FACULTY OF CIVIL AND ENVIRONMENTAL ENGINEERING
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Design of Traffic-Actuated Plan Selection Road Signal Control

Supervisor:
Prof. Gaetano Fusco

Correlator:
Dr. Gabriele Randelli

Graduate:
Angelika Wierzchowska

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ABSTRACT

Traffic congestion is a growing problem in urban zones nowadays. It affects air quality, causes delay and jeopardizes safety on the road. Management of large amount of vehicles in metropolitan areas is a problem to be considered and requires an efficient traffic planning and control. The maintenance of safe and efficient signal timing is mightily important, especially as the fuel pricing and the value of time increase. Signal timing improvements are crucial to handle traffic congestion.

Following this further, it is very difficult to improve the performance of urban traffic signal control system efficiently by using traditional methods of modelling and control because of time-variability, non-linearity and indeterminacy of the system. Unfortunately, these methods often do not represent reality in adequate way, because of continuous traffic changes and the frequent presence of unexpected events along the streets.

Traffic congestion can be reduced with effective traffic signal control system. Closed-loop traffic signal control system is an example of such a system. It can be operated primarily in Time of Day Mode (TOD) or Traffic Responsive Plan Selection Mode (TRPS). TRPS mode, if properly configured, can easily handle time independent variation in traffic volumes. Moreover, it can reduce the effect of time aging.

Despite these advantages, TOD mode is used more frequently than TRPS mode. The reason being a lack of formal guidelines or methodology for implementation of TRPS mode. In this research systematic design method of TRPS mode is presented, introducing machine learning techniques to effectively discriminate different traffic scenarios, hence applying best signal plan every time. This methodology when compared with Time of Day mode and evaluated on a closed-loop system along Guglielmo Marconi artery produced an average travel time reduction of 19%.

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Chapter 1

INTRODUCTION

1.1. PROBLEM STATEMENT

The purpose of traffic signals is to separate conflicting movements in time for a given space. A signal timing plan is developed for safe and efficient operation of the signal control for a given traffic demand. In other words, it is necessary to reduce the total delay by optimizing significant traffic parameters according to changes in traffic flow, such as: cycle length, green splits, phase sequences and offset. Implementation of an optimal signal-timing plan in response to traffic demand has always been challenging. This becomes a major concern when traffic demand varies widely and is highly unpredictable.

In closed-loop traffic control systems, there are two control modes for the selection of particular timing plan at a given instant: Time of Day mode (TOD) and Traffic Responsive Plan Selection mode (TRPS). The difference between TOD and TRPS mode is that in TOD mode the timing plans are based on the historical traffic conditions, while in TRPS mode the plans change according to actual traffic demand.

TOD mode assumes that traffic patterns are iterative, hence plans are implemented and actuated at the same time every day, regardless of the existing traffic condition. It works very well on networks with predictable traffic conditions. However, with unexpected traffic flows, signal timing plan working in TOD mode can be inappropriate. Moreover, timing plans have to be continuously updated to match temporal traffic trends.

In order to implement the most suitable signal-timing plan for a current demand and overcome the aforementioned limits of TOD implementation, this thesis introduces and presents a novel traffic responsive plan selection mode, which is mostly based on TRPS mode. TRPS mode relies on system detectors to estimate demand on the network and choose the optimal traffic plan in response to actual conditions.

The Traffic Responsive Plan Selection mode, if correctly configured, provides an efficient operation due to its capacity to adapt anomalous traffic conditions such as special events, incidents, and holiday traffic. The TRPS mode is able to reduce the need for frequent updates or redesign the signal timing plans. It can operate more optimally and efficiently than TOD mode, but its parameters have to be set up correctly for proper operation of the system. Otherwise, inappropriate timing plans will be selected or system will run in a continuous transitioning state. Due to the cumbersome configuration procedure of optimal TRPS system parameters and thresholds, traffic engineers usually choose the time of day mode of operation for its easiness of setup. However, this approach significantly limits the potential benefits that traffic monitoring could gather by combining system detectors with TRPS mode.

Past research regarding TRPS mode only considers only the trivial threshold mechanisms to implement traffic responsive in traffic networks, because this approach is widely adopted and supported by many controller manufacturers. Nonetheless, the machine learning community has developed many smarter decision support systems approaches, which are mostly based on pattern recognition. There is still a very limited research in applying such methodologies to the signal control problem, and this is one of the major contribution of this thesis.

This thesis provide guidelines regarding the design of a new TRPS mode for urban artery, where a pattern recognition approach has been adoped to identify a much more meaningful and descriptive set of traffic features, rather than mere thresholding. Moreover, a second contribution of this thesis has been the evaluation and implementation of traffic responsive control on Guglielmo Marconi arterial network in Rome, analysed through the video detector SmartEye, by mesoscopic simulation using DYNASMART-P.

1.2. PURPOSE AND SCOPE

The purpose of this thesis was to develop a systematic procedure and a general framework to implement Traffic Responsive Plan Selection control mode by applying pattern matching techniques in order to reduce travel time and congestion on urban arterial networks. The main research steps are described below:

1. Detection of traffic data by using any video detector (in this thesis, it has been adopted SmartEye intelligent traffic system);

2. Identification of traffic features and scenarios using K-means clustering;

3. Design of optimum timing plan for each traffic scenario using SYNCHRO 8;

4. System validation by SimTraffic and DYNASMART-P simulation to compare the TOD and TRPS performance in Guglielmo Marconi arterial network.

1.3. THESIS LAYOUT

This thesis is organised into eight chapters. The second chapter presents a background regards to traffic engineering, precisely signal timing control concept, different types of volume detection and simulation models. The third chapter provides a summary for the offline and online control strategies and a review of the previous related work concerning such area of concern. Chapter four presents description and methodology of implementation TRPS mode with the use of pattern matching mechanism. The fifth chapter describes traffic conditions of Guglielmo Marconi signalized intersections. Chapter six presents the cluster analysis in order to identification of traffic scenarios in examined arterial network. In the seventh chapter, optimal signal timing plans are designed for each traffic state. Chapter eight presents system validation by SimTraffic and DYNASMART-P simulation in order to compare TOD and TRPS mode. Finally, the last chapter concludes with the conclusions of the research and recommendations for further study

Chapter 2
BACKGROUND

2.1. SIGNAL TIMING

Traffic signals play a relevant role in the transportation network. The use of them at a busy intersection in a standard urban area can direct the movement of as many as 100,000 vehicles per day. Many of these signals might be improved by modernizing timing plans. Poor or outdated traffic signals give rise to a large portion of delay on urban arterials. Retiming of traffic signals is one of the most fundamental strategies to improve traffic flow and to reduce congestion. Despite the relevant role of traffic signal in traffic management, they are often not proactively managed. Maintenance works are often cancelled or delayed, in order to cut back on budget and staffing.

Signal timing suitable to the condition of the specific intersection gives the opportunity to improve the traffic flows and safety of the transportation network and brings further environmental advantages.

When establishing a signal timing plan we consider the dual objective - efficiency and safety of the service. This requires a plan that assigns right-of-way to the different users. A signal timing plan should adapt fluctuations in mobility demand during each day, week, and year. A suitable plan will reduce road-user costs while systematically serving each movement in an equitable way and without causing an intolerable level of service to any movement.

The signal timing plan should be periodically updated to maintain intersection safety and efficiency, because travel demand patterns change over time. Safety can often be perceived as a superfluous element in order to reach improvements in efficiency and meet steadily growing demand. In reality, traffic signals must serve both, safety and operational efficiency, based on the circumstances. Traffic control signals that are bad designed, improperly operated, inefficiently placed, or poorly maintained, result in excessive delay, insubordination of the indication, and increases of collisions on the roads (1).

Traffic signals correctly timed and designed can provide following benefits:

- Efficient movement of people;
- Maximization of the volume movements operated at the intersection;
- Reduction of the severity and frequency of crashes; and
- Proper levels of accessibility for side street traffic and pedestrians.

There are many signal timing factors that influence efficiency of the intersection including green time movement, cycle length and clearance intervals. On the one hand, thanks to the increase of a traffic movement's green time it is possible to diminish delay and the amount of stopped vehicles. On the other hand, an extension of one movement's green time usually causes increased delay and stops to second movement. A good signal timing plan should allocate time suitably based on the traffic demand at the intersection while maintaining minimum cycle lengths.

The relationship between signal timing and safety is also connected with design of the junction and particular timing parameters. The yellow change interval has to simplify safe transmission or right-of-way from one movement to another. The safety benefit of this interval is realized when its duration is equivalent to the needs of drivers approaching the intersection at the onset of the yellow indication. This need refers to the driver's capacity to perceive the yellow sign and to gauge their capacity to stop before the stop line, or to cross the intersection safely. Many factors, especially speed, influence the decision. Appropriately, timed yellow indication minimizes intersection accidents.

The timing settings at the intersection are designed and implemented by the traffic signal controller. It has to respond to the needs of users and meet objectives defined by the policies of a reliable agency.

2.2. BASIC SIGNAL TIMING PARAMETERS

Figure shown below illustrates the basic operation of vehicular movement through a signalized junction (2). The instantaneous flow of vehicles is shown on the vertical axis, while the traffic signal is on the horizontal axis. During the time of a red indication there is no flow, vehicles arrive and form a queue. Upon receiving a green indication, the driver of the first vehicle need a few seconds to perceive that the signal change and to start the motion. Similarly, the next few vehicles also take some time to accelerate. This is commonly assumed to 2 seconds and defined as the start-up delay or start-up lost time and. After it, the flow rate tends to stabilize at the maximum flow rate, known as the saturation flow rate. Upon termination of the green indication, which should be sustained until the last vehicle of the queue departs the intersection some vehicles still pass through the junction during the yellow change interval. Effective green time for the movement is the usable amount of green time with a duration between the end of the start-up delay and the end of the yellow extension. Clearance lost time is the unused portion of the yellow change and the red clearance interval.

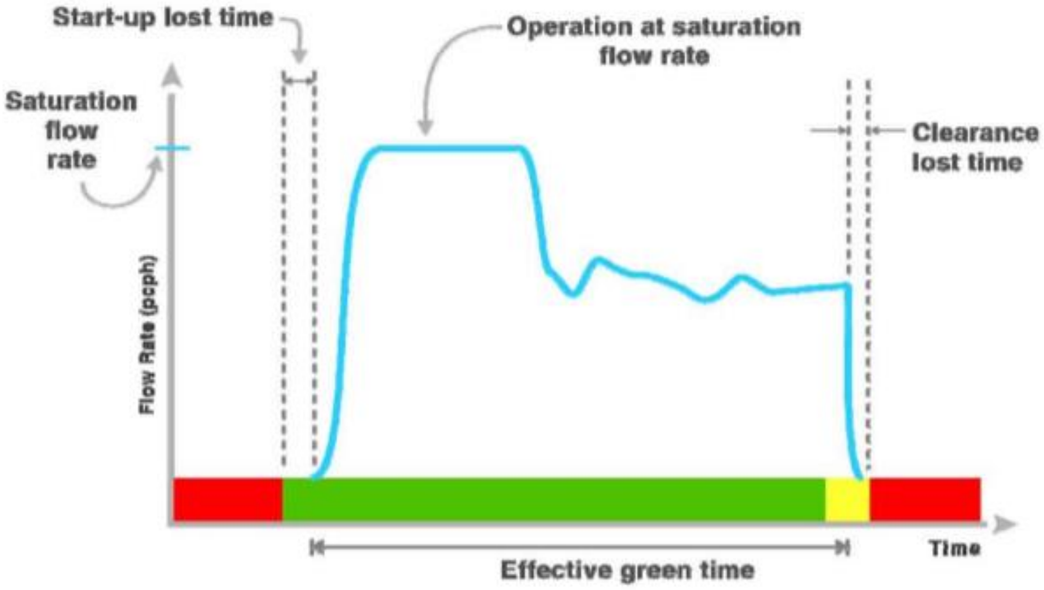


Figure 1: Vehicular movement through a signalized intersection

SATURATION FLOW RATE

Performance of a specific movement is estimated by the saturation flow rate parameter. In other words, it is simply the headway in seconds between vehicles moving from a queue, divided into 3600 seconds per hour. Saturation flow rate for a lane group is a direct function of vehicle speed and separation distance. These are in turn functions of a variety of parameters, such as the number and width of lanes, lane use grades, factors that constrain vehicle movement including presence or absence of conflicting vehicle and pedestrian traffic, bus movements and on-street parking. Consequently, saturation flow rates vary by location, time and movement. Usually it varies from 1,500 to 2,000 passenger cars per hour per lane. Ideal saturation flow rate is commonly assumed 1,900 passenger cars per hour per lane and may not be achieved during each signal cycle. To maximize vehicular movement through the intersection and get optimal efficiency, traffic flow should be sustained at or near saturation flow rate on each approach.

LOST TIME

As described previously, lost time is a time not used by vehicles, which is the sum of the beginning of each green period and a portion of each yellow change plus red clearance period. Lost time is utilized in evaluation of the overall capacity of the junction by removing the sum of the lost times for each of the critical movements from the overall cycle length. The default value of total lost time is defined by HCM as 4 seconds per phase. The equation below defines the resulting effective green time:

$$g = G + Y + R - (l_1 + l_2)$$

g = effective green time;
G = actual green interval;
Y = actual yellow change interval;
R = actual red clearance interval;
l_1 = start-up lost time;
l_2 = clearance lost time.

CAPACITY

The capacity for a particular movement is determined by two elements: saturation flow rate and the ratio of time, during which vehicles may enter the intersection. The flow rate mentioned above is known as the maximum rate at which vehicles can pass through an intersection in an hour under prevalent conditions. The capacity equation is presented below:

$$c = s\left(\frac{g}{c}\right)$$

c = capacity;
s = saturation flow rate of the lane group;
C = cycle length.

Figure 2 shows the capacity as the area bounded by effective green time and saturation flow rate. The area under the flow rate curve represents the volume.

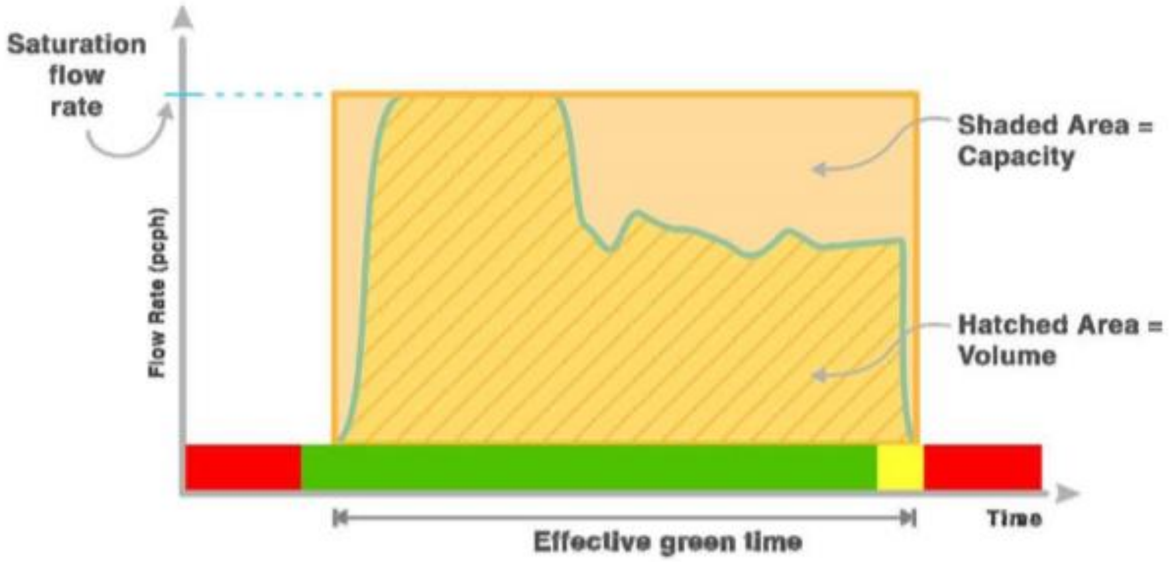


Figure 2: Capacity of the movement

VOLUME-CAPACITY RATIO

The volume-capacity ratio, also known as the degree of saturation X, is calculated for each movement through the following equation:

$$v/c = \frac{v}{s\left(\dfrac{g}{C}\right)} = \frac{vC}{sg},$$

v = demand volume of the subject movement

The volume-capacity ratio represents the proportion of the area, in Figure 2, defining the capacity that is occupied by volume. Unsaturated movements with stable operations and sufficient capacity have volume-capacity ratios less than 0.85. Less stable traffic flow is observed in movements with ratio from 0.85 to 1.00, due to natural cycle-to-cycle variations in the flow. If movement is close capacity, fluctuation in traffic flow may provoke the demand during the cycle to exceed the green time for that cycle. Thus, it will create a queue that is carried over to the next cycle. In cases when demand exceeds capacity meaning volume-capacity ratio for a lane group is more than 1.00 over the entire analysis period, it is possible to observe an accumulation of vehicles not served by the signal during each cycle that affects adjacent intersection or causes shifts in demand patterns. These oversaturated conditions require significantly different approaches for signal timing.

Grazis (3) expanded this concept to diagnose oversaturation, for a single intersection with two competing demands, by proposing the following model:

$$\frac{q_1}{s_1} + \frac{q_2}{s_2} > 1 - \left(\frac{L}{C}\right)$$

q_1, q_2 = arrival rates for two directions
L = total lost time

This equation corresponds only for junctions with fixed cycle length and lost time, thus Green (4) modified it to the equation called "absolute" oversaturation dealing with not fixed value of cycle length:

$$\frac{q_1}{s_1} + \frac{q_2}{s_2} > 1$$

Because of the difficulty to measure the arrival flow using current data collection systems especially under congested situations and due to the uncertainty of the capacity and saturation flows, a direct application of the above models is difficult. Alternatively, some characteristics of oversaturation can be used to diagnose this condition.

2.2.1. INTERSECTION-LEVEL PERFORMANCE MEASURES

Users do not easily perceive capacity measures, which are essential for determining the capability of the intersection to accommodate existing and projected demand. Delay and queues are two primary user-perceived performance values.

DELAY

There are two types of delay: total delay and control delay. The total delay can be defined as the difference between the travel time actually experienced by the users and the reference travel time. This latter is considered the travel time in the absence of traffic control, speed changes due to geometric conditions, the interaction with any other road users and any incidents. Control delay is the portion of delay that is attributable to the control device plus the time decelerating to a queue condition, waiting in queue, and accelerating time from a queue.

For typical through movements at a signalized intersection, total delay and control delay are the same in the absence of any incidents. Equations for calculating control delay for each movement, with main factors such as volume and capacity of the lane group, effective green time and cycle length can be calculated from the Highway Capacity Manual. Control delay of the intersection is an average delay of all movements, weighted by volume.

QUEUE LENGTH

Queue length occurs when vehicle is waiting to proceed through the intersection. It is an essential measurement commonly used to estimate the value of storage needed for turn lanes and to establish if vehicles will physically spill over from one intersection into an adjacent one. Queue length as well as residual queuing for a given phase are the preferred indicators for diagnosis of oversaturation. Several queue length estimations are commonly used with signalized intersections. However, accurate estimation of queue typically depends on the information of arrival flow and requires installation of upstream detectors. Typically, advance detectors for vehicle actuation purposes are installed before the stop line. In case, when the queue length spills over the location of advance detector, most traditional input-output approaches to estimating queue length will not work appropriately. Therefore, either additional upstream detectors should be installed or alternative methods to estimate queue length needs to be developed.

2.1.3 TYPES OF TRAFFIC SIGNAL CONTROL

Traffic signals are timed with the goal to improve traffic flow and to make the traffic system as safe as possible. Each traffic signal controller is programmed with different timing settings, depending on the time of day or according to the current situation at the intersection at that moment. Traffic signal control may be grouped into strategies for individual intersections and strategies for groups of intersections as follows:

STRATEGIES FOR INDIVIDUAL INTERSECTIONS

Local intersection control is the strategy residing in the local controller that manages traffic flow independently of other traffic signals. Two types of local control exist, pre-timed and actuated. Since each signal in uncoordinated control or isolated intersection control operates independently, offset is not a controlled parameter when isolated intersection control is implemented.

- Pre-timed Control;
- Traffic Actuated Control

In ***PRE-TIMED CONTROL,*** sensors are not required when right-of-way is assigned based on a predetermined fixed time duration. It can be determined from historic data, for all signal display intervals. This type of control is generally inefficient when there are changes in demand. Pre-timed control may be used in conjunction with traffic adjusted or traffic responsive timing plan selection, where closely spaced signals dictate fixed offsets, such as with diamond interchanges or central business districts. It is possible to distinguish off-line techniques such as TRANSYT. These techniques are useful in generating the parameters for fixed timing plans for conventional pre-timed urban traffic control systems, which are based on the deterministic traffic conditions in different time periods of the day.

In ***ACTUATED CONTROL***, sensors are utilized to provide data to a local traffic signal controller. The most common sensors used for this application are inductive-loops or video image processors that are in continuous development. Here, it is possible to differentiate semi-actuated or fully actuated control. In semi actuated, green is always present and the phase is recalled to the major street unless a minor street actuation is received. Therefore, sensors are required only for the minor cross-street phases, where the vehicles approach the intersections in a random manner, that is, where platoons cannot be sustained. Fully actuated control implies that all intersection is detected and all phases, which are determined by traffic demand, are actuated. This type of control can be utilized at intersections with sporadic and varying traffic distribution since cycle length varies from cycle to cycle.

STRATEGIES FOR GROUPS OF INTERSECTIONS:

Concerning groups of intersection it is possible to determine different types of strategies for designing a traffic signal control, such as (5):

- Uncoordinated control;
- Time-based coordinated control TBC;
- Interconnected control;
- Traffic adjusted control;
- Traffic responsive control; and
- Traffic adaptive control

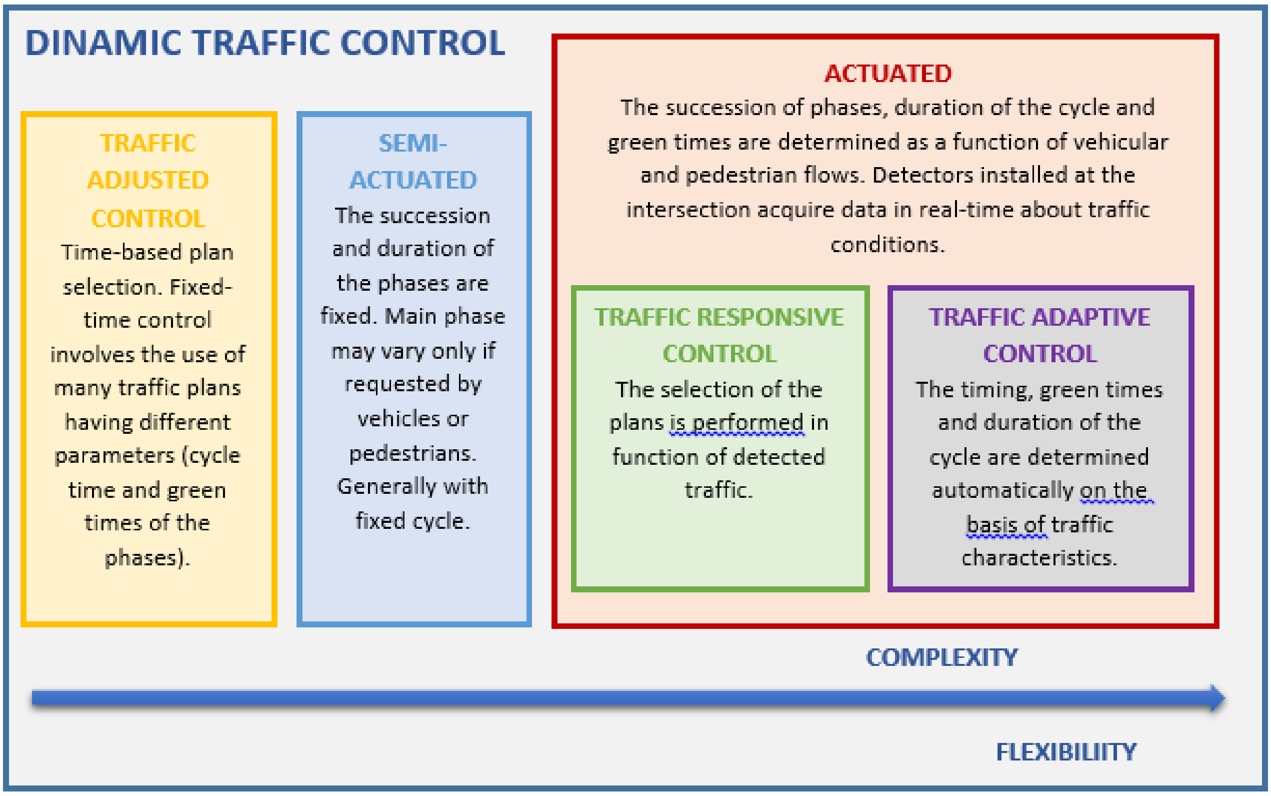


Figure 3: Types of Traffic Signal Control

Traffic flow in ***UNCOORDINATED CONTROL*** is measured without considering the operation of adjacent traffic signals.

TBC SYSTEMS have no sensors in the system. They provide signal progression that allows platoons of vehicles to proceed along arterial routes without stopping. It also minimizes the number of stops over an entire network and total delay. Furthermore, control information on the time-of-day/day-of-week (TOD/DOW) plan is provided by means of a real time clock. Most modern controllers have this capability built in.

INTERCONNECTED CONTROL allows platoons of vehicles to proceed along arterial routes without stopping thanks to proper signal progression. Local signal timing of offset and cycle or actuated timing based on TOD/DOW is determined. Pre-stored plans are generated offline from historical data or average. The operator can choose and download timing plans and changes as well as monitor and record system status. System response to traffic changes is measured in weeks or months.

In ***TRAFFIC ADJUSTED CONTROL***, the system uses fixed timing plans, a selection of which is adapted to 15 minutes or more, and can be generated online. The system operates with some sensors, which measure volume and weight lane occupancy or predominant direction of flow. After, pattern matching is used to choose from among a set of pre-calculated plans.

TRAFFIC RESPONSIVE CONTROL uses automatic traffic detectors that provide the system with real-time measurements. It can be implemented in two different ways: plan selection or plan generation. The first method selects the most appropriate pre-calculated plan according to the traffic conditions observed in real-time, whereas, the plan generation method applies a control logic that adjusts signal settings online, according to real-time traffic counts.

The last method is theoretically the most effective, since it is flexible enough to carry out quick adjustments of signals to better accommodate traffic at each junction. However, it is difficult to obtain stable solutions for all possible traffic conditions. For these reasons, plan selection methods are even more often applied and off-line optimization methods for traffic signal synchronization are still widely utilised in order to pre-compute the optimal plans. These systems typically use macroscopic measures of traffic flow on individual links such as platoon and other characteristics. System response to traffic changes is prompt. Systems operate with at least one sensor per link, up to one sensor per lane per link.

While, ***TRAFFIC ADAPTIVE CONTROL*** systems do not have cycle, split, and offsets in the classic sense, the system proactively re-optimizes selection, sequence and duration of phases every several seconds through forecasting of the traffic to the near future. Flow is measured for every individual vehicle in order to predict the future flow. Generally, for maximum efficiency, the system operates with two sensors per lane per link.

Poor traffic signal timing contributes to traffic congestion and delay. The traditional signal timing process requires substantial amounts of manually collected traffic data and is time consuming. Pre–programmed, daily scheduled signal timing plans do not accommodate variable and unpredictable traffic demands. This leads to customer complaints, frustrated drivers, and a poor level of safety. In the absence of complaints, months or years might pass before inefficient traffic signal timing settings are updated.

2.3. DETECTORS

Continued increase in traffic volume and the limited construction of new highway facilities in urban, rural areas requires the maximization of the efficiency and capacity of existing transportation networks. Limited construction of new roads and fast growing demand has caused recurrent congestion. Nowadays the implementation of strategies for Intelligent Transportation Systems (ITS) is needed, which promote more efficient utilization of current road transportation facilities. Intelligent Transportation Systems roadway programs are intended to reduce travel time, easing congestion and delay, reducing pollutant emissions and improving safety. Electronic surveillance, communications, and traffic analysis and control technologies, which contain ITS, bring benefits to transportation system users and maintainers. ITS applications rely on traffic flow sensors to provide vehicle detection, incident detection, real-time traffic adaptive signal control, automatic traveller surveillance, emergency information services, archival data and traveller data.

The updated knowledge of the network traffic conditions is a prerequisite of intelligent transport systems, which have been defined as adaptive systems able to change their operating characteristics as a function of external conditions for the pursuit of certain goals.

Traffic monitoring systems are used to detect the total number of vehicles or transited users in a section of the road network. The position of a vehicle or user on the network can be determined through localization system. Identification systems can recognize the presence of a given vehicle or user, belonging to a set of previously coded and identified system users. They are also used to identify vehicles authorized to stop in a restricted traffic zone or for monitoring a fleet of vehicles travelling exclusively on a known arcs of the network, detecting the passage at certain points as in the case of a fleet of buses used for scheduled service.

