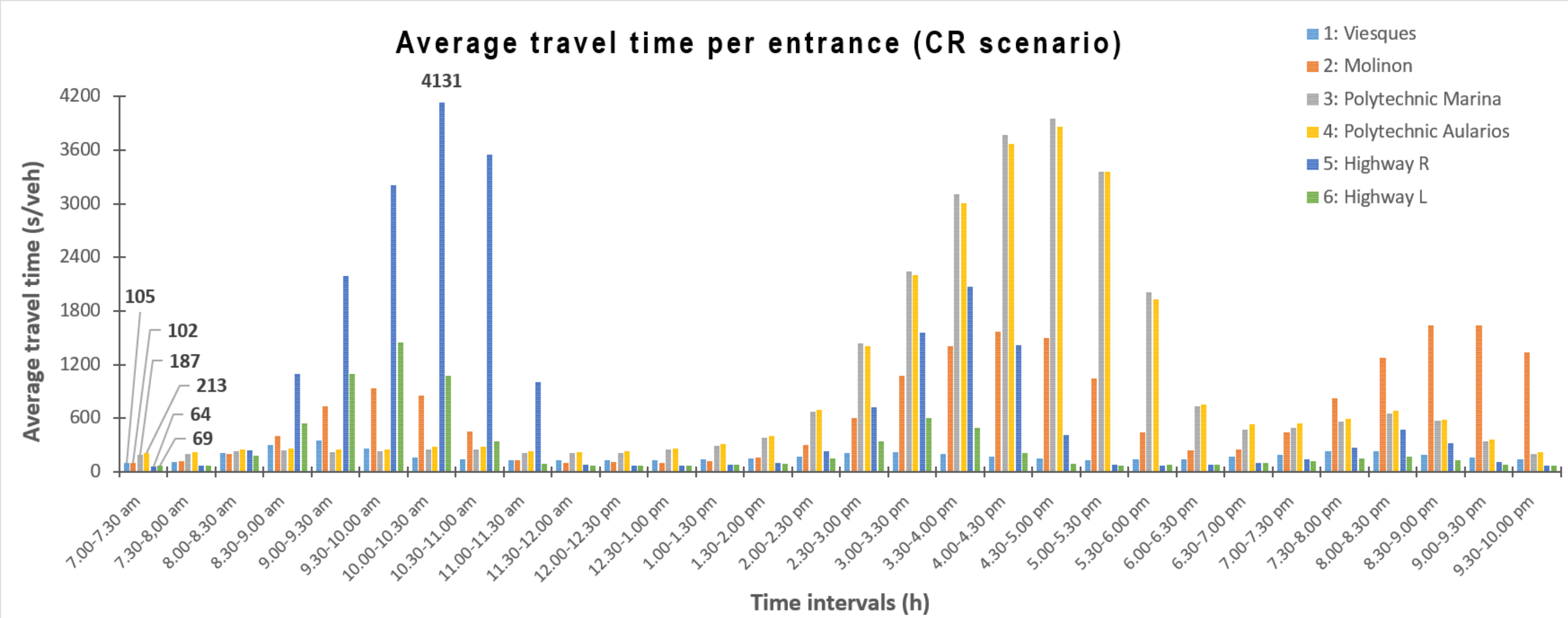


Chart 6.3. Travel time (per entrance) vs. time interval in CR scenario

Chart 6.3 shows the average travel time spent by a vehicle from the beginning of the link (i.e. where the vehicle is created) to the yield line of the roundabout (in each entrance and during each time interval). At the beginning of the day, between 7:00-7:30 am, the average travel time spent in each entrance is very low (105 s/veh in Viesques entrance, 102 s/veh in Molinón entrance, 187 s/veh in Polytechnic Marina entrance, 213 s/veh in Polytechnic Aularios entrance, 64 s/veh in Highway R entrance and 69 s/veh in Highway L entrance). These results of travel time are very meaningful because they show the optimal traffic conditions. Therefore, the authors of this thesis consider appropriate that these results were the reference to compare other results with, in order to measure the efficiency of the network.

For instance, the average travel time in Highway R entrance during 10:00-10:30 am is 4131 s/veh. If this result is compared with the optimal one (64 s/veh), it could be said that the travel time spent is 65 times longer according to the ideal travel time.

6.2.3. Capacity

Chart 6.4 shows the total number of vehicles that have already reached their destination and left the network during each time interval (called *total outgoing vehicles)* and the total number of remaining vehicles in the network at the end of each time interval (called as *total remaining vehicles*). It is a very interesting chart because it somehow shows the maximum capacity of the roundabout.

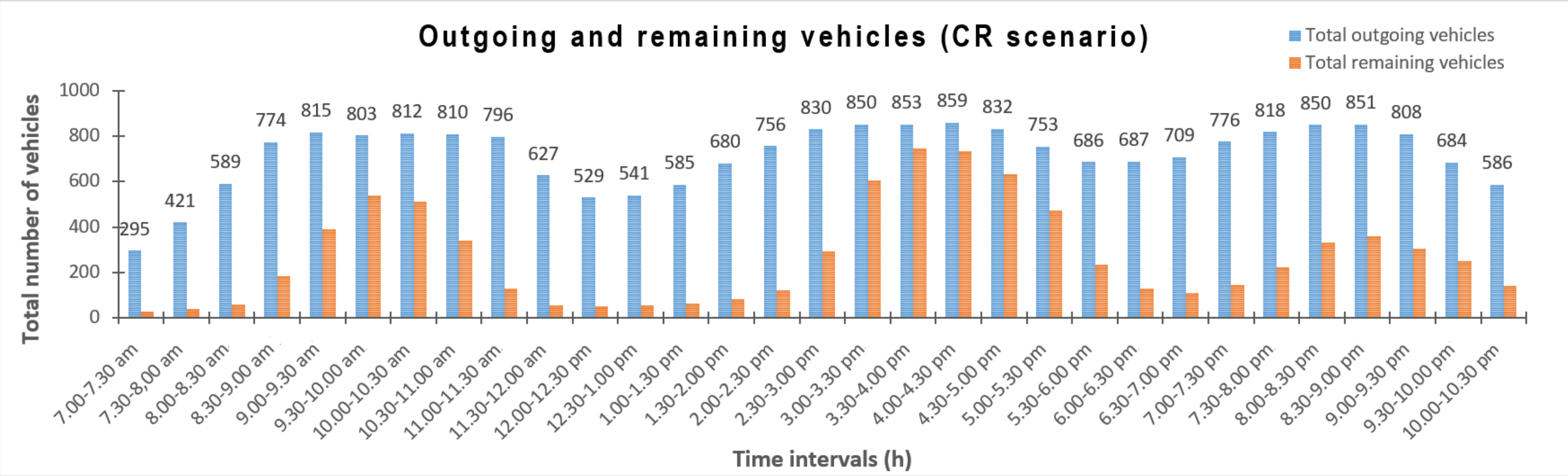


Chart 6.4. Outgoing and remaining vehicles vs. time intervals in CR scenario

As it has been discussed before (in Chapter *2.3 Capacity*), calculating the capacity of a roundabout is not something easy because it depends on many factors. Traffic simulation is a very good tool for estimating the capacity of a roundabout under specific conditions. In this case, *total outgoing vehicles* indicates the amount of vehicles that have used the roundabout during the time interval considered. In Chart 6.4 it can be seen that the number of outgoing vehicles tend to stabilize in the peak moments, when the capacity of the roundabout is exceeded. During these time intervals the roundabout saturates reaching its maximum capacity. The **roundabout maximum capacity** is around 850 vehicles per half an hour, which means **around 1700 vehicles per hour**.

7. Analysis of location change of the zebra crossing

7.1. Introduction

Several studies on the effect of pedestrians on roundabout entry capacity have been conducted (see chapter *2.3.2. Influential parameters in the entry capacity of a roundabout*). Such studies have shown that the presence of pedestrians crossing crosswalks in a roundabout reduces the entry capacity because incoming vehicles must yield to pedestrians and they miss gaps in the circulating traffic.

This fact can be verified observing the simulation. Figure 7.1 illustrates an instant of the simulation in which vehicles in Molinón entrance are waiting while the pedestrians are crossing. If this zebra crossing did not exist, the vehicles would have been looking for an available gap to enter the roundabout. Instead of that, vehicles in Molinón entrance must firstly yield to pedestrians and secondly they can look for a gap in the circulating traffic. As a result, the entry capacity in such approach decreases.

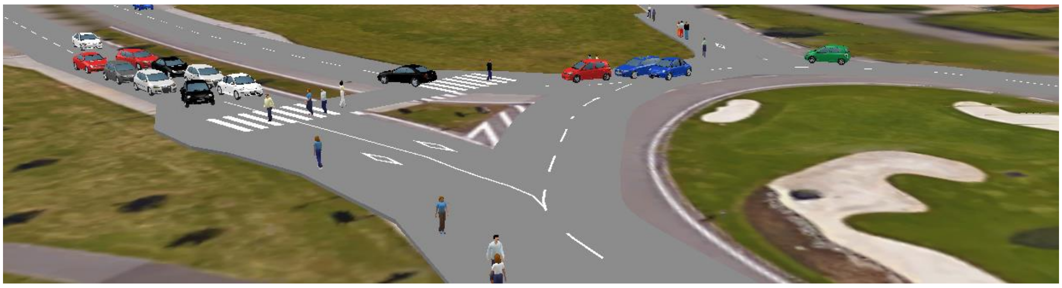


Figure 7.1. Vehicles yielding to pedestrians in the zebra crossing of Molinón entrance

Furthermore, pedestrians can cause the congestion of the roundabout. As Figure 7.2 shows, a queue is formed in the circulatory roadway because several vehicles that want to take Molinón exit must yield to pedestrians in the zebra crossing.

Figure 7.2. Queue formed in the circulatory roadway because of pedestrians

7.2. Changes in the simulation model

The first improvement that is going to be implemented in the simulation model is the change of the zebra crossing location. The idea is basically to place the crosswalk 120 meters upstream of the road (as it can be seen in Figure 7.3). This new scenario is called **ZCLC** (Zebra Crossing Location Change).

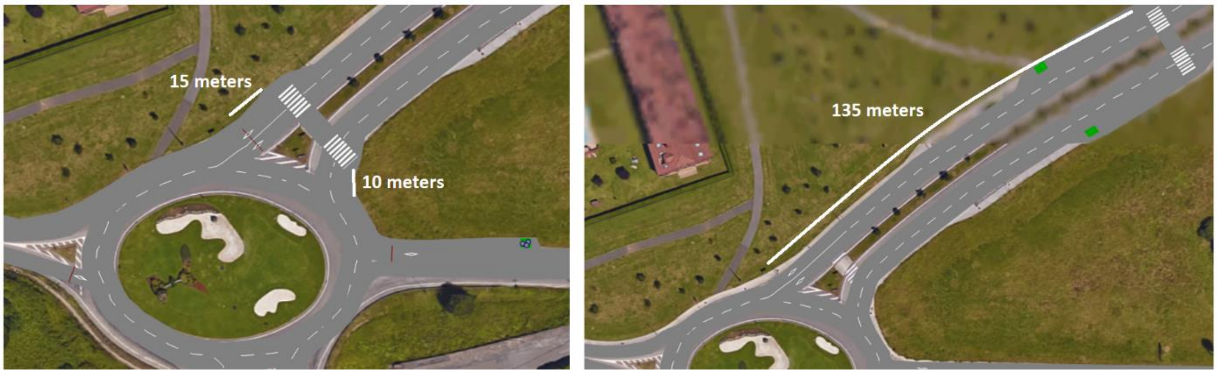


Figure 7.3. Location change of zebra crossing

By this improvement the roundabout collapse (as happened in Figure 7.2) is avoided in Molinón exit because there is more available space (135 m instead of 10 m) to form a queue without interfering in the circulatory roadway.

In addition, the entry capacity of Molinón entrance is higher during the rush hours because there is a bigger amount of available vehicles to enter the roundabout from Molinón entrance. Before changing the crosswalk location vehicles had to wait while yielding to pedestrian and later for entering the roundabout. That caused large queues upstream the zebra crossing and the lack of available vehicles between the crosswalk and the roundabout to enter the roundabout (as Figure 7.1 shows).

It could be concluded that the further away the zebra crossing is located from the roundabout, the less negative effects cause over the roundabout capacity.

7.3. Simulation results

7.3.1. Queues

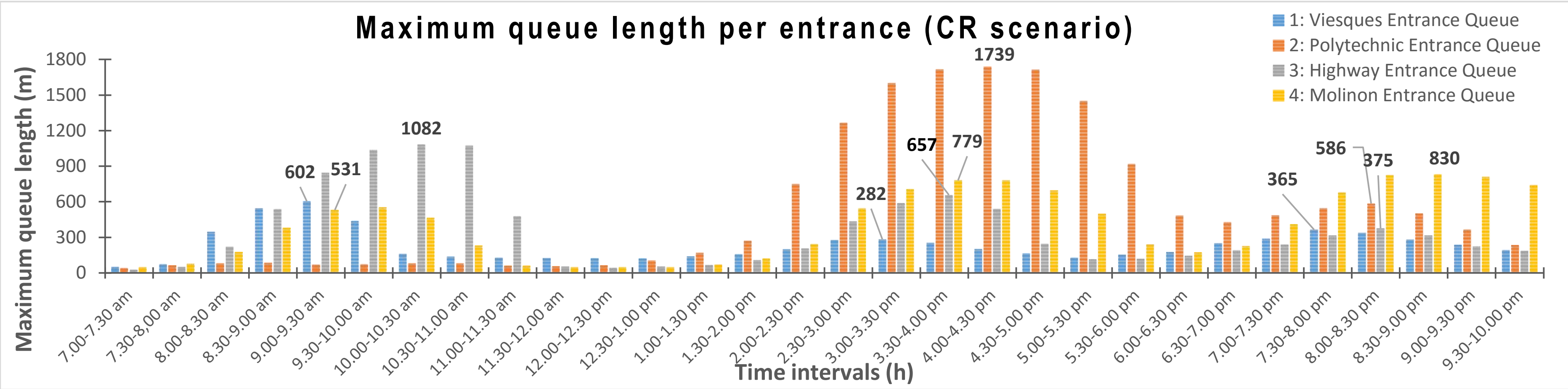


Chart 6.1. Maximum queue length (per entrance) vs time intervals in CR scenario

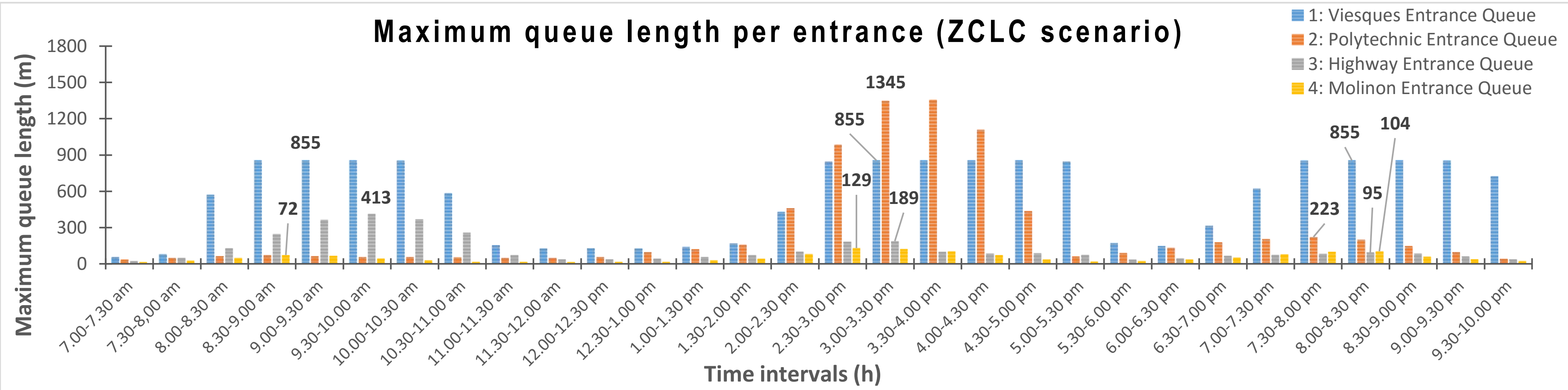


Chart 7.1. Maximum queue length (per entrance) vs time intervals in ZCLC scenario

Comparing Chart 7.1 “Maximum queue length (per entrance) vs time intervals in ZCLC scenario” with Chart 6.1 “Maximum queue length (per entrance) vs time intervals in CR scenario” the next findings can be highlighted:

- **During the morning peak or first peak (8:00-9:30 am approximately)**, the maximum queue lengths of Molinón and Highway entrances have decreased considerably. The main reason that explains the reduction in the maximum queue of Molinón entrance (from 531 m in CR scenario to 72 m in ZCLC scenario) is the change of the location of the zebra crossing. In addition, the reduction in the maximum queue of Highway entrance (from 1082 m to 413 m) can be a consequence of the lack of blocking upstream the zebra crossing in Molinón exit (see Figure 7.2). The analysis of the maximum queue lengths of Polytechnic entrance shows that there have not been significant changes. However, it is observed that the maximum queues in Viesques entrance have increased. This is because the location change of the zebra crossing allows vehicles from Molinón entrance a better use of gaps and consequently a more fluidity of traffic entering from such entrance. This results in a reduction of available gaps for vehicles wishing to enter the roundabout from Viesques entrance.

- **During the afternoon peak or second peak (2:00-3:30 pm approximately)** Molinón entrance maximum queue decreases from 779 m to 129 m. Highway entrance maximum queue length decreases from 657 m to 189 m. Maximum queues in Polytechnic entrance slightly decrease (from 1739 m to 1354 m). The maximum queues in Viesques entrance have increased (from 282 m to 855 m). Reductions and increases are caused by the same reasons given for the morning peak.

- **During the evening peak or third peak (6:30-9:00 pm)**, the results are similar to the both previous peaks due to the same reasons. The maximum queues in Molinón entrance decrease (from 830 m to 104 m). In Highway entrance the queues also decrease (from 375 m to 95 m). The maximum queues also decreases in Polytechnic entrance (from 586 m to 223 m). In Viesques entrance the maximum queues have increased (from 365 m to 855 m).