The network travel time tracing vehicles identified is estimated. Traffic monitoring systems can also respond to many other needs, such as detection of external conditions relevant for safety and the reduction of other traffic externalities, in particular air and noise pollution. An automatic monitoring system principally consists of: also sensor, detector, transmitter and processor.

2.3.1. COUNTS AND VEHICULAR TRAFFIC MEASURES

The traffic monitoring devices perform the counting measures of vehicles, which transit or present in the section of the detection area. Subsequently, is possible to estimate the vehicular flow variables and then apply the analytical or traffic regulation models.

FLOW (q)- quantity of vehicles passing across a road section during the time interval.
SPEED or velocity (v)- average speed of vehicles in a segment of unitary length in a given time instant.
DENSITY (k)- quantity of cars which are on a segment of unitary length in a given time instant.

The counting device, positioned in correspondence to a road section, counts the number n of transit vehicles and therefore provides a measure of the flow q in a time T:

$$q = \frac{n}{T}$$

T = time interval
n = road section

Two detections in two consecutive sections, also allow performing a measurement of the speed.

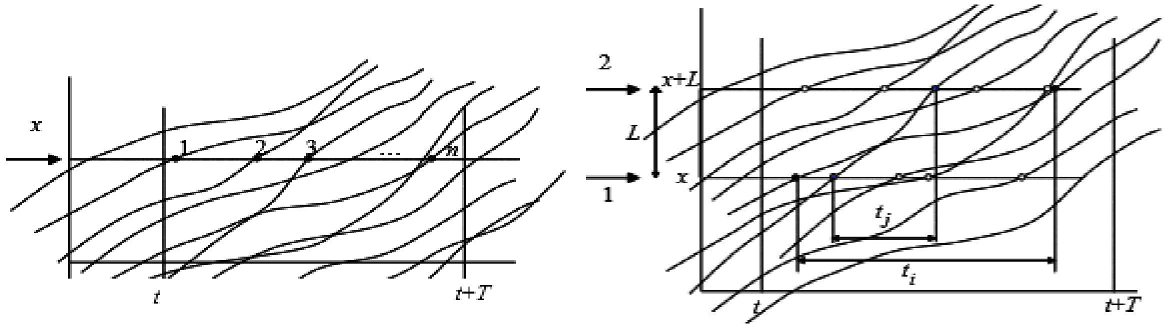


Figure 4: Propagation of traffic flow

Travel time is provided as the time interval between the passage of the generic vehicle in the first and second section:

$$t_i = t_{2i} - t_{1i}$$

t_i = *travel time*
t_{1i} = *travel time in first section*
t_{2i} = *travel time in second section*

The total time spent in the vehicle detection is the sum of the individual travel times of vehicles entering and leaving in time T:

$$\sum_{i=1}^{n}(t_i) = \sum_{i=1}^{n}(t_{2i} - t_{1i}) = \sum_{i=1}^{n}(t_{2i}) - \sum_{i=1}^{n}(t_{1i})$$

The relationship between the residence time of the vehicles and the observation time T is said to be the rate of occupancy and is one of the main variables provided by the non-point detectors:

$$\tau = \frac{1}{T}\sum_{i=1}^{n} t_i$$

τ = *occupancy rate*

The total time spent in the vehicle detection is used to determine the spatial average speed of the vehicles detected (space-mean speed):

$$v_s = \frac{L}{\frac{1}{n}\sum_{i=1}^{n} t_i} = \frac{n}{\frac{1}{n}\sum_{i=1}^{n}\frac{t_i}{L}} = \frac{n}{\frac{1}{n}\sum_{i=1}^{n}(\frac{1}{v_i})}$$

v_s = *spatial average speed*

It is possible to estimate the vehicular density as the ratio between the average occupancy rate and the length of the detection area by applying the state equation and disregarding the change in speed over time:

$$k = \frac{q}{v_s} = \frac{n}{T}\frac{\left(\sum_{i=1}^{n}\frac{t_i}{L}\right)}{n} = \left(\frac{1}{T}\sum_{i=1}^{n} t_i\right)\frac{1}{L} = \frac{\tau}{L}$$

k = density
L = length of detected area

2.3.2. SENSORS TECHNOLOGY

It is possible to observe the continued evolution of sensors that are applied to traffic management and control. Originally, sensors were utilized for signalized intersection control, and now are used to provide data in real-time for traffic adaptive signal control. During the last years, many advances in traffic control technology have been supported by the development of microprocessors and other electronic elements. The scope of continuous research and development of new and advanced technologies is an obtainment of more accurate results and, in the end, the reduction of congestion. Strategies of traffic control that rely on measurement of traffic flow data from sensors are common to central business districts, arterial, and freeway traffic management strategies.

Necessary data generally can be measured by one or more sensor technologies. Below, are presented and shortly described the most common technologies for traffic flow data measurement (6).

INDUCTIVE-LOOP DETECTORS

Figure 5: Inductive-Loop Detector

An inductive-loop detector senses the presence of a conductive metal object by inducing currents in the object, which reduce the loop inductance. Inductive-loop detectors are installed within roadway surface. They consist of four parts: a wire loop of one or more turns of wire embedded in the roadway pavement; a lead-in wire running from the wire loop to a pull box; a lead-in cable connecting the lead-in wire at the pull box to the controller; and an electronics unit housed in the controller cabinet. When a vehicle passes over the wire loop or is stopped within the area enclosed by the loop, it reduces the loop inductance; it unbalances the tuned circuit of which the loop is a part. The resulting increase in oscillator frequency is detected by the electronics unit and interpreted as vehicle detection by the controller.

MAGNETIC SENSORS

Figure 6: Magnetic Sensors

Magnetic sensors are passive devices that detect the presence of a ferrous metal object through the perturbation they cause in the Earth's magnetic field. The metal object creates a magnetic anomaly by the magnetic dipoles on a steel vehicle when it enters a magnetometer's detection zone. Two types of magnetic field sensors are used for traffic flow parameter measurement. The first type, the two-axis fluxgate magnetometer, detects changes in the vertical and horizontal components of the Earth's magnetic field produced by a ferrous metal vehicle. The second type of magnetic field sensor is the magnetic detector, more properly referred to as an induction or search coil magnetometer. It detects the vehicle signature by measuring the distortion in the magnetic flux lines induced by the change in the Earth's magnetic field produced by a moving ferrous metal vehicle. Magnetic detectors are inserted horizontally below the roadway. Since they provide only passage data and not occupancy or presence data, their use is limited to special applications.

MICROWAVE RADAR SENSORS

Figure 7: Microwave Radar Sensor

A radar transmits electromagnetic signals and receives echoes from objects of interest within its volume of coverage. It transmits energy by an overhead-mounted microwave radar toward an area of a roadway. The beam width is controlled by the size and the distribution of energy across the aperture of the antenna. When a vehicle passes through the antenna beam, a portion of the transmitted energy is reflected back towards the antenna. The energy enters the receiver, which calculates the speed, volume and vehicle length. Microwave sensors that transmit a continuous wave (CW) Doppler waveform detect vehicle passage and provide measurements of vehicle count and speed. They cannot detect stopped vehicles. Microwave sensors that transmit a frequency modulated continuous wave (FMCW) detect vehicle presence as well as vehicle passage.

INFRARED SENSORS

Active infrared sensors illuminate detection zones with low power infrared energy transmitted by laser diodes operating in the near infrared region. A portion of the transmitted energy is reflected or scattered by vehicles back towards the sensor. Detection zones are illuminated with IR energy transmitted by laser diodes. This energy reflected from the vehicle is focused by an optical system onto a detector array mounted at the focal plane of the optics. Real-time signal processing is used to analyse the received signals and to determine speed, presence, count and vehicle class. Passive infrared sensors transmit no energy of their own and detect energy emitted from roadways and vehicles or energy that is reflected from them. The energy captured by the sensors is focused by an optical system onto an infrared-sensitive material mounted at the focal plane of the optics. This material converts the reflected and emitted energy into electrical signals. Only the passage of vehicles is detected. The sensors are mounted overhead to view approaching or departing traffic.

Figure 8: Infrared Sensor

LASER RADAR SENSORS

Figure 9: Autovelox

Laser radars are active sensors in that they transmit energy in the near infrared spectrum. They scan infrared beams over one or two lanes or use multiple laser diode sources to emit a number of fixed beams that cover the desired lane width. Laser radars provide vehicle presence at traffic signals, speed, volume, queue measurement, lane assessment and classification. Multiple units can be installed at the same intersection without interference from transmitted or received signals. Modern laser sensors produce two- and three-dimensional imagery of vehicles suitable for vehicle classification.

As the traffic congestion grows and begins to present a huge problem for urban areas, a proper detection system is needed to control and to monitor this phenomenon. Large urban traffic, which provokes overload on the metropolitan areas, is a problem that must be carefully managed and controlled. The main motivation is to detect and obtain the precise traffic flow parameters with the most suitable method for such a fast-growing environment.

Single inductive-loop detectors give information concerning vehicle presence and/or volume, but other parameters such as speed and density must be deduced from algorithms that analyse or interpret the measured data. The values of parameters calculated from inductive loop data may not be suitable to support specific application or they might not have enough accuracy. Magnetic sensors, which use models with small detection zones, need more devices in order to detect a full lane, which leads to higher costs. Moreover, in-pavement devices are buried in or under the pavement, which requires cuts in the road and, during the installation and maintenance process, closure of the line. This leads to delays at intersections and dissatisfaction on the part of users. That is why modern over-roadway sensors provide alternative solutions to inductive-loop detectors. These devices present suitable, non-invasive installation over the road. The installation location must ensure a clear view of vehicles for best performance that in result will allow monitoring of multiple lanes. Most over-roadway sensors directly provide multilane or all intersection traffic volume, vehicle length, speed, occupancy, and classification that are not ordinarily available from other detectors. Information gained from one camera can be easily linked to another one. Below are described the operations and basic functions of Video Image Processors.

VIDEO IMAGE PROCESSORS

Video image processing automatically analyses the scene at the area of interest and extracts information for traffic surveillance and management. A video image processor (VIP) system typically consists of one or more cameras, a microprocessor-based computer for digitizing and analysing the imagery, and software for interpreting the images and converting them into traffic flow data. A VIP can replace several in-ground inductive loops and provide detection of vehicles across several lanes. Some VIP systems process data from more than one camera and further expand the area over which data is collected. VIPs can classify vehicles by their length and report vehicle presence, volume, lane occupancy, and speed for each class and lane. They have the capacity to register lane changes and turning movements. Analysing data obtained from a series of image processors can provide us with parameters such as link travel time, vehicle density and origin-destination matrix, which are suitable for arterial and freeway applications. The system detects vehicles through the analysis of black

Figure 10: Video Image Processor

and white or colour imagery gathered by cameras at a chosen section. Image analysis is performed by algorithms that examine the variation of grey levels in groups of pixels contained in the video frames.
Among available techniques, Video Image Processing is considered superior due to its ease in installation, maintenance, upgrade, and visualizing results while processing recorded videos. For this reason, in the present thesis, a traffic measurement will be performed with the SmartEye System based on Video Detection.

2.4.TRAFFIC MODELS

An urban transportation system is a large complex, non-linear system. It consists of surface-way networks, freeway networks, and ramps with mixed traffic flow of vehicles, pedestrians and bicycles. Frequent congestion affects daily life and poses all kinds of problems and challenges. Reduction of traffic congestions improves travel safety, efficiency, but also reduces environmental pollution. There are many factors responsible for traffic problems, such as unreasonable traffic infrastructures and planning, weak public awareness of traffic; however, the major factor is that existing urban traffic signal control systems do not adequately fulfill an optimal traffic control and management role.

Urban and rural road modeling have attracted the attention of many scholars and researcher in the field in the last decade. Research on traffic flow modelling started some fifty years ago. Lighthill and Whitham (1955) presented a model based on the analogy of vehicles in traffic flow and particles in a fluid. Since then, traffic flow mathematical description has been the theme of research and debate for traffic engineers.
Nowadays the interest in solving roadway traffic problems is growing more and more, because of increased overloading of the road network. That is why traffic engineering is focusing essentially on causes of congestion, congestion propagation through network and the definition of the location and time of traffic breakdown.

For this reason mathematical models that could faithfully reproduce reality were developed. These mathematical models of sufficient accuracy are required for traffic data processing and estimation, the design of control strategies, and the testing of control strategies via simulation. Thanks to obtained information it is possible for two types of analyses: simulative and predictive. Simulative analyses observe how the model behaves if stressed by particular external conditions, which must not necessary occur in reality. Predictive analyses consist of minute by minute monitoring of traffic conditions, and in result will draw conclusion about user behaviour, which will serve for the near future and will help to prevent negative situations on the road such as delays or traffic jams.
That is why traffic models play a fundamental role in identifying the traffic conditions and criticality of the system. Applications of realistic flow models in model-based control to improve techniques and capacity of the computation to solve control problems become feasible. There are plenty of traffic models in transport literature and in the following subsection; some of them will be explained.

2.4.1. CLASSIFICATION

Apart from the scientific problem of reproducing traffic flows, safety and costs also play an important role as well. Analytical approaches do not always provide the desired solutions, due to the complexity of the traffic. Traffic flow models are designed to characterise the behaviour of the complex traffic flow system. “The challenge of traffic flow researcher is to look for useful theories of traffic flow that have sufficient descriptive power, where sufficient depends on the application purpose of their theories” (7).

Nowadays, different categories of traffic analysis tools exist. Among them as one of the most powerful approach to traffic analysis research, is the simulation approach. Models can be classified based on different criteria:

- Operationalisation: analytical, simulation;
- Scale independent variable: discrete or continuous;
- Physical interpretation;
- Process representation: deterministic or stochastic; and
- Level of detail.

With the reference to level of detail, we can distinguish:

- Microscopic models;
- Mesoscopic models; and
- Macroscopic models.

MICROSCOPIC MODELS describe the behaviour of every single vehicle, interaction between vehicles and between the vehicle and the infrastructure. Models predict the character of the network through speed, origin-destination movement, types of vehicles, accelerations and decelerations rates and behaviour of drivers. Simulation consists of loading the network with flow of vehicles, which interact with each other and with infrastructure along their path. Microscopic models use standard simulation models based on discrete events, among other things. They are used to model short periods of time with a very high level of details.

MESOSCOPIC MODELS are derived from gas-kinetic theory, which describe the dynamics of velocity distributions. Traffic in the mesoscopic model is represented by groups of traffic entities, the activities and interactions of which are described at a low detail level. These models do not trace or differentiate individual vehicles, but specifies the behaviour of individuals, for instance in probabilistic terms. Lane-change manoeuvre might be represented for an individual vehicle as an instantaneous event, where the decision to perform a lane-change is based on relative lane densities or speed differentials.

MACROSCOPIC MODELS describe traffic at a high level of aggregation as a flow without distinguishing its parts. For instance, the traffic stream is represented by use of characteristics such as velocity, flow-rate and density. Models can be classified according the number of partial differential equations. Further distinction can be made within this class of models in the order of the models themselves: the first-order model of Lighthill, Witham and Richards (1955), which uses only the density as a variable to represent the state of the system; the model of Payne (1971) of the second order recurs to the density and average speed, and finally, a traffic model of third order Helbing (1996), uses density, average velocity and variance of the speed as status variables.

The most important behavioral submodels used in simulation of microscopic models are:

- Car following;
- Gap acceptance; and
- Lane changing.

The macroscopic models, working with aggregate variables, are suitable to rapid traffic simulations of transport networks, since they require fewer parameters to estimate than microscopic models. It is also possible to define a cost function, in terms of aggregate variables, which simplify the calculation of the cost associated with a particular state of traffic.

A natural classification is the time-scale for traffic models since all of them describe dynamic systems. It is possible to distinguish between the continuous and discrete time-scale. A continuous model describes how the traffic system's state changes in continuous way over time in response to continuous stimulus. Discrete models assume that state changes occur discontinuously overtime at discrete time instants. Also other independent variables as position and desired velocity, can be described by either continuous or discrete variables. Smulders (1990) Mixed models have also been proposed (Smulders, 1990)

In this respect, we will distinguish stochastic and deterministic models. Stochastic models incorporate processes that include random variables. Deterministic models have no random variables; all actors in the model are defined by exact relationships.

The following section will focus on macroscopic models since they are better suited than microscopic models to the design of regulations and the control of traffic.

2.4.2. MACROSCOPIC TRAFFIC FLOW MODELS

In macroscopic traffic flow models aggregate behaviour of drivers depends on the traffic conditions in the drivers' environments. They deal with traffic flow in terms of aggregate variables. Usually, the models are derived from the analogy between vehicular flow and flow of continuous media, yielding flow models with a limited number of equations that are relatively easy to handle. Models describe the dynamics of macroscopic variables as flow, density and velocity using partial differential equations.

The independent variables of a continuous macroscopic flow model time instant t and location x. It considered a small segment of a roadway due to introducing the dependent traffic flow variables.

Most macroscopic models describe the dynamics of the density d = d(x,t), flow q = q(x,t) and the velocity v = v(x,t). The flow equals the expected number of vehicles flowing past x during [t,t+dt) per time unit. The density describes the expected number of vehicles on the roadway segment [x,x+dx) per unit length at instant T. The velocity equals the expected velocity of vehicle. Some macroscopic traffic flow models also contain partial differential equations of the velocity variance Θ= Θ (x,t), or the traffic pressure P = P(x,t) = r Θ.

Two types of macroscopic models will be discussed, namely:

- Lighthill-Whitham-Richards models; and
- Payne-type models

Before analysing in detail the various models, the conservation law of vehicles and fundamental diagram, one of the most important tool of traffic engineering, will be discussed.

2.4.2.1. NETWORK FUNDAMENTAL DIAGRAM

Various theories were proposed to describe vehicular traffic on an aggregate level in the last years. Recently, it was found that the notion of a fundamental diagram, in the form of a flow-density curve, could be applied to two-dimensional urban road networks as well. In the two-dimensional diagram as the accumulation or density is increasing (free flow condition), the traffic flow increases up to the capacity of the network (yellow region). After this, the network starts degrading and enters into the over-saturated region indicated by the red rectangle. The aim of traffic engineers is to maintain the overall traffic state, by applying to the saturated condition area range various traffic management tools, such as traffic signal optimization, gating, route guidance, etc., in order to avoid spill over and gridlock creation. The concept is sometimes called MFD (macroscopic fundamental diagram).

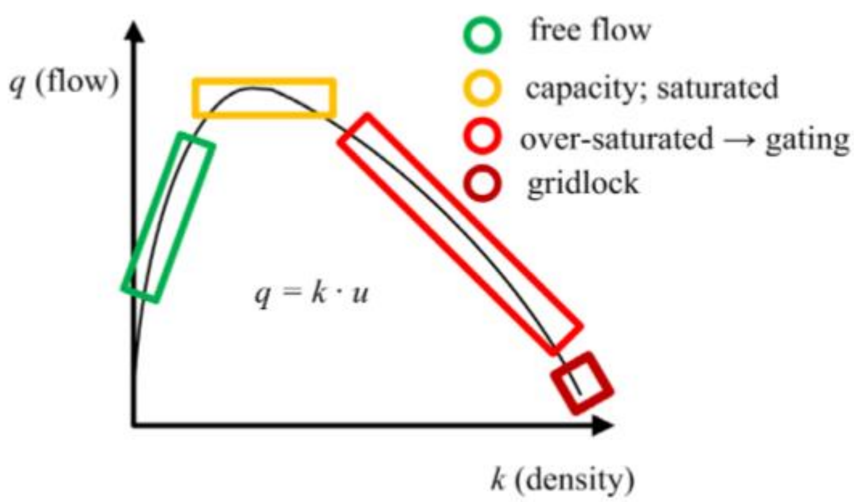


Figure 11: Network Fundamental Diagram

There are further possibility of the diagram (q,k) derived from the need to adapt the curves to the experimental data. Various formulations have been proposed, after Greenshields' work, based on a single circulation regime

that is single expression between the characteristic quantities of the outflow for all the density values. Also proposed were multi-regime models, which present points of discontinuity and allow a better fit to the data.
More sophisticated functional forms between density and flow have been proposed over the past years: the work of Smulders (1988), a two-regime model that provides a nondifferentiable point curve (q, k) in correspondence to critical density, the model METANET of Messmer and Papageorgiou (1990) single-regime that introduces a inflection point of the fundamental diagram corresponding to the maximum density.
Because there is, no consensus among traffic engineers on the correct shape of the diagram, the functional form variations will continue to be suggested.

2.4.2.2. CONSERVATION OF VEHICLES

A traffic stream is treated like a viscous and compressible fluid: the dependent traffic flow variables are differentiable functions of time and space. It is possible to derive models that present a limited number of equations, easy to manipulate, utilizing analogy to a stream of fluid.

The first step is the adoption of conservation of the law of vehicles, which is a differential equation of the first order that can be derived by assuming that the numbers of vehicles present on a road with the length of the section that remains constant:

$$\frac{dk(x,t)}{dt} + \frac{dq(x,t)}{dx} = 0$$

The state equation is presented as:

$$q = kv$$

Therefore, at the steady state the characteristic parameters of the outflow are dependent of each other, but linked by the state equation, which allows expressing one parameter in the function of the other two. In non-steady state conditions, the state equations continue to be valid locally and is presented as:

$$q(x,t) = \mathrm{k(x,t)} * \mathrm{v(x,t)}$$

2.4.2.3. LIGHTHILL-WHITHAM-RICHARDS (LWR) TYPE MODELS

Lighthill, Whitham and Richards (1956) have first introduced macroscopic traffic flow models. All three authors converge on the following hypothesis: assuming the continuity approach that the flow model can describe the behaviour of vehicles in congested traffic.

Flow is considered a local function of the density, q (k (x, t)), and both in presence of low intensity of traffic and in congested conditions. The dynamics of traffic are described by a partial differential equation, which models the conservation of vehicles:

$$\frac{dk(x,t)}{dt} + \frac{dq[k(x,t)]}{dx} = 0$$

The dynamic model of Lighthill and Whitham is a sufficient model to describe the dynamics of vehicular traffic without the need to introduce other hypotheses or equations, if not the necessary boundary conditions. Because of its simplicity and ability to reproduce characteristics of traffic flow, it is the most popular model applied in practice (8).
The important drawback of the LWR-model is that vehicles are assumed to attain the new equilibrium velocity immediately after a change the traffic state, which implies infinite acceleration (9). Moreover, it does not yield a unique continuous solution; in this case, the density assumes two different values, each belonging to specific characteristics. This mean that, in a given point in space, the flow velocity should have two values at the same time.

The first order dynamic model reaches more satisfactory degree of accuracy than purely static approach. Its strong point is undoubtedly the ability to provide analytical and graphical interpretations solutions, which avoids the use of numerical resolutions in certain cases. In addition, the input parameters are limited to those provided in the calibration of the fundamental diagram. Rapid and appropriate numerical scheme allows approximations to solve some practical problems. This model uses the fundamental diagram, in which the traffic is smooth, steady and deterministic.

Over the years, the LWR model has been extended and refined: various improvements have been proposed to take into account the heterogeneous nature and not steady stream of traffic in real conditions. Each extension requires certain assumptions and introduction of new parameters, which increase the template input.

2.4.2.4. PAYNE-TYPE MODELS

The second order models were born from the need to reduce weaknesses of the first order models. They add the relations that take into account the inability of drivers to instantly adjust their speed to density variations. They also assume for traffic stream behaviours similar to this of follower vehicles that adapt their speed variations in dependence of the inducements that they receives from the leader vehicle:

$$v(x,t) = V[k(x + \Delta x, t)], \Delta x > 0 \qquad V = \text{ speed of equilibrium;}$$

Expanding in a Taylor series, respectively the first and the second member of the above equation, following Payne (1979), is obtained:

$$\underbrace{\frac{\partial v(x,t)}{\partial t}}_{\text{Gradient w.r.t time}} = \underbrace{-v\frac{\partial v}{\partial x}}_{\text{Convection Term}} + \frac{1}{T}\left[\underbrace{V(k) - v(x,t)}_{\text{Relaxation Term of speed v(x,t) w.r.t. stationary state V(k)}} - \underbrace{\frac{\mu}{k}\frac{\partial k(x,t)}{\partial x}}_{\text{Anticipation term}}\right]$$

Where $\mu = \frac{\xi}{\tau}$ is the anticipation constant in which $\xi = \frac{-\,\mathrm{d}v^2}{dk}$ is the rate of decrease of the equilibrium speed with increasing density. Assuming a constant rate is obtained a linear relationship between the density and the speed $V(k)$. For the function $c_0(k)$, which identifies the speed of propagation of the shock waves, is adopted the following expression:

$$k\frac{dV(k)}{dk}$$

Payne identified the following convection, relaxation, and anticipation terms:

- Convection (C): describes changes in the mean velocity due to inflowing and outflowing vehicles;
- Relaxation (R): describes the tendency of traffic flow to relax to an equilibrium velocity; and
- Anticipation (A): describes the drivers' anticipation on spatially changing traffic conditions downstream.

The model presented is used to describe the traffic behaviour under dynamic conditions in aggregate form regardless of the individual vehicle and representing the state of the system through the performance of the macroscopic flow variables. The model requires the knowledge of the incoming demand of the analysed section, the initial conditions of the system and the conditions of entry, representing into and exit flows from the ramps. The artery of study is divided into a number of road segments and the determination of the characteristic takes place at each of the simulation advance interval. To achieve this, is necessary to pass from the study of the phenomenon itself to the study of its modelling, which allows to simulate the flow on the road, on N arcs to vary time intervals.

The system allows two different analyses: simulative and predictive. The quality of the estimation of traffic variables depends not only on the reconstruction of the initial state to which the model is particularly sensitive, but also on spatial and temporal interval which determine the state of current or planned network traffic. A good practice that can guide the choice is to consider the length of the sections L and a time progress t so the sections will not be skipped. The accuracy of the model strongly depends on the spatial and temporal intervals with which the simulation is performed. The comparison of the results with traffic data showed an excellent performance of second-order models.

Chapter 3
RELATED WORK

3.1. SIGNAL CONTROL STRATEGIES

The installation of traffic signal system on an intersection has as objective to provide safe and equitable right-way to a number of competing movements. In general, it is possible to classify purposes of the intersection control into two basic categories, progression bandwidth maximization and delay minimization. Bandwidth-oriented approaches, which will be used in this research, require optimization of four signal-timing parameters (e.g., green split, cycle length, phase sequence and offset) in order to maximize network progression. While, delay-oriented control approaches target minimizing delay, queue length and stops.

This chapter aim is to provide an overview of the models used in the design and evaluation of traffic lights systems. These are essentially based on knowledge of the flows: strategies using mathematical programming techniques or optimal control for the determination of the control parameters that consider as a hypothesis the distribution of flows within the intersection or network.

The models proposed in the literature can be distinguished based on the relationship between the traffic demand and the capacity of the intersections. The traffic demand in signalized intersections varies during the day. During peak periods, when demand exceeds the capacity of the intersection, is possible to observe the queue formation. When these conditions are attained, the intersection is oversaturated. The duration of oversaturation periods is a function of the demand and the capacity of the intersection. Thus, the models for the control of the intersections can then be distinguished on the basis of the state of intersections as follows:

- Models for oversaturated intersections; and
- Models for undersaturated intersections

Regarding to the characteristics of the models, these can be classified based on the level of aggregation, which is represented with the flow of traffic, or the mode with which the advancement of the time variable is handled, as it was possible to see in the previous subchapter. The table below shows other classification, regards to their field of application. There are models for isolated simple intersection, arterial intersections and models developed for networks. This research deals with model for arterial intersections.

Table 1: Design models of signal setting (source: (10))

SIGNAL SETTING STRATEGIES				
		Flows		*Arrival times*
	Variables	*Min delay*	*Max capacity factor*	
Isolated Intersections	*Green timing* *Green timing and scheduling*	SIGSET SICCO	Webster, SIGCAP, SICCO	MOVA
Arteria	*Offset*		MAXBAND, MULTIBAND	
Network of intersections	*Green timing and offset*	TRANSYT, SCOOT	As isolated intersections	OPAC, UTOPIA

3.1.1. MODELS FOR ISOLATED INTERSECTION

The simple isolated intersections are characterized by the ability to design and verify the traffic signal timing without having to take into account the relationships that arise with other intersections, in terms of constraints due to the queues, type of arrival of the platoons to the approaches, traffic light coordination etc. The models for the design and evaluation of isolated intersections are multiple and have had considerable developments over the past years. There has been a development of mathematical models of analysis and very accurate optimization of the traffic light timing, thanks to the pressure due to increase of traffic volume together with the introduction of more adaptable control processors of the system.

The green timing and the green timing and scheduling are two design variables, which are defined for isolated intersections. In this case, the strategy based on flows may have as objectives: minimization of total delay, with fixed or variable cycle time duration.

Webster (1958, 1966) has developed early theories. Later, thanks to methods of SIGCAP (11) and SIGSET (12) of Allsop (1971, 1972), was possible to overtake some limitations and constraints of these models.