Chart 7.2 shows the overall maximum queues per entrance in CR and ZCLC scenarios. This chart bring to light that the overall effects of the change of the zebra crossing location are positive. Even though the overall maximum queue length has increased in Viesques entrance, the same value has considerably decreased for the remaining three entrances. Table 7.1 shows the percentage change of the overall maximum queue length for each entrance in ZCLC scenario compared to the CR base scenario.

	Molinón entrance	**Highway entrance**	**Polytechnic entrance**	**Viesques entrance**
Increase (+) / Decrease (-)	- 87.2 %	- 66.1%	- 50.0%	+ 137.3%

Table 7.1. Percentage change of the overall maximum queue length for each entrance in ZCLC scenario compared to the CR base scenario

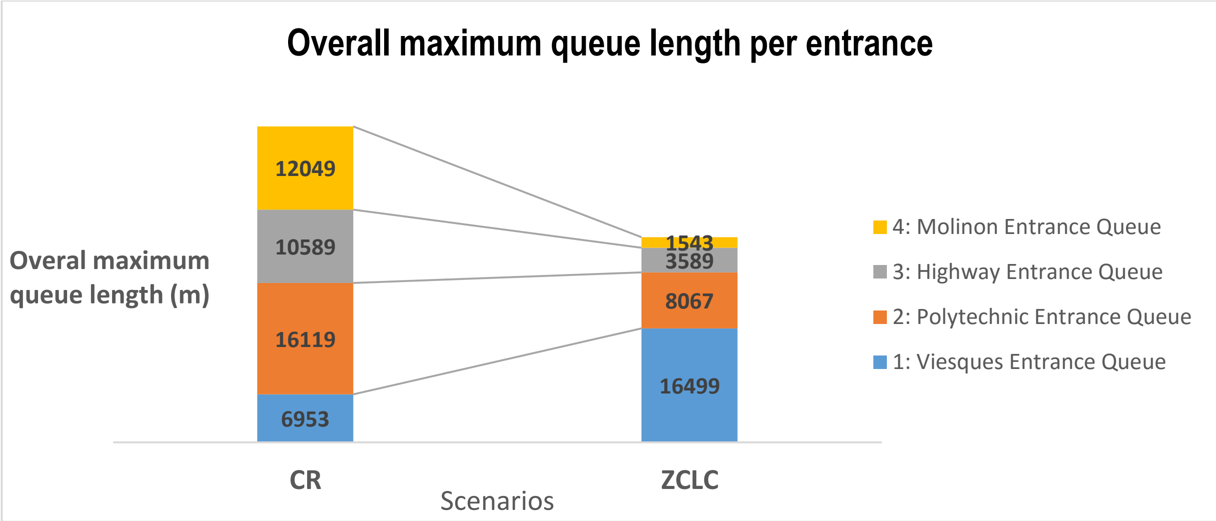


Chart 7.2. Comparison of the overall maximum queues per entrance between CR and ZCLC scenarios

Now a comparison of the average queue length for each entrance in CR and ZCLC scenario is performed (see Chart 7.3 "Average queue length (per entrance) vs time intervals in ZCLC scenario" and Chart 6.2 "Average queue length (per entrance) vs time intervals in CR scenario"). As expected based on previous maximum queue findings it can be concluded that:

- In Viesques entrance average queue lengths significantly increase during the three peaks (morning, afternoon and evening peak).
- Polytechnic average queue lengths decrease considerably in the evening peak but less strongly in the afternoon peak.
- Average queue lengths in Highway entrance decreases considerably during the three peaks.
- The entrance which experiences a highest decrease in average queue lengths in all the peaks is Molinón entrance.

Table 7.2 shows a comparison of the maximum values of average queue length in each entrance during the peak hours between CR and ZCLC scenarios.

	Molinón entrance			Highway entrance			Viesques entrance			Polytechnic entrance		
Peak	First	Second	Third	First	Second	Third	First	Second	Third	First	Second	Third
CR scenario	467	716	803	1020	553	278	354	129	194	0	1704	448
ZCLC scenario	12	44	23	300	96	29	826	839	835	0	1196	59

Table 7.2. Comparison of the maximum values of average queue length in each entrance during the peak hours between CR and ZCLC scenarios

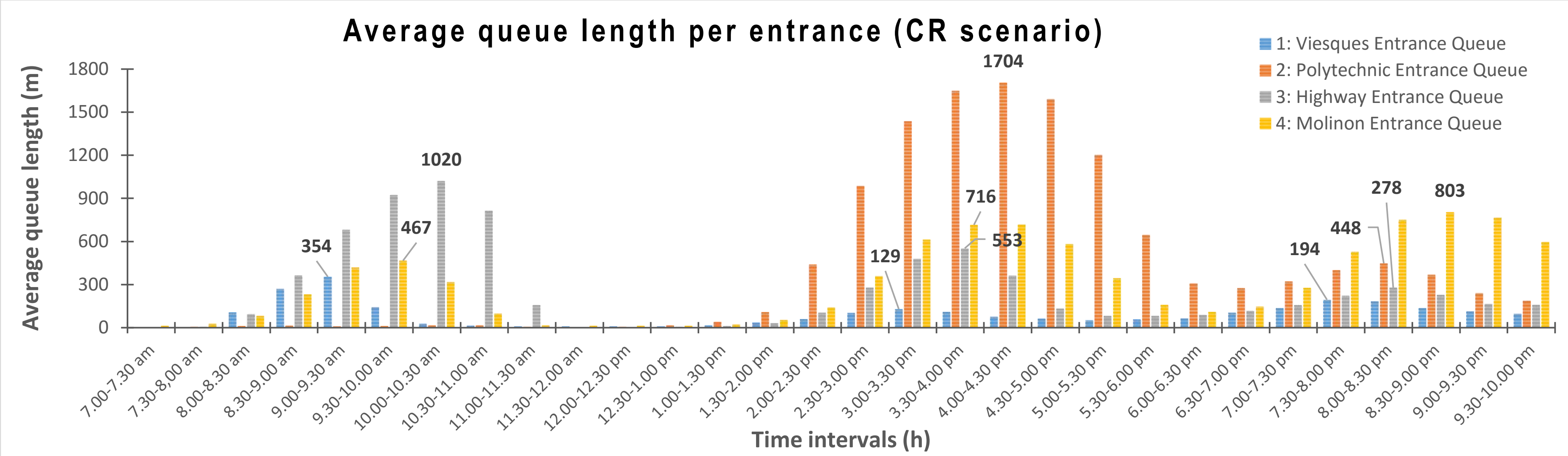


Chart 6.2. Average queue length (per entrance) vs. time intervals in CR scenario

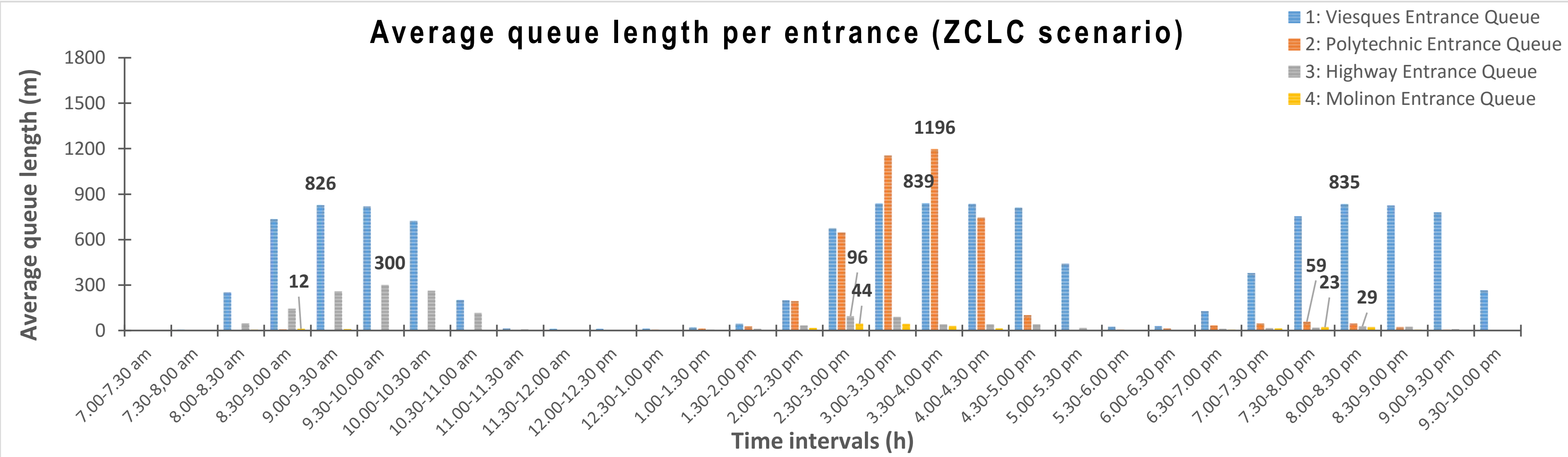


Chart 7.3. Average queue length (per entrance) vs time intervals in ZCLC scenario

7.3.2. Travel times

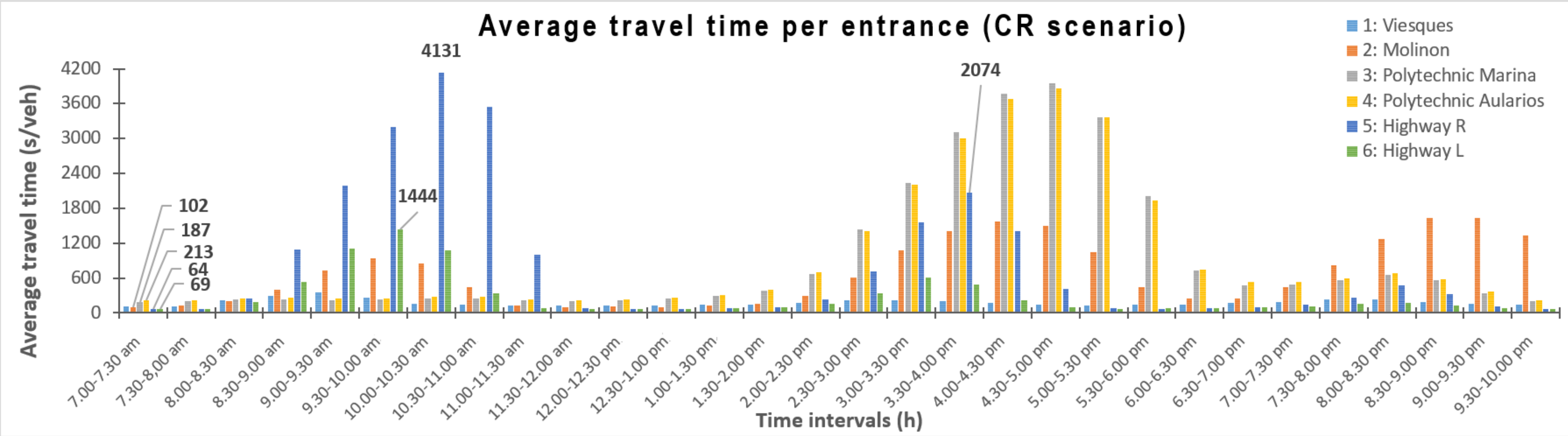


Chart 6.3. Average travel time (per entrance) vs. time interval in CR scenario

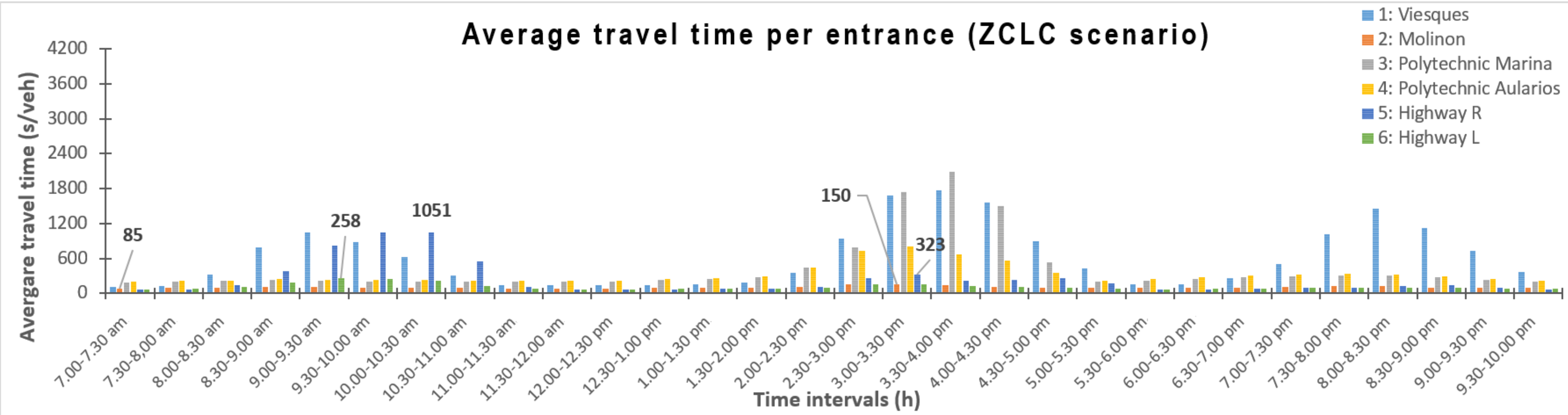


Chart 7.4. Average travel time (per entrance) vs time interval in ZCLC scenario

Chart 7.4 shows the “Average travel time (per entrance) vs time interval in ZCLC scenario”. Comparing the results of Chart 7.4 with the results of the Chart 6.3 “Average travel time (per entrance) vs. time interval in CR scenario” the following results can be outlined:

- Average travel time in Viesques link increases considerably during the three peaks (morning, afternoon and evening peak).
- Average travel time in Molinón link decreases significantly and keeps in low values steadily throughout the day (ranging from a minimum of 85 s/veh to a maximum of 150 s/veh) close to the optimum value 102 s/veh.
- Average travel time in Polytechnic Aularios and Polytechnic Marina links decreases in the three peaks, especially during the afternoon peak. However, the average travel times during the afternoon peak (even though the reduction suffered) are far away from the ideal travel time (213 s/veh and 187 s/veh for Polytechnic Aularios and Polytechnic Marina respectively).
- Average travel time in Highway R link decreases sharply during the morning peak (from 4131 s/veh to 1051 s/veh, both values not close to the optimum 64 s/veh) and during the afternoon peak (from 2074 s/veh to 323 s/veh). During the evening peak average travel time values also decreases and are very close to the optimum one.
- Average travel time in Highway L link decreases considerably in the morning peak (from a maximum on 1444 s/veh to a maximum of 258 s/veh) and slightly during the afternoon and evening peak reaching values near to the optimal one (69 s/veh).

The queue length in the entrances of the roundabout is directly related to the travel time spent by the vehicles. The longer queue, the bigger travel time. Therefore, the reasons cited above to explain variations in maximum queue length are the same that explain the changes in the average travel time between CR scenario and ZCLC scenario.

Chart 7.5 shows a comparison of the overall average travel time per entrance between CR and ZCLC scenarios. This chart clearly shows that although the overall average travel time on Viesques link has sharply increased the same value has considerably decreased for the remaining 5 links.

Table 7.3 shows the percentage change of the overall average travel time for each entrance in ZCLC scenario compared to the CR base scenario.

	Highway L	**Highway R**	**P. Aularios**	**P. Marina**	**Molinón**	**Viesques**
Increase (+) / Decrease (-)	-60.9 %	-70.7 %	-33.0 %	-9.5 %	-82.5 %	+ 217.5 %

Table 7.3. Percentage change of the overall average travel time for each entrance in ZCLC scenario compared to the CR base scenario

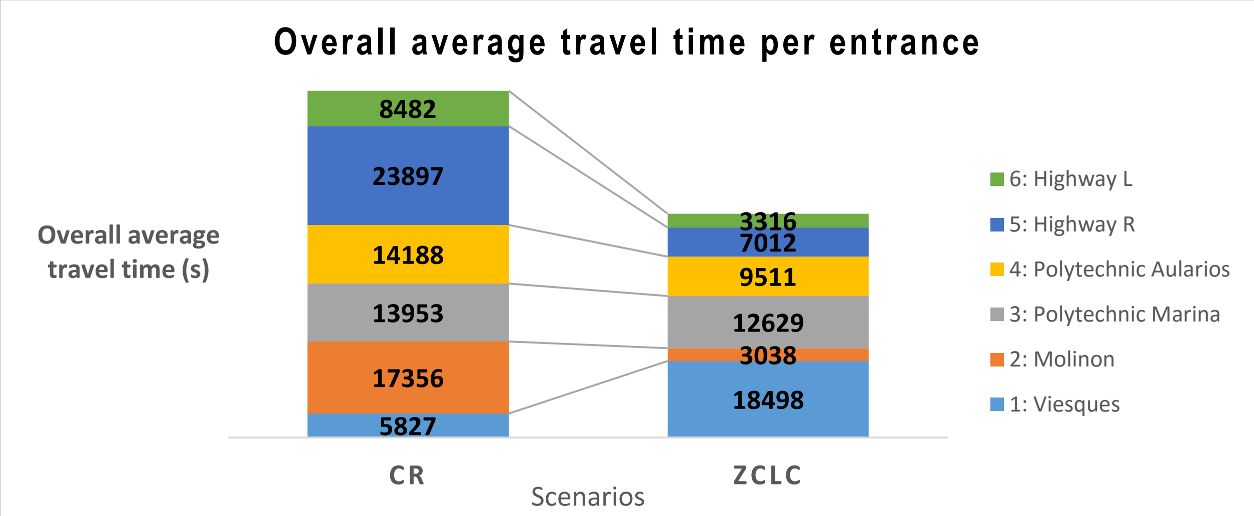


Chart 7.5. Comparison of the overall average travel time per entrance between CR and ZCLC scenarios