Optimization variables are real and linear constraints, while the phases and their sequence must be established. The strategy based on arrival time with exogenous structure of stages has been studied in several methods like MOVA model. Other models allow overcoming this limitation and allow determining with a unique optimization process the structure of the phases and the traffic light timing. Among these are the works of Heydecker and Dudgeon (1987) and Improta and Cantarella (1984, 1988), offering models with formulas of linear (or convex) binary mixed integer mathematical programming with linear constraints and real variables. The indexes that are used to evaluate the performance of the intersection are generally constituted by the waiting time and the capacity, or better from the capacity factor f, which represents the greatest common multiplier for all incoming flows for which is still verified in the under saturation condition. Minimization of the total delay or maximization of the capacity factor of the intersection generally characterize the objective function.

The described models and other models have been a basis for the implementation of different software for the analysis of signalized intersections. The most common are the HCS, OSCADY, LINSING, SIDRA and SIGSIGN.
The most important methods for signalized intersection control is the Webster model, very widespread and historically significant, and the HCM model (Highway Capacity Manual), important in the process of evaluating and defining the notion of an intersection service level.

3.1.1.1. WEBSTER MODEL

Due to fact that direct observation of delay in the field is difficult by uncontrollable variations and theoretical calculation of delay is very complex, Webster (1958) developed a model to estimate intersection delay using a deterministic queueing analysis and empirical result from simulation. Webster's Delay minimization model (13) is a fundamental equation of signal timing for an individual intersection. In the first term of equation, Webster considered uniform arrival rate of vehicles. The second term of delay formula assumes random arrivals. The last part, purely empirical, is aimed to bridge the gap between practical and theoretical results.

$$d = \frac{c(1-\lambda)^2}{2(1-\lambda x)} + \frac{x^2}{2q(1-x)} - 0.65(\frac{c}{q^2})^{\frac{1}{3}} x^{(2+5\lambda)}$$

f = arrival flow;
x = saturation degree;
λ = g/c

This method allows obtaining the duration of the cycle and the green times for a single intersection of which is known the matrix of the phases. The procedure is simple and it based on the representation of each phase using a single stream, characterized in the higher value of the flow ratio.

The effective green time g of each phase, with cycle length C and total lost time L:

$$\sum_{i=1}^{n} g_i = \mathrm{C} - \mathrm{L}$$

are determined assuming that all the representative streams of each phase have the same capacity factor μ_i, which is assumed as the capacity factor of the intersection μ*. This is known as the principle of equi-saturation:

$$\mu_i = \frac{g_i/c}{y_i} = \mu *$$

Moreover, if we assume that:

$$Y = \sum_{i=1}^{n} y_i$$

Effective green time will result:

$$g_i = \frac{y_i}{Y}(c - L)$$

The following expression estimates the value of the duration of the cycle that minimizes the total delay:

$$c = \frac{1.5L+5}{1-Y}$$

Finally, it should be noted that the minimum cycle, in order to ensure no oversaturation, is obtained by placing the intersection capacity factor as equal to 1:

$$c_{min} = \frac{L}{1-Y}$$

The Webster model is simple in its application, but does not allow the introduction of the constraints of minimum green and maximum red and requires individuation of the representative stream of vehicles. These limits are exceeded in the models of Allsop.

The delay model incorporated into the HCM calculates intersection delay based on Webster uniform delay equation, Akcelik's overflow delay model, and initial queues delay equation (14). Thus, the delay formula of HCM consists of three terms, as the following equation shows.

$$d = d_1\, PF + d_2 + d_3$$

$$PF = \left(\frac{1-\mathrm{P}}{1-\frac{\mathrm{g}}{\mathrm{c}}}\right) * f_p$$

$$d_1 = \frac{\mathrm{c}}{2} * \frac{(1-\frac{g}{c})^2}{1-[\min(1,X)\left(\frac{g}{c}\right)]}$$

d = *control delay;*
d_1= *uniform delay component;*
PF = *progression adjustment factor;*
d_2 = *overflow delay component;*
d_3 = *delay due to pre-existing queue;*
k = *incremental delay factor for actuated controller settings;*
l = *upstream filtering/metering adjustment factor;*
P = *proportion of vehicles arriving during the green interval;*
f_p= *supplemental adjustment factor for platoon arriving during the green.*

$$d_2 = 900\mathrm{T}[(\mathrm{X}-1) + \sqrt{(X-1)^2 + \frac{8\mathrm{klX}}{\mathrm{cT}}}\,]$$

$$d_3 = \frac{1.8 * Q_b * (1+u) * t}{C * T}$$

Q_b = *size of initial queue*
t = duration of oversaturation within T
u = delay parameter

3.1.1.2. HCM MODEL

For the study of the signalized intersections, HCM provides two analytical procedures: Operational Analysis and Planning Analysis. Second one is used mainly in the design phase, while the Operational Analysis is a verification tool. HCM allows determining a suitable traffic light timing: the duration of the cycle, the type of the phases and their duration. Model requires the volume of traffic for each manoeuvre, peak hour factor, the position of the intersection and parking manoeuvres. The procedure to determine the traffic light timing involves for the determination of the critical volume and calculation is based on the principle of equi-saturation. Subsequently, the Operational Analysis evaluate obtained timing. This procedure allows evaluating the performance of the intersection based on traffic volumes, intersection geometry and performed traffic light design.

HCM determines the capacity and the level of service of each group of lanes and the entire intersection. The analysis require an accurate data of the geometry of the intersection, the volume of traffic and the traffic signal plan. The sequence of operations consists of five modules: data collection, evaluation of the flows, the evaluation of the saturation flows, the calculation of the capacity and the calculation of the level of service (15); it is shown in the Figure below:

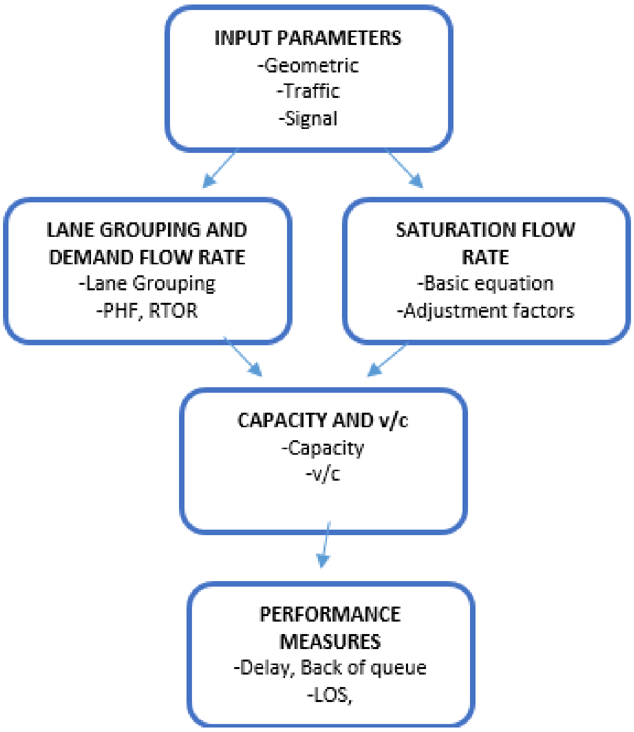


Figure 12: Highway Capacity Manual Operation (source: (15))

The first module defines the input data that include geometric data, traffic data, signal timing plan.

In the second module, the calculation flows to be used for analysis of the intersection are determined based on the input data, by correcting the values depending on the peak hour factor PHF. The flows are then divided according to the manoeuvres for lane groups. Subsequently, a further coefficient is applied to the flows in order to take into account not perfectly balanced distribution of the flows within multiple lanes that effects the same manoeuvre.

The third module defines the method of calculating the saturation flow s of each manoeuvre:

$$s = s_0 * N * f_w f_{HV} f_g f_p f_{bb} f_a f_{rt} f_{lt}$$

$s_0 =$ *ideal saturation flow per lane;*
N = *number of lanes of the considered lane group*
$f_w, f_{HV}, f_g, f_p, f_{bb}, f_a, f_{rt}, f_{lt}$ = *all the others corrective coefficients determined on the basis of experimental formulas and tables*

In the fourth module is carried out the capacity calculation, an essential element for the evaluation of a signalized intersection performance. It allows the evaluation of the relationship between the demand through an intersection and the possibility of the same to absorb that traffic.

The fifth module determines the level of service (LOS) of analysed intersection. The average delay of vehicles passing by intersection is calculated to determine LOS. Delay values are obtained by the delay of all vehicles at the intersection in the period of analysis. Below is presented the table with Level of Service for signalized intersections in terms of control delay defined by HCM:

Table 2: Level of Service and its equivalent delay at the Intersection

LOS	***Control Delay per Vehicle [sec/veh]***
A	≤ 10
B	> 10-20
C	> 20-35
D	> 35-55
E	> 55-80
F	> 80

3.1.2. MODELS FOR NETWORK OF INTERSECTIONS

The regulation methods for a network of intersections can be distinguished according to the type of network that must be taken into consideration: the networks or signalized arteries.
Roess (16) defined the bandwidth as “The time between the first and the last vehicle that pass through the entire arterial system without stopping”.

The most popular methods have been proposed in the work of Morgan and Little (1964) and Little (1966) based on a heuristic-combinatorial optimization technique, which was the base of development of many computer programs.
In these methods, a green wave is realized along the arteria: when a vehicle travels within green wave meets all signals arranged to green. The time interval, in which there is the green wave within the cycle time, identifies the green band. The band, which is sufficiently large, allows all of the vehicles to built-up vehicles to the first

access of artery to complete its path without further arrests. The mentioned methods search to maximize the bandwidth. The Little method allows, among other things, to obtain different bandwidths and to assume as variables to optimize cycle time and the crossing speed.
For arteria of intersections, design variables are common cycle time duration, green time duration on each access, offset between each couple of adjacent intersections. The first two variables are the same that involve the green timing setting. Offset design is analysed in MAXBAND model (17) and MULTIBAND model developed by Gartner (1990).

Regarding to network of intersections, in order to minimize total delay, two approaches may be considered:

- Coordination- after signal setting design of intersection with equal cycle length, method consists in offset design through maximization of capacity factor which is independent from offset;
- Synchronization- consists in the simultaneous design of green time, cycle length and offset.

The most popular software, using strategy based on flows, is TRANSYT (18) that uses heuristics optimization techniques. Other software to mention is SCOOT that can use real-time monitored flows. OPAC, dynamic time optimization model, bases on arrival time strategy. Instead, UTOPIA is a bi-level model, which divide the problem in two sub problems: one for whole network and one for single intersection.
For small networks and with short distance between signals Camus, A. D'Amore and Ukovich (1983) developed an optimization model. The complex intersection reduced distances between nodes are rarely sufficient to ensure the accumulation of queues without influencing spillback, leading to collapse the entire system.

The next chapter provides the relevant literature on signal optimization models with subdivision into online and off-line control strategies. A traffic flow model and optimization process are the major components of signal control plan generating method. An optimization process is a search procedure, which analyse a solution space for an optimality. Computational efficacy of optimization is of main concern to the underlined method, particularly for real-time applications. Traffic flow model is used to evaluate performance indices of optimization process. Usually, traffic flow model is either analytical or simulation-based (microscopic, macroscopic or mesoscopic) with stochastic and/or deterministic characteristics in nature. Its usefulness depends on its ability of realistic replication of real-world traffic flows subject to traffic operations, signal control decision, geometric configurations and incidents.

The purpose of the next subchapters is to review several existing online and off-line methods of signal control plan generation. For each method, the main interests are as follows:

- The effectiveness of its optimization process;
- The capability of its traffic flow model;
- Its ways to generate signal control plan; and
- Its performances in terms of strengths and weaknesses.

3.2. OFF-LINE SIGNAL CONTROL STRATEGIES

This section reviews popular models in off-line signal optimization in order to learn the experiences from those models. This subchapter introduces the most popular existing off-line signal optimization models: MAXBAND and TRANSYT-7F.

3.2.1. TRANSYT-7F

TRANSYT (Traffic Network Study Tool) is a macroscopic simulation and optimization model fits into category of disutility-oriented methods developed by Roberston (1967). In this model, a hill-climbing optimization procedure changes splits, offsets of signal control and determines whether the specified performance index is improved. A macroscopic, deterministic traffic flow model is used to compute a specified performance index for a given set of signal control settings and for a given traffic network. TRANSYT model evaluates arrival flow for each approach in the network. Uniform arrival rate is assumed for link without upstream intersection. Otherwise, TRANSYT models arrival traffic as cyclic flow profile (CFP) and automatically update input and output flows of the link. Platoon dispersion model is used to laterally disperse vehicles in a platoon that is proportion to the distance to the downstream intersection (19). In recent version, in order to improve congestion modelling, queues are estimated horizontally. Therefore, delay, stops, queues size, storage bay overflow and link's spillback are accurately evaluated.

Hill-climbing technique searches for optimal signal settings, which optimize the chosen performance index. This heuristic process consists of series of optimization stages. First, an optimal cycle length is determined from a pre-specified interval. Then, phase splits for each intersection can be specified or automatically calculated based on equalizing degree of saturation of conflicting movements. Optimization proceeds as a series of several sequential offset adjustments at each intersection in the network. During these procedures, the phase splits are constant. The algorithm performs line search, for each intersection, to improve a global objective function. It continues incrementally adjusting the offset as long as the value of objective function is improving. When the adjustment produces decreasing of the objective function, the adjustment' direction is reversed with the same step size. The search is alternated with different step sizes to escape from local minimum region. Thus, climbing-hill procedure does not guarantee convergence to the optimal solution.

In order to improve and accelerate the convergence to optimal solution, TRANSYT-7F incorporates genetic algorithm search technique with climbing-hill. It allows simultaneously optimization of the cycle length with offset and splits.

The main target of TRANSYT is to optimize wide variety of objective functions, such as progression opportunity maximization, throughput maximization, disutility minimization and other hybrid objectives. It is possible to specify only one-performance measures as objective function for entire analysis period. The objectives of TRANSYT-7F are listed by acronyms below and explained subsequently:

- DI;
- PROS;
- PROS then DI;
- PROS/DI;
- QR x DI;
- THRU/DI;
- THRU then DI;
- THRU V/C.

DI refers to a disutility index that is the traditional delay and stop minimization objective. PROS, the progression opportunity, is defined as the number of successive green signals that vehicle may expect to progress through without stopping when driving at the design speed. The queuing ratio QR is a measure of the average back of the queue on a link divided by the maximum number of vehicles that is possible to accommodate on the link. While, THRU is a measure of throughput and the v/C objective penalizes by link-wise deviation above specified saturation degree. The primary objective function is defined as follows:

$$DI = \sum_{i=1}^{n} [(w_{di}d_i + K.w_{si}s_i) + U_i(w_{di-1}d_{i-1} + K.w_{si-1}s_{i-1}) + QP]$$

d_i = delay on link i
s_i = stops on link i
w_{di} = link-specific weighting factors for delay and stops on link i
U = binary variable
K = user-specified stop penalty factor
QP = queuing penalty

3.2.2. MAXBAND

Models such us PASSER II and IV (Progression Analysis and Signal System Evaluation Routine), MULTIBAND and MAXBAND (Maximal Bandwidth Traffic Signal Setting Optimization Program) fit to the category of progression-based methods.

Bandwidth maximization is one of the oldest methods of traffic synchronization. Morgan and Little in 1964 explored synchronization of traffic signals along an arterial in two scenarios, and presented the first method for optimally maximizing arterial bandwidth with potentially different green splits at each signal. In the first scenario, traffic flow is equilibrated in both directions, in the second one during rush hour, the traffic flow biases in one direction.

The objective of MAXBAND, bandwidth-based signal optimization model, is to achieve maximal progression bandwidths on arterial streets and networks. In the model, phase splits at each intersection are assigned according to Webster's theory and all traffic signals have a common cycle time. Mixed integer linear programming (MILP) formulation is a base of the model in order to obtain maximum bandwidth signal settings (20).

Later, various researcher as Messer (1987), Tsay and Lin (1988), Gartner (1990) enhanced the model in terms of an arterial model. Stamatiadis and Gartner (1996) promoted MULTIBAND approach, which is an extension of MAXBAND and better adapt the bandwidth to the flow variations. For MAXBAND as a network model, Chaudhary (1991) promoted two heuristic methods to improve the computational efficiency and accelerate optimization process of the model. Pillai (1998) developed numerically stable and fast heuristic method for the maximum bandwidth signal-setting problem based on restricted search of the integer variables in the solution space. This method, being computationally efficient, can generate optimal or near-optimal solutions.

MAXBAND model calculates cycle time, progression speed, offsets and left-turn phase sequence, which maximizes the weighted sum of bandwidths subject to interference constraints, bandwidth ratio constraint, cycle time constraint, speed and speed-change constraints and loop integer constraints. Model uses the mathematical programming system MCODE (Land and Powell, 1973) to solve the MILP problem formulation.

One of the strengths of MAXBAND model is that in comparison with other disutility-oriented methods, it requires relatively little input and provide traffic engineers and drivers with easily visualized and understandable progression bands. Moreover, it requires no starting solution, achieves a global optimum, and optimize phase sequence and cycle length (Cohen, 1983). Model, compared to PASSER II, has wider set of decision capabilities, more rigorous mathematical programming model, capability to handle other objective functions and expandability to network formulations.
In the other hand, it is possible to report the weakness of model such as requirements of extensive computation time since it bases on MILP formulation with employment of branch-and-bound techniques for its solution. This is infeasible for realistic network problems (Little, 1981; Chaudhary, 1991; Pillai, 1998). Model has no capability of reporting traffic measures such LOS, stops, delay and it does not optimize green splits (Chaudhary & Messer, 1993). Furthermore, generated progression schemes have uniform width, which does not always hold. Because of using average through-movement volume for allocating the total bandwidth, the green band can be either deficient at intersection with higher than the average through moving-volume, or be wasted at intersections with lower than the average through-moving volume. This has been the most important drawback of progression method and optimum results cannot be guaranteed (Stamatiadis & Gartner, 1996). The next disadvantages of the model can be numerical instability results in suboptimal or no solutions for network problems with a range of variable cycle lengths.

MULTIBAND model overcomes the limitations of MAXBAND model by relaxing the assumption of a uniform platoon moving through all the signals. In MULTIBAND model is possible to specify a variety of flow-dependent objective functions, and optimize the same variables as in MAXBAND. It optimizes all signal control variables, such as offsets, cycle time, phase lengths, and phase sequence, and generates variable bandwidth progressions on each arterial in the network that correspond to the specified objective. It uses MINOS mathematical programming package to solve MILP problem, and in result to attain a global optimality.

3.3.UTCS

Urban traffic control system (UTCS) are specialized form of management systems that provides signal timing in response to changing in traffic conditions measured by detectors (21). The most important benefit from UTC system is high traffic performance in a road network by reduction of unplanned stops and delays of vehicles.
It consists of two components: software and hardware. The software is composed of the arrival model, departure model, control algorithm, stops and queue estimation models. While, hardware component includes controllers, signal heads, detecting device, central device and communication lines.
Over the last 40 years, it was possible to notice the evolution of UTC systems, in response to the needs of different cities around the world, the advances in control technologies, detection and communications. Efficient urban control improve road safety and air quality, increase economic efficiency and reduce a congestion (22).

UTC systems have been classified into five control generations based on fix-time or real-time control: 1GC, 1.5GC, 2GC, 3GC and 4GC. The first generations implement pre-calculated signal timing plans. This is the oldest method, but many systems are still using it. Drawback of the fixed timing plans is that those systems does not follow changes in traffic and will not automatically answer to incidents or variations that may affect the system's capacity. 1.5GC systems select signal timing plans offline or generates time plans online. While, 2GC, 3GC and 4GC systems are capable to calculate the optimal timing plans dynamically, yet they differ greatly in their response frequencies and optimization interval.
According to the traffic request, city's size and singular cases in urban areas, the authorities can choose to implement a specific type of control system with desired level of optimization of traffic conditions. The UTC systems have very similar objectives such us minimization of delay. These systems can be centralized or decentralized. Centralized adaptive control systems is one of two fully adaptive traffic control systems, which are available nowadays. This system link all the junctions through their signal controllers to a central computer located in the traffic control centre. Having all data available in real time and in single place, plus cutting-edge processing equipment in the data centre, offer important benefits, such us the possibility of integrate the urban control system with public transport prioritization system, incident detection systems or traffic information systems within a complex management system. The main difference of second type of adaptive systems, distributed adaptive systems, is in its communication architecture. Distributed system uses a full or partial

mesh topology between the intersection's controllers and a routing module to direct traffic flow through intermediate nodes.

UTC systems different in detectors coverage and deployment in the network and level of optimization. Moreover, optimization in UTC systems can use different approach, such as:

- Mixed integer linear programming
- Hill-climbing technique
- Dynamic rolling horizon
- Forward dynamic programming

For better understanding the traffic situations in the controlled area and simplify the flow analysis, UTC systems is made of a number of graphical display facilities, which include diagrams, dispersion data and queue creation, diagrams of the travel distances and display of the individual operations in the intersection. The following figure displays the typical block structure of a modern UTC system:

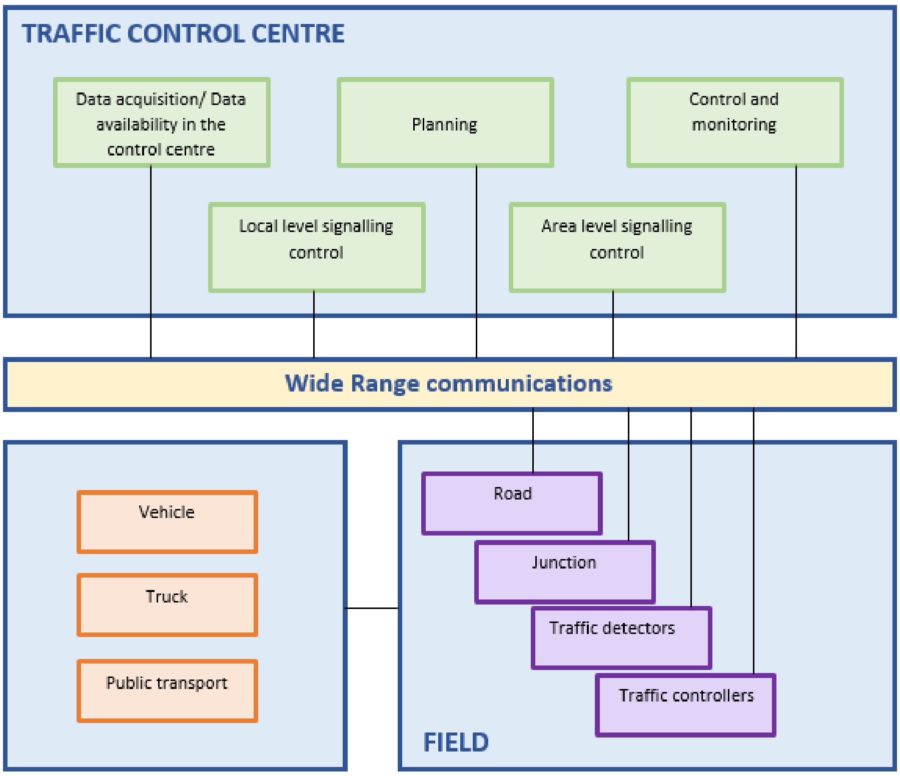


Figure 13: Block structure of UTC systems

Many UTC systems have been implemented throughout the world, each with individual strength and weakness. Some of them are listed below:

- TR2	- CALIFE	- UTOPIA	- OTSCS
- UTCS-1	- SCOOT	- CRONOS	- MOTION
- UTCS-2	- SCATS	- OPAC	- ALLONS-D
- UTCS-3	- RHODES	- SYNCHRO	- PRODYN
- GERTRUDE	- MOVA	- SIGOP	- TRANSYT

Ineffectiveness of many UTC systems is often connected with detectors errors, malfunction, and failure to respond to short-term traffic fluctuations as a matter of robust prediction models. Moreover, the transition between optimal plans may offset the benefit achieved.

The aim of the UTC strategy is to provide, at each cycle, dynamic signal settings, taking into account the overall traffic conditions. Some of strategies implemented in different urban areas are gating and matering, force and hold greens, negative offset, maximum capacity flow, shorter or longer cycle length, green waves with cross streets. Metering and extended green strategy is one of the most effective techniques used to recover from congestion or to reduce its effect on the urban network. Negative offset is using within predetermined plans for long queue formations. The next sub chapter reviews some of existing UTC online systems.

3.4. ON-LINE SIGNAL CONTROL STRATEGIES

Models described in previous subchapters reviewed off-line signal control methods. In this section, on-line traffic signal control will be discussed.

Adaptive Traffic Control System (ATCS) continuously makes adjustments, in real-time, of signal timing parameters in order to respond to current traffic conditions, demand and system capacity. Typical ATCS bases on the traffic model, highly developed control algorithms, centralized or decentralized architecture and various detector configuration. ATCS are well suited for under saturated and unpredictable traffic conditions by dealing split, cycle length, phase sequence and offset adjustment. The benefits of adaptive system are not easily observable in oversaturated traffic conditions. However, they can delay the oversaturation and reduce its duration.

During the last years, various systems were developed, where widely used are SCOOT (Split Cycle Offset Optimization Technique) and SCATS (Sydney Coordinated Adaptive Traffic System). Some of ACT systems abandoned standard signal timing structures constrained by offset and cycle length and offer new approach based on different techniques of mathematical programming. Some of these models are OPAC (Optimization Policies for Adaptive Control), PRODYN (Programming Dynamic), SPOT (System for Priority and Optimisation of Traffic), etc. Some of these strategies are discussed in the next subchapters.

3.4.1. SCOOT

SCOOT (Split Cycle Offset Optimization Technique) was initiated by the British Transport and Road Research laboratory in the 1970s and it is a 2th Control Generation model. Nowadays, it operates in over 170 cities worldwide. It is parametric, cyclic, centralized and fully traffic responsive signal control system. SCOOT automates the TRANSYT traffic signal optimisation model (Robertson, 1969) by using on-line surveillance information to incrementally update the signal timings. This gradual approach is adaptive, but not prone to overreacting and is less disruptive than the process of transitioning between two distinct plans as typical time of day scheme. System is less sensitive to detectors failure and there is no need to predict arrival flows. A SCOOT system divides the traffic network into "regions", each consisting of a number of "nodes" (set of signalised intersections with a common cycle length which permit a progression).

SCOOT predicts the traffic arrival pattern based on the flow information collected at detectors placed downstream of the upstream intersections. This location can also detect imminent spillback conditions. Moreover, detector is less useful when it is covered by a queue. SCOOT is not capable to detect and model condition, when queueing occurred right up to the exit detector.
SCOOT converts information, about passing vehicles through the upstream detector, into "link profile units", a hybrid measure of link occupancy and flow. This unit is used over time to estimate "cyclic flow profiles" for each link of the intersection.

Arrival and departure profiles are compared, and the difference between them represents the queued vehicles at the junction. System uses a combination of queued vehicles, the time to clear the queue, and the impact of the split and offset adjustment to calculate the traffic flows for each cycle. The SCOOT includes dynamic control algorithms of individual intersections, arterials and networks. This optimisation algorithm works at three levels: Split, Offset and Cycle in order to minimize stops and delays.

3.4.2. OPAC

OPAC (Optimised Policies for Adaptive Control), is the example of third Control Generation models, which optimise parameters such as cycle time, offsets and splits, to non-parametric models in which the decision to switch between phases is based on actual arrival data at the junction. This flexibility in the setup of signal timing enables the controller to generate "acyclic" signal settings and hence it is more appealing for real-time signal control implementation. Gartner (1983) formulated the problem in OPAC for a single intersection as a discrete-time optimal control problem. This formulation that was not practically solvable using DP-based methods, therefore Gartner suggested to use a restricted search heuristic that enumerates a few alternative feasible solutions for a two phase intersection.

OPAC considers the saturation flow and queue formation on each link and in the result maximises the intersection throughput. System first determines the next phase to activate in cases where no critical link is identified. Loop detector are used to measurement to predict traffic arrival rates, which are then fed into the algorithm to evaluate the necessity for revisiting the neighbouring intersection timings in light of the intersection throughput and queue formation at neighbouring intersections.

More recently, OPAC has been extended to accommodate arterial networks and uses a local level and a network level of control in a decentralised fashion. At the local control level system calculate the next phase at the intersection. At the network control level, OPAC provides progression through the intersection. OPAC identifies critical intersections by traffic flows measurements from all intersections within the controlled area, and then determine a "virtual common cycle" length once every few minutes. A virtual fixed cycle is determined on-line and is fixed between intersections to enable progression. The length of this cycle varies according to the needs of either the critical intersection or the majority of intersections. Therefore, OPAC provides local progression by considering flows into and out of an intersection in selecting its splits and offset.

OPAC went through several developmental stages that ranged from OPAC-I to OPAC-VFC (Gartner, 1983; Gartner et al., 1995, 1999, 2001). OPAC-I optimised intersection performance using DP and it could not be implemented in real-time because of the extensive time needed to compute the optimal parameters. OPAC-II used optimal sequential constraint search OSCS to calculate the total delay for all possible phase switching options. The optimal solution was the phase switching, which minimises the total intersection delay. It predicts arrival traffic flows throughout the planning horizon. OPAC-III employ a rolling horizon approach on a simple two-phase intersection and in result overcome the limitation of previous systems. Later, was extended to an eight-phase intersection, with possibility of phase skipping. OPAC-VFC include an algorithm to achieve progression along corridors.
Although OPAC attempts to achieve theoretical optimum signal timing plans, it does not guarantee global optimality due to the approximation done to DP using the restricted search heuristic OSCS. Furthermore, the application scale of OPAC is limited due to the tremendous computational effort involved in the OSCS search.

3.4.3. REINFORCEMENT LEARNING

The 4-GC systems are principally built on self-learning capabilities, which are based on experience under real-time conditions and reasonable computational requirements to be implemented in real-time. These

traffic systems are still under continuous research and development. Promising potential for "self-learning" ATSC shown Reinforcement Learning, an artificial intelligence technique. RL not only achieves as much as DP but also requires less calculation and does not need a perfect model of the environment.
Moreover, RL-based control methods learn from direct interaction with the traffic network, and can consequently capture the stochastic variations in traffic flow without the necessity for model-based traffic prediction.

The basic concept of RL is concerned with a signalised intersection interacting with traffic network in a closed-loop system in which the intersection acts as the controller of the process . The agent iteratively observes the state of the environment, takes an action accordingly, receives a feedback reward for the actions taken and adjusts the policy until it converges to the optimal mapping from states to optimal actions (optimal policy or control law) that maximises the cumulative reward. Accumulating the maximum reward not only requires the traffic signal control agent to exploit the best-experienced actions, but to also explore new actions to possibly discover better actions in the future. The interaction between the agent and the environment can be viewed as two processes performed repeatedly: a learning process and decision making process. In the learning process, the agent adjusts the policy by updating the value associated with each state-action pair using, in-part, the immediate reward value. In the decision making process, the agent chooses its action by balancing exploration and exploitation using action selection algorithms.

Chapter 4
METHODOLOGY

4.1. OVERVIEW

Today, adaptive control is one of the most effective way to manage traffic networks. From the previous chapter, it was possible to see different adaptive control strategies. Each of them has been studied and analysed to determine the most appropriate way to select the parameters required by each method. Nowadays, second and third generation control are progressively development. However, adaptive systems need superior investments in terms of infrastructure and communication hardware. Existing TRPS mode can be utilized in order to provide an operation, which theoretically are equivalent to adaptive control. Traffic responsive control is one of the most popular, efficient and powerful adaptive control modes. First generation control concept is termed as being a closed-loop system, which consist of series of traffic signals controllers connected to the master controller. Linkage between traffic controllers occurs by means of fibre-optic cables, hard wire connections or spread spectrum radio. Master controller implements suitable timing plans stored in the local controllers of individual intersections. Moreover, it can also send complete traffic condition report to the traffic management centre through telephone or other communications channel.

Previous study have proven that coordination of traffic signals in a closed-loop system and implementation of newly optimized timing signal plans can provide reduction in delay and decrease of travel time of 10-20 percent (23). Furthermore, travel time reduction will also decrease the number of total stops, value of vehicle emission and fuel consumption.

The sequent research in Texas regards to evaluation of impact of correct timing a closed-loop system confirmed reduction of above values as follows: 29.6 percent of delay, 11.5 percent in stops and 13.5 percent in fuel consumption (24). Total savings to the public was estimated of approximately 252 million dollars in one year.

In order to obtain listed benefits it is required that timing plans, operating in the closed-loop, are proper to the existing traffic conditions and able to change in a timely manner with variation of traffic volume.

There are two control modes for the selection of particular the timing plan at a given instant:

- Time of Day mode (TOD)
- Traffic responsive plan selection mode (TRPS)

The difference between TOD and TRPS mode is that in TOD the timing plans based on the historical traffic conditions, while in TRPS mode the plans are changing with variation of traffic demand. TRPS mode are more efficiently in cases of holidays, special events or other randomly occurred conditions.

The TOD mode, commonly used, assumes that traffic patterns are iterative and in result, particular TOD plan is implemented at the same time every day, regardless of the existing traffic condition. It works very well on the networks with predictable traffic conditions. However, in networks where demands has dynamic unexpected traffic flows, signal timing plan working in TOD mode can be inappropriate for current traffic patterns. Moreover, timing plans have to be continuously updated to match to temporal traffic trend.

TRPS mode, subject of this thesis, has capacity to implement proper timing plans, which are suitable to actual traffic condition. TRPS mode uses system detectors to measurement counts and/or occupancy in the closed-loop system network.

This information is aggregated to certain TRPS parameters and subsequently the master controller continuously compares them to the corresponding thresholds and select adequate timing plan form a pre-stored library of signal timing plans. In comparison of TOD mode, traffic responsive plan selection mode can efficiently reduce total system delay, minimize number of stops and in result improve the system performance. Moreover, the TRPS mode can reduce the need for frequent redesign of signal timing plans.

Despite all the advantages from implementation of Traffic responsive plan selection mode, it has been rarely applied in the field. Moreover, in the literature, there is limited information about methodology for implementation and setting up of this mode. Therefore, time of day mode is preferred.

4.2.TRAFFIC RESPONSIVE CONTROL

Timing plans typically based on historical vehicle demand data. In reality, demands presented at specific time on specific day are random values from some statistical distribution, which is not constant and changes over time in regards to variation in the zone or in population. People can change theirs routes, modes of transport or departure times because of environmental impacts, such as weather. All this modification increases travel times and shifts arrival times at intersection and in results varies the traffic demand. TRPS assigns current demand in one of the pre-established demand states and selects a suitable signal-timing plan. Demand states were determined by clustering the approach volumes of the network collected in the field. Clustering techniques identify a number of the best-separated groups existing in the data set. These groups are demand states to which subsequently is assigned proper timing plan.

Traffic responsive plan selection mode provides a mechanism by which is possible to change timing plans in real time in response to variation in traffic demand. Traffic controller chooses and implements optimal timing plan to actual traffic conditions.

In order to set up TRPS mode, it is necessary to proper numbers of detectors distributed on the traffic network. Position and number of system detectors that can be supported by traffic controllers varies depending on the manufacturer. Efficiency of traffic control is connected with type of system detectors used for measurement. In general volume and/or occupancy data are collected from chosen system detectors. There are different methodologies used by various manufacturers to proceed with collected data, but the concept of all methods is the same. Threshold mechanism and pattern matching mechanism are two methods for implementation of traffic responsive mode in any network.

TRPS threshold mechanism utilizes detector data (volume and/or occupancy) that is aggregated into Computational Channel parameters (CC) by multiplying each system detector by its corresponding weight. Subsequently, CC parameters are aggregated into plan selection parameters (PS) in order to arrive at the final timing plan. The master controller uses smoothing, scaling and weighting factors in order to aggregate information from the detectors. The master controller compares each PS parameters to the predefined set of thresholds in order to determine appropriate PS level. If based on the traffic variation, values of PS parameters are different, new pre-stored timing plan is implemented.

In order to determine the thresholds with the best separation between levels, different researches are performed, such as discriminant analysis, decision-tree classifiers, artificial neural networks, etc.
Each controller manufacturer uses various CC parameters and different mechanism for applying the traffic responsive plan selection mode. This thesis develops modern approach to implement TRPS mode into SmartEye system in order to provide an optimal operation of traffic signal in the urban artery. This innovative system will directly identify the best traffic state for every 15-minutes traffic volume and subsequently, it will send information to the controller with corresponding pre-stored timing plan.

Pattern matching mechanism, implement only weighting factors for system detectors. In this method, weight assigned to corresponding detector is different. Master controller change timing plan applied in networks based the sum of the deviations of individual count and occupancy data from those stored in the master controller for every timing plan. Values of stored counts and occupancy data simulate the thresholds in the threshold mechanism. All detectors data are combined together with pre-stored detectors value in only one parameter Fj for each timing plan. The combined Fj parameter is calculated for every stored plan and depends on various factors such as the VPLUSKO weighting factor K. This factor is the weight factor for every system detector and the global factor for all detectors and all times of day. In order to calculate various Fj plan values some of the controllers use following formula (25):

$$F_j = \sum_{n=1}^{\infty} |W_i[(V_i + \mathrm{K} * O_i) + (V_{ij} + \mathrm{K} * O_{ij})]|$$

Fj= sum overall detectors (i) of the absolute value of the weighted difference between current and pre-stored data accompanied with each plan;

Vi and Oi= volumes and occupancies of detector (i);

Vij and Oij= the volumes and occupancies stored with plan (j) for detector (i);

K= a user supplied VPLUSKO weighting factor whose value is between 0 and 100;

Wi= specific weighting factor of detector used to emphasize occupancies and volumes measured by selected detectors if their outputs are more significant. These value ranging between 1 and 10.

Traffic responsive control mechanism has some limitations implemented by traffic controller manufacturers. The most important restrictions are impossibility of implementation many timing plans and limited number of system detectors that can be assigned to the network. Moreover, usual traffic monitoring cameras cannot detect traffic flow volume from all approaches, so there is a need to use of larger number of sensors. The SmartEye system, proposed in this thesis as detection and operation methodology of TRPS mode overcomes this drawbacks.

Furthermore, with many fluctuations in traffic during a typical day, many different times could be implemented. Because of frequent changes of timing plans, the effect of transition should be considered.

4.3.TRANSITION

Traffic responsive systems examine traffic fluctuation during the day and apply new timing plans, when different conditions occur. This mode is very sensitive to actual traffic demands on the network, because use of real-time information. When the traffic fluctuations is highly variable, timing plans should be changed frequently. It is important to consider the transition when changing timing plans. In order to reach new setting, the timing plans are adjusted by sub tracking or adding time during certain intervals. During transition the traffic is disrupted, because of phase is lengthened or shortened. This provoke an increase in delay. In this case, the offset for through progression should be adjusted to re-establish good traffic progression. In signal traffic control, there is a period of time, when the traffic signal operates with less optimal signal settings every time a new timing plan is enacted in a system. These signal settings can extend over multiple cycles until the new timing plan can be implemented and in results increase the amount of delay on the artery.

The main problem with transition from one plan to another is to avoid the following:

- so short green times that drivers are confused and have rear-end crashes as one stops but others does not;
- so short red times that pedestrians cannot cross the road;
- extended red intervals provoke excessive queues on the intersection approaches;
- some approaches do not have enough vehicles due too long red displays upstream of the signal.

In order to ward off these problems, traffic signal controllers have incorporated strategies allowing transition from one plan to another. A transition process covers changes in timing, phasing and offsets in a coordinated signal system in specific period of time that is required for transition from one timing plan to another.

There are different methods of transition between timing plans depend on controller manufacturers (26). The most used methods of effecting an offset change, such as:

- Shortway;
- Shortway Add Only;
- Infinite Dwell;
- Dwell with Interrupt;
- Smooth;
- Add Only;
- Dwell.

A shortway transition method implements a new offset by the shortest way possible. This method subtracts or adds time to different phases until achievement of the new offset. The time required to transition to a new offset is no more than 50 percent of the cycle length. The transition can occur over multiple cycles. Based on the total amount of transition time, that is the time difference between the existing and the proposed offset, a decision if add or subtract time during transition is made. The time is added until the proposed offset is reached, if the time difference is less than 50 percent of the cycle length. In the case, this difference is higher, the time is subtracted. When time is being added, it is added only to the coordinated phase. When the time is being subtracted, an equal portion of the total transition time is subtracted from all phases, subject to availability of time. 18.75 percent of cycle length is a maximum amount of time that can be subtracted or added during each cycle. If the new offset cannot be reached within five cycles by subtracting time from the phases, the offset is affected by adding time.

A shortway Add Only transition method is a variation of Shortway method, where the offset transition is realised by dwelling in the green portion of the coordinated phase. Likely, in the previous method, 18.75 percent of the cycle lengths is the maximum time, when controller can dwell in the coordinated phase. After dwelling, the controller releases and begins timing the other timing plan phases. If the desired offset is not reached during first dwell time, the process is repeated until the new offset is reached.

In Infinite Dwell method, the controller dwells in the coordinated phase until it receives a proper synchronization pulse from the master controller. In this transition method, the master controller need an offset interrupter, which imposes a number of shifting interrupter pulses onto the interconnect line containing the real synchronization pulse. Until the desired offset is achieved, the interrupter keeps the controller from receiving the adequate synchronization pulse. When the adequate offset is reached, the controllers receive a synchronisation pulse and rest of the phasing is allowed to occur.

The next available method of transition is Dwell with Interrupt, which is similar to Shortway Add Only in the fact that the controller is forced to dwell in the coordinated phase. The difference is that the user decide the maximum amount of time (from the range between 1 second and 999 seconds) that the controller can dwell in the coordinated phase. After dwelling of the controller in the coordinated phase for the allotted time, it services the remainder of the phases in the cycle. This process is continuously repeated up to achieve desired offset.

A smooth transition option change the current offset to the desired offset in the shortest time possible. In this method, it is possible to add a maximum of 20 percent or to subtract a maximum of 17 percent of the cycle length to the coordinated phase. The controller calculates the difference between the current and new offset, after each transition. The controller will add time to the coordinated phase, when the new offset is higher than current by more than 50 percent of the cycle. In case, if the new offset is less than current by more than 50 percent of the cycle, the controller will subtract time to the coordinated phase. The controller forces the offset change to occur by adding time, when the controller determines that sub tracking time from the coordinated phase results in cycle length less than minimum cycle length.

In the Add Only transition method, changes in offsets are affected by only adding time to the coordinated phase, in regards to the magnitude of the offset change. The maximum of 20 percent of the cycle length is added to the coordinated phase every cycle until new offset is accomplished.

The last Dwell method gives the possibility to the controller to holds the coordinated phase at the beginning of the green portion for a time interval specified by the user. The user can set up a dwell time in seconds (0-255 seconds) or as percentage of the cycle length (0-99 percent). After expiring a dwell interval, the controller releases the coordinated phase and normal timing resumes. This method repeats dwell interval once each cycle till the new offset is reached.

4.4.PROPOSED APPROACH

Traffic Responsive Plan Selection system requires an effort and significant amount of time. This is a reason why traffic engineers usually revert to time of day mode. Outdated TOD plans may provoke delays and excessive number of stops. Proposed approach provide good performance and reduce the "aging" of timing plane. The approach discussed in this thesis proposes that only a few timing plans are needed for certain traffic arteria. TRPS parameters must be selected in order to design and choose the most suitable plans and match them to the existing traffic conditions (27).

Because of huge number of traffic pattern levels and conditions, it is necessary to group them together and later match a proper timing plan. This method is very similar to TOD approach, which functioning with limited number of timing plans assigned to certain period of time. There are typically am-peak, off-peak, pm-peak, etc.

Selection of representative timing plans in this thesis has a congruent methodology by clustering similar traffic conditions into smaller number of groups and after assigning traffic plan to each of them. The most important difference is that approach described in this thesis is not limited to clustering traffic patterns that are temporally adjacent.
As follow, differ issues related to TRPS mode are described:

- Detection of existing traffic data;
- Generation of traffic levels and pattern matching;
- Development of signal timing plans;
- Simulation and evaluation.

The first issue is detection and collection of the existing traffic volume data from the detectors positioned along the chosen intersections. TRPS control mode requires the input of many detectors to measure the traffic flow changes. Detectors have to ideally represented traffic volume and possess consistent data. SmartEye video-detection system was used to monitor and record the volume data. This system is able to perform the analysis of the distribution of the traffic flows along the whole road intersections in real time and through implemented algorithms manage TRPS mode in the most suitable way. The sensor detects the traffic volume of every movement of every approach on the junctions and automatically discard the unusual spikes or dips in traffic flow from further consideration.

The second problem is generation of different traffic scenarios, which can encounter in the future, and definition of threshold for each of this state. Detailed plot of the 15-minutes volume data by time of day over specific weekday was prepared and subjected to the next examinations. TRPS mechanism selects and associates timing plans to traffic state, then activate a specific timing plan after recognition of this adjacent traffic state. The activation mode is applied through a pattern matching mechanism, where new timing plan is implemented, when the traffic volume is associated to its similar to state. K Nearest Neighbour classification method was used as pattern matching mechanism. It is a simple algorithm that stores all available cases and classifies new cases based on a similarity measure such us distance functions. Therefore, it is very important to cluster the traffic states into groups with similar characteristics and after associate suitable traffic plan, lest fail through activation of inadequate timing plan. In this thesis, K-means clustering is used as traffic state classification method. This analysis groups each observation from n-dimensional space that are closer in their attributes K-means clustering determine thresholds, as the mid-points between different group centres. In the field application, this cluster algorithm described in the thesis will implemented in SmartEye system in order to directly associates 15-minutes traffic volume to the closest group and applies the most suitable timing plan for the next 15-minutes.

Subsequently, optimal timing plans for all traffic states was designed. The purpose of this step was to select the best timing plans, which ensure good progression in both directions along the artery. Existing phase sequence at the intersections is kept the same as the sequence in the current time of day mode, because this sequence is governed by the geometry at each intersection. Synchro 8 was selected and used in order to develop the optimal solutions. This programme is described with details in the next chapter.

The last step was the simulation and evaluation of time of day mode and traffic responsive control. Evaluation of created timing plans with traffic scenarios was performed using SimTraffic simulation implemented in Synchro 8. This evaluation is very important since it provides estimated values for numbers of stops and total delay for both modes, TOD and TRPS. Afterwards, was carried out the comparison of two discussed modes.

4.4.1. SYNCHRO STUDIO 8

Trafficware Inc. develops Synchro and it has the best user interface of signal-timing software currently available. Synchro is a complete software for design and optimization of traffic signal timing plans. SYNCHRO bases on Highway Capacity Manual to analyse the intersection capacity and improve signal timing through optimization of cycle lengths, offsets and splits. This eliminates the need to search the best signal timing plans by trying multiple plans. The software reduce delays in the network and is able to model actuated signals (28).

Synchro provides optimization of cycle length by analysing all cycles in the user defined range and increment. In order to determine network cycle length, Synchro minimizes the performance index (PI). It will be chosen the cycle length with the minimum performance index, calculated from following formula:

$$PI = [(D * 1) + (St * 10)]/3600$$

PI = Performance Index;

D = Total delay (s);

St = Vehicle stops (vph).

Optimization of offset has multiple stages, which depend on the optimization level selected by user (number of stages and search step size). Synchro tests all possible offsets. The optimal signal timing plans minimize the delays between the intersection and its immediate neighbouring intersections. Synchro recalculates the delay based on the departure patterns for a junction and its adjacent node.

Split optimization is accomplished by first attempting to service critical lane's 90th percentile traffic flow. Synchro attempts to serve 70th or 50th percentile traffic flow, if the cycle time is too short to achieve this. Main phases get any extra green time.

Synchro assumes random arrivals that follow a Poisson distribution and uses 10th, 30th, 50th 70th and 90th scenarios for the delay calculations, in order to calculate the variability of traffic flow. The delay output by the model is the average of these five scenarios weighted by the percentile flow rates, which gave similar results to Webster's formula. However, the method is more indicated to actuated signals in the presence of skipped or pedestrian phases.

Timing plans are developed in Synchro based on detected volumes from SmartEye sensors. These timing plans were subsequently adjusted manually after observation of the real traffic conditions in the field. Timing plans were then simulated in two software SimTraffic and DYNASMART in order to test their level of performance.

4.4.2. SIMTRAFFIC

SimTraffic is a tool implemented in Synchro 8 and developed in order to model signalized and un-signalized intersections in the network. The most important purpose of the simulator is to check and fine tune traffic signal controls before implementing them in the field. Software includes the driver and vehicle performance characteristics developed by the Federal Highway Administration for use in traffic modeling. SimTraffic is especially useful for simulation of complex situations, which cannot be easily modeled macroscopically including closely spaced intersection with blocking or lane change problems, the affects of signals on nearby unsignalized intersections. SimTraffic is able to model pretimed and actuated signals, large traffic circles, roadway bends, cars, trucks, buses and pedestrians.

SimTraffic is capable of simulating traffic conditions read in from outside files. These files are based on volume data at every 15-minute intervals and the simulation effectively mimics the trend of traffic conditions according to these historical volumes. SimTraffic is also able to simulate transitions between timing plans by reading in the plan files output from Synchro that correspond to the times being simulated.

A major drawback with SimTraffic is that only 19, 15-minute intervals can be simulated at one time; however, there are no restrictions on the number of intersections in the network.

4.4.3. DYNASMART-P

DYNASMART is a state-of-the-art Traffic Estimation and Prediction System, which supports operations decision and transportation network planning. The following figure illustrate the model structure of the software.

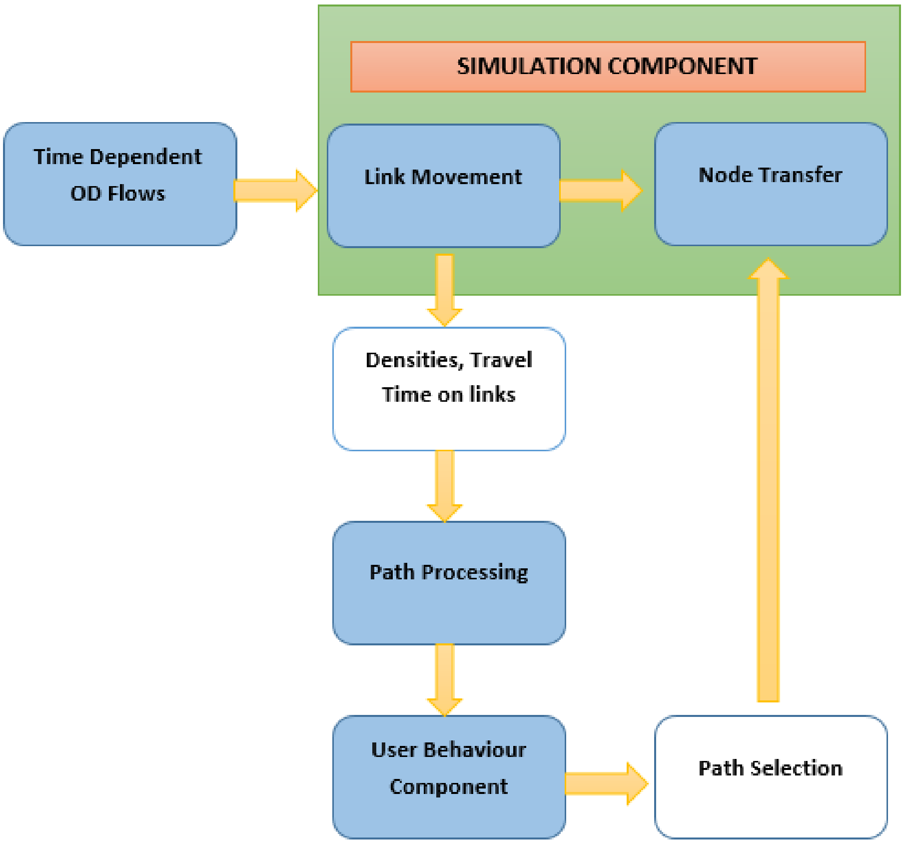


Figure 14: DYNASMART-P model structure

After design of the network and control settings, the simulation component will load OD flow matrix depended on time and process the movement on and between links. Software introduces instructions with user behavior in order to determine every individual path decision of the users in the network. Alternatively, DYNASMART can be used as a simulator in the context of algorithmic procedures (e.g. system optimal dynamic traffic assignment), where path decisions may be pre-assigned for all or some users under particular assignment scheme.

There are two version of DYNASMART. The real-time version DYNASMART-X also supports the ATMS/ATIS capabilities in the ITS environment. In this thesis, offline version DYNASMART-P will be used. It is a dynamic transportation network design, evaluation, planning and traffic simulation tool. The software models the traffic demand in the network based on travel decisions of individual drivers, which seeks to fulfil a chain of activities over a given planning horizon. Model of DYNASMART-P is an efficient hybrid traffic simulation-assignment approach due to explicit representation of traffic network elements, the explicit description of time-varying traffic processes and its richer representation of travel behaviour decisions. The modeling features of the software achieve a balance between computational efficiency, representation detail and input data requirements. The OD flow matrix in DYNASMART-P are input externally and are fixed for the period of the analysis.

The DYNASMART simulation model moves vehicle in discrete macro particles at the prevailing local speeds determined from the established speed-density relations. This concept is adapted from plasma physics, exhibiting similar properties. The first macro particle simulation model was developed as a special-purpose code for experimental research of commuter behaviour dynamics in congested traffic networks. From 5 to 20 vehicles were used as a macro particle. The DYNASMART simulation model is the extension of this macro particle model. It uses only one vehicle as a macro particle, which mean that it can effectively track the location and movement of individual vehicles through a network. However, the model does not track microscopic details of individual's movements, such as in car-following models. Therefore, the model is a mesoscopic simulation due to the combined aspects of microscopic details and macroscopic relationship.

The traffic simulation uses the equilibrium of the speed-density relationships together with the conservation law in order to represent the traffic flow, which is practically LWR-type macroscopic traffic flow theory. The continuity equation is solved numerically using discrete time steps. Virtually, both average link speed and average link volume are eligible to transfer the vehicles in the simulation since the identity "volume = speed × density" is always hold. However, for links of finite lengths, the model moves vehicles through a corridor at the prevailing local speeds determined from the equilibrium speed-density relations, in order to avoid physically unrealistic speeds.

Node transfer and link movements, described in subsequent sub chapters, are two primary modules of the simulation.

4.4.3.1. LINK MOVEMENT

On the link movement module vehicles move on links during every simulation time step or scanning time interval in the simulation. Links of the network are subdivided into smaller segments for purposes of traffic simulation. The concentration of the vehicles prevailing in a section over a simulation time step is defined from the solution of the finite difference form of the continuity equation, given the concentration as well as outflows and inflows over the previous time-step.

The corresponding section's speeds are calculated using the current concentration and according to a speed-density relation:

$$Vi^t = (Vf - Vo) * (1 - \frac{Ki^t}{Kj})^{\alpha} + Vo$$

Vi^t, Ki^t = mean speed and concentration in section i during the t-th time step,

Vf , Vo = mean free speed and minimum speed, respectively,

Kj = jam concentration,

α = a parameter used to capture the sensitivity of speed to the concentration.

4.4.3.2. NODE TRANSFER

Module of the node transfer performs the section-to-section or link-to-link vehicles transfer at nodes. It allocates the right of way according to the control strategy at this intersection. The node transfer module determines the vehicle numbers, which traverse each intersection and number of entering/exiting vehicles to the network at each simulation time step. As an output node transfer gives the number of vehicles remained in queue, added or subtracted from each link section for each simulation step. A wide range of traffic control measures for intersections are reflected in the outflow and inflow capacity constraints of this module. The maximum number of vehicles that leave each lane at an intersection is limited by the outflow capacity constraints, described in the following equation:

$$VIi = \min(VQi, VSi)$$

i: link index;

VIi : number of vehicles that can enter the intersection from link i during AT;

VQi : number of vehicles in queue on link i at the end of AT;

*VSi : maximum number of vehicles can enter the intersection from link i during AT, i.e. Si *Gi · ;*

Gi : remaining effective green time during simulation interval for the movement from link i

Si : saturation flow rate for the movement from link i; and

AT: the simulation interval.

This formula states that the total number of vehicles entering an intersection depends on vehicles waiting in queue at the end of current simulation interval and the capacity of this approach. The capacity definition follows the HCM, and consists of the maximum number of vehicles that can be served under prevailing traffic signal operation. The maximum number of vehicles allowed to enter a link is determined by inflow capacity constraints, which bound the total number of vehicles from all approaches that can be accepted by the receiving link; they include the maximum number of vehicles from all upstream links wishing to enter the receiving link, the section capacity constraint of the receiving link and the available physical space constraint.

$$VOj = \min(\sum_{k \in U} VIkj, VEj, Cj\Delta T)$$

j: link index;

VOj : number of vehicles that can enter link j;

U: set of inbound links into link j;

VIkj : number of vehicles wish that to move from k to j;

VEj : the available space on link j;

Cj : the approach capacity of link j;

ΔT : duration of a simulation interval;

Chapter 5

CASE STUDY: GUGLIELMO MARCONI STREET

5.1. DESCRIPTION OF THE ROAD SECTION

Marconi Street network, because of its extended size, is divided into two functional synchronized part. First part that is discussed in this thesis consists of six intersections and it is 914 m. in length. Speed limit for the main arterial and side streets is 50 km/h. Marconi Street is one of the most congested artery in Rome. The picture below presents the street in the question.