7.3.3. Capacity

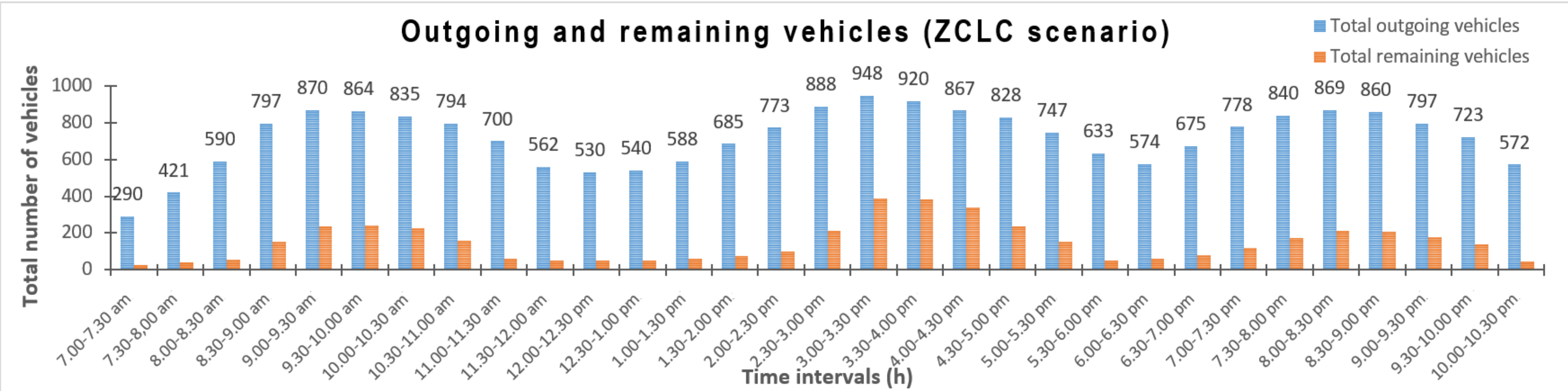


Chart 7.6. Outgoing and remaining vehicles vs. time intervals in ZCLC scenario

Chart 7.6 shows the total number of vehicles that have already reached their destination (total outgoing vehicles*)* and the total number of remaining vehicles in the network. As it was expected, roundabout capacity increases from 850 vehicles per half an hour in CR base scenario to 948 vehicles per half an hour in ZCLC scenario. That means a change from 1700 to **1896 vehicles per hour**. Therefore, **the change of the location of the zebra crossing has increased capacity by 11.5%.**

7.4. Summary of results and conclusions

After the detailed analysis of results carried out in this chapter the following conclusions about the effect of the change of the zebra crossing location can be highlighted:

- Overall, this improvement has a very positive effect on all entrances except in Viesques entrance.
- This improvement reduces the **maximum queue lengths** in Molinón and Highway entrance, it does not significantly affect the maximum queue lengths in Polytechnic entrance and increases the maximum queue lengths in Viesques entrance.
- This change reduces the **overall maximum queue lengths** in Molinón, Highway and Politechnic entrance and increases such value in Viesques entrance.
- The **average queue length** is reduced in Molinón and Highway entrance, it does not significantly change in Polytechnic entrance and it increases in Viesques entrance.
- The **average travel time** decreases in all the links (Molinón, Polytechnic Aularios, Polytechnic Marina, Highway R and Highway L) except in Viesques link. The same happens with the **overall travel time**.
- **Capacity increases by 11.5%**, from 1700 veh/h in the base scenario to **1896 veh/h** in the new scenario.

For all these reasons and taking into account the negative effect in Viesques entrance (which is explained by the fact that this change allows vehicles from Molinón entrance a better use of gaps than in the base scenario and consequently a reduction of available gaps for vehicles wishing to enter the roundabout from Viesques entrance) the authors of this thesis consider that changing the location of the zebra crossing affects positively the overall roundabout performance.

8. Analysis of immediate exits and flared entries

8.1. Introduction

In order to increase the roundabout capacity immediate exits in all the roundabout entrances are going to be added. In addition, Viesques and Polytechnic entrances are going to be flared. This new scenario is called **IEFE** (immediate exits and flared entries)

Immediate exits benefit vehicles entering the roundabout which want to take the adjacent exit since they can directly access to said exit. A priori, it is thought that this improvement increases the number of available gaps in the circulatory roadway, which results in an increase of traffic fluidity, a reduction of congestion and an increase of capacity. It also allows to reduce queues and delays at the roundabout entrances.

It should be noted that the more upstream in the entrance the immediate exit is located, the better results are going to be achieved regarding queues and delays.

Flaring Viesques and Polytechnic entrances and adding one lane in each entrance is a way of increasing the entering traffic when there are available gaps in annular traffic, allowing the entry of more vehicles and thus increasing capacity (see chapter *2.3.2. Influential parameters in the entry capacity of a roundabout*).

8.2. Changes in the simulation model

Figure 8.1 shows the immediate exits (yellow lines) that have been added to the simulation model. As a consequence, Highway entrance is connected directly with Polytechnic exit, Polytechnic entrance is connected directly with Molinón exit , Molinón entrance is connected directly with Viesques exit and Viesques entrance is connected directly with Highway exit.

Figure 8.1. Immediate exits added to the simulation model

As it can be seen in Figure 8.2, in Viesques and Polytechnic entrance one extra entering lane has been added in each entrance.

Figure 8.2. Flaring of Viesques and Polytechnic entrances

It should be noted that IEFE scenario also includes the improvement of ZLCZ scenario. Therefore, IEFE scenario has the following improvements over the CR base scenario:

- Change of the location of the zebra crossing.
- Immediate exits in all the entrances.
- Addition of one extra lane in Viesques and Highway entrances.

8.3. Simulation results

8.3.1. Queues

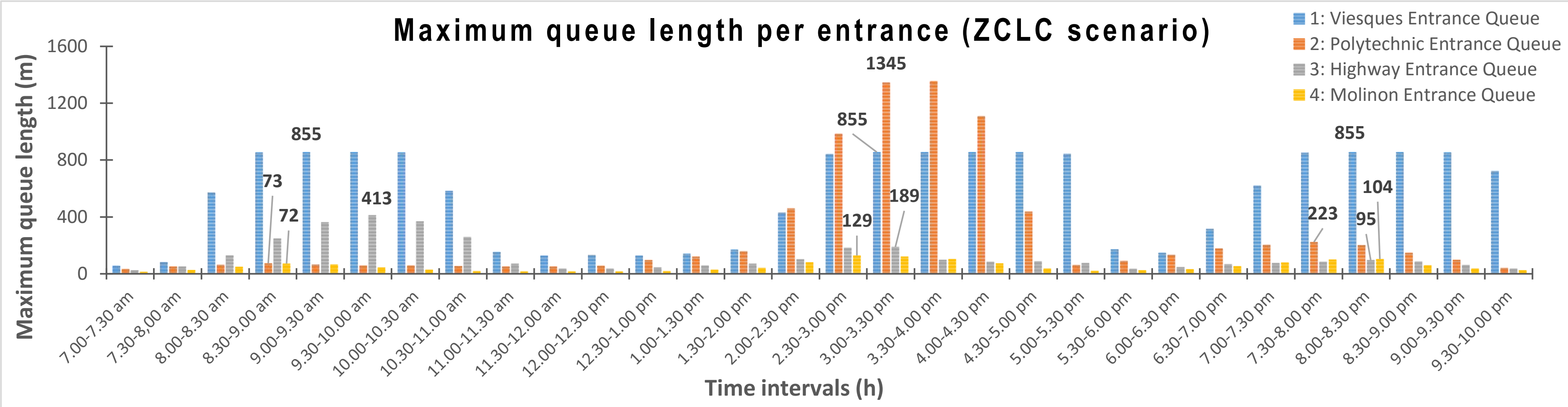


Chart 7.1. Maximum queue length (per entrance) vs time intervals in ZCLC scenario

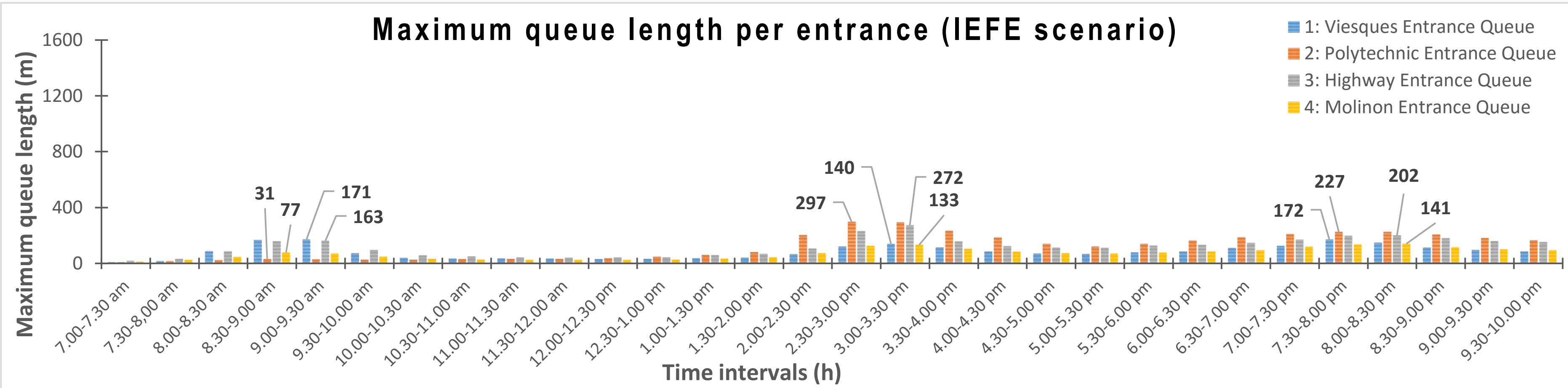


Chart 8.1. Maximum queue length (per entrance) vs time intervals in IEFE scenario

Comparing the results of Chart 7.1 “Maximum queue length (per entrance) vs time intervals in ZCLC scenario” with the results of Chart 8.1 “Maximum queue length (per entrance) vs time intervals in IEFE scenario” it can be concluded that **there is a general reduction in the maximum queue lengths in all the entrances during all day**. An example of such improvement is the reduction of the maximum queue length from 1354 m (Polytechnic entrance in ZCLC scenario) to 297 m (Polytechnic entrance in IEFE scenario).

Chart 8.2 shows the comparison between CR, ZCLC and IEFE scenario considering the addition of the maximum queue lengths per entrance in each time interval. As it can be seen in Chart 8.2, IEFE scenario shows a drastic reduction in the overall maximum queue length comparing with the CR base scenario.

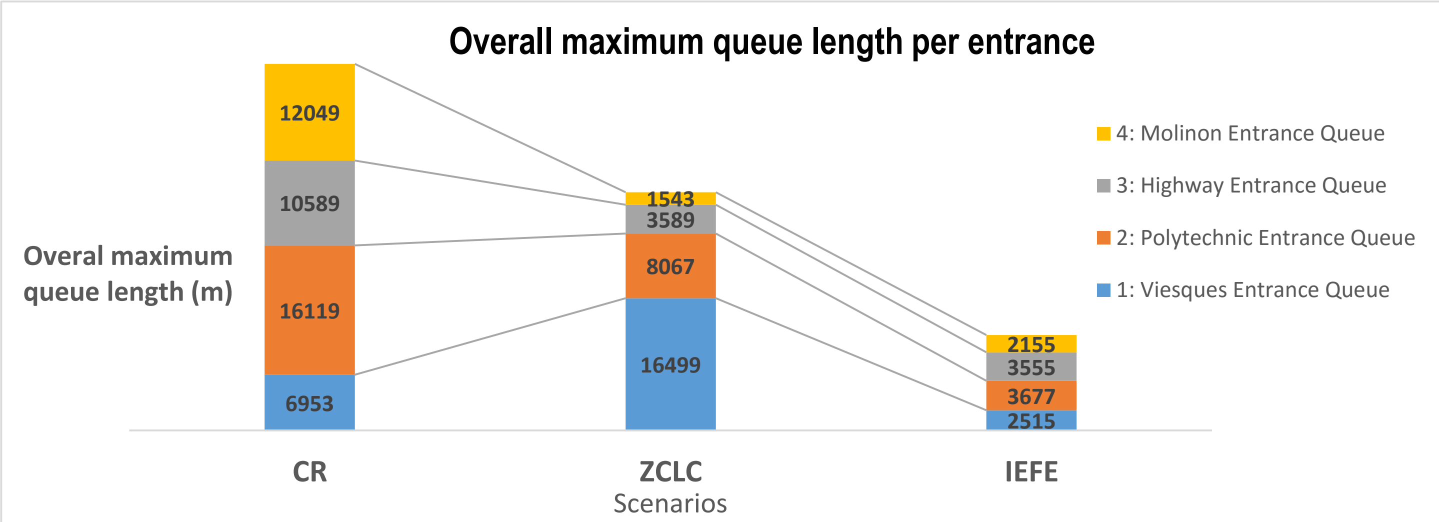


Chart 8.2. Comparison of the overall maximum queues per entrance between IEFE, ZCLC and CR scenarios

Table 8.1 shows the percentage change of the overall maximum queue length for each entrance in IEFE scenario compared to the ZCLC scenario. Even though there is an increase in overall maximum queue length in Molinón entrance, the overall values in the rest of entrances decrease in a higher proportion. It can be concluded that IEFE scenario improves the previous scenario (ZCLC) regarding maximum queue length.

	Molinón entrance	Highway entrance	Polytechnic entrance	Viesques entrance
Increase (+) / Decrease (-)	+39.7%	-0.9%	-54.5%	-84.8%

Table 8.1. Percentage change of the overall maximum queue length for each entrance in IEFE scenario compared to the ZCLC scenario

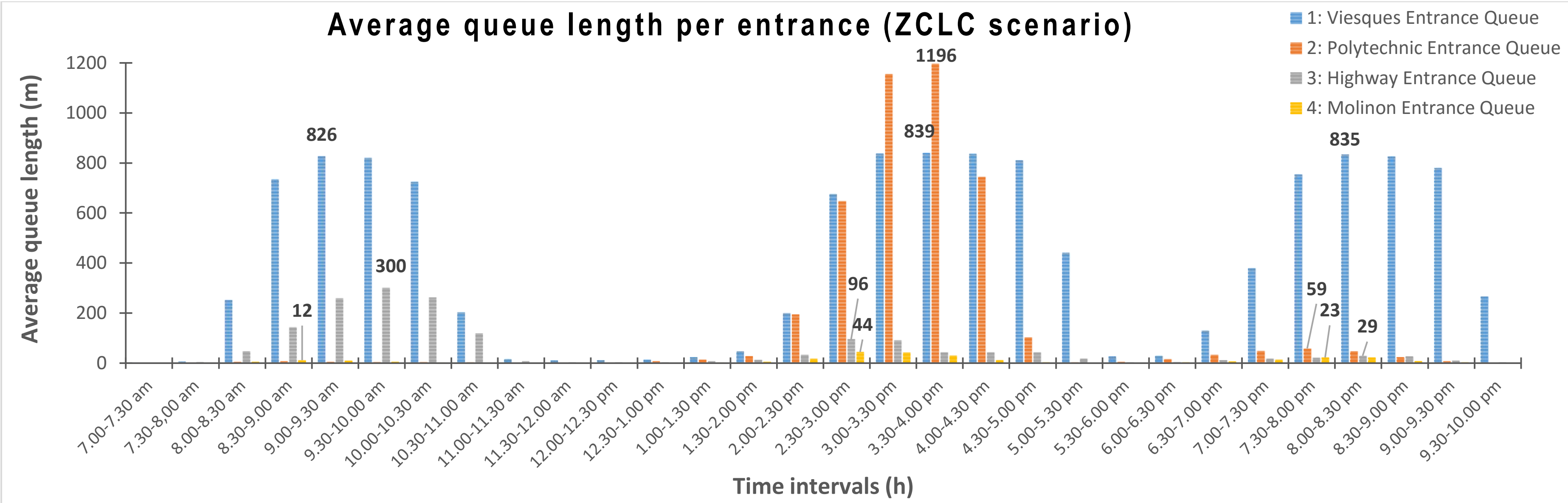


Chart 7.3. Average queue length (per entrance) vs time intervals in ZCLC scenario

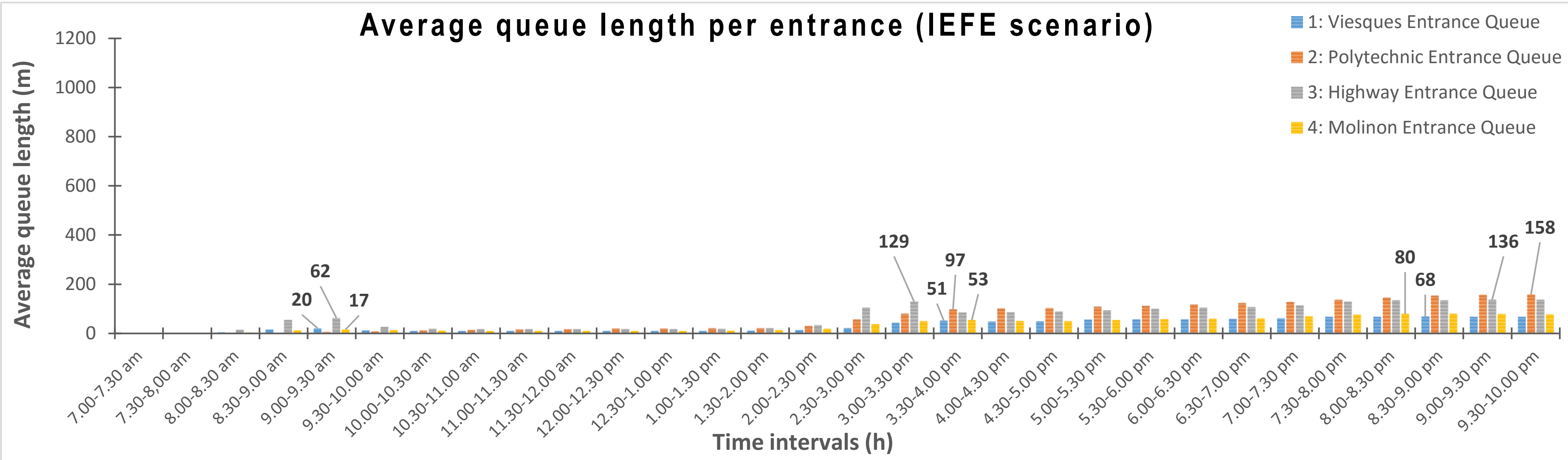


Chart 8.3. Average queue length (per entrance) vs time intervals in IEFE scenario

Comparing Chart 8.3 and Chart 7.3, both related to average queue length in ZCLC and IEFE scenario respectively, the following findings can be emphasised:

- During the **morning peak** the average queue length of Viesques and Highway entrance decrease considerably: Viesques entrance from 826 m to 20 m and Highway entrance from 300 m to 62 m. Regarding average queue length of Polytechnic and Molinón entrances, the average queue formed in these entrances had already been reduced to very low values during ZCLC scenario. However, with the implementation of the IEFE scenario improvements a slightly increase can be told in such entrances.
- The **afternoon peak** shows a severe reduction in average queue length in Polytechnic entrance (from 1196 m to 97 m) and in Viesques entrance (from 839 m to 51 m). On the other hand, Highway and Molinón entrance experience a slightly increment, from 96 m to 129 m and from 42 m to 53 m respectively.
- Finally, the **evening peak** shows a great decrease in the maximum average queue length of Viesques entrance (from 835 m to 68 m). Moreover, a slightly increase in the maximum average queue in the rest of entrances can be caused due to the sharing of capacity in a more homogenous way between all the entrances of the roundabout.

Table 8.2 shows a comparison of the maximum values of the average queue length formed in each entrance during the peak hours between CR, ZCLC and IEFE scenarios.

	Molinón entrance			**Highway entrance**			**Viesques entrance**			**Polytechnic entrance**		
Peak	First	Second	Third	First	Second	Third	First	Second	Third	First	Second	Third
CR scenario	467	716	803	1020	553	278	354	129	194	0	1704	448
ZCLC scenario	12	44	23	300	96	29	826	839	835	0	1196	59
IEFE scenario	17	53	80	62	129	136	20	51	68	9	97	158

Table 8.2. Comparison of the maximum values of average queue length in each entrance during the peak hours between CR and ZCLC scenarios

In conclusion, the improvements of IEFE scenario cause a really positive effect in the average queue lengths. Such improvements have allowed a high reduction on the average queue length in Viesques and Polytechnic entrances during the rush hours.

8.3.2. Travel times

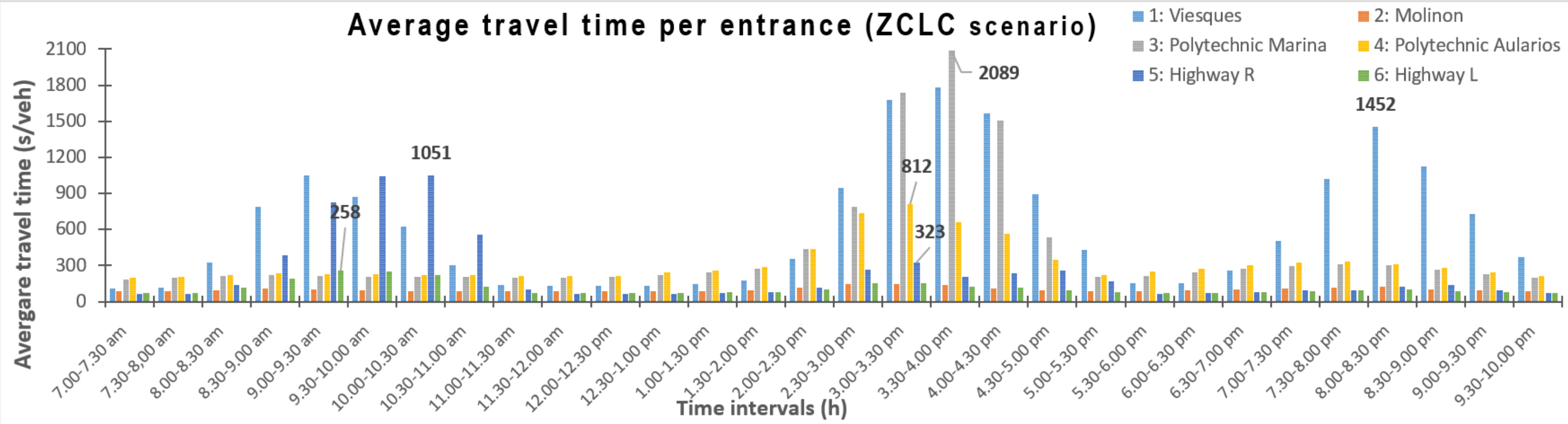


Chart 7.4. Average travel time (per entrance) vs time interval in ZCLC scenario

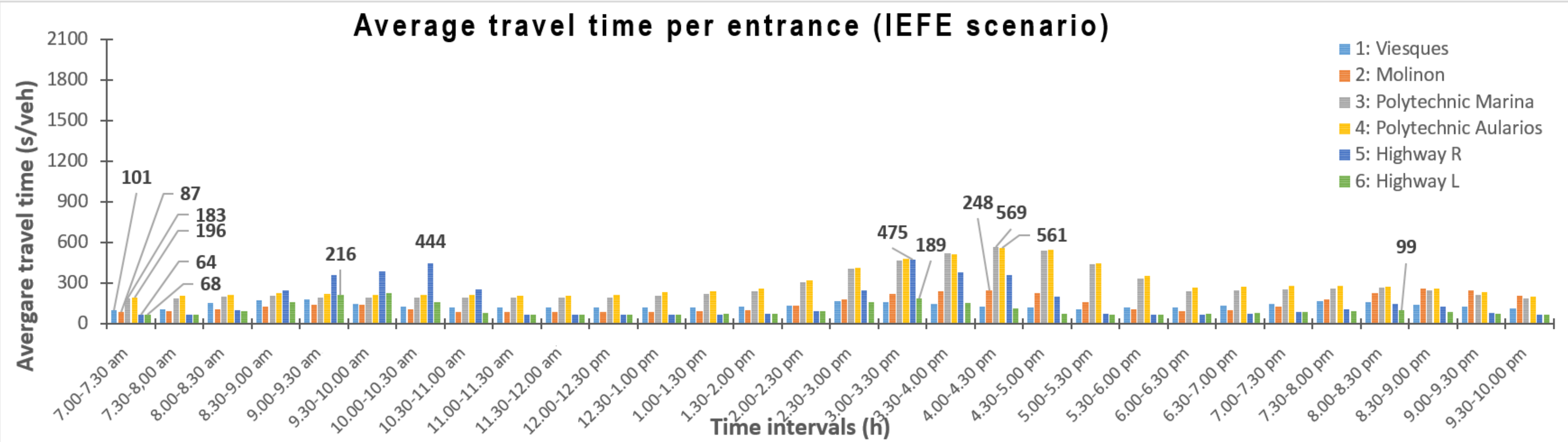


Chart 8.4. Average travel time (per entrance) vs time interval in IEFE scenario

Chart 8.4 shows "Average travel time (per entrance) vs time interval in IEFE scenario". Comparing Chart 8.4 with the results of Chart 7.4 "Average travel time (per entrance) vs. time interval in ZCLC scenario" the following points can be emphasised:

- Average travel time in Viesques link decrease considerably during the three peaks. These values are close to the ideal travel time.
- Average travel time in Molinón link does not experience significant changes from ZCLC scenario. These values are close to the ideal travel time during the morning peak whereas during the afternoon and evening peak are near the double.
- Average travel time in Highway R link decreases during the morning peak. During the afternoon peak average travel time values slightly increase. In the evening peak average travel time values do not experience significant changes.
- Average travel time in Highway L decrease slightly during the morning and afternoon peak. During the evening peak, average travel time values are near to the optimum value.
- Average travel time in Polytechnic Marina and Polytechnic Aularios link decreases during the afternoon peak and does not experience significant changes during the morning and evening peak.

Chart 8.5 shows a comparison of CR, ZCLC and IEFE scenarios considering the overall average travel times per entrance. This chart shows how the overall sum of the travel time in IEFE scenario decrease in comparison with ZCLC scenario.

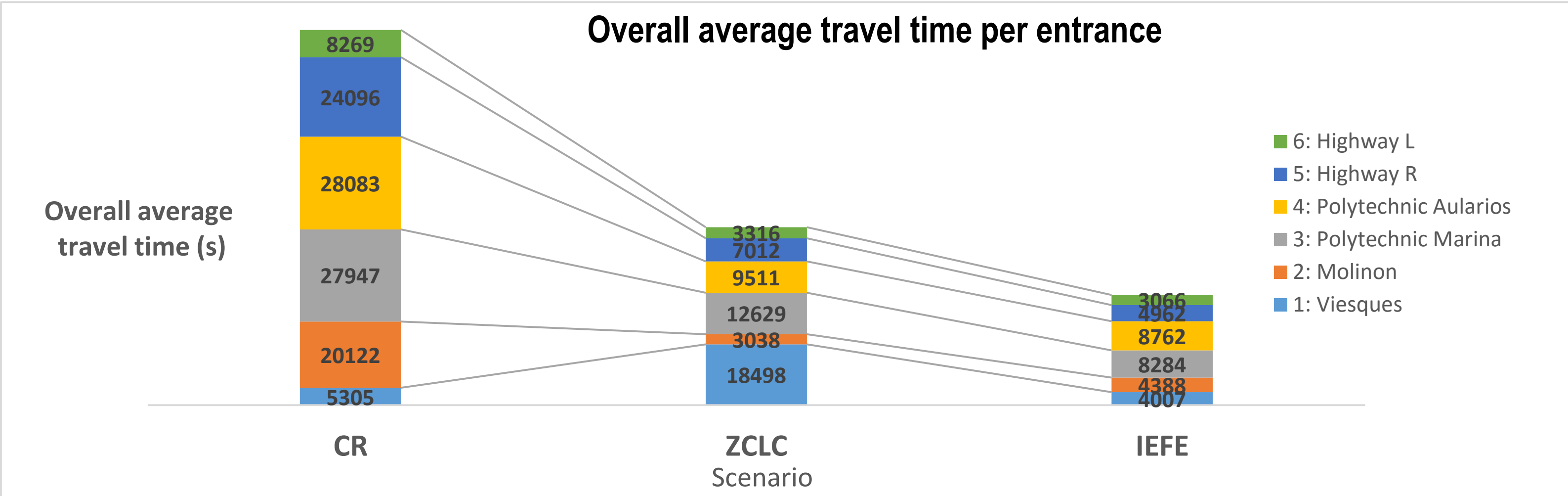


Chart 8.5. Comparison of the overall average travel time per entrance between CR, ZCLC and IEFE scenarios

Table 8.3 shows the percentage change of the overall average travel time for each entrance in IEFE scenario compared to the ZCLC scenario.

	Highway L	Highway R	P. Aularios	P. Marina	Molinón	Viesques
Increase (+) / Decrease (-)	+7.5 %	-29.2 %	-7.9 %	-34.4 %	+44.4 %	-78.3 %

Table 8.3. Percentage change of the overall average travel time for each entrance in IEFE scenario compared to the ZCLC base scenario

Even though there is an increase in the overall average travel time in Molinón entrance of 44.4% and 7.5% in Highway entrance, the decrease in the rest of entrances is higher. It can be concluded that there is an improving (regarding travel times) using IEFE scenario instead of ZCLC.

8.3.1. Capacity

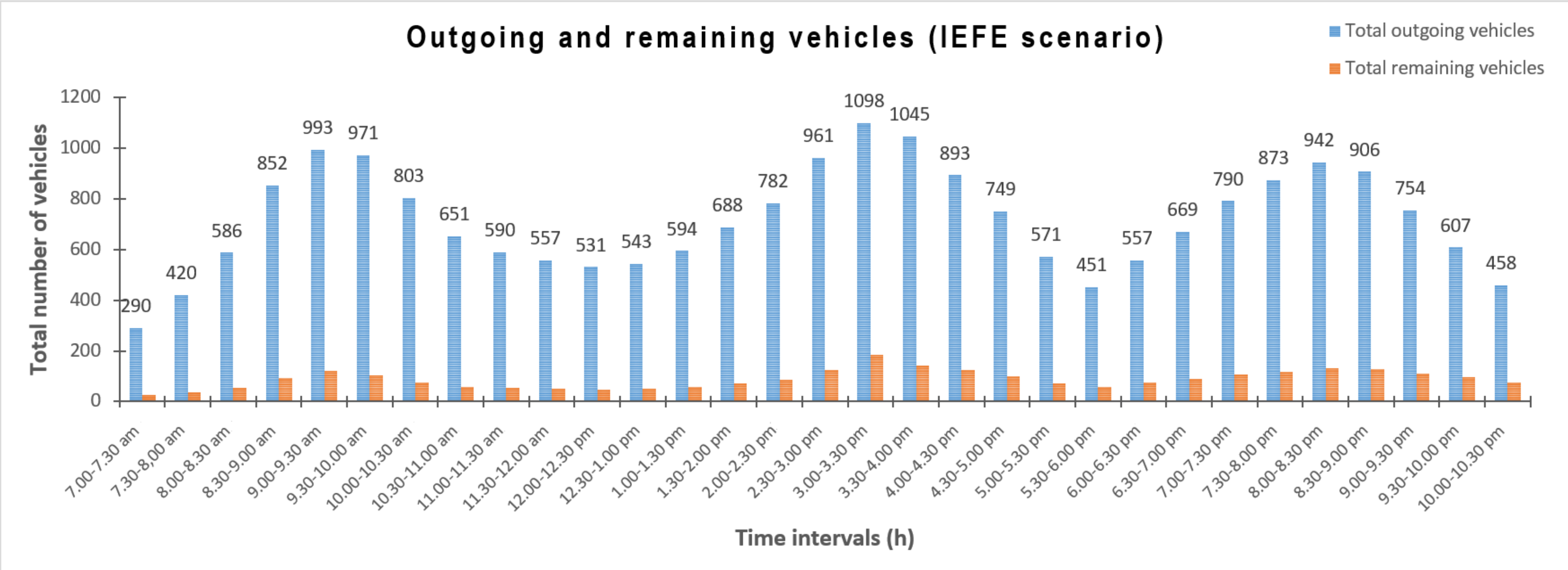


Chart 8.6. Outgoing and remaining vehicles vs. time intervals in IEFE scenario

Chart 8.6 shows outgoing and remaining vehicles in each time interval. Roundabout **maximum capacity** increases from 948 (ZCLC scenario) to 1098 (IEFE scenario) vehicles per half an hour, that means from 1896 to **2196 vehicles per hour**. However, for this value it has been taking into account the immediate exits as a part of the roundabout itself. Therefore, it can be concluded that the difference of maximum capacity between ZCLC and IEFE scenario (300 vehicles per hour) is the extra capacity provided by the immediate exits and the flaring of Viesques and Polytechnic entrances. Analysing remaining vehicles at the end of each time interval it can be concluded that the improvements implemented by IEFE scenario have a good effect in decreasing the amount of vehicles at the end of each time interval. This is because roundabout capacity has been increased, allowing more vehicles to use it.

8.4. Summary of results and conclusions

After the detailed analysis of results carried out in this chapter, the following conclusions about the effect of immediate exits and flaring the entries can be highlighted:

- Overall, changes implemented in this scenario has a very positive effect on all entrances except in Molinón entrance in which the performance gets slightly worse.
- This improvement reduces the **maximum queue lengths** in all the entrances except in Molinón entrance. The same happens with the **overall maximum queue lengths.**
- Analyzing the **average queue length** in every entrance, it can be said that there is a global reduction in average queue length in all the entrances except in Molinón entrance in which they slightly increase.
- The **average travel time** descreases in all the links (Polytechnic Aularios, Polytechnic Marina links, Highway R, Highway and Viesques) except in Molinón link. The same happens with the **overall travel time**.
- **Capacity increases by 15.8%**, from 1896 veh/h in the ZCLC scenario to **2196 veh/h** in the new scenario (IEFE).

For all these reasons and taking into account the minor negative effect in Molinón entrance (which can explained by a different distribution of overall roundabout capacity) the authors of this thesis consider that implementing immediate exits and flaring Viesques and Highway entrances affects positively to overall roundabout performance.

9. Analysis of traffic lights control actuated by detectors

9.1. Introduction

The main novelty of this thesis is the implementation of traffic lights control based on detectors actuated by vehicles. The idea is to let the roundabout work without any traffic light control as long as the traffic situation is not congested and there are no long queues in the roundabout entrances. However, during the rush hours the capacity of the roundabout may not be enough to manage all the traffic. It is at this point when the traffic light system is required and therefore it is activated by the vehicles in the network. So it is a self-regulated system, not being needed any kind of person (such as police) to control the roundabout.

The aim of not controlling the roundabout during the less congested hours is because a roundabout is a very efficient way of managing the traffic when its capacity is not exceeded. If the traffic lights system is working every time (for instance, when the traffic density is low) the own system creates queues and delays that otherwise they would not be formed. For this reason, the authors of this thesis think that a dynamic system which reacts to the traffic situation is the most efficient way of managing the traffic in a roundabout.

It should be highlighted that this scenario is called **ITL** (Implementation of Traffic Lights) and it includes the improvements of IEFE scenario. Therefore, ITL scenario has the following improvements over the CR base scenario:

- Change of the location of the zebra crossing.
- Immediate exits in all the entrances.
- Addition of one extra lane in Viesques and Highway entrances.
- Traffic lights control actuated by vehicles.

9.2. Changes in the simulation model

In order to implement the traffic lights control using detectors it is necessary to implement the following changes in the simulation model:

- Detectors
- Traffic lights
- Control logic

From now on, an explanation of these three points is given in order to understand the simulation model.

9.2.1. Detectors

The system uses presence detectors placed in the roundabout entrances. In Figure 9.1 the location of these detectors can be seen (presence detectors are represented by blue rectangles). The main function of these detectors is to be the link between the traffic light system and the situation of the roundabout regarding the queues formed in the entrances.