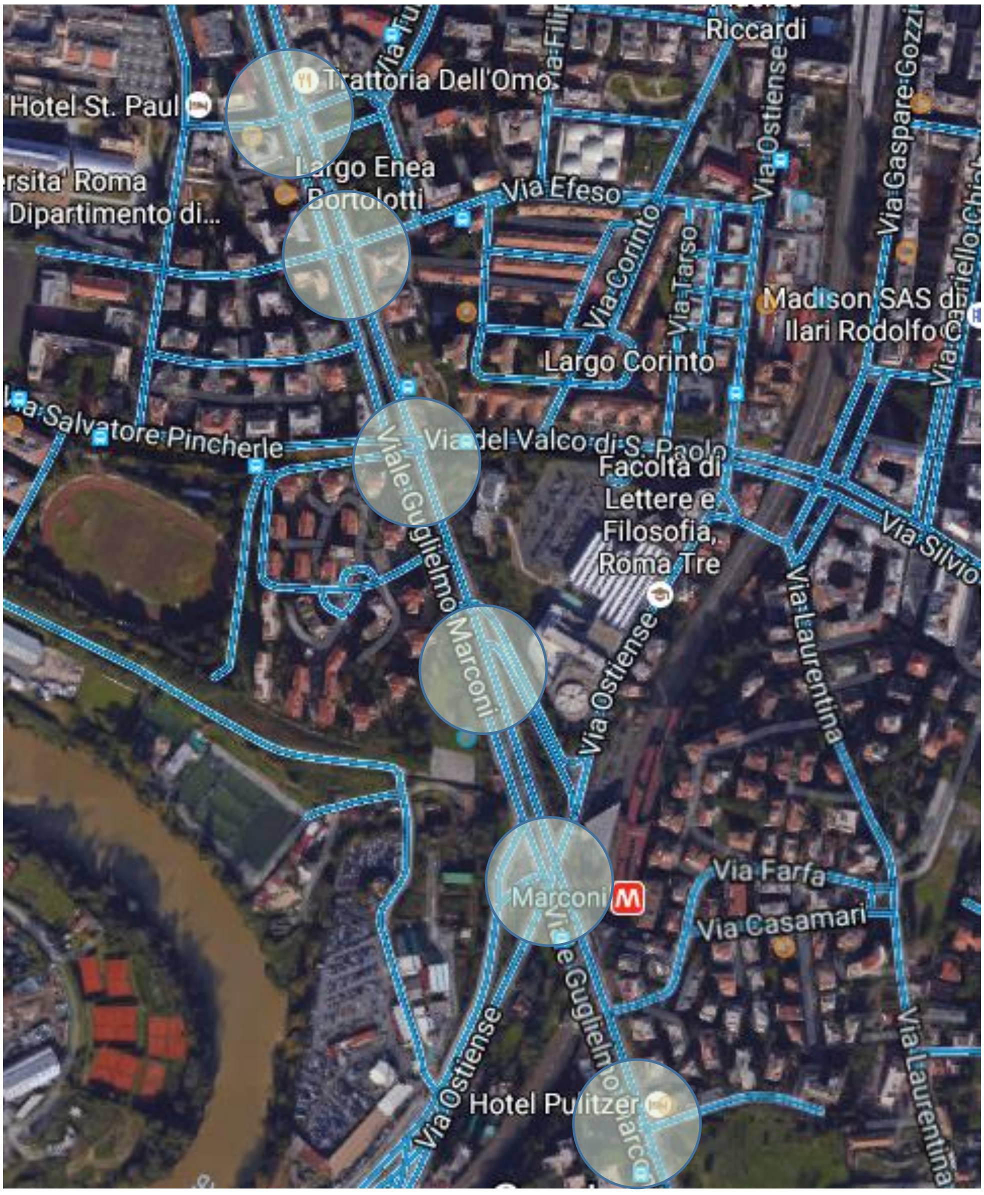


Figure 15: Guglielmo Marconi Street with corresponding analysed cross sections

A campaign of traffic analysis to measure the traffic volume and its composition at the signalized intersections of the artery was conducted in the days from 14 to 15 April 2016 through SmartEye sensors. These detectors have been positioned in three intersections of the artery and they have recorded the traffic flow every 15 minutes. On the other three intersection, the traffic stream have not been detected, but for purposes of the thesis, it was assumed as the values balancing the difference of the flows between the two signalized intersections connected directly with the intersections in question. On the two of these intersections (code 11090 and 12044), artery flow conflicts only with pedestrian volume therefore, second phase is imposed to the minimum in order to permit the safety passage for pedestrian flow. Moreover, the junction with code 11056, where Marconi Street and Melloni Street intersect, is located in a short distance from the intersection coded 11019, because of this, both intersections share the same phasing and cycle length. Other three intersection in their current state, where the traffic flow has been measured, are discussed in this chapter.

Discussed road artery is currently operated using Time of Day mode. Five different timing plans control the entire network, depending on the specific time of day:

- Plan 1: lasts 132 seconds, working from 6.00 am to 10.00 am;
- Plan 2: lasts 102 seconds, functioning from 10.00 am to 03.00 pm;
- Plan 3: lasts 120 seconds, working from 03.00 pm to 09.00 pm;
- Plan 4: lasts 93 seconds, working from 09.00 pm to 01.00 am;
- Plan 5: lasts 81 seconds, functioning from 01.00 am to 06.00 am.

Marconi Street is located in a strictly urban context and is characterized by flow between EUR district and the city Centre and by transversal flows resulting from residential areas such as San Paolo, Garbatella and Ostiense. The existing urban movements are mainly due to Marconi Street, which collects southern part of the city such as EUR and Laurentino district with Ostiense, Garbatella and San Paolo urban districts. Vehicles coming from the south (G.R.A. approach) can enter into the city Centre. Conversely, due to the large number of residents and commercial activities as well as students that are directed to the faculty of the University Roma 3, during all day is encountered a large volume of traffic in both directions. The Tiber River, which parallels and intersects the road axis for relevant sections, creates a natural barrier towards the Magliana and Ostiense district, forcing vehicular flows to join the main axis of Marconi Street.
The present subchapter describe in details the analysis done in order to implement traffic responsive control mode along Marconi Artery.

Section of the road analysed in this thesis has been defined in the section between the intersection of the Gibilmanna Street and the intersection with Bartolotti Street, on both carriageways, and contains the following signalized intersections:

Table 1: Analysed network with corresponding codes of intersections

Code IS	Location	
11022	Guglielmo Marconi Street	Gibilmanna Street
12044	Guglielmo Marconi Street	del Mare Street
11090	Guglielmo Marconi Street	Metro B station
11013	Guglielmo Marconi Street	Valco S. Paulo Street
11056	Guglielmo Marconi Street	Rosa Melloni Street
11019	Guglielmo Marconi Street	Bartolotti Street

The following picture shows location of concerned intersection.

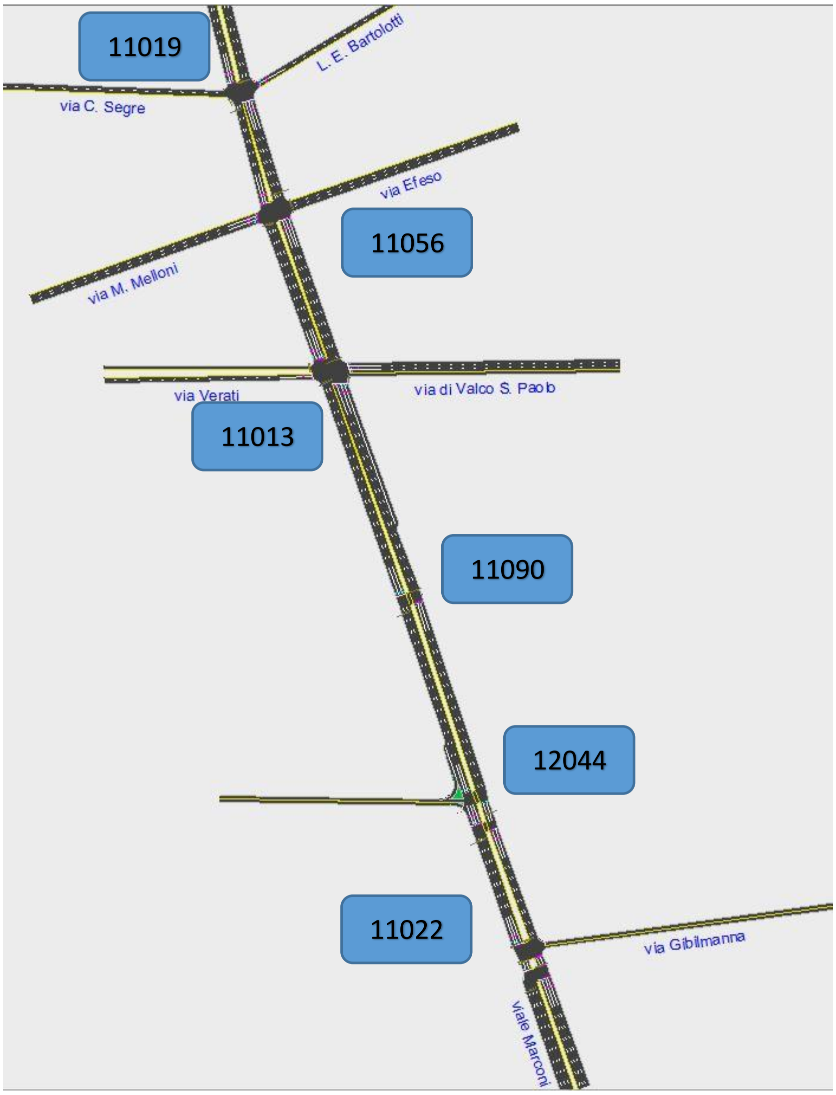


Figure 16: Synchro model of the examined network

5.2. INTERSECTION 11019: MARCONI STREET – LARGO BORTOLOTTI

The intersection Marconi Street- Largo Bortolotti is located in a strictly urban context and is characterized by traffic flow between EUR district and the city center and transversal flows of the residential areas through Largo Bortolotti /Ephesus Street, and to the University of Rome 3 through Segre Street /Melloni Street.

The pictures below present the intersection in the question.

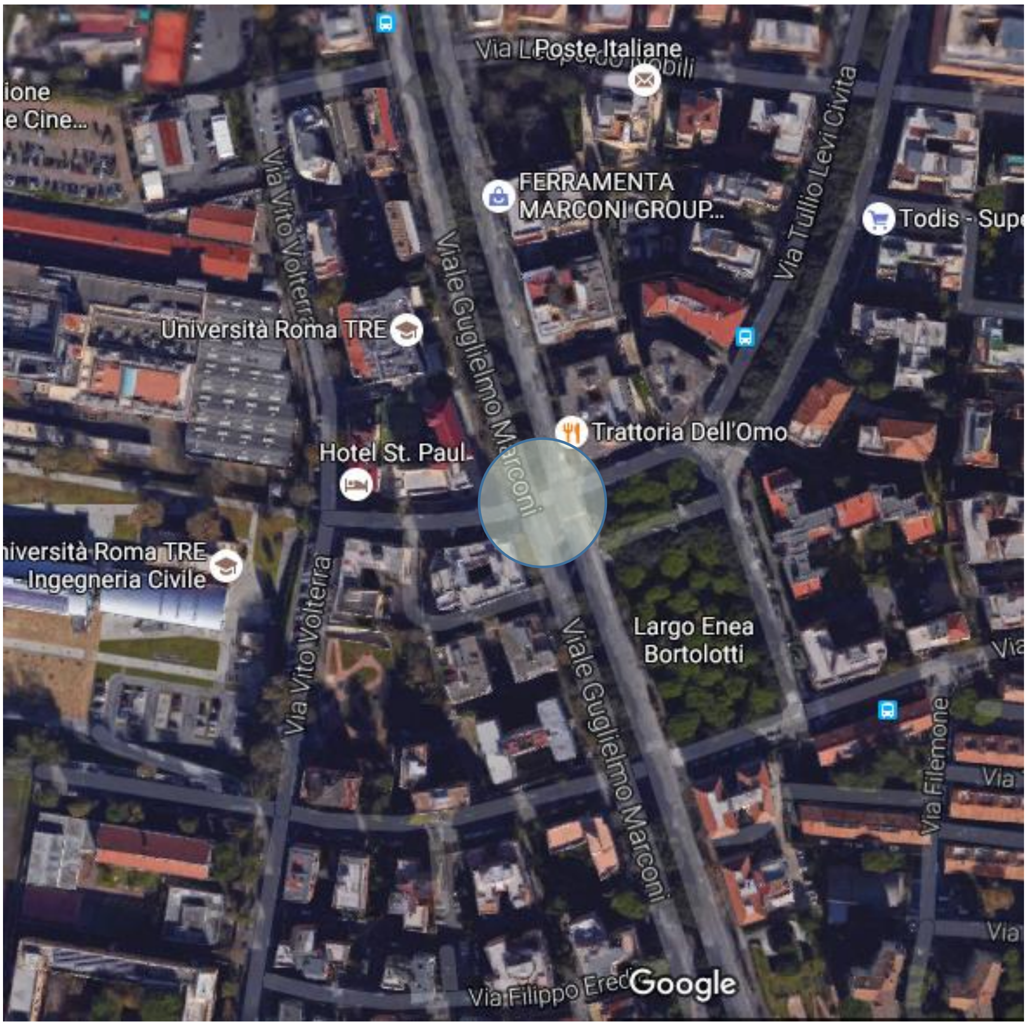


Figure 17: Intersection 11019

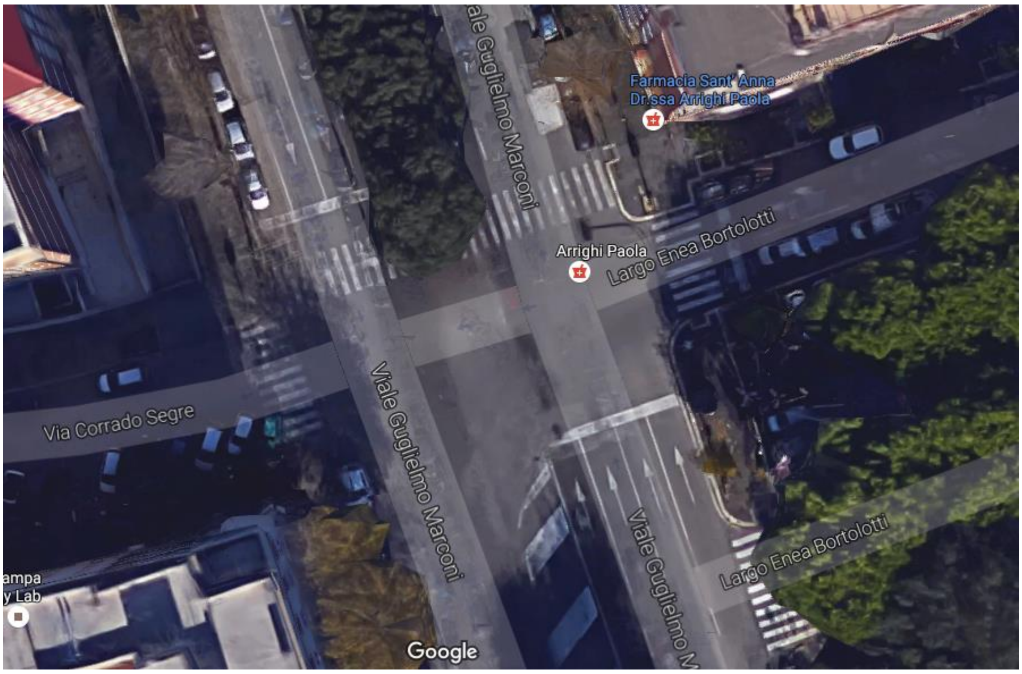


Figure 18: View of intersection 11019

The following plan shows the functional organization of the signalized node at issue.

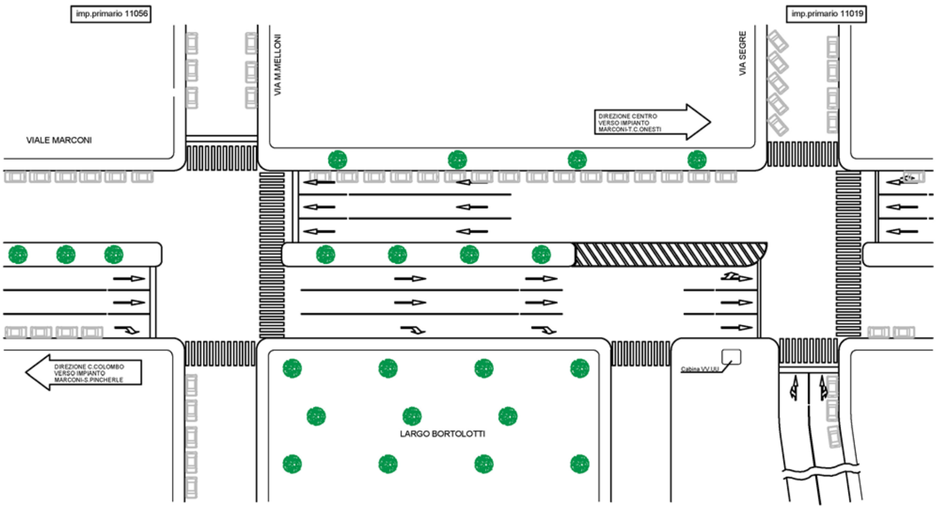


Figure 19: Functional organization of the intersections 11019/11056

5.2.1. TRAFFIC FLOW ANALYSIS

The figure below highlights images captured by the sensor SmartEye, which indicate the type of installation and the sensor's ability to determine, through the definition of virtual targets, the traffic flow along the intersection.

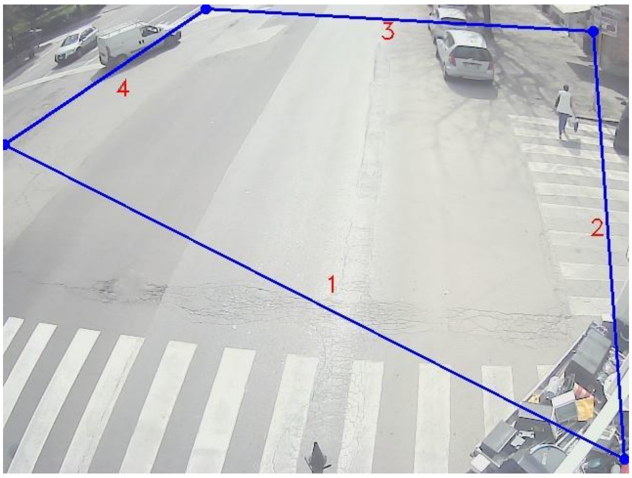


Figure 20: Image of the node 11019 captured by the sensor SmartEye

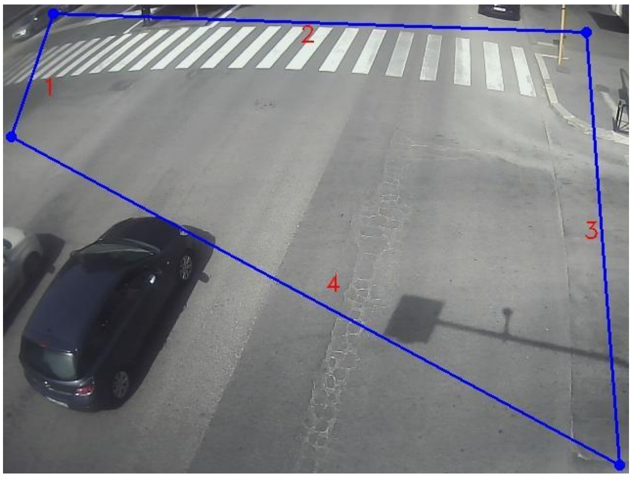


Figure 21: Virtual targets of the sensor

At the intersection under examination, given the wide road surface and the inability to install the sensors at the appropriate heights, two SmartEye sensors have been installed, in order to cover the entire study area.

As shown in the figure below, the node is affected by a traffic flow that has medium-high values, with two maximum peaks in the morning and afternoon hours in correspondence of the opening and closing of the offices. Also during non-peak daily hours, the intersection is characterized by medium volume of the vehicular traffic. Traffic stream is decreasing significantly during a night.

The figure below illustrates a trend of vehicular flows on the intersection in the exam. During morning peak hour the value of vehicle at the junction arrive up to 5050 [veh/h], while during afternoon peak is increasing up to 6200 [veh/h]. At the night traffic stream oscillates near 2000-1000 [veh/h].

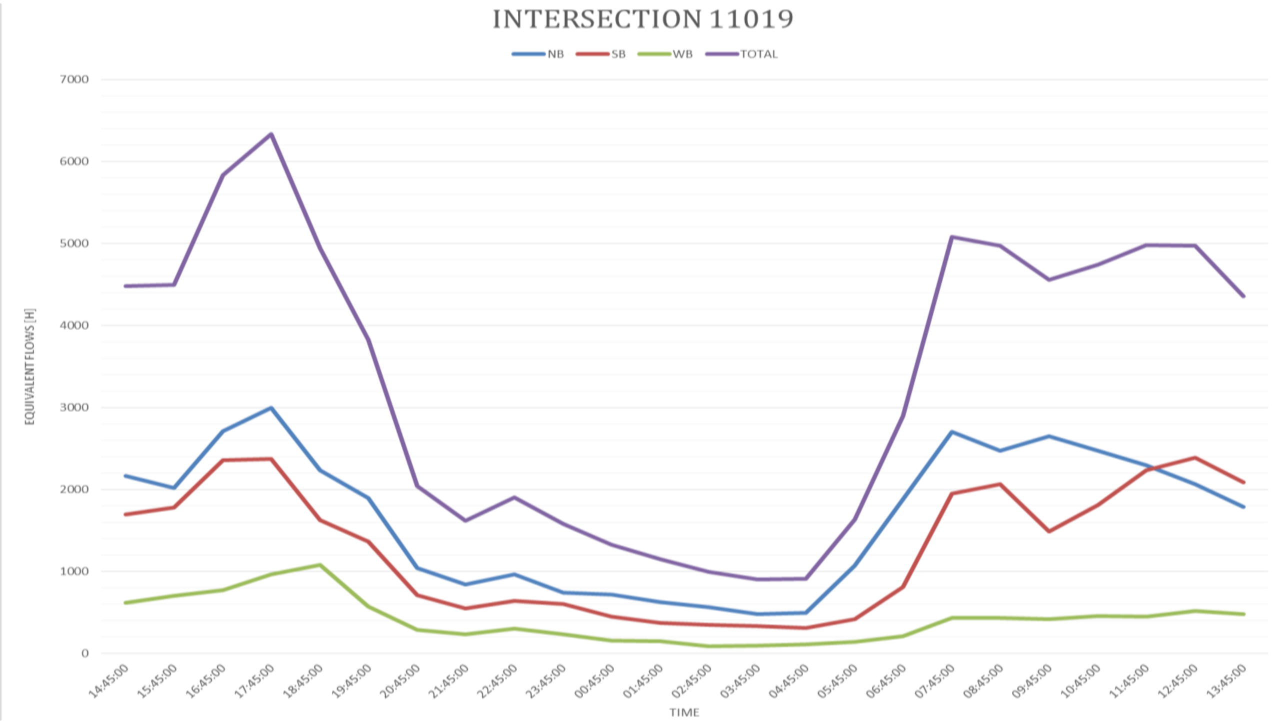


Figure 22: Vehicular flow [h] at the intersection 11019

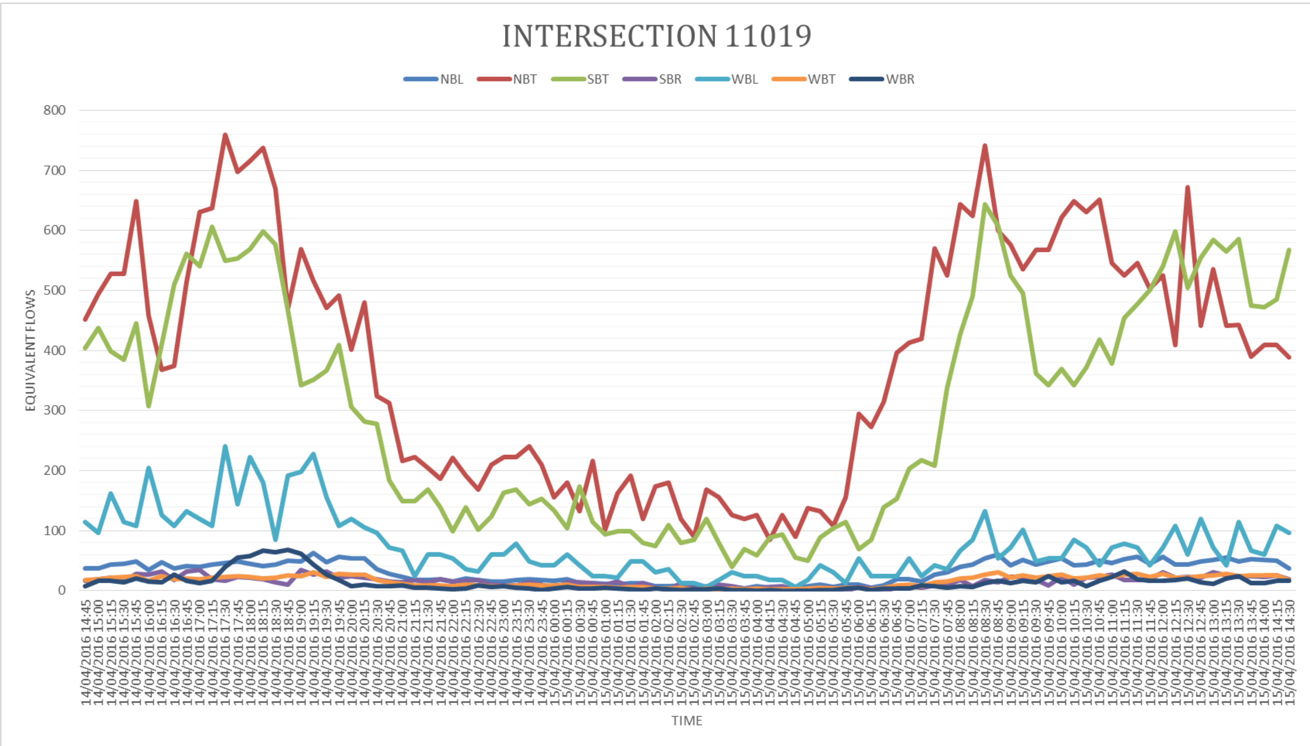


Figure 23: 15-minute's vehicular flow at the intersection 11019

The analysis of the vehicular characteristics, used to determine the total and partial equivalent volumes along the approaches, In addition highlights a flow distribution on vehicle classes, divided as follows: 73% cars, heavy vehicles 13%, 14% motorcycles.

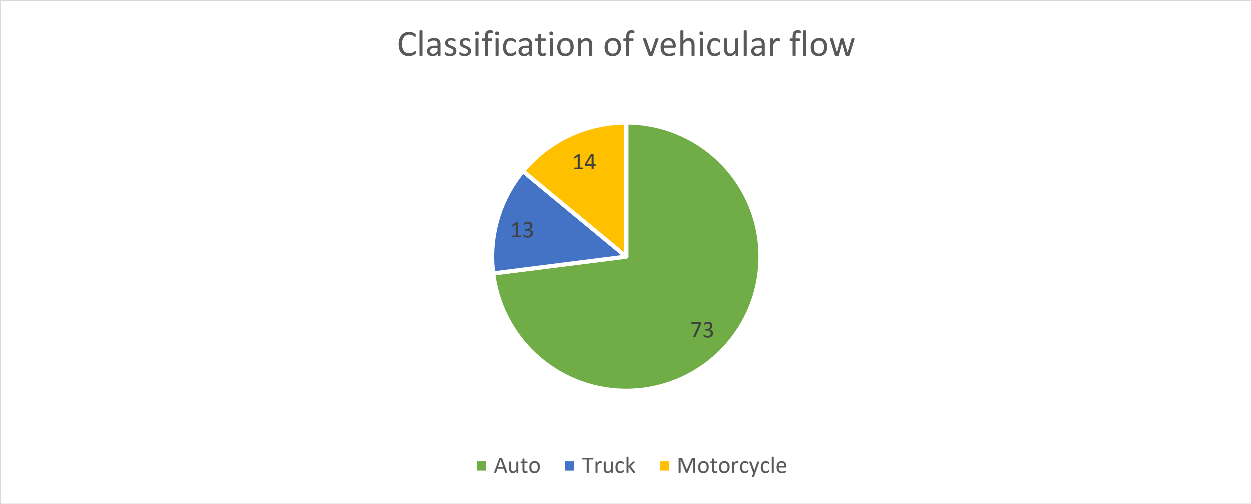


Figure 24: Classification of flow distribution on vehicle classes

Through the data collected by the sensors, it is possible to have the necessary information to differentiate the intersection vehicular flow along various approaches, and have the helpful information to optimize traffic lights.

Approaches at the intersection, are shown below:

- Approach of Bartolotti Street
- Approach of Marconi Street, direction to the Centre (northbound)
- Approach of Marconi Street, GRA direction (southbound)

As shown in the figure, the overall traffic volume is distributed along the approaches predominantly along Marconi Street. Bartolotti Street does not generate a significant contribution of traffic volume at the intersection.