Figure 9.1. Detectors

There is one detector in Viesques entrance and another one in Polytechnic entrance. On the other hand, there are two detectors in Molinón entrance (because this entrance has two lanes) and 3 detectors in highway entrance (because there are two lanes and the other detector is placed in Highway R incorporation). Although there are in total 7 detectors, only 4 groups of detectors are considered, belonging each group to one entrance (Highway entrance detectors, Viesques entrance detectors, Molinón entrance detectors and Polytechnic entrance detectors).

The role of the sensors is to detect the formation of a queue. However, detectors are activated as long as a vehicle passes in front of the sensor, but that does not mean that a queue has been formed. For this reason, it is necessary to define when the system considers that a queue has been formed. In order to achieve that, the control system considers that there is a queue between the entrance and the roundabout sensor when the sensor is activated more than a certain amount of time: **10 seconds**.

9.2.2. Traffic lights

The traffic light system consists of 8 traffic lights situated strategically in each entrance of the roundabout. Figure 9.2 shows the location of traffic lights in the roundabout. As it can be seen in such figure, this system allows to control the traffic flux not only in the roundabout entrances but also in the circulatory roadway. Each entrance has the same configuration of traffic lights:

- One traffic light in the yield line, which regulates the traffic which enters from such entrance to the roundabout.
- Another traffic light in the circulatory roadway just in the left of the yield line of such entrance, which regulates the traffic flux in the circulatory roadway (just upstream of each entrance).

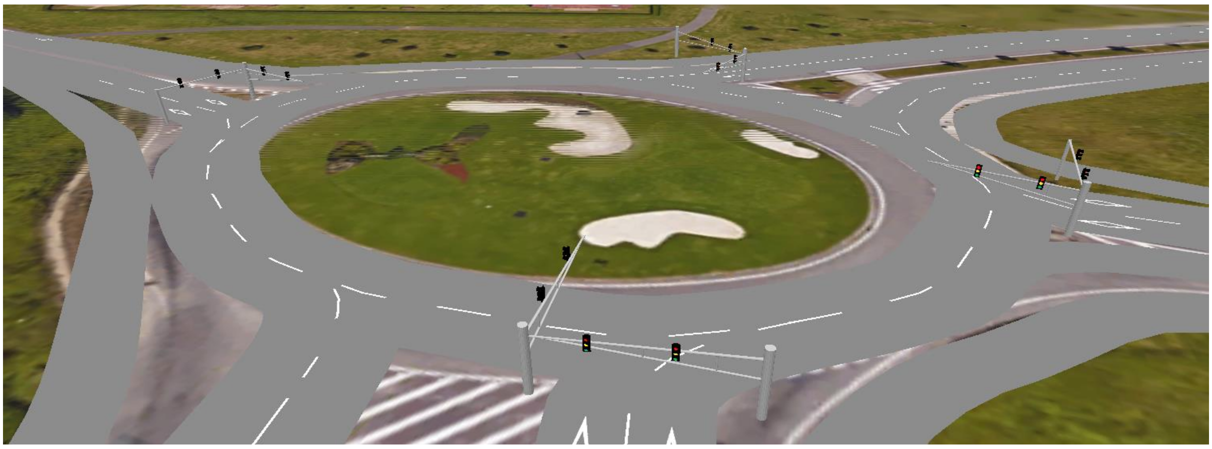


Figure 9.2. Traffic lights location in the roundabout

9.2.3. Control logic

The control system checks the state of the detectors in a fixed cycle (Highway entrance - Polytechnic entrance – Molinón entrance – Viesques entrance). If the detector of an entrance is activated more than 10 seconds the control system relieves congestion in such entrance ("Congestion" state of the control system) and then the system continuously checks the rest of the entrances according to the aforementioned fixed cycle.

It should be noted that in case of two or more detectors are activated more than 10 seconds at the same time, the priority is a function of the aforementioned fixed cycle. For instance, if the detectors of Highway entrance and Viesques entrance are activated (more than 10 seconds) at the same time, Highway entrance is the first of being decongested. Then Polytechnic entrance is checked, Molinón entrance is checked and finally Viesques entrance is decongested.

Two different states can be distinguished in the logic followed by the control system:

- **“Good traffic conditions” state** (no information sent by the detectors, which implies that the traffic light system is not required).
  In this situation the roundabout under study works as a normal roundabout (without any type of traffic light control as long as the traffic system considers there are no queues formed in the roundabout entrances). That means that the traffic conditions are good and the roundabout can self-regulate.
  During this stage, all the traffic lights located in the circulatory roadway are in green mode (allowing the traffic inside the roundabout without restriction) and the traffic lights in the yield lines (which regulate the access to the roundabout) are in flash-amber mode, as it can be seen in Figure 9.3.

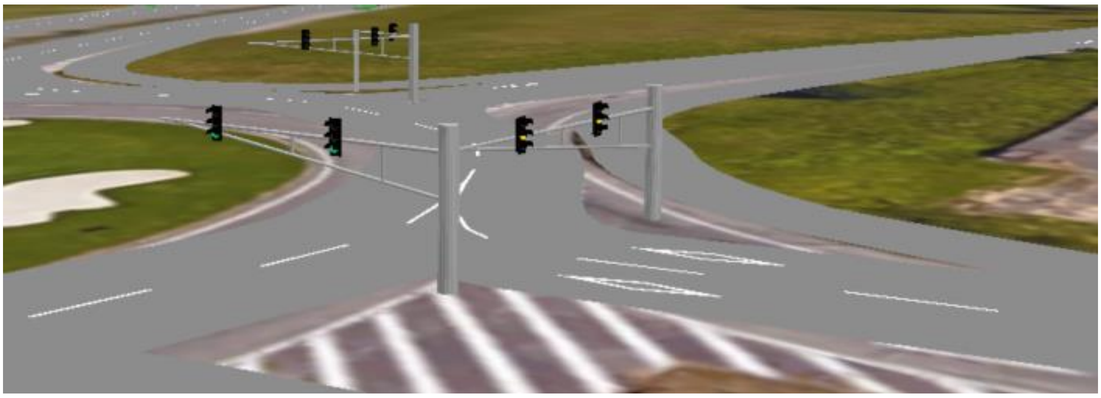


Figure 9.3. Highway traffic lights in “good traffic conditions”

- **“Congestion” state** (traffic density increases and queues are formed so the traffic light system is activated).
  The traffic light system considers that a queue has been formed in a specific entrance when a vehicle remains activating the sensor placed in that entrance more than a certain amount of time (10 seconds). That means that a queue has been formed between the yield line and the detector location. In order to reduce such queue and delays the traffic light system is activated.
  As it has been already mentioned, there are in total 4 groups of detectors (one group per entrance). If the system detects that one of the detectors of a group is activated more than 10 seconds (queue condition), the system is activated. For instance, if one detector of Highway entrance is activated (in the circumstances illustrated by Figure 9.4), the traffic light control system understands that there is traffic congestion in this entrance. Afterwards, the traffic light control is activated allowing the relief of traffic in this entrance.

Figure 9.4. Queue formed in Highway entrance

Continuing with the same example, the traffic light which regulates the entry from Highway entrance to the roundabout is turned green (see Figure 9.5) allowing the vehicles in such entrance to have the priority and to enter the roundabout. Consequently, the traffic light placed just in the left in the circulatory roadway is turned red blocking the traffic in the circulatory roadway, as it can be seen in Figure 9.5.

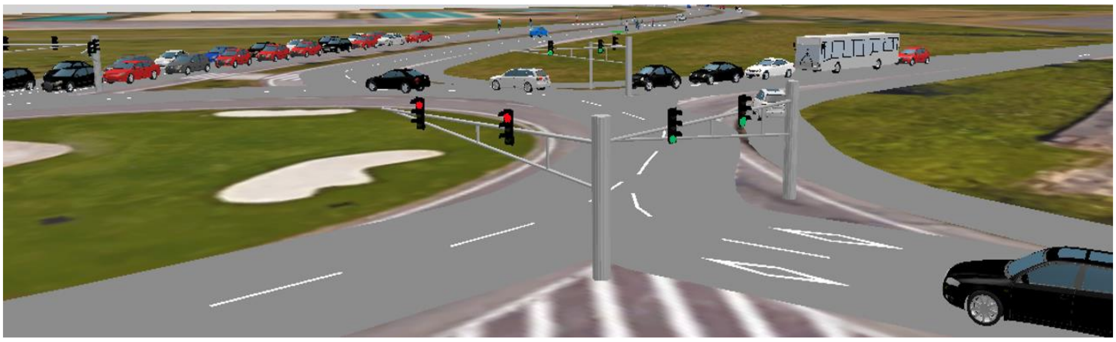


Figure 9.5. Highway traffic lights in “congestion”

In addition, the traffic light in the yield line of Viesques entrance is turned red. The rest traffic lights remain in the same state (in “green” the traffic lights in the circulatory roadway and in “flash-amber” the traffic lights in the yield lines).

Now, it is necessary to define the duration of “Congestion” state. The authors of this thesis think that an appropriate duration is the one that allows the vehicle detected by the sensor to enter the roundabout. Based on the simulation model it has been calculated that such vehicle takes **approximately 30 seconds** to go inside the circulatory roadway once the traffic light is turned green.

Once the traffic congestion has been lightened in such entrance (Highway entrance in the example) the control system checks the state of the others detectors in a fixed cycle (Polytechnic entrance – Molinón entrance – Viesques entrance). If none of the detectors are activated the traffic systems returns to “Good traffic conditions” state. Otherwise, if one or more sensors are activated the control system runs “Congestion” state, according to the priority established in the aforementioned fixed cycle.

The same procedure is followed no matter in which entrance the queue is detected. Figure 9.6, 9.7, 9.8, 9.9 and 9.10 show the different states of the traffic lights depending in which entrance the queue is detected by the control system.

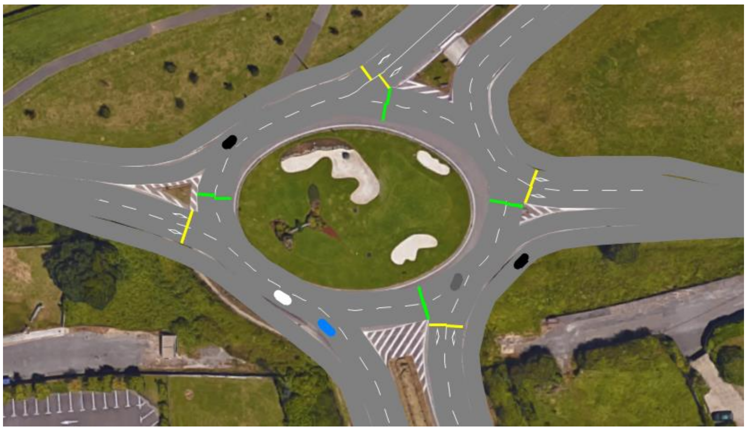


Figure 9.6. No queue detected

Figure 9.7.Viesques queue detected

Figure 9.8. Highway queue detected

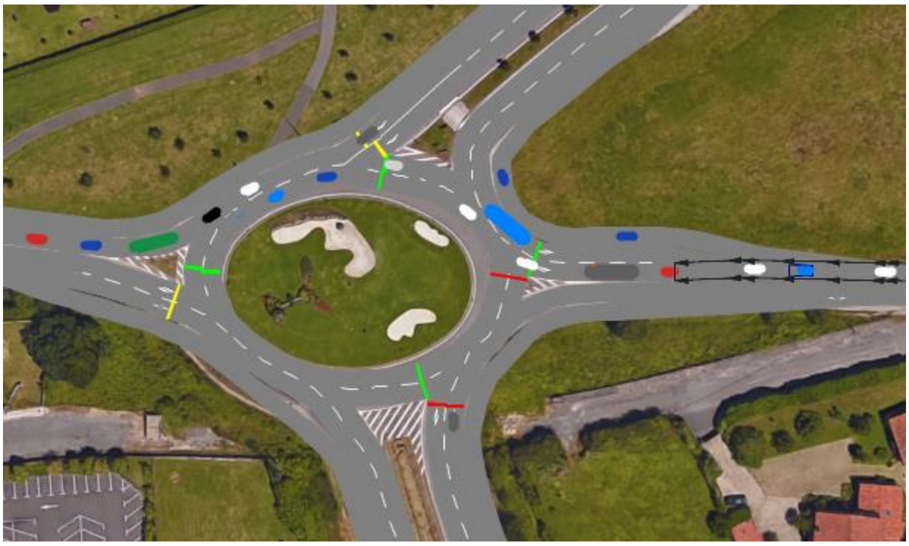


Figure 9.9.Polytechnic queue detected

Figure 9.10.Molinón queue detected

9.2.4. VisVAP module and .PUA file

Once the operation of the control system has been explained the following step is the implementation of the control logic in PTV Vissim. There are two important files which have to be made to define the control logic in the model (*.vap file and *.pua file), as it can be seen in Figure 9.11.

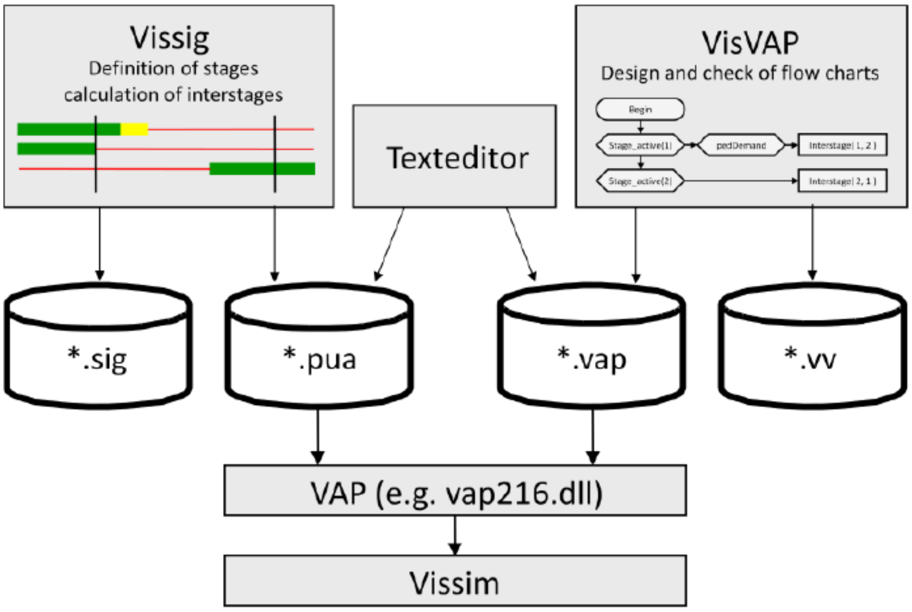


Figure 9.11. How to set the logic control in PTV Vissim [58]

First file ***.vap** can be programmed using VisVAP module, which offers a friendly tool for creating and editing the control logic program of the simulation model as flow charts.

The logic control of the simulation model under study can be seen in Figure 9.12. That file contains the logic related to the commands and assignments that must be fulfilled depending on the logical conditions. Basically, the flow chart of Figure 9.12 establish that the control system checks the state of the detectors in a fixed loop (Highway entrance - Polytechnic entrance – Molinón entrance – Viesques entrance). If the detector of an entrance is activated more than 10 seconds the control system relieves congestion in such entrance during 30 seconds and then, the system continuously checks the rest of the entrances according to the aforementioned fixed loop.

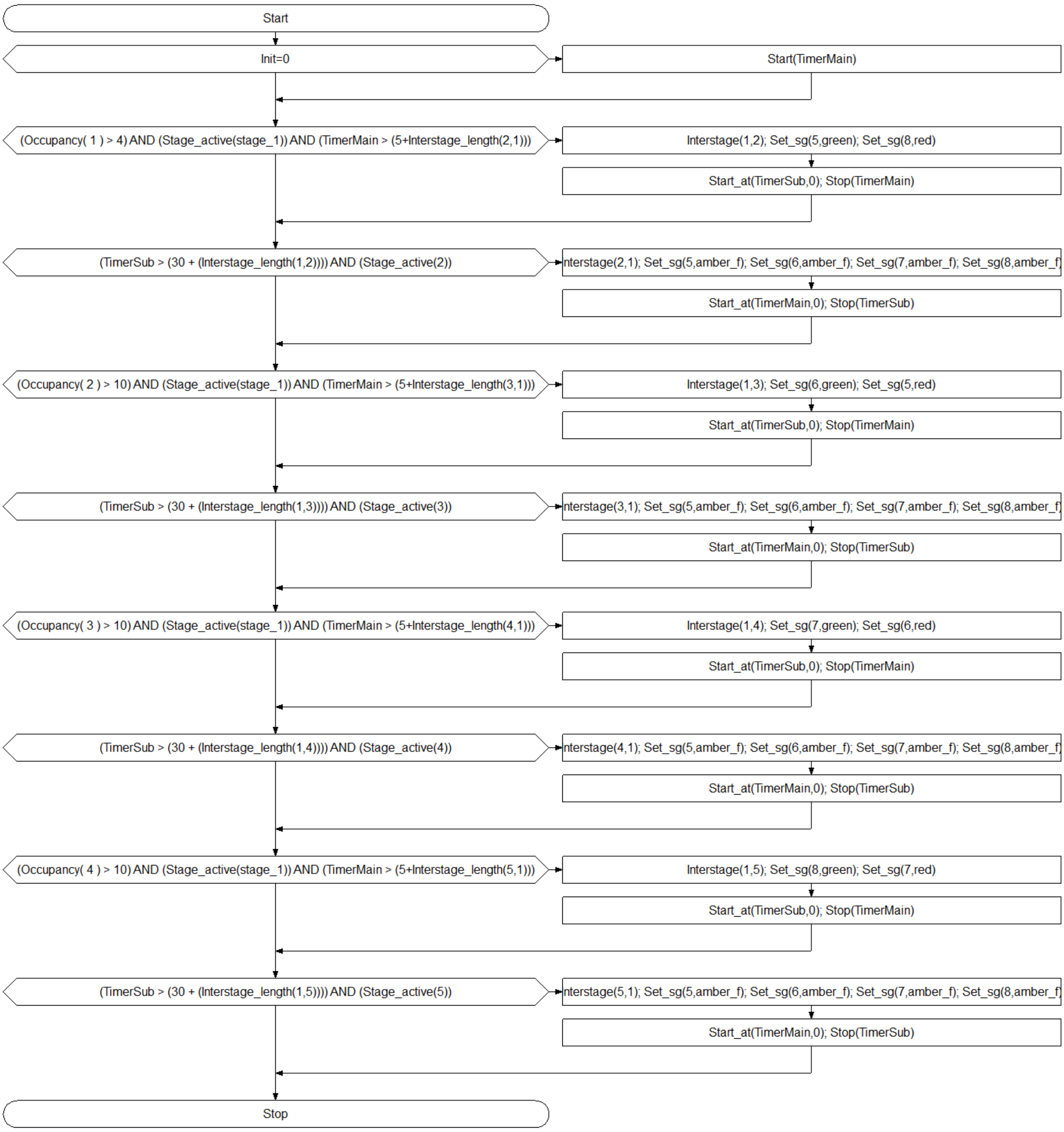


Figure 9.12. Logic control program using VisVAP module

In addition, there is the possibility to program using text editors for those who prefer direct programming of code. The programmed code is shown below.

```
IF Init=0 THEN
Start(TimerMain)
END;
IF (Occupancy(1) > 4) AND (Stage_active (stage_1)) AND (TimerMain
>    (5+Interstage_length(2,1))) THEN
Interstage(1,2); Set_sg(5,green);
Start_at(TimerSub,0); Stop(TimerMain)
END;
IF (TimerSub > (30 + (Interstage_length(1,2)))) AND
(Stage_active(2)) THEN
Interstage(2,1);Set_sg(5,amber_f); Set_sg(6,amber_f);
Set_sg(7,amber_f); Set_sg(8,amber_f);
Start_at(TimerMain,0); Stop(TimerSub)
END;
IF (Occupancy( 2 ) > 10) AND (Stage_active(stage_1)) AND
(TimerMain > (5+Interstage_length(3,1))) THEN
Interstage(1,3); Set_sg(6,green);
Start_at(TimerSub,0); Stop(TimerMain)
END;
IF (TimerSub > (30 + (Interstage_length(1,3)))) AND
(Stage_active(3)) THEN
Interstage(3,1);Set_sg(5,amber_f); Set_sg(6,amber_f);
Set_sg(7,amber_f); Set_sg(8,amber_f);
Start_at(TimerMain,0); Stop(TimerSub)
END;
IF (Occupancy( 3 ) > 10) AND (Stage_active(stage_1)) AND
(TimerMain > (5+Interstage_length(4,1))) THEN
Interstage(1,4); Set_sg(7,green);
Start_at(TimerSub,0); Stop(TimerMain)
END;
IF (TimerSub > (30 + (Interstage_length(1,4)))) AND
(Stage_active(4)) THEN
Interstage(4,1);Set_sg(5,amber_f); Set_sg(6,amber_f);
Set_sg(7,amber_f); Set_sg(8,amber_f);
Start_at(TimerMain,0); Stop(TimerSub)
END;
IF (Occupancy( 4 ) > 10) AND (Stage_active(stage_1)) AND
(TimerMain > (5+Interstage_length(5,1))) THEN
Interstage(1,5); Set_sg(8,green);
Start_at(TimerSub,0); Stop(TimerMain)
END;
IF (TimerSub > (30 + (Interstage_length(1,5)))) AND
(Stage_active(5)) THEN
Interstage(5,1);Set_sg(5,amber_f); Set_sg(6,amber_f);
Set_sg(7,amber_f); Set_sg(8,amber_f);
Start_at(TimerMain,0); Stop(TimerSub)
END
```

The file ***.pua** defines the traffic signal groups, their stages and interstages. Due to the lack of PTV Vissig software (Figure 9.6), this file has been edited manually. The structure of the file is the following:

- Definition of all signal groups
- Definition of all stages
- Definition of start stage
- Definition of all interstages

```
$SIGNAL_GROUPS
$
K1  1
K2  2
K3  3
K4  4

$STAGES
$
stage_1 K1,K2,K3,K4
stage_2 K2,K3,K4
red     K1
stage_3 K1,K3,K4
red     K2
stage_4 K1,K2,K4
red     K3
stage_5 K1,K2,K3
red     K4

$STARTING_STAGE
$
stage_1

$INTERSTAGE
Interstage_number     :12
length [s]            :3
from stage            :1
to stage              :2

$
K1    -127   1
K2    -127   127
K3    -127   127
K4    -127   127

$INTERSTAGE
Interstage_number     :21
length [s]            :3
from stage            :2
to stage              :1

$
K1    3      127
K2    -127   127
K3    -127   127
K4    -127   127

$INTERSTAGE
```

```
Interstage_number      :13
length [s]             :3
from stage             :1
to stage               :3

$
K1     -127   127
K2     -127   1
K3     -127   127
K4     -127   127

$INTERSTAGE
Interstage_number      :31
length [s]             :3
from stage             :3
to stage               :1

$
K1     -127   127
K2     3      127
K3     -127   127
K4     -127   127

$INTERSTAGE
Interstage_number      :14
length [s]             :3
from stage             :1
to stage               :4

$
K1     -127   127
K2     -127   127
K3     -127   1
K4     -127   127

$INTERSTAGE
Interstage_number      :41
length [s]             :3
from stage             :4
to stage               :1

$
K1     -127   127
K2     -127   127
K3     3      127
K4     -127   127

$INTERSTAGE
Interstage_number      :15
length [s]             :3
from stage             :1
to stage               :5

$
K1     -127   127
K2     -127   127
K3     -127   127
K4     -127   1

$INTERSTAGE
```

```
Interstage_number      :51
length [s]             :3
from stage             :5
to stage               :1

$
K1     -127   127
K2     -127   127
K3     -127   127
K4     3      127

$END
```

9.3. Simulation results

9.3.1. Queues

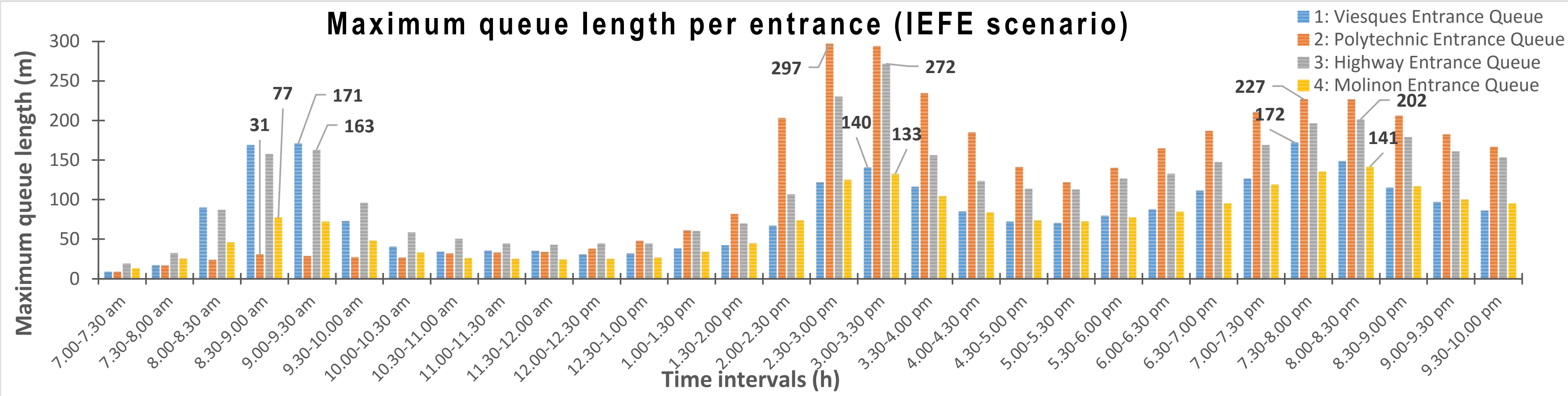


Chart 8.1. Maximum queue length (per entrance) vs time intervals in IEFE scenario

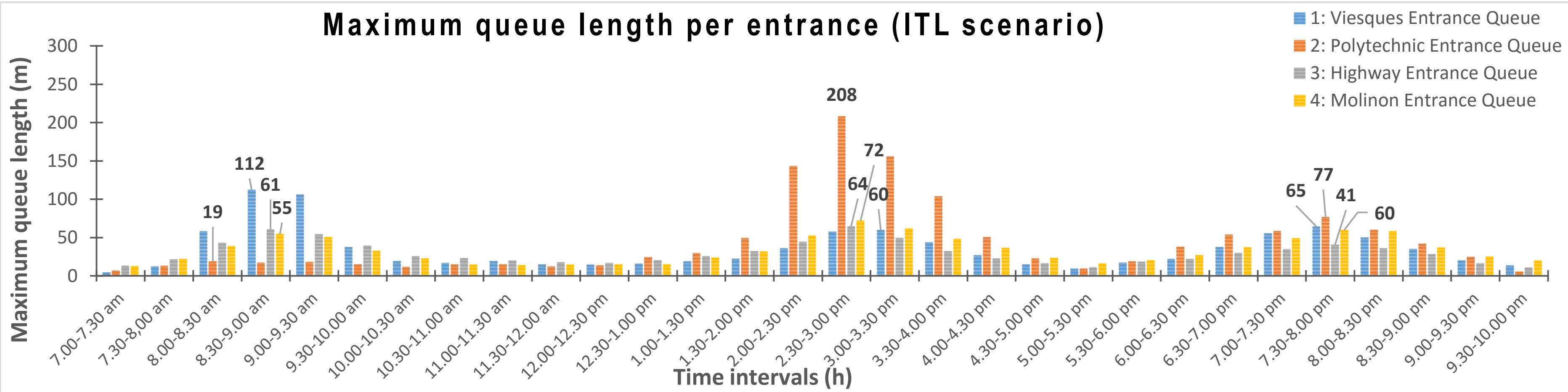


Chart 9.1. Maximum queue length (per entrance) vs time intervals in ITL scenario

Comparing the results of Chart 9.1 “Maximum queue length (per entrance) vs time intervals in ITL scenario” with the results of Chart 8.1 “Maximum queue length (per entrance) vs time intervals in IEFE scenario”, the following points must be underlined:

- Maximum queue length in Viesques entrance decreases during the three peaks (from 171 m to 112 m in the morning peak, from 140 m to 60 m in the afternoon peak and from 172 m to 65 m during the evening peak).
- Maximum queue length in Polytechnic entrance decreases as well during the three peaks (from 31 m to 19 m in the morning peak, from 297 m to 208 m in the afternoon peak and from 227 m to 77 m during the evening peak).
- Maximum queue length in Highway entrance also decreases during the three peaks (from 163 m to 61 m in the morning peak, from 272 m to 64 m in the afternoon peak and from 202 m to 41 m during the evening peak).
- Maximum queue length in Molinón entrance decreases during the three peaks (from 77 m to 55 m in the morning peak, from 133 m to 72 m in the afternoon peak and from 141 m to 60 m during the evening peak).

Chart 9.2 shows the comparison between CR, ZCLC, IEFE and ITL scenarios considering the overall addition of the maximum queue length per entrance. This graph shows the improvement caused by ITL scenario regarding other scenarios.

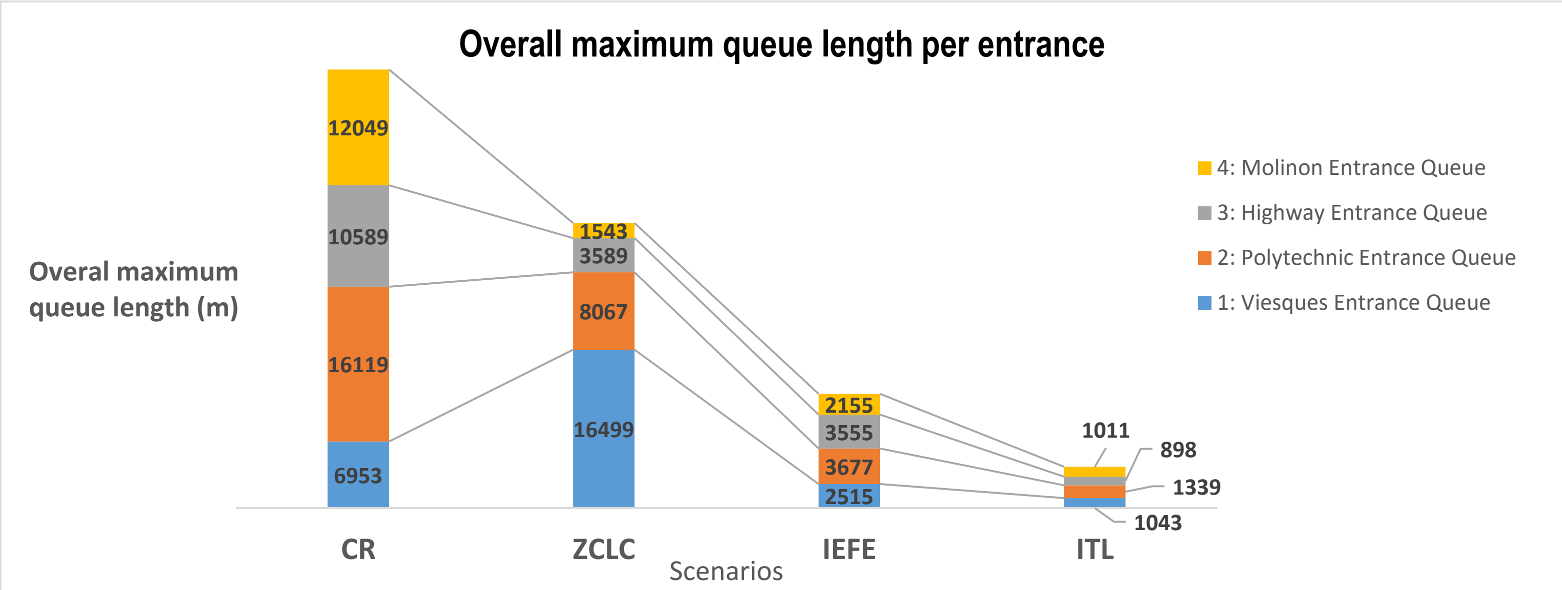


Chart 9.2. Comparison of the overall maximum queues per entrance between CR, ZCLC, IEFE and ITL scenarios

Table 9.1 shows the percentage change of the overall maximum queue length for each entrance in ITL scenario compared to IEFE scenario. It should be highlighted the reduction of the overall maximum queue length in all the entrances.

	Molinón entrance	Highway entrance	Polytechnic entrance	Viesques entrance
Increase (+) / Decrease (-)	-53.1 %	-74.7 %	-63.6 %	-58.5 %

Table 9.1. Percentage change of the overall maximum queue length for each entrance in ITL scenario compared to the IEFE scenario

Comparing the results of Chart 9.3 "Average queue length (per entrance) vs time intervals in IEFE scenario" with the results of the Chart 8.3 "Average queue length (per entrance) vs time intervals in ITL scenario" it can be concluded that there is a general reduction of the average queue length in all the entrances during all day.

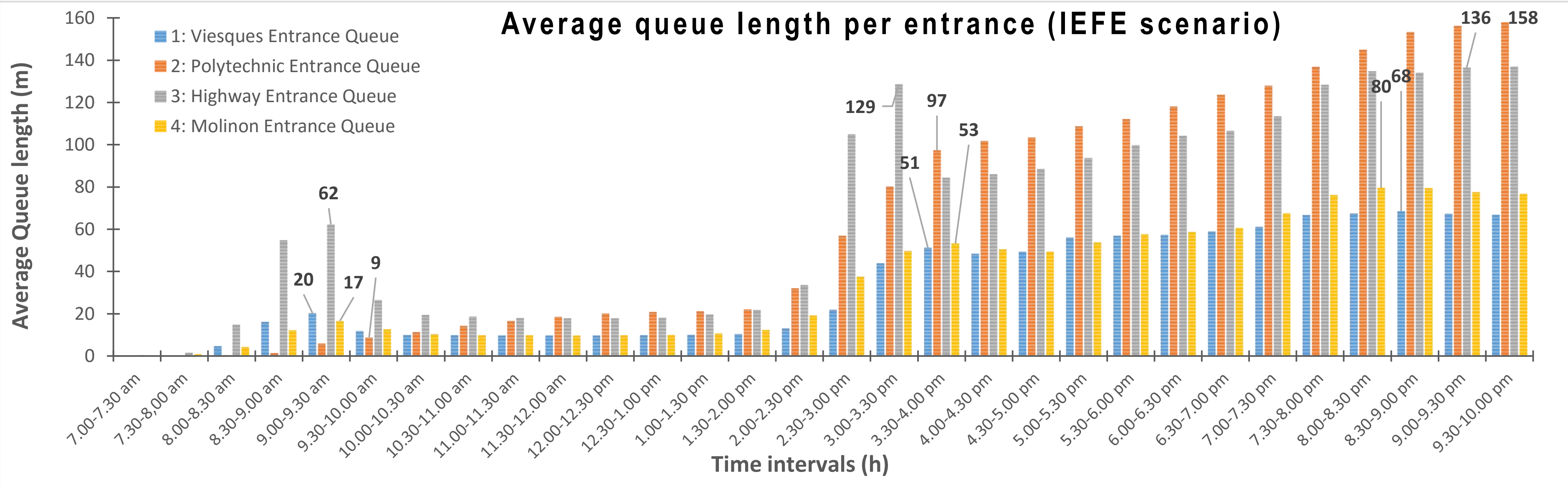


Chart 8.3. Average queue length (per entrance) vs time intervals in IEFE scenario

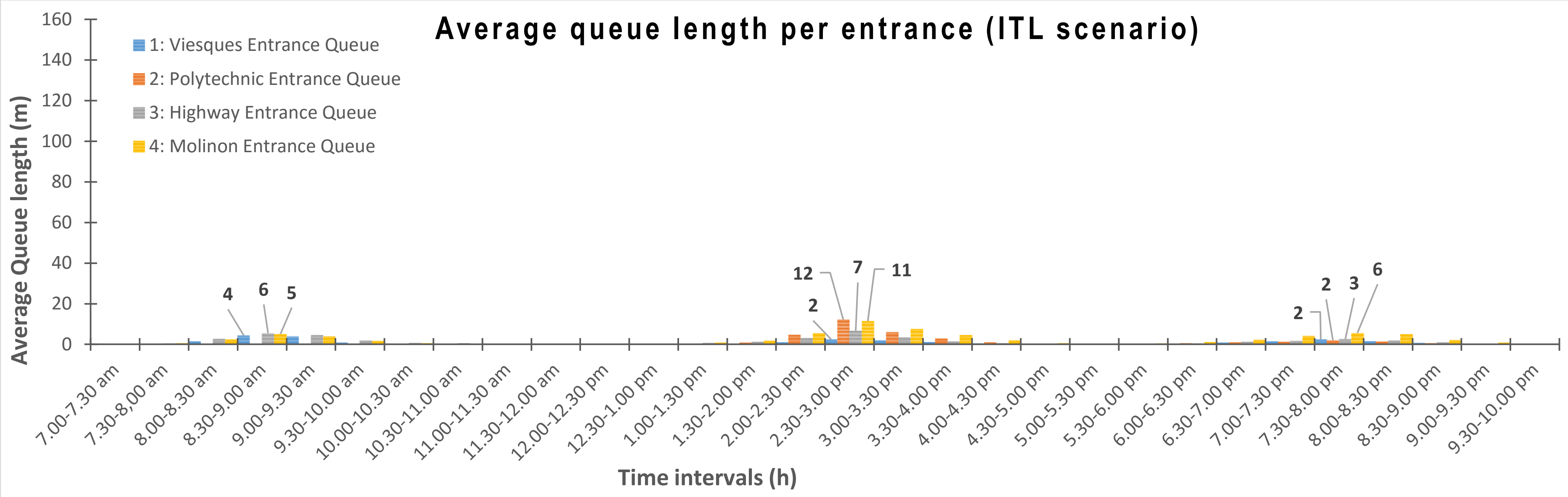


Chart 9.3. Average queue length (per entrance) vs time intervals in ITL scenario

Table 9.2 shows a comparison of the maximum values of average queue length in each entrance during the peak hours between CR, ZCLC, IEFE and ITL scenarios.