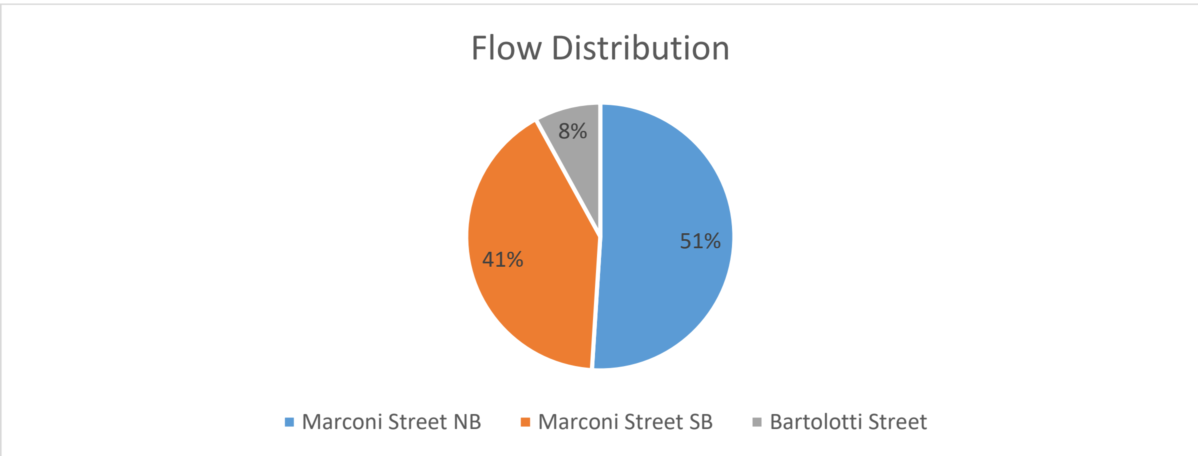


Figure 25: Approach flow distribution

Moreover, subsequent figures represent the flows disaggregated by approach, which highlights:

- Approach of Marconi Street (northbound), the trend of flows is very variable during the day, fluctuating between high (3000 veh/h) and low values (1500 veh/h). During the night, the traffic flow turns out to be poor, with volumes slightly below 1000 units / hour.
- Approach of Marconi Street (south bound), the trend of flows is very variable during the day, fluctuating between high (2500 veh/h) and low values (1400 veh/h). The number of vehicles during the night decreases progressively up to reach 400 vehicles / hour around 4:45 am;
- Approach of Bartolotti Street presents very low flow values oscillating around 1000 veh/h during morning peak hour. The number of vehicles during night hours is less than 100.

Subsequent figures show the diagrams for single approach with the representation of the different manoeuvres relating to each of them.

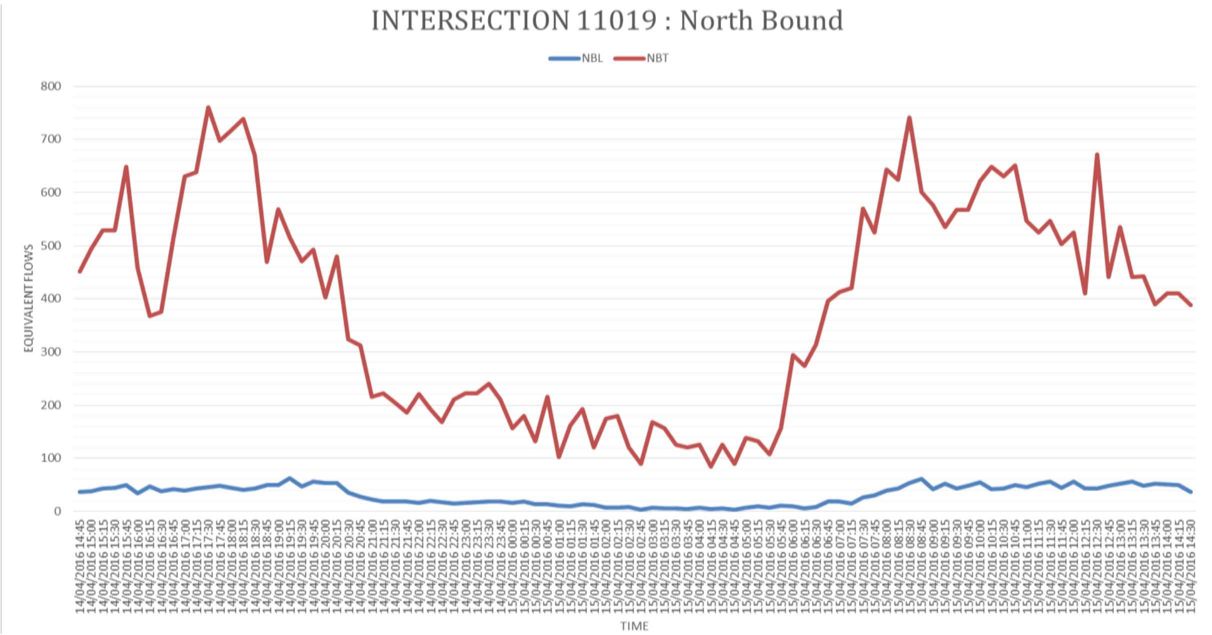


Figure 26: North Bound approach flow distribution. Intersection 11019

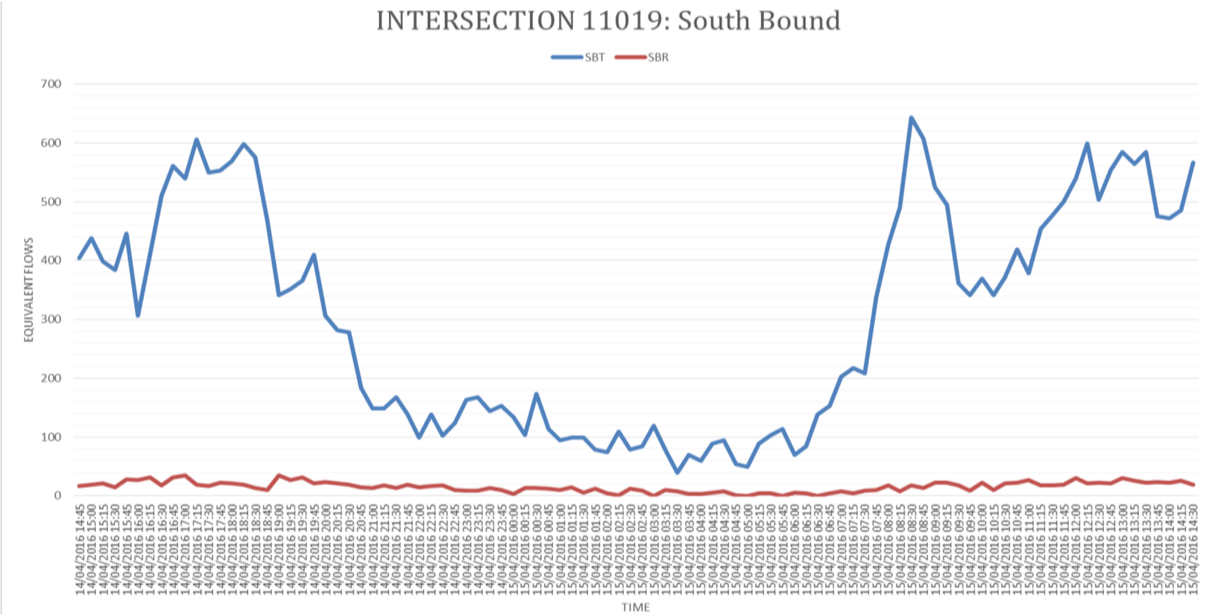


Figure 27: South Bound approach flow distribution. Intersection 11019

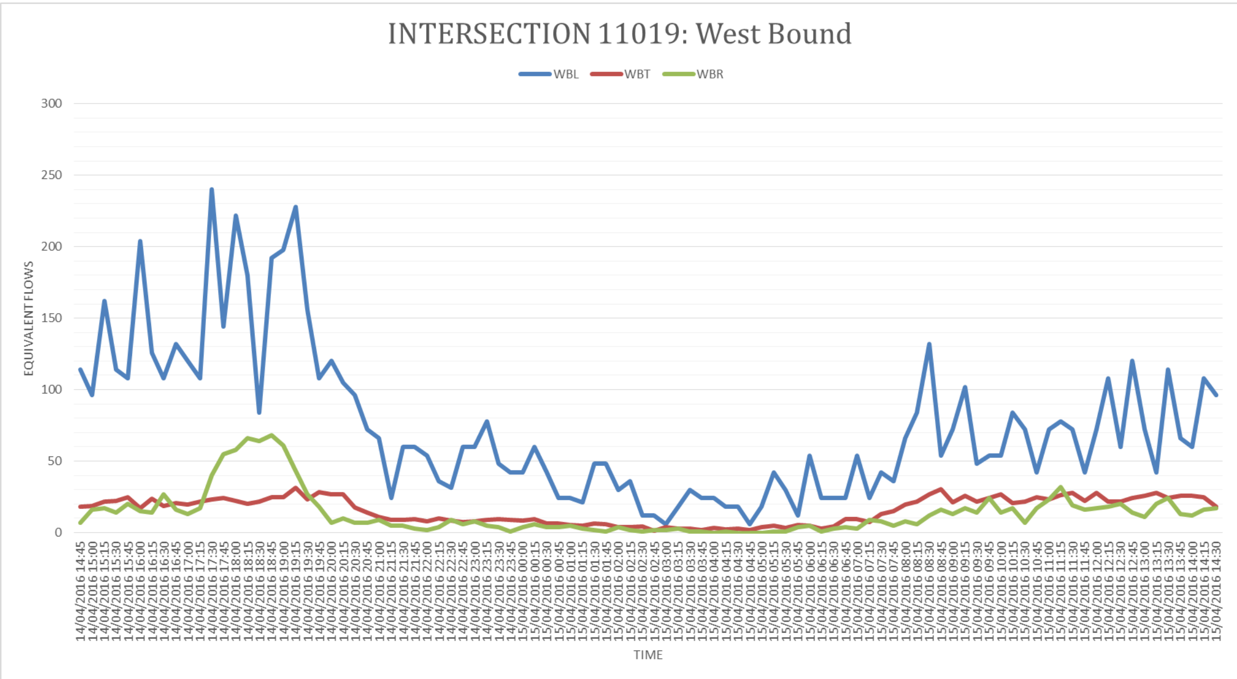


Figure 28: West Bound approach flow distribution. Intersection 11019

5.2.2. CURRENT TIMING DIAGRAM

The timing diagram of the current state, reported below, shows the following characteristics:

- All manoeuvres, presented in figure below, are subject to traffic signal control;
- The system works with two traffic signal phases:

- Phase 1: the manoeuvres along Marconi Street in two opposite directions (SB and NB) and the turning manoeuvre toward Segre Street.
- Phase 2: the manoeuvres from Bartolotti Street and the manoeuvre of crossing along the main axis at the intersection 11056.

- The phases are further divided into two sub-phases, to enable the correct outflow of the vehicles between the two intersections, taking also into account the travel time between junctions.
- The cycle length is variable along 5 different plans based on time of day;
- Two neighbouring intersections 11019 and 11056 operate with the same phasing: 11056 and 11019;
- The assigned green times are presented in the following timing diagram:

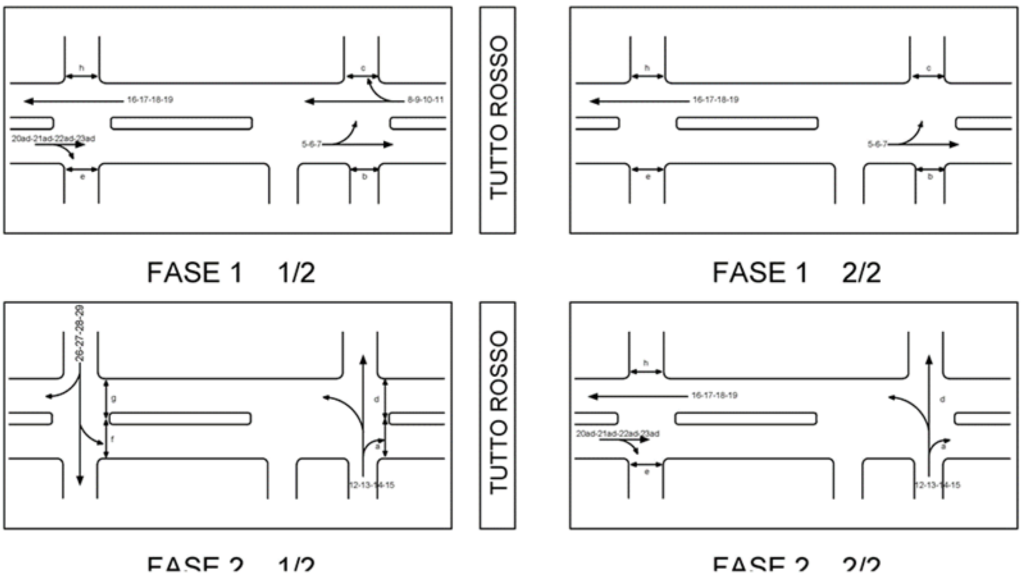


Figure 29: Scheme of movements through intersections 11019/11056

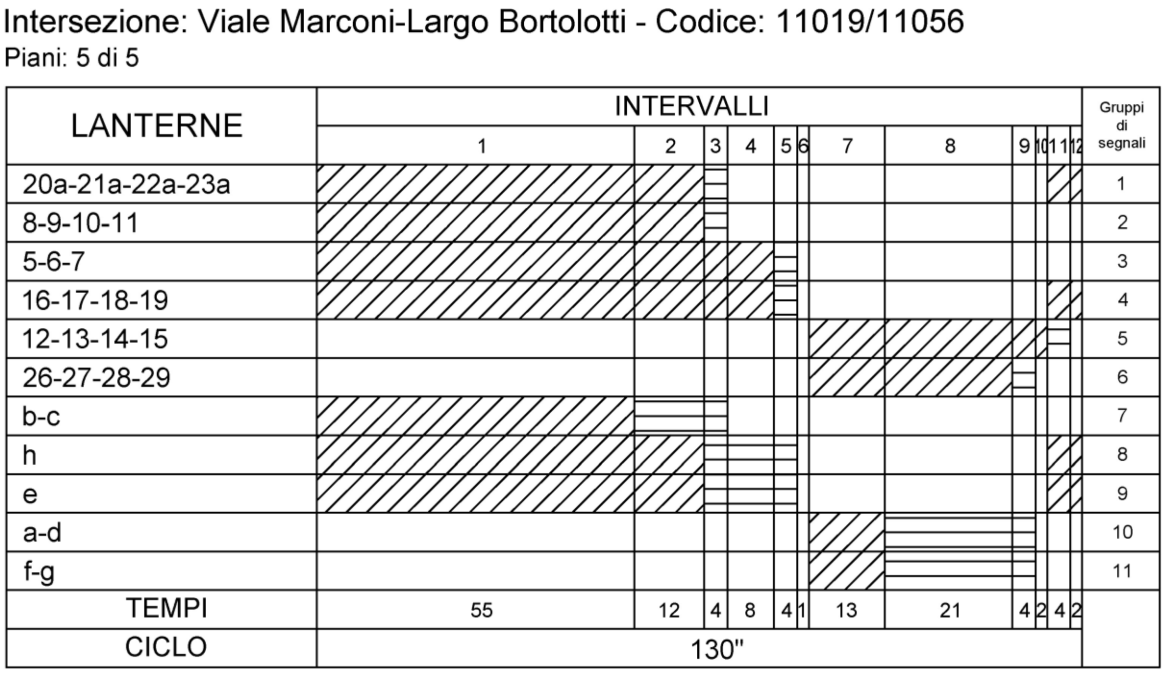


Figure 30: Current timing diagram of the intersections 11019/11056

5.3. INTERSECTION 11013: MARCONI STREET – PINCHERLE STREET - VALCO SAN PAULO STREET

The intersection Marconi Street- Pincherle Street and Valco San Paulo Street is located in a strictly urban context and is characterized by main flow between EUR district and the city center and transversal flow of the residential areas and flow directed to Ostiense/Sao Paulo districts.

The pictures below present the intersection in the question.

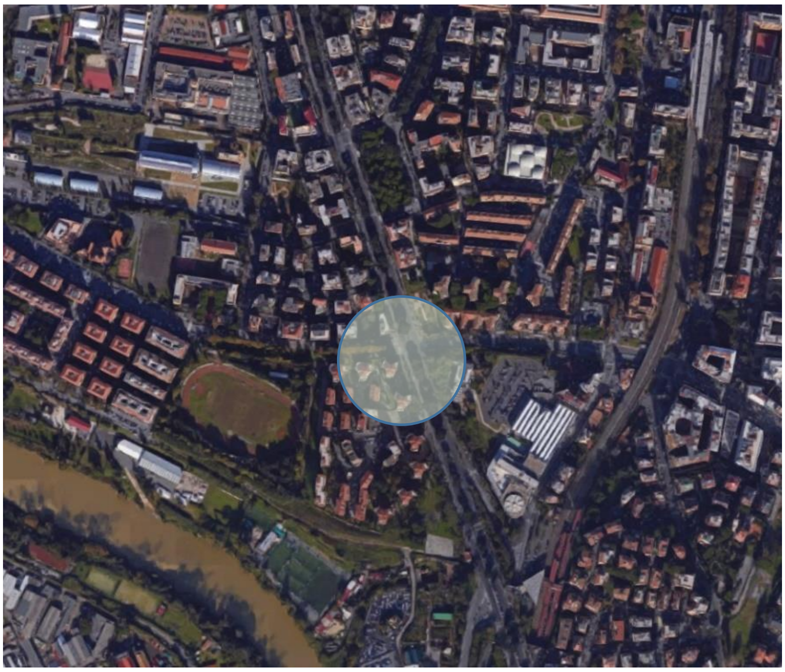


Figure 31: Intersection 11013

Figure 32: View of Intersection 11013

The following plan shows the functional organization of the signalized node at issue:

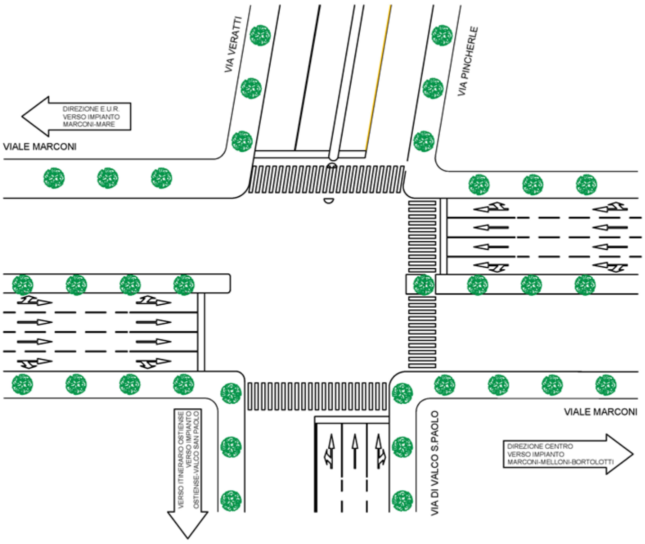


Figure 33: Functional organisation of intersection 11013

5.3.1. TRAFFIC FLOW ANALYSIS

The figure below highlights images captured by the sensor SmartEye, which indicate the type of installation and the sensor's ability to determine, through the definition of virtual targets, the traffic flow along the intersection.

Figure 34: Image of the node 11013 captured by the sensor SmartEye

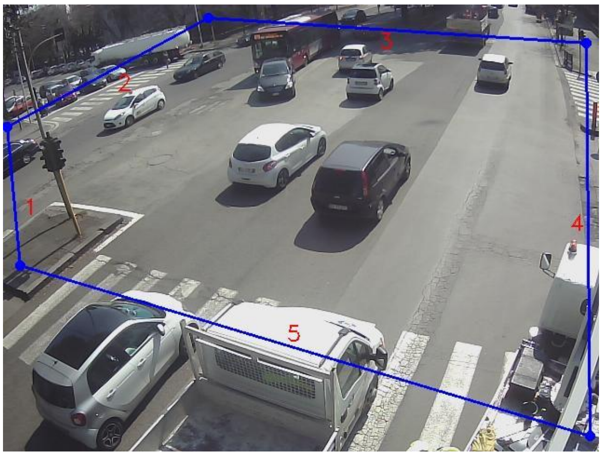


Figure 35: Virtual target of sensor

At the intersection under examination, given the wide road surface and the inability to install the sensors at the appropriate heights, two SmartEye sensors have been installed, in order to cover the entire study area. As shown in the figure below, the node is affected by a traffic flow that has medium-high values, with two maximum peaks in the morning and afternoon hours in correspondence of the opening and closing of the offices. Also during non-peak daily hours, the intersection is characterized by medium volume of the vehicular traffic. Traffic stream is decreasing significantly during a night.

The figure below illustrates a trend of vehicular flows on the intersection in the exam. During morning peak hour the value of vehicle at the junction arrive up to 6000 [veh/h], while during afternoon peak is increasing up to 7200 [veh/h]. At the night traffic stream oscillates between 2000 and 1000 [veh/h].

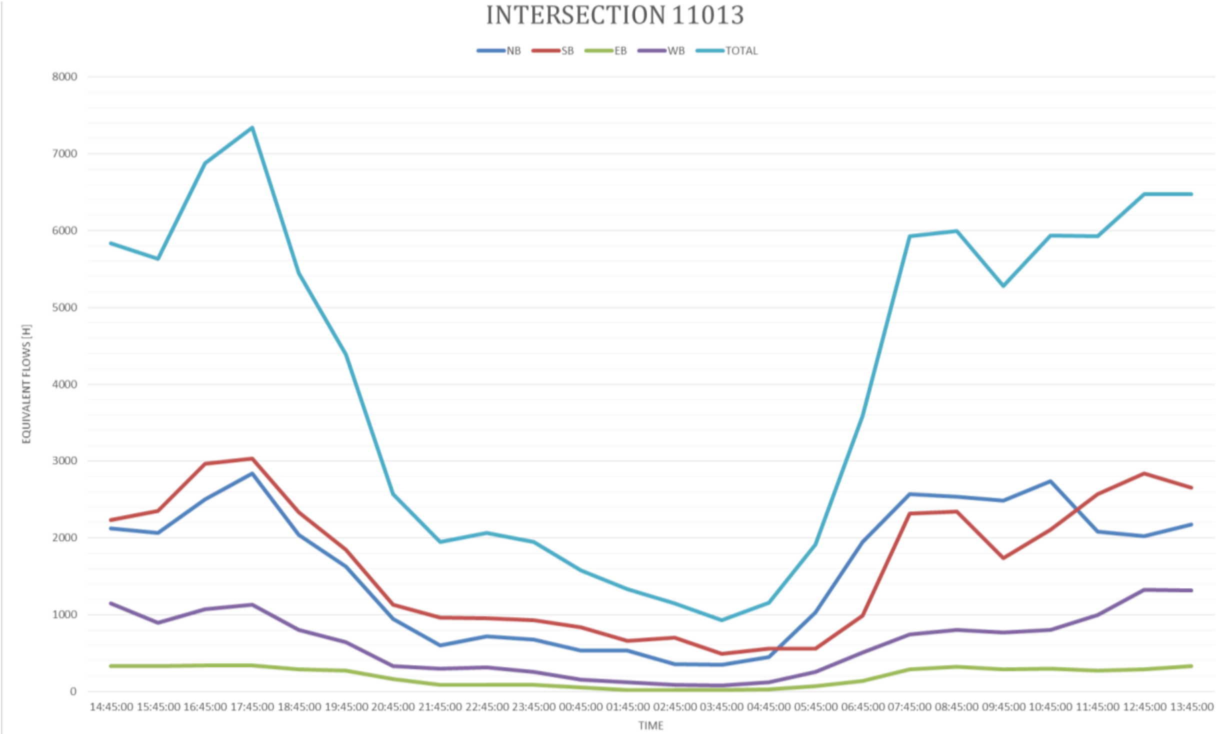


Figure 36: Vehicular flow [h] at the intersection 11013

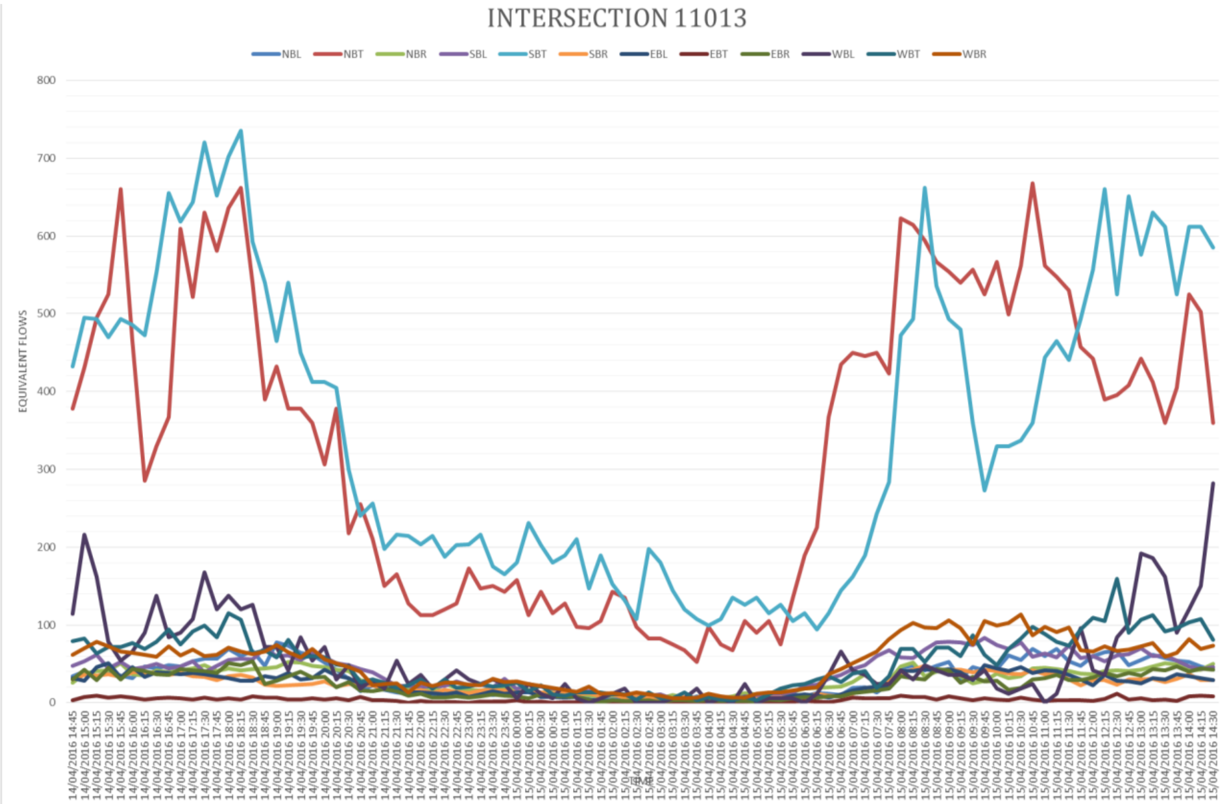


Figure 37: 15-minute vehicular flow at the intersection 11013

The analysis of the vehicular characteristics, used to determine the total and partial equivalent volumes along the approaches, In addition highlights a flow distribution on vehicle classes, divided as follows: 77% cars, heavy vehicles 13%, 10% motorcycles.

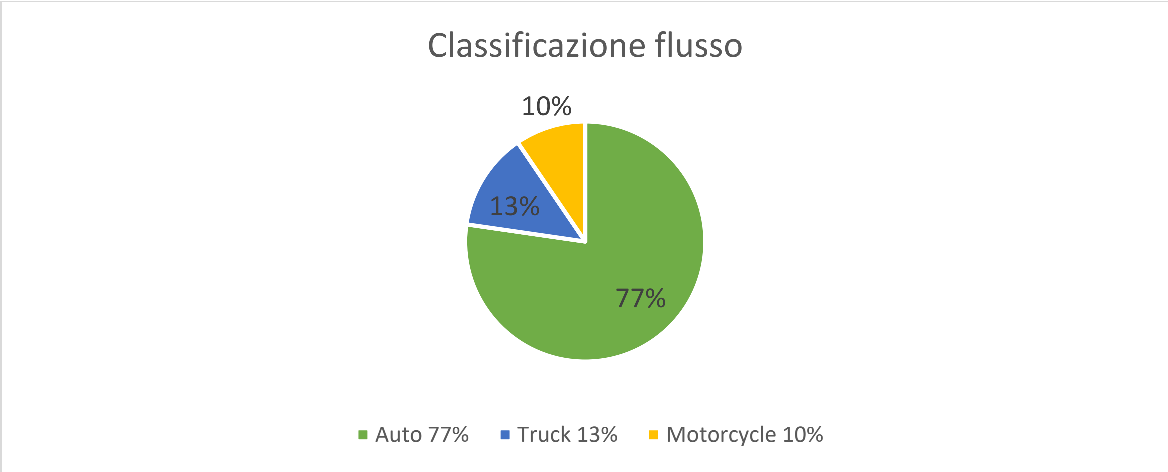


Figure 38: Classification of flow distribution on vehicle classes. Intersection 11013

Through the data collected by the sensors, it is possible to have the necessary information to differentiate the intersection vehicular flow along various approaches, and have the helpful information to optimize traffic lights. Approaches at the intersection, are shown below:

1. Approach of Valco S. Paulo Street;
2. Approach of Pincherle Street;
3. Approach of Marconi Street, direction to the Center (NB);
4. Approach of Marconi Street, GRA direction (SB)

As shown in the figure, the overall traffic volume is distributed along the approaches predominantly along Marconi Street. Valco S. Paulo Street generate 22% of the total flow, while Pincherle Street generate only 10% of traffic volume at the intersection.