	Molinón entrance			Highway entrance			Viesques entrance			Polytechnic entrance		
Peak	First	Second	Third	First	Second	Third	First	Second	Third	First	Second	Third
CR scenario	467	716	803	1020	553	278	354	129	194	0	1704	448
ZCLC scenario	12	44	23	300	96	29	826	839	835	0	1196	59
IEFE scenario	17	53	80	62	129	136	20	51	68	9	97	158
ITL scenario	5	11	6	6	7	3	4	2	2	0	12	2

Table 9.2. Comparison of the maximum values of average queue length in each entrance during the peak hours between CR, ZCLC, IEFE and ITL scenarios

9.3.2. Travel times

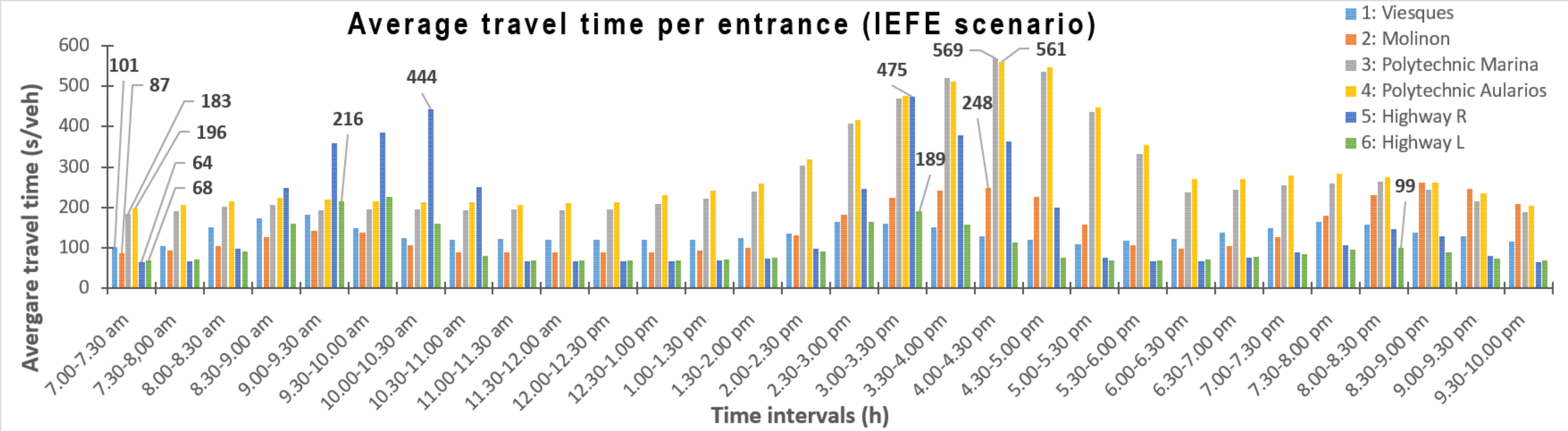


Chart 8.4. Average travel time (per entrance) vs time interval in IEFE scenario

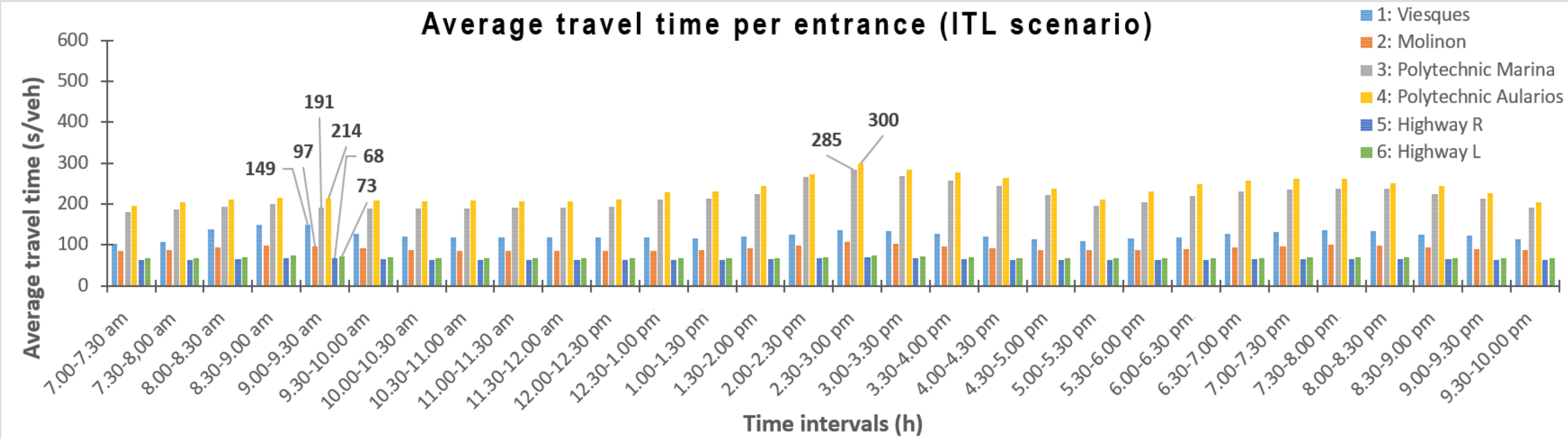


Chart 9.4. Average travel time (per entrance) vs time interval in ITL scenario

Chart 9.4 shows “Average travel time (per entrance) vs time interval in ITL scenario”. Comparing Chart 9.4 with the results of Chart 8.4 the following points can be highlighted:

- It has been a general decrease of the average travel times in all the entrances, overall during the peak hours.
- Homogenous values of average travel time have been achieved during all day. In addition, these values are very close to ideal travel time values (although there is a little increase of travel times during the afternoon peak). That means that ITL scenario is able to manage the traffic influx efficiently.

Chart 9.5 shows a comparison between CR, ZCLC, IEFE and ITL scenarios considering the overall average travel times per entrance. This chart shows how the sum of the overall travel time per entrance in ITL scenario decrease in comparison with IEFE scenario.

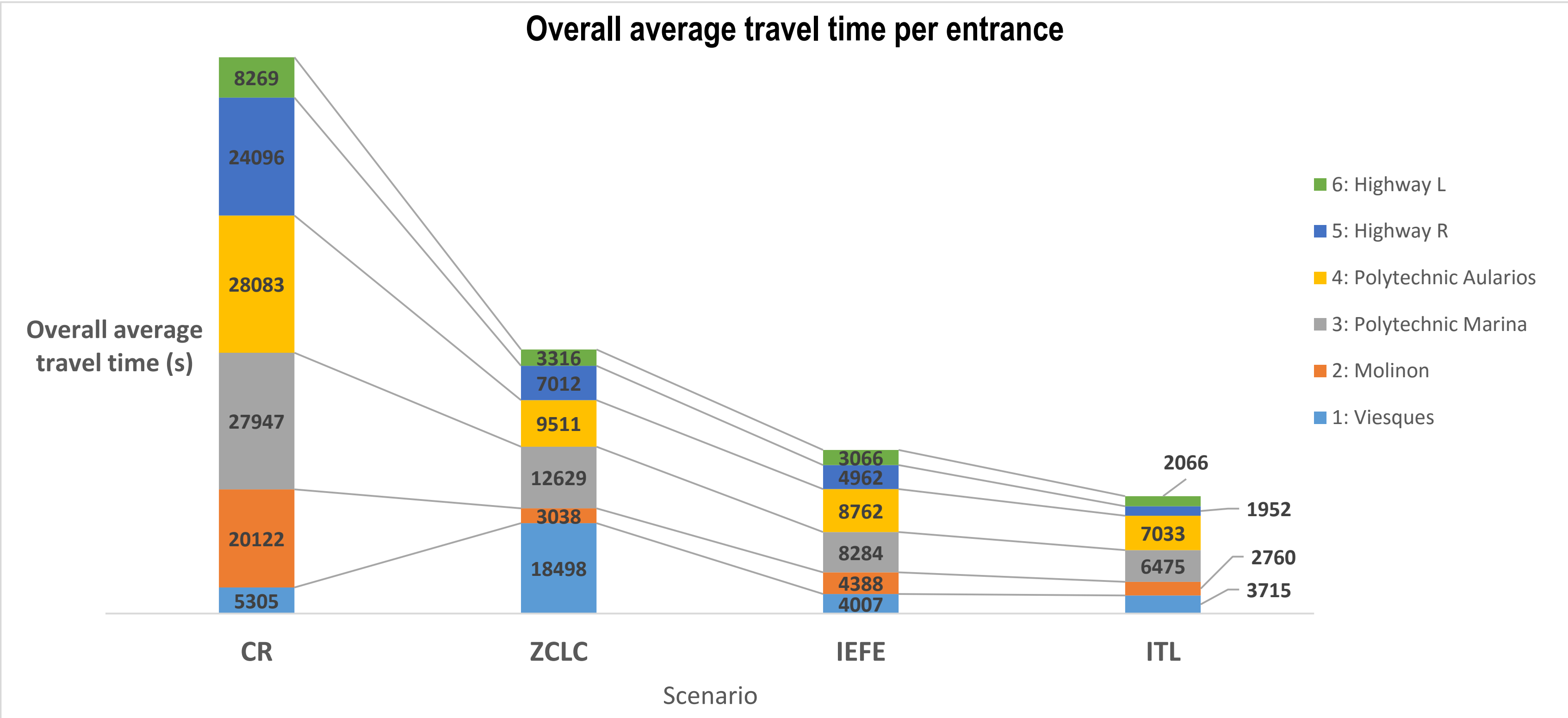


Chart 9.5. Sum of average travel time in ZCLC, CR, IEFE and ITL scenarios

In order to quantify the previous improvement, table 9.3 shows the percentage change of the overall average travel time for each entrance in ITL scenario compared to IEFE scenario.

	Highway L	Highway R	P. Aularios	P. Marina	Molinón	Viesques
Increase (+) / Decrease (-)	-32.6 %	-60.7 %	-19.7 %	-21.8 %	-37.1 %	-7.3 %

Table 9.3. Percentage change of the overall average travel time for each entrance in ITL scenario compared to the IEFE base scenario

9.3.3. Capacity

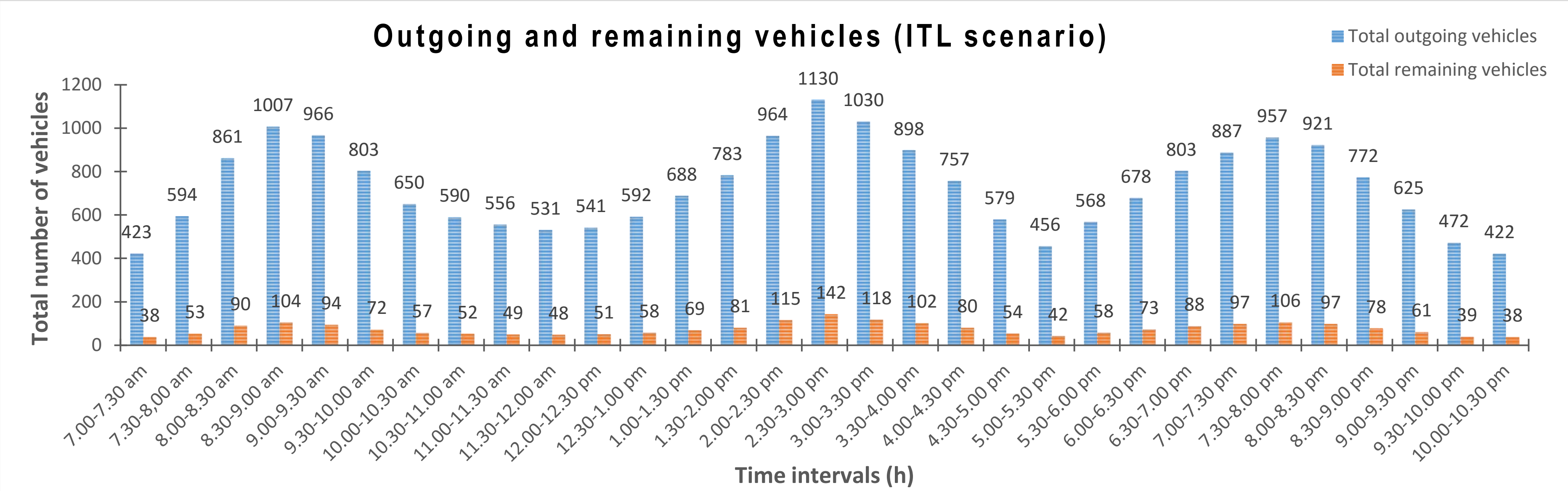


Chart 9.6. Outgoing and remaining vehicles vs. time intervals in ITL scenario

Chart 9.6 shows outgoing and remaining vehicles in each time interval in ITL scenario. Roundabout maximum capacity increases from 1098 (IEFE scenario) to 1130 (ITL scenario) vehicles per half an hour, that means from 2196 to 2260 vehicles per hour. Such increase in capacity (164 vehicles hour) is because of the implementation of the traffic light system.

The increase of capacity is not the only upgrade of ITL scenario but it also helps to make equitable (according queues formed) the use of the capacity of the roundabout between the entrances. The implemented traffic light system avoids the excessive use of the roundabout capacity by only one entrance. That happens when some of the entrances have more traffic volume in comparison with the rest (see chapter *2.6.1. The problem of unbalanced entry flows*). In short, this scenario allows distributing roundabout capacity in function of the amount of vehicles waiting in each entrance, sharing the capacity in relation to the queues formed in each entrance. Therefore, long queues, long delays and the saturation of the roundabout (because of unbalanced entry flows) are avoided.

Regarding to remaining vehicles at the end of each time interval, it can be concluded that the improvements implemented by ITL scenario have a good effect in reduction the amount of vehicles at the end of the afternoon peak comparing with IEFE scenario (see Chart 8.6). This is because the roundabout capacity has increased and is better shared.

9.4. Summary of results and conclusions

After the detailed analysis of results carried out in this chapter the following conclusions about the effect of traffic lights control actuated by vehicles can be highlighted:

- Overall, this improvement has a very positive effect on all entrances.
- This scenario helps to make equitable (according queues formed) the use of the capacity of the roundabout between the entrances.
- This improvement reduces **maximum queue lengths** and **average queue lengths** in all the entrances.
- This change reduces the **overall maximum queue lengths** in all the entrances.
- The **average travel time** descreases in all the links, as well as the **overall average travel time**.
- **Capacity increases by 2.9%**, from 2196 veh/h in the base scenario to **2260 veh/h** in the new scenario.

For all these reasons, the authors of this thesis consider that ITL scenario affects positively to overall roundabout performance because the queues and waiting times decrease considerably.

10. Budget

In this chapter a budget of each scenario is estimated in order to make an approach to the real cost of materializing each scenario. This is interesting because it brings to light if the improvements are worthy regarding the expenses required. Moreover, it should be noted that **these budgets are only an approximation** to the real cost subject to a considerable variability.

10.1. ZCLC scenario budget

Table 10.1 shows the budget for the scenario in which the location of the zebra crossing is changed (Zebra Crossing Location Change scenario). In order to change such location it is necessary to carry out civil works following these steps:

1. Removing the previous zebra crossing lines.
2. Painting new zebra lines.
3. Adapting the access between the sidewalk and the road.

Product description	Cost (€/unit)	Units	Total cost
1. Removing previous zebra crossing lines			
Scraper renting	60 €/day	1 day	60,00 €
Workforce	14 €/h	5 h	70,00 €
2. Painting new zebra crossing lines			
Asphalt white paint	51,5 €/L	4 L	206,00 €
Workforce	14 €/h	6 h	84,00 €
3. Adapting access between sidewalk and road (Sidewalk and road levelling)			
Concrete HM-20/P/40/I	60 €/m^3	1 m^3	60,00 €
Paving stone 1 m	30 €/u	8 u	240,00 €
Workforce	14 €/h	15 h	210,00 €
		Total civil work	**930,00 €**
Quality control (1%)			9,30 €
		Budget of material execution	**939,30 €**
General company expenses (17%)			159,68 €
Industrial profit (6%)			56,36 €
		Budget work execution	**1.155,34 €**
VAT (18%)			207,96 €
		TOTAL ZCLC Budget	**1.363,30 €**

Table 10.1. ZCLC scenario budget

10.2. IEFE scenario budget

Table 10.2 shows the budget for the scenario in which one extra lane is added in Viesques and Polytechnic entrances and direct exits are added in all the entrances (Immediate Exits and Flaring Entrances scenario). It should be recall that IEFE scenario also includes the improvements of ZCLC scenario. Therefore, IEFE scenario budget is the budget of ZCLC scenario adding the cost of the following works:

1. Preparing the ground and removing old asphalt with an excavator.
2. Spreading and compacting artificial gravel (this is a layer of 20 cm of thickness).
3. Asphalting (layer of 8 cm of thickness).
4. Painting new lines.

Product description	Cost (€/unit)	Units	Total cost
1. Preparing ground and removing old asphalt			
Renting excavator (workforce included)	50 €/h	40 h	2.000,00 €
2. Artificial gravel (spreading and compacting)			
➢ Flaring Viesques entrance 8mx40m=320 m^2			
➢ Direct exit Viesques 149mx4m=596 m^2			
➢ Flaring Polytechnic entrance 8mx45m=360 m^2			
➢ Direct exit Polytechnic 144mx4m= 576 m^2			
➢ Direct exit Highway 137mx4m= 548 m^2			
➢ Direct exit Molinon 180mx4m= 720 m^2			
Total area to gravel and asphalt……3120 m^2			
Total volume to gravel: 3120 m^2x0,2m=624m^3			
Artificial gravel (workforce and machinery included)	15€/m^3	624m^3	9.360,00 €
3. Asphalting			
Total volume to asphalt: 3120m^2x0,08mx2,4tm/m3=599 tm			
Asphalt (MBC D-1225) (workforce and machinery included)	45 €/tm	599 tm	26.955,00 €
4. Painting new lines			
Asphalt white paint	51,5 €/L	8 L	412,00 €
Workforce	14 €/h	12 h	168,00 €
		Total civil work	**38.895,00 €**
Quality control (1%)			388,95 €
		Budget of material execution	**39.283,95 €**
General company expenses (17%)			6.678,27 €
Industrial profit (6%)			2.357,04 €
		Budget work execution	**48.319,26 €**
VAT (18%)			8.697,47 €
		Base Budget	**57.016,73 €**
		Base Budget ZCLC	**1.363,30 €**
		TOTAL Budget IEFE	**58.380,03 €**

Table 10.2. IEFE scenario budget

10.3. ITL scenario budget

Table 10.3 shows the budget for the scenario in which a control system of traffic lights is implemented (Implementation of Traffic Lights scenario). It should be recall that ITL scenario also includes the improvements of IEFE scenario. Therefore, ITL scenario budget is the budget of IEFE scenario adding the cost of the following works and materials:

1. Civil work
   - Channelization of the wires needed for the traffic lights
   - Channelization of the wires needed for the detectors
   - Foundation of traffic lights columns
2. Traffic light system
   - 8 circular traffic light leds (ambar/green/red states) located in the yield lines (1 traffic light led per lane, see Figure 10.1 and Figure 10.2)
   - 8 circular traffic light leds (red/green states) located in the circulatory roadway (1 traffic light led per lane, see Figure 10.1 and Figure 10.2)
   - 8 traffic light structures (1 structure in each entrance and 4 structures in the circulatory roadway, see Figure 10.2)
   - 7 detectors (see Figure 9.1)
   - 1 PLC
   - 1 exterior cabinet for the PLC
   - Installation and assembly of the control system

Figure 10.1. Traffic light leds per lane

Figure 10.2. Total number of traffic light leds and traffic light structures

Product description	Cost (€/unit)	Units	Total cost
1. Civil work			
− Channelization of the traffic lights wires below the road (40x60 cm) including PVC tubing of 10 cm diameter. Backfilling with concrete protection, asphalting the surface and workforce also included in the cost.	111 €/u	8	888,00 €
− Channelization of the detectors wires below the road (40x60 cm) including PVC tubing of 10 cm diameter. Backfilling with concrete protection and ground in the surface and workforce also included in the cost.	10 €/m	250 m	2.500,00 €
− Column foundation 0.5x0.5 m of concrete. Excavation materials, workforce and anchor bolts included in the cost.	47 €/u	8	376,00 €
		Total civil work	**3.764,00 €**
2. Traffic light system			
− traffic lights circular leds ambar/green/red states (yiel lines)	210 €/u	8	1.680,00 €
− traffic lights circular leds red/green states (circulatory roadway)	210 €/u	8	1.680,00 €
− Traffic light structure (assembly and placement included in the cost)	1100 €/u	8	8.800,00 €
− Photoelectric sensor, Diffuse system, reflex, Range 11	129,5 €/u	7	906,78 €
− Schneider Electric SR2 A101BD PLC-aansturingsmodule 24 V/DC	135,18 €/u	1	135,18 €
− Galvanized steel exterior cabinet for the PLC 530x375x280 cm	145 €/u	1	145,00 €
− Installation and assembly of the control system (workforce)	20 €/h	40	800,00 €
		Total traffic light system	14.146,96 €
		Total civil work+ Total traffic light system	17.910,96 €
Quality control (1%)			179,11 €
		Budget of material execution	18.090,07 €
General company expenses (17%)			3.075,31 €
Industrial profit (6%)			1.085,40 €
		Budget work execution	22.250,79 €
VAT (18%)			4.005,14 €
		Base Budget	26.255,93 €
		Base Budget IEFE	58.380,03 €
		TOTAL Budget ITL	84.635,96 €

Table 10.3. ITL scenario budget

11. Conclusions

The aim of this chapter is to summarize the most meaningful results obtained in this Master's Dissertation for each scenario.

11.1. Comparing capacity

Chart 11.1 shows a comparison of the roundabout maximum capacity for each scenario. As it can be seen, the capacity increases from 1718 vehicles/h in the base scenario to 2260 vehicles/h in ITL scenario. Moreover, the highest increase of capacity is between ZCLC scenario and IEFE scenario (300 vehicles/h), followed by the increase between CR and ZCLC scenario (178 vehicles/h) and, finally, between IEFE and ITL scenario (64 vehicles/h).

Therefore and according to the previous results, it can be noticed that the increase of capacity decreases as long as the improvements are added.

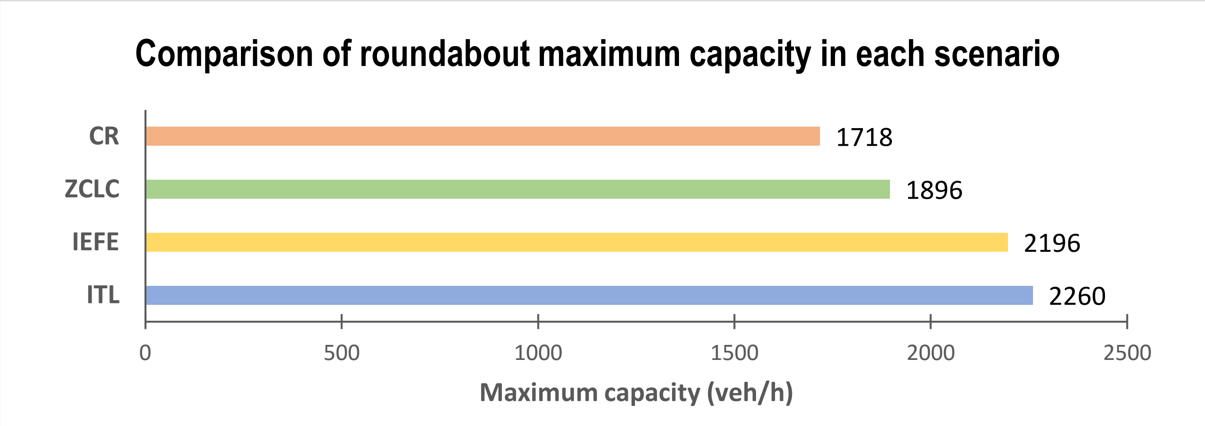


Chart 11.1. Roundabout maximum capacity per scenario

11.2. Comparing delay time

According to PTV, "the delay time of a vehicle in one time step is the part of the time step spent because the actual speed is lower than the desired speed. It is calculated by subtracting the quotient of the actual distance travelled in this time step and the desired speed from the length of the time step" [58].

Chart 11.2 shows the sum of all delay times due to all the vehicles in the network in each scenario. Analysing such chart, it can be concluded that delay time decreases as long as the scenarios are implemented. For example, there has been a delay time reduction of 57.7% from CR scenario to ZCLC, 86.3% reduction from CR to IEFE scenario and 92.2 % reduction from CR to ITL scenario.

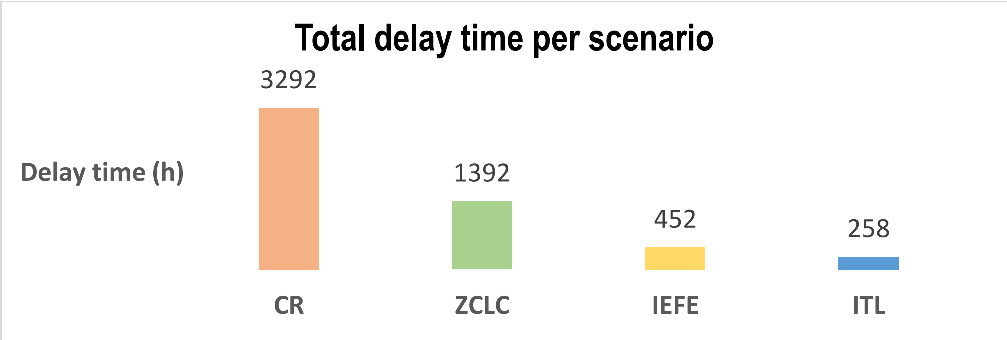


Chart 11.2. Delay time per scenario

Taking into account that on average about 22500 vehicles arrive at the roundabout, an average delay time per vehicle can be calculated. Chart 11.3 shows the result of dividing "total delay time" between the average of vehicles arrived.

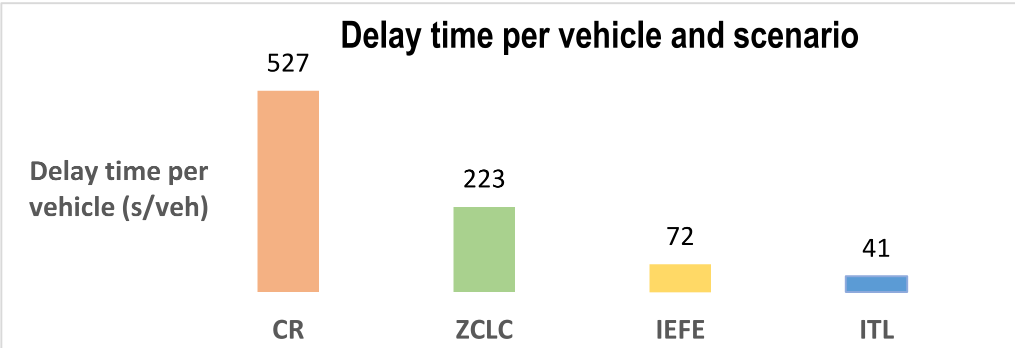


Chart 11.3. Delay time per vehicle and scenario

11.3. Comparing budgets

From an economic perspective, it is important to analyse the cost of each scenario compared to the improvement in traffic conditions.

Chart 11.4 shows the budget of each scenario. It can be noticed that there is a small increment in the budge between CR and ZCLC scenario (1363,30 €), a high increase in the budge between IEFE and ITL scenario (26255,93 €) and the maximum increase takes place between ZCLC and IEFE scenario (57016,73 €).

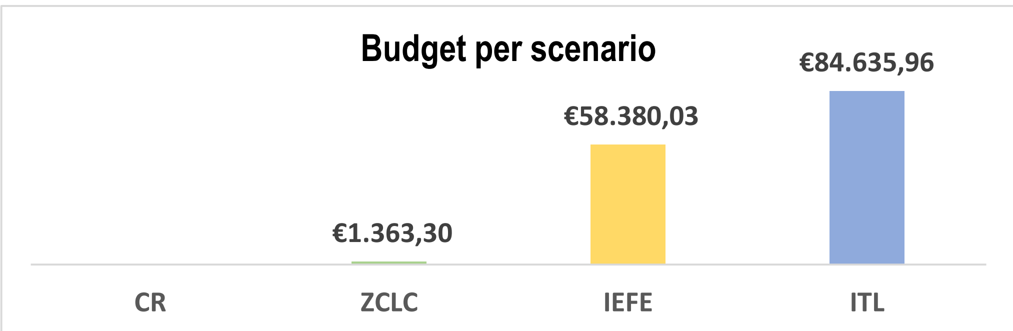


Chart 11.4. Budge per scenario

Even more interesting is to analyse the scenarios from an economic perspective comparing the budget with the expected improvement in each one. Chart 11.5 shows the cost of improving one capacity unit in each scenario (i.e. the cost of incrementing 1 vehicle/h). The increment in the capacity cost is calculated using the following equation:

$$C_{1\,veh/h} = \frac{\Delta budget}{\Delta capacity}$$

Where $\Delta budget$ is the increment of budget needed to go from one scenario to another and $\Delta capacity$ is the increment of capacity between both scenarios.

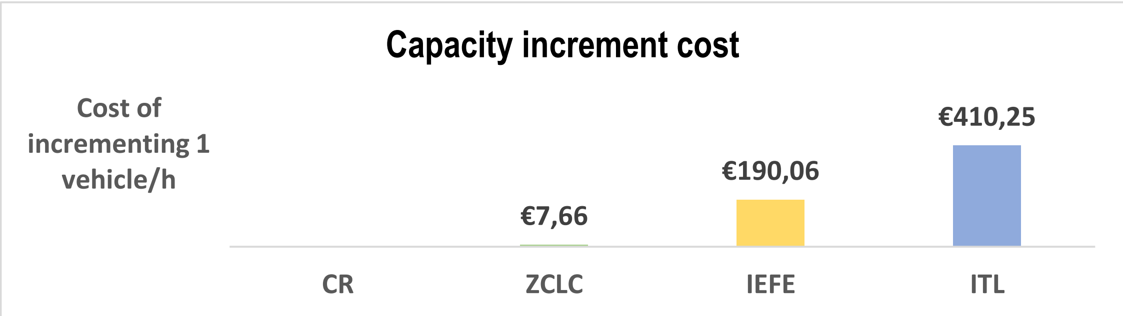


Chart 11.5. Capacity increment cost

As it is expected, as more improvements are implemented in each scenario, the higher the cost is to increment one capacity unit. For instance, the price of increasing one capacity unit from CR to ZCLC scenario is 7,66 € and the total capacity increases in 178 vehicles/h. The cost of improving 1 capacity unit from ZCLC to IEFE scenario is 190,06 € being the total capacity increased by 300 vehicles/h. The maximum cost per unit incremented is between IEFE and ITL scenario (410,25 €) and the capacity improves by 64 vehicles/h.

It should be noted that there are other improvements related with ITL scenario that can justify the high inversion of such scenario. The most important improvement is the roundabout self-control capacity when the traffic conditions are heavy or when there are unbalanced entry traffic flows. The traffic conditions are heavy during the peak hours and unbalanced entry flows take place every time that the local team plays at Molinón stadium, during the Sunday street market or when an event is organized in the International Trade Fair enclosure. For all these reasons, it is common to see police trying to regulate the traffic in the roundabout.

ITL scenario not only would avoid police presence in the roundabout but also it would improve the traffic management due to a more objective distribution of the roundabout capacity between the entrances.

Although ITL budget could seem a bit high, it should be highlighted that such inversion is mainly an initial investment without high expenses after it (only the maintenance of the traffic lights).

It is interesting to calculate the expenses of the current situation of the roundabout related to the local police presence in order to regulate the traffic. With the aim of making an approximation to these expenses the following points have to be considered:

- A local police agent earns around 20 €/h (before taxes).
- Two agents are needed to regulate the traffic flow.
- Peak hours take place every working day (20 days per month).
- Police presence is required during 1 hour in each peak hour (morning, afternoon and evening), that means 60 hours per month.
- Twice per month the local team plays in Molinón stadium. That means 2 hours of police presence before and after the match. In total 8 hours per month.
- Every Sunday, 4 hours of police presence are required due to the street market: in total 16 hours of police presence per month.
- It is supposed that 1 extraordinary event per month takes place, with 5 hours of police presence in the roundabout.

All these considerations sums 89 hours per month. Taking into account that two agents are required, a total of 178 hours of police presence are needed. Therefore, an expense of 3560 € per month (42720 € per year) are required to regulate the roundabout by police presence. Taking into account all these estimations, and comparing police presence cost with the budget of ITL scenario (84635,96 €), the repayment period of ITL scenario is 23,8 months (1,98 years).

11.4. Suggestions for further research

In order to improve or even further develop this Master's Dissertation the authors propose to consider the following points:

- The best (and most obvious) way to improve this thesis lies in the validation of the results. As it was said in *Chapter 5.5. Determining sample size. Results validation*, using a microsimulation software as PTV Vissim which models each vehicle as an independent entity results in a high variability of results. Therefore, a high number of replicas are needed to obtain significant results, necessitating a high simulation time.
  A suggestion for further research is to simulate the four scenarios (CR, ZCLC, IEFE and ITL) the required number of replicas required to obtain a confidence level of 95% and an error margin of 5%. In addition, it should be analysed how the results change.

- Moreover, the scenarios presented in this thesis have infinite possibilities of analysis. The authors highlight the following proposals for future research:
  - Regarding ZCLC scenario, the authors propose to analyze how the results vary depending on the distance between the location of zebra crossing and the yield line of Molinón entrance.
  - Regarding IEFE scenario, the authors suggest to study what combination of direct exits optimizes more the results while minimizing the cost.

- Regarding ITIL scenario, the authors propose to analyse what combination of traffic lights presents a better relationship between optimizing roundabout performance and minimizing the cost.

  Also, it could be analysed how the roundabout performance varies when varying the distance between the yield line and the detectors.

11.5. Final conclusions

The development of this Master's Dissertation has shown that the proposed improvements increase roundabout capacity and reduce queues and waiting times. The decision of which improvement should be applied (depending on the available investment) is subjective and ultimately corresponds to the town of Gijón.

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