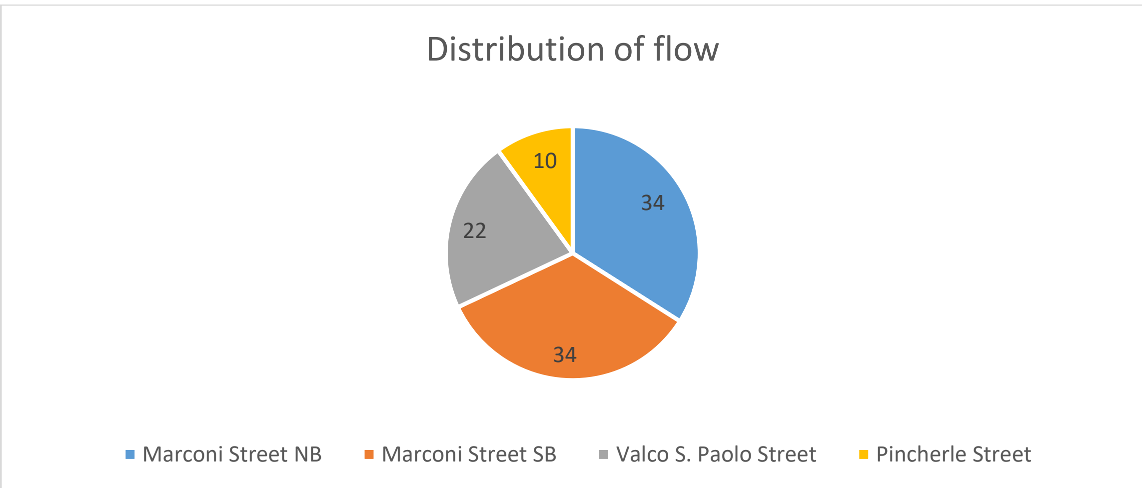


Figure 39: Approach flow distribution on intersection 11013

Moreover, subsequent figures represent the flows disaggregated by approach, which highlights:

- Approach of Marconi Street (NB), the trend of flows is very variable during the day, fluctuating between high (2800 veh/h) and low values (1500 veh/h). During the night, the traffic flow turns out to be poor, with volumes slightly below 1000 veh/h.
- Approach of Marconi Street (SB), the trend of flows is very variable during the day, fluctuating between high (3000 veh/h) and low values (1300 veh/h). The number of vehicles during the night decreases progressively up to 600 veh/h;
- Approach of Pincherle Street presents very low flow values oscillating around 300 veh/h during morning peak hour. The number of vehicles during night hours is less than 100;
- Approach of Valco San Paolo Street, being a connecting point with the San Paolo and Ostiense district, has medium size of traffic volume. The trend shows the most dense traffic flow during the afternoon hours, mainly targeted towards the areas of the EUR district.

Subsequent figures show the diagrams for single approach with the representation of the different manoeuvres relating to each of them.

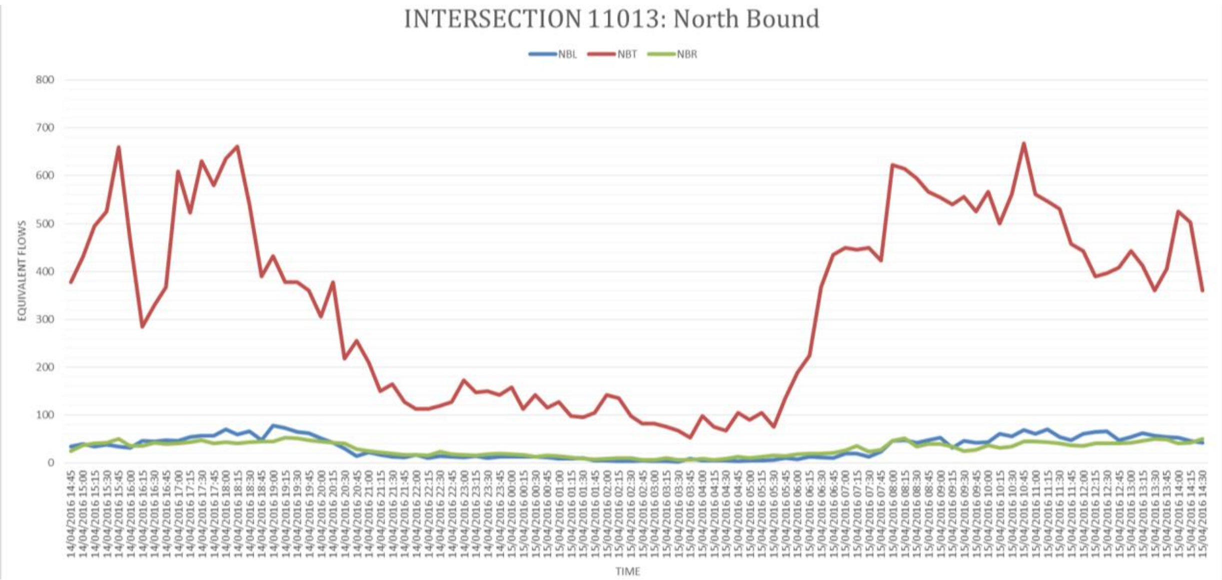


Figure 40: North Bound approach flow distribution. Intersection 11013

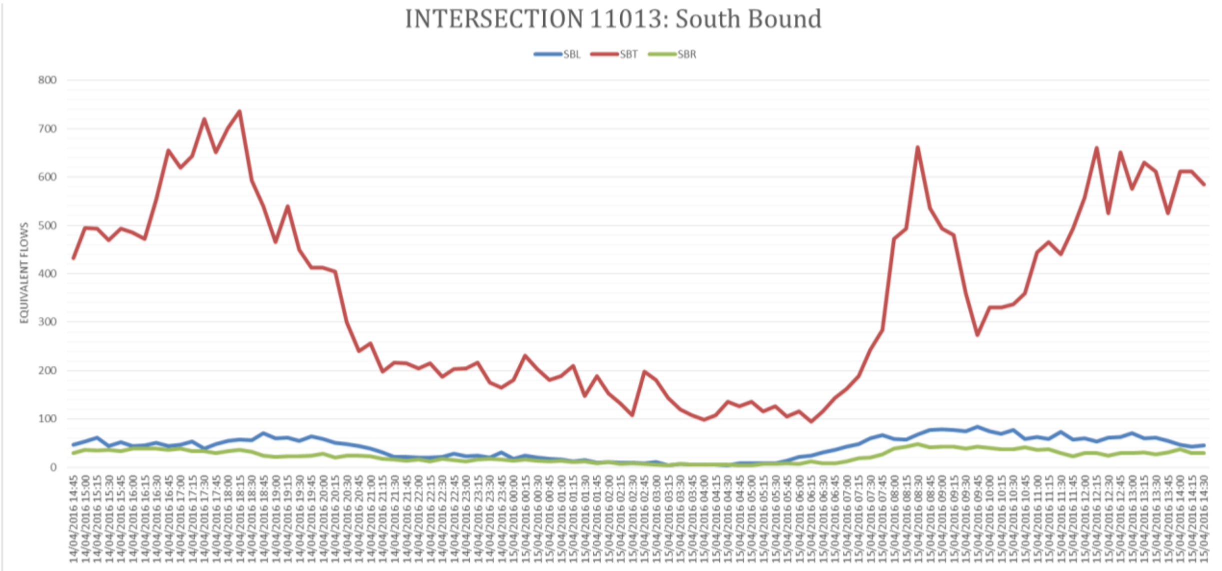


Figure 41: South Bound approach flow distribution. Intersection 11013

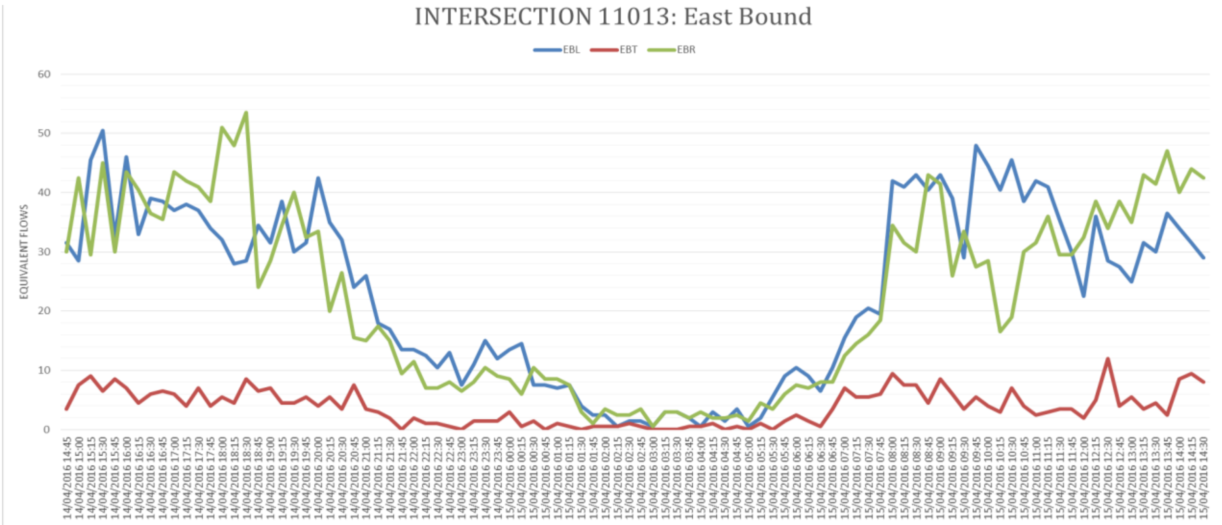


Figure 42: East Bound approach flow distribution. Intersection 11013

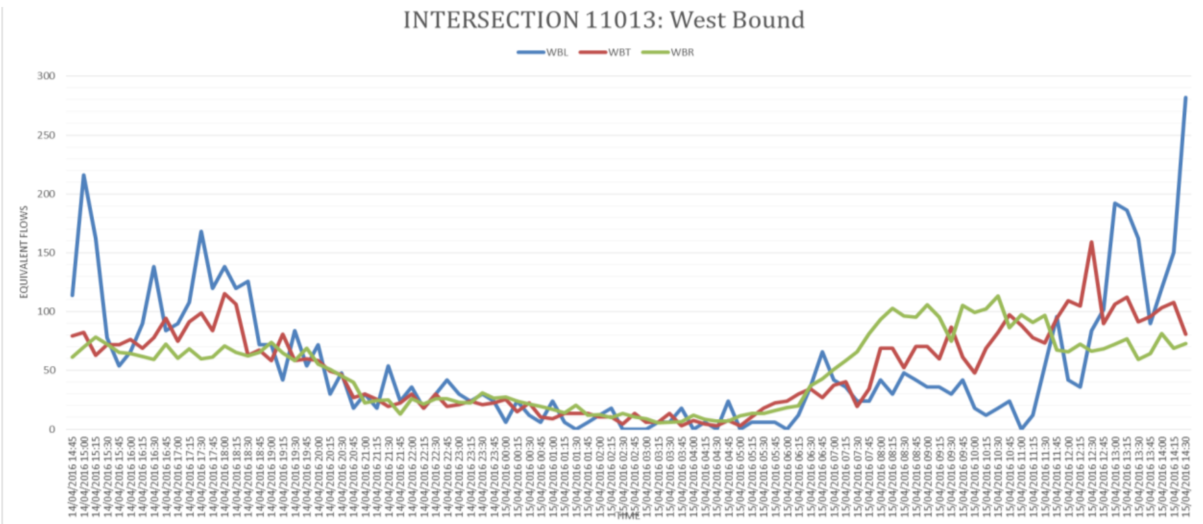


Figure 43: West Bound approach flow distribution. Intersection 11013

5.3.2. CURRENT TIMING DIAGRAM

The timing diagram of the current state, reported below, shows the following characteristics:

- All manoeuvres, presented in figure below, are subject to traffic signal control;
- The system works with two traffic signal phases:

- Phase 1: the manoeuvres along Marconi Street in two opposite directions (SB and NB) and the turning manoeuvre toward Pincherle Street and San Paolo Street.
- Phase 2: the manoeuvres from Pincherle Street and San Paolo Street in all directions;

- The cycle length is variable along 5 different plans based on time of day, which are described subsequently;
- Group 6 regarded to pedestrian phase working only with pedestrian call;
- The assigned green times are presented in the following timing diagram:

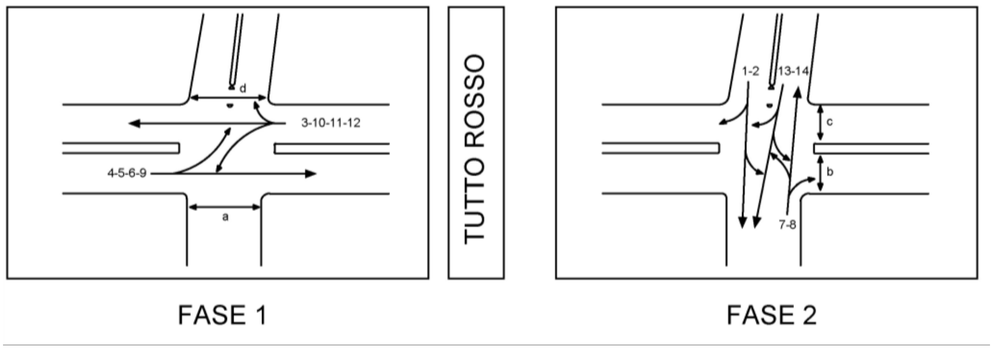


Figure 44: Scheme of movements through intersection 11013

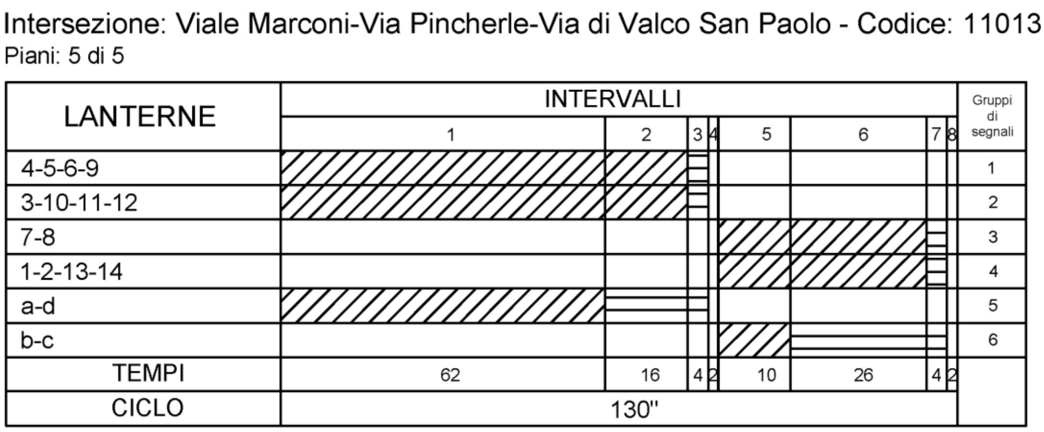


Figure 45: Current timing diagram of the intersection 11013

5.4. INTERSECTION 11022: VIALE MARCONI – VIA GIBILMANNA

The intersection between Marconi Street and Gibilmanna Street is characterized by high flows from/to EUR district and city Center on the main artery and by limited traffic flow from residential zone of Gibilmanna Street. The pictures below present the intersection in the question.

Figure 46: Intersection 11022

Figure 47: View of intersection 11022

The following plan shows the functional organization of the signalized node at issue:

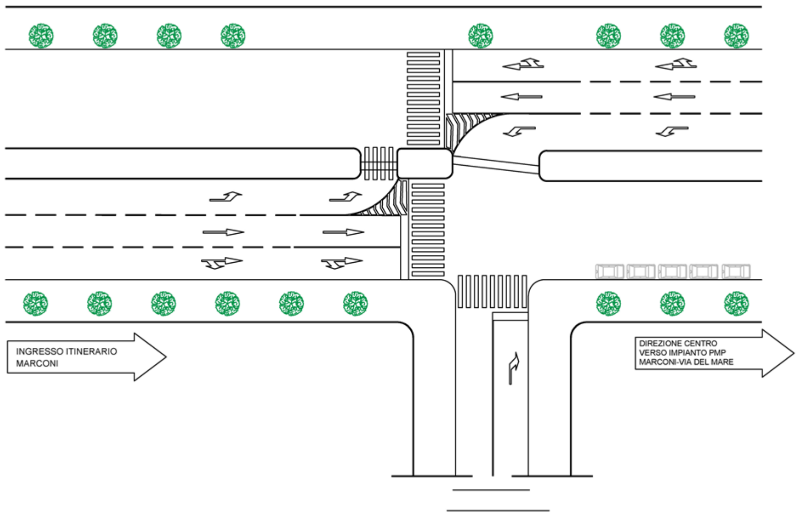
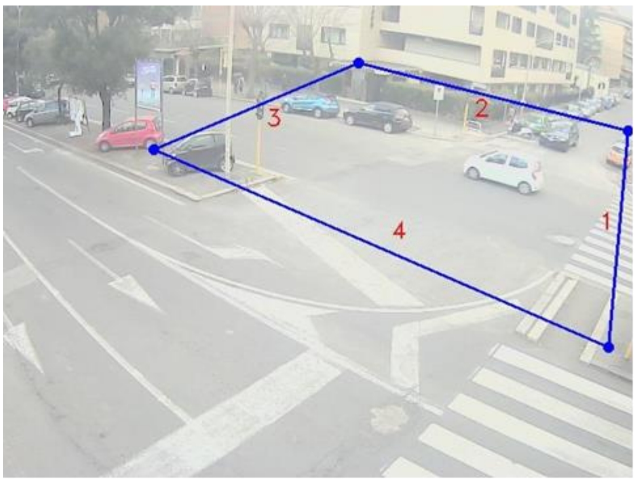


Figure 48: Functional organisation of the intersection 11022

5.4.1. TRAFFIC FLOW ANALYSIS

The figure below highlights images captured by the sensor SmartEye, which indicate the type of installation and the sensor's ability to determine, through the definition of virtual targets, the traffic flow along the intersection.



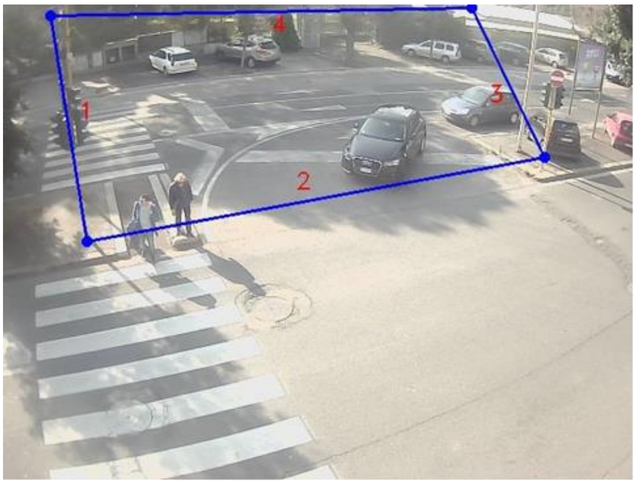


Figure 49: Image of the node 11022 captured by the sensor SmartEye　*Figure 50: Virtual target of sensor*

At the intersection under examination, given the wide road surface and the inability to install the sensors at the appropriate heights, two SmartEye sensors have been installed, in order to cover the entire study area.

As shown in the figure below, the node is affected by a traffic flow that has medium-high values, with two maximum peaks in the morning and afternoon hours in correspondence of the opening and closing of the offices. Also during non-peak daily hours, the intersection is characterized by medium volume of the vehicular traffic. Traffic stream is decreasing significantly during a night. The figure below illustrates a trend of vehicular flows on the intersection in the exam. During morning peak hour the value of vehicle at the junction arrive up to 5000 [veh/h], while during afternoon peak is increasing up to 6000 [veh/h]. At the night traffic stream oscillates near 800-2000 [veh/h].

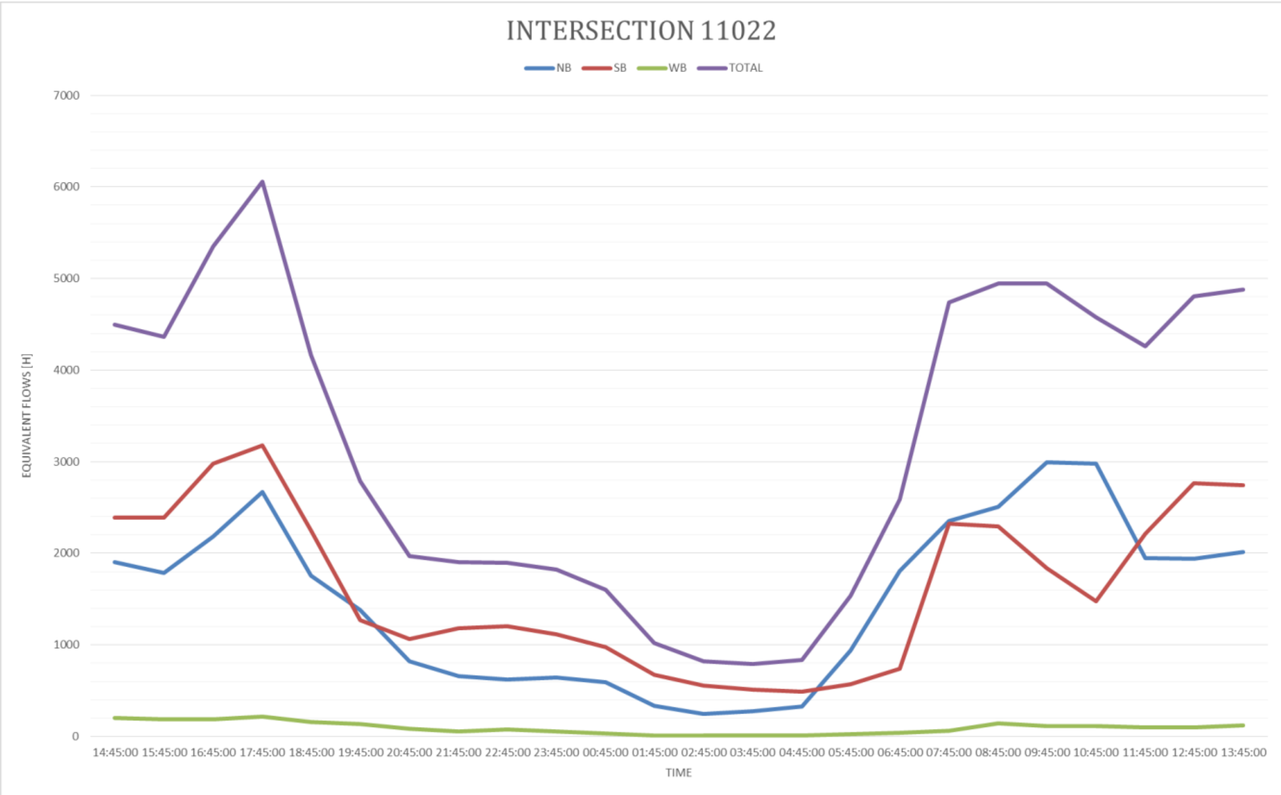


Figure 51: Vehicular flow [h] at the intersection 11022

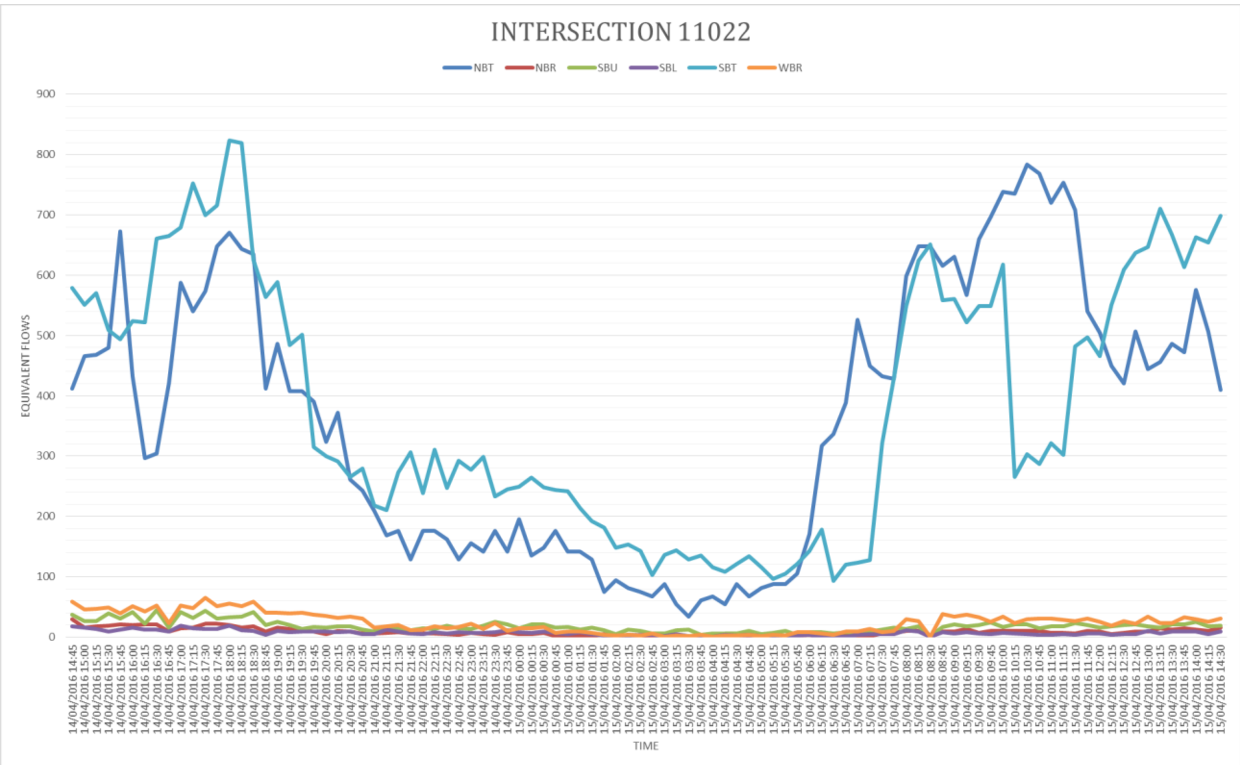


Figure 52: 15-minutes vehicular flow at the intersection 11022

The analysis of the vehicular characteristics, used to determine the total and partial equivalent volumes along the approaches, in addition highlights a flow distribution on vehicle classes, divided as follows: 69% cars, heavy vehicles 18%, 13% motorcycles.

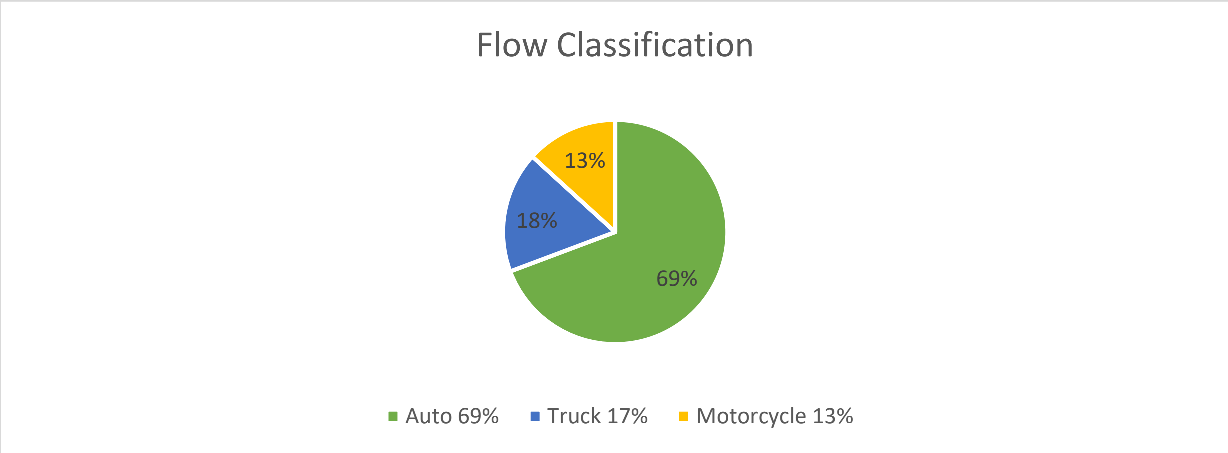


Figure 53: Classification of flow distribution on vehicle classes. Intersection 11022

Through the data collected by the sensors, it is possible to have the necessary information to differentiate the intersection vehicular flow along various approaches, and have the helpful information to optimize traffic lights.

Approaches at the intersection, are shown below:

1. Approach of Gibilmanna Street
2. Approach of Marconi Street, direction to the Center (NB)
3. Approach of Marconi Street, GRA direction (SB)

As shown in the figure, the overall traffic volume is distributed along the approaches predominantly along Marconi Street. Gibilmanna Street generate very low traffic volume at the intersection.

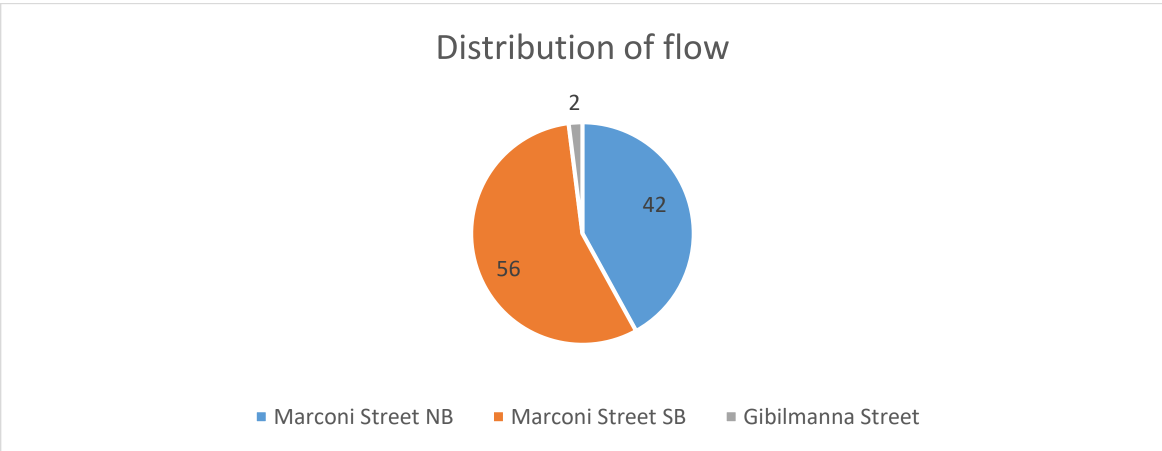


Figure 54: Approach flow distribution on the intersection 11022

Moreover, subsequent figures represent the flows disaggregated by approach, which highlights:

- Approach of Marconi Street (NB), the trend of flows is very variable during the day, fluctuating between high (3000 veh/h) and low values (1500 veh/h). During the night, the traffic flow turns out to be poor, with volumes slightly below 1000 units / hour.
- Approach of Marconi Street (SB), the trend of flows is very variable during the day, fluctuating between high (3200 veh/h) and low values (1400 veh/h). The number of vehicles during the night decreases progressively up to reach 600 veh/h around 4:45 am;
- Approach of Gibilmanna Street presents very low flow values oscillating around 200 veh/h during peak hours. The number of vehicles during night hours is near zero.

Subsequent figures show the diagrams for single approach with the representation of the different manoeuvres relating to each of them.

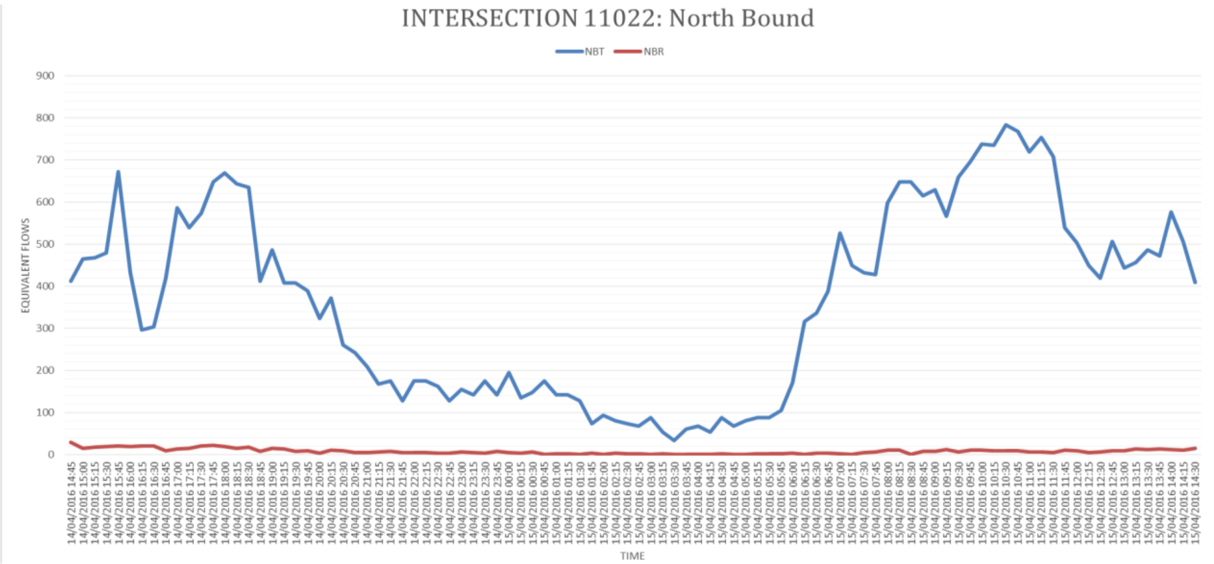


Figure 55: North Bound approach flow distribution. Intersection 11022

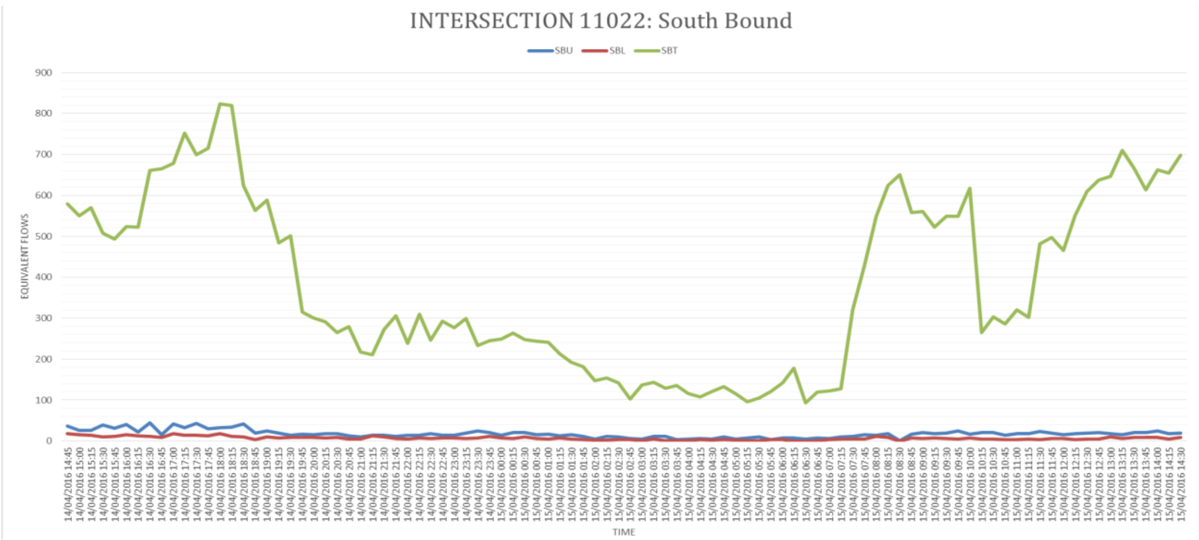


Figure 56: South Bound approach flow distribution. Intersection 11022

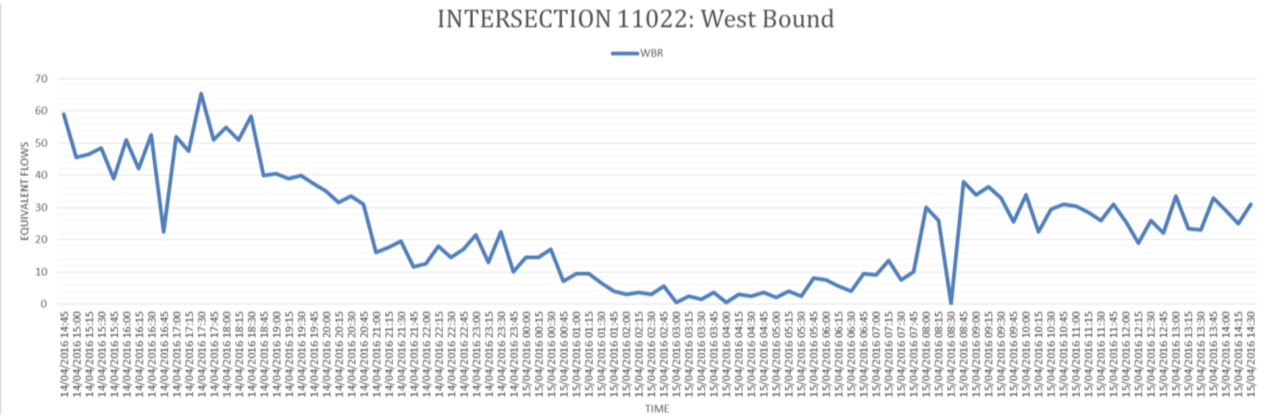


Figure 57: West Bound approach flow distribution. Intersection 11022

5.4.2. CURRENT TIMING DIAGRAM

The timing diagram of the current state, reported below, shows the following characteristics:

- All manoeuvres, presented in figure below, are subject to traffic signal control;
- The system works with two traffic signal phases:
- Phase 1: the manoeuvres along Marconi Street in two opposite directions (SB and NB) and the turning manoeuvre toward Gibilmanna Street from south approach of Marconi Street;
- Phase 2: the manoeuvres from Gibilmanna Street and the left/U turning movements from Marconi Street;
- The cycle length is variable along 5 different plans based on time of day, which are described previously;
- Pedestrian phase of group 7 and 8 from timing diagram is working only after pedestrian call;
- The assigned green times are presented in the following timing diagram:

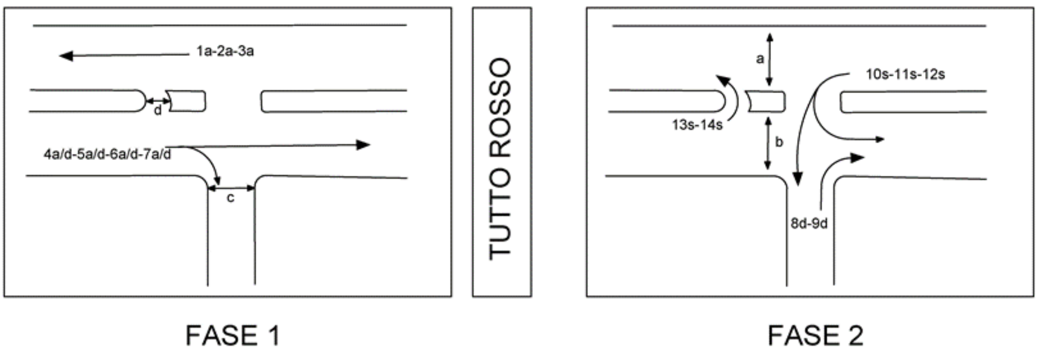


Figure 58: Scheme of movements through intersection 11022

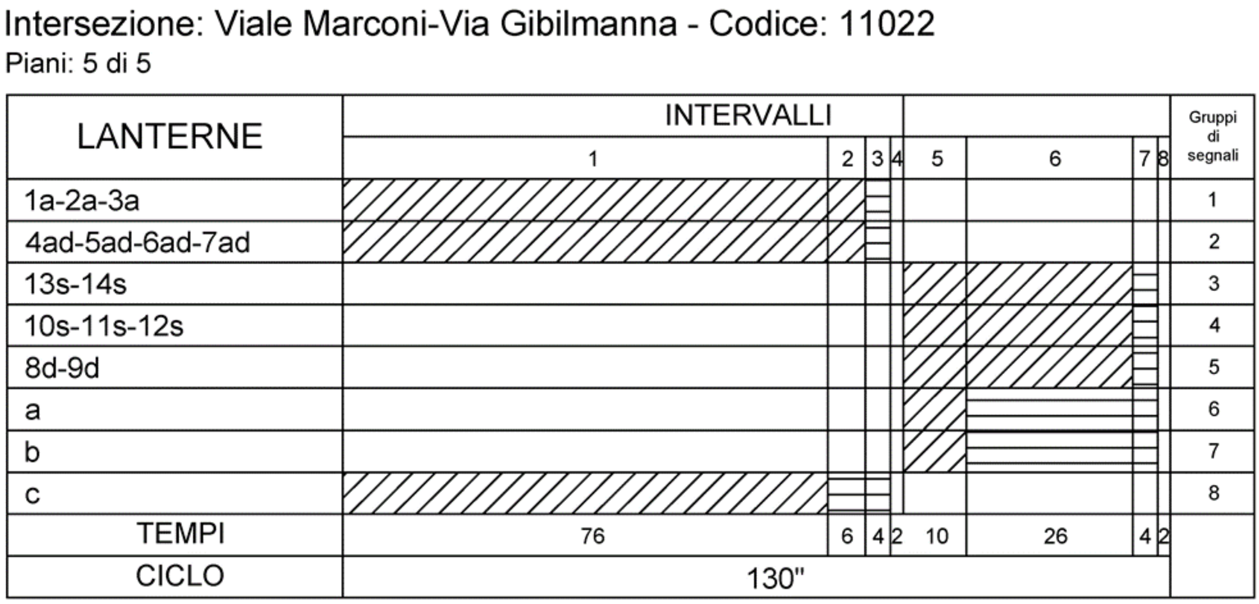


Figure 59: Current timing diagram of the intersection 11022

Chapter 6

CLUSTER ANALYSIS

6.1.INTRODUCTION

The scope of this chapter was the determination of existing demand states. The accuracy of TRPS mode depends on efficient clustering of existing traffic demands. A single timing plan was associated to each traffic demand state.

The cluster analysis group together the demand patterns having similar attributes in order to apply suitable signal timing plan. A dimensional vector of movement volumes represent the demand along the network.

The human perception system used for classification of the data are accurate and effective for grouping to three-dimensional space. However, in reality the networks has many approaches, therefore many dimensions that is why automated algorithm need to be used for accuracy.

Cluster analysis has essentially three steps:

- Identification of feature vectors;
- Normalization of feature vectors;
- Clustering of feature vectors with common attributes.

In this chapter, the details of the proposed approach to determine the traffic levels of Marconi Street are presented.

6.2. DATA COLLECTION

Common volume states on all movements of each approach was clustered together and they are to associate a signal timing plan to each cluster. 15 minutes volume measurements was chosen as a feature vector. Collected field data represent all demand variations existing in the field. Data should be collected during a normal day, a weekend and any special event or anomalous traffic conditions.

For the purpose of this thesis, the traffic volumes were measured in a normal weekday on 14th and 15th of April 2016 by video detection technique with subsequent manual verification in the field. The tables below show 15-minutes traffic volumes of all movements at every intersection. At the intermediate intersections such as 11056, 11022 and 12044 the traffic flow measurements were not performed, but for the goal of the thesis were established in order to equilibrate the different values of the flows between the intersections where traffic flow was detected.

Table 2: 15-minutes traffic volumes. Part 1

Actual timing plan	Clustered timing plan	Proposed timing plan	Time	11019							11056					
				NBL	NBT	SBT	SBR	WBL	WBT	WBR	NBT	NBR	SBT	EBL	EBT	EBR
2	5	6	14/04/2016 14:45	36	452	404	17	114	18	7	454	17	518	0	18	9
3	6	6	14/04/2016 15:00	38	494	438	20	96	19	16	527	2	534	0	19	50
3	6	6	14/04/2016 15:15	43	528	399	21	162	22	17	619	0	561	48	22	29
3	5	5	14/04/2016 15:30	44	528	384	15	114	22	14	648	0	498	76	22	53
3	3	5	14/04/2016 15:45	49	648	446	29	108	25	20	758	0	554	61	25	26
3	5	5	14/04/2016 16:00	34	458	307	27	204	17	15	576	0	511	84	17	58
3	5	5	14/04/2016 16:15	47	368	411	32	126	24	14	345	35	537	0	24	21
3	6	5	14/04/2016 16:30	38	375	510	18	108	19	27	429	0	618	16	19	24
3	5	5	14/04/2016 16:45	41	513	561	32	132	21	16	403	76	693	0	21	43
3	1	1	14/04/2016 17:00	39	630	540	35	120	20	13	707	0	660	37	20	44
3	1	1	14/04/2016 17:15	43	638	606	20	108	22	17	576	52	714	0	22	17
3	1	1	14/04/2016 17:30	46	760	549	17	240	23	40	648	79	789	0	23	3
3	1	1	14/04/2016 17:45	48	698	554	23	144	24	55	606	70	698	0	24	31
3	1	1	14/04/2016 18:00	45	716	569	21	222	22	58	717	22	791	0	22	1
3	1	1	14/04/2016 18:15	41	738	599	20	180	20	66	731	24	779	0	20	50
3	1	1	14/04/2016 18:30	44	670	576	14	84	22	64	549	82	660	0	22	21
3	5	5	14/04/2016 18:45	50	470	467	11	192	25	68	461	29	659	0	25	25
3	5	5	14/04/2016 19:00	49	569	342	35	198	25	61	457	80	540	0	25	8
3	5	5	14/04/2016 19:15	62	516	351	27	228	31	43	385	97	579	0	31	44
3	5	5	14/04/2016 19:30	47	471	366	32	156	23	27	415	51	522	0	23	6
3	5	5	14/04/2016 19:45	56	492	410	21	108	28	18	373	88	518	0	28	16
3	2	2	14/04/2016 20:00	53	402	306	24	120	27	7	353	51	426	0	27	72
3	2	2	14/04/2016 20:15	54	480	282	21	105	27	10	394	70	387	0	27	90
3	2	2	14/04/2016 20:30	35	324	278	20	96	18	7	231	64	374	0	18	2
3	2	2	14/04/2016 20:45	28	312	183	15	72	14	7	298	21	255	0	14	52
4	4	4	14/04/2016 21:00	22	216	149	14	66	11	9	259	0	215	20	11	103
4	4	4	14/04/2016 21:15	18	222	149	18	24	9	5	145	48	173	0	9	75
4	4	4	14/04/2016 21:30	18	204	168	14	60	9	5	192	15	228	0	9	27
4	4	4	14/04/2016 21:45	18	186	139	20	60	9	3	104	50	199	0	9	50
4	4	4	14/04/2016 22:00	16	221	99	15	54	8	2	68	84	153	0	8	88
4	4	4	14/04/2016 22:15	20	192	139	17	36	10	4	81	66	175	0	10	73
4	4	4	14/04/2016 22:30	17	168	102	18	32	9	9	128	29	134	0	9	93
4	4	4	14/04/2016 22:45	15	210	124	11	60	8	6	108	59	184	0	8	61
4	4	4	14/04/2016 23:00	16	222	163	9	60	8	8	168	35	223	0	8	16
4	4	4	14/04/2016 23:15	18	222	168	9	78	9	5	121	59	246	0	9	9
4	4	4	14/04/2016 23:30	19	240	144	14	48	9	4	133	63	192	0	9	21
4	4	4	14/04/2016 23:45	18	210	153	11	42	9	1	134	47	195	0	9	16
4	4	4	15/04/2016 00:00	16	156	134	3	42	8	4	199	0	176	26	8	35
4	4	4	15/04/2016 00:15	19	180	104	14	60	9	6	103	48	164	0	9	107
4	4	4	15/04/2016 00:30	13	132	173	14	42	6	4	172	0	215	27	6	21
4	4	4	15/04/2016 00:45	13	216	114	12	24	6	4	56	86	138	0	6	73
5	4	4	15/04/2016 01:00	11	102	94	11	24	6	5	152	0	118	38	6	101
5	4	4	15/04/2016 01:15	10	162	99	15	21	5	3	66	53	120	0	5	112
5	4	4	15/04/2016 01:30	13	192	99	6	48	6	2	36	84	147	0	6	27
5	4	4	15/04/2016 01:45	12	120	79	12	48	6	1	107	13	127	0	6	80
5	4	4	15/04/2016 02:00	8	174	74	5	30	4	4	133	24	104	0	4	71
5	4	4	15/04/2016 02:15	8	180	109	2	36	4	2	103	42	145	0	4	3
5	4	4	15/04/2016 02:30	8	120	79	12	12	4	1	97	16	91	0	4	35
5	4	4	15/04/2016 02:45	3	90	84	9	12	1	2	95	0	96	2	1	118
5	4	4	15/04/2016 03:00	7	168	119	0	6	4	2	9	83	125	0	4	71
5	4	4	15/04/2016 03:15	5	156	79	11	18	3	3	6	78	97	0	3	56
5	4	4	15/04/2016 03:30	6	126	40	8	30	3	1	21	56	70	0	3	64
5	4	4	15/04/2016 03:45	4	120	69	3	24	2	1	0	63	93	0	2	25
5	4	4	15/04/2016 04:00	7	126	59	3	24	3	0	87	23	83	0	3	27
5	4	4	15/04/2016 04:15	4	84	89	6	18	2	1	85	2	107	0	2	11
5	4	4	15/04/2016 04:30	5	126	94	8	18	3	0	21	55	112	0	3	32
5	4	4	15/04/2016 04:45	3	90	54	2	6	2	0	116	0	60	22	2	79
5	4	4	15/04/2016 05:00	7	138	50	0	18	4	0	59	43	68	0	4	80
5	4	4	15/04/2016 05:15	10	132	89	5	42	5	1	99	22	131	0	5	0
5	4	4	15/04/2016 05:30	6	108	104	5	30	3	1	74	20	134	0	3	7
5	4	4	15/04/2016 05:45	10	156	114	0	12	5	4	154	6	126	0	5	1
1	4	4	15/04/2016 06:00	10	294	69	6	54	5	5	132	86	123	0	5	21
1	4	4	15/04/2016 06:15	5	273	84	5	24	3	1	230	24	108	0	3	22
1	2	2	15/04/2016 06:30	9	314	139	0	24	4	3	411	0	163	89	4	7
1	2	2	15/04/2016 06:45	18	396	153	5	24	9	4	489	0	177	74	9	11
1	2	2	15/04/2016 07:00	19	413	203	8	54	10	3	517	0	257	85	10	40
1	2	2	15/04/2016 07:15	15	420	218	5	24	7	9	523	0	242	88	7	15
1	2	2	15/04/2016 07:30	26	570	208	9	42	13	8	477	60	250	0	13	73
1	2	2	15/04/2016 07:45	30	525	338	11	36	15	5	493	31	374	0	15	4
1	3	3	15/04/2016 08:00	40	644	428	18	66	20	8	758	0	494	75	20	76
1	3	3	15/04/2016 08:15	43	624	490	8	84	22	6	758	0	574	91	22	19
1	1	3	15/04/2016 08:30	53	741	644	18	132	27	12	673	61	776	0	27	2
1	3	3	15/04/2016 08:45	61	600	608	14	54	30	16	703	0	662	42	30	7
1	3	3	15/04/2016 09:00	42	576	525	23	72	21	13	703	0	597	85	21	17
1	3	3	15/04/2016 09:15	52	536	495	23	102	26	17	675	0	597	87	26	3
1	3	3	15/04/2016 09:30	43	567	361	18	48	22	14	661	0	409	50	22	64
1	3	3	15/04/2016 09:45	48	567	342	9	54	24	24	679	0	396	64	24	3
2	3	3	15/04/2016 10:00	54	621	369	23	54	27	14	711	0	423	36	27	22
2	3	3	15/04/2016 10:15	42	648	342	11	84	21	17	595	47	426	0	21	11
2	3	3	15/04/2016 10:30	44	630	371	21	72	22	7	721	0	443	47	22	8
2	3	3	15/04/2016 10:45	50	651	419	23	42	25	17	793	0	461	92	25	0
2	3	3	15/04/2016 11:00	46	546	378	27	72	23	23	701	0	450	109	23	94
2	3	3	15/04/2016 11:15	52	525	455	18	78	26	32	679	0	533	102	26	28
2	3	3	15/04/2016 11:30	56	546	477	18	72	28	19	663	0	549	61	28	5
2	5	5	15/04/2016 11:45	45	503	500	20	42	22	16	555	0	542	8	22	31
2	5	5	15/04/2016 12:00	56	525	540	30	72	28	17	481	50	612	0	28	34
2	5	5	15/04/2016 12:15	43	410	599	21	108	22	18	499	0	707	46	22	36
2	5	5	15/04/2016 12:30	44	672	504	23	60	22	20	266	225	564	0	22	46
2	5	5	15/04/2016 12:45	49	441	554	21	120	24	14	504	0	674	14	24	69
2	6	6	15/04/2016 13:00	52	536	584	30	72	26	11	493	47	656	0	26	20
2	6	6	15/04/2016 13:15	56	441	564	26	42	28	20	521	0	606	24	28	115
2	6	6	15/04/2016 13:30	48	443	585	23	114	24	24	409	41	699	0	24	0
2	5	6	15/04/2016 13:45	52	390	475	24	66	26	13	506	0	541	64	26	69
2	6	6	15/04/2016 14:00	51	410	473	23	60	26	12	641	0	533	180	26	163
2	6	6	15/04/2016 14:15	50	410	485	26	108	25	16	603	0	593	144	25	92
2	6	6	15/04/2016 14:30	36	389	567	20	96	18	17	462	0	663	37	18	3

Table 3: 15-minutes traffic volumes. Part 2

11013												11022		12044		11022					
NBL	NBT	NBR	SBL	SBT	SBR	EBL	EBT	EBR	WBL	WBT	WBR	NBT	SBT	NBT	SBT	NBT	NBR	SBU	SBL	SBT	WBR
34	378	26	47	432	30	32	4	30	114	80	62	438	576	507	634	412	29	37	18	579	59
40	431	37	53	495	36	29	8	43	216	83	70	507	754	537	593	466	16	26	16	551	46
34	495	41	62	494	35	46	9	30	162	63	79	570	685	541	610	468	18	26	14	570	47
39	525	42	44	470	37	51	7	45	78	72	73	606	593	568	558	480	19	40	10	509	49
35	660	51	52	494	34	33	9	30	54	72	66	745	578	742	537	672	21	31	12	494	39
32	465	36	44	486	39	46	7	44	66	77	65	533	596	524	581	432	20	41	16	524	51
46	285	35	46	473	39	33	5	41	90	69	62	366	603	361	557	297	21	22	13	522	42
44	330	42	50	553	38	39	6	37	138	78	60	416	727	401	718	304	21	45	12	661	53
48	368	40	44	655	36	39	7	36	84	95	73	455	775	458	688	420	9	15	9	665	23
46	609	41	46	619	39	37	6	44	90	75	61	697	752	681	739	587	15	42	19	679	52
54	522	44	54	644	34	38	4	42	108	92	69	620	794	620	799	540	16	32	14	753	48
57	630	48	39	720	33	37	7	41	168	99	60	735	929	682	756	574	22	43	14	700	66
57	581	41	48	652	30	34	4	39	120	84	62	678	810	730	759	648	22	31	13	715	51
70	636	44	55	702	34	32	6	51	138	116	71	749	891	758	875	670	20	33	19	824	55
60	662	42	57	736	36	28	5	48	120	107	66	763	904	729	865	644	15	34	12	819	51
66	540	43	56	593	32	29	9	54	126	63	63	650	772	735	676	635	18	42	10	625	59
48	390	44	70	540	24	35	7	24	72	68	66	482	636	471	588	412	9	20	4	564	40
78	432	46	60	465	22	32	7	29	72	59	74	555	566	552	624	486	15	25	11	588	41
73	378	53	61	540	23	39	5	35	42	81	65	504	617	467	512	408	14	20	8	485	39
64	378	52	55	450	23	30	5	40	84	59	59	494	574	462	524	408	9	14	9	502	40
62	360	48	64	413	24	32	6	33	54	60	69	469	499	444	341	390	10	17	9	315	38
52	306	46	58	413	28	43	4	34	72	59	56	404	518	374	324	324	5	15	9	300	35
42	378	43	51	405	21	35	6	20	30	50	51	463	455	421	316	372	11	18	8	291	32
30	218	41	48	300	24	32	4	27	48	47	46	289	375	312	292	261	9	18	9	266	34
15	255	29	44	240	24	24	8	16	18	27	40	299	274	287	296	243	6	13	5	279	31
23	210	25	39	257	22	26	4	15	30	30	23	258	302	235	232	209	6	10	5	218	16
16	150	22	31	198	18	18	3	18	18	26	25	188	234	200	237	169	7	14	13	211	18
14	165	20	22	216	17	17	2	15	54	20	25	199	285	210	297	176	8	15	10	273	20
11	128	17	21	215	13	14	0	10	24	23	13	156	248	151	324	128	6	12	6	306	12
16	113	18	20	204	17	14	2	12	36	30	26	146	252	203	258	176	6	15	5	239	13
11	113	15	21	215	13	13	1	7	18	18	22	139	240	207	332	176	6	14	8	311	18
15	120	24	22	188	17	11	1	7	30	30	26	159	225	195	272	162	5	19	6	247	15
13	128	19	28	203	15	13	1	8	42	20	26	159	253	159	315	128	4	14	8	293	17
12	173	17	23	204	13	8	0	7	30	21	23	201	241	190	297	155	7	14	7	277	22
14	147	16	24	216	16	11	2	8	24	24	23	177	248	174	324	142	5	19	7	299	13
10	150	18	20	176	17	15	2	11	30	21	31	178	216	223	266	176	4	25	8	233	23
13	143	19	31	165	16	12	2	9	24	23	27	175	198	172	277	142	8	21	12	245	10
14	158	18	18	180	14	14	3	9	6	26	28	189	195	225	272	196	5	15	8	250	15
13	113	17	24	231	16	15	1	6	24	15	24	142	261	171	292	135	5	21	7	264	15
12	143	13	21	203	13	8	2	11	12	23	22	168	225	187	279	149	7	21	10	248	17
12	116	16	18	180	13	8	0	9	6	11	20	144	195	198	266	176	2	16	7	244	7
9	128	14	16	189	14	7	1	9	24	9	17	151	222	168	263	142	3	17	5	241	10
9	98	12	12	210	11	8	1	8	6	14	14	118	224	164	234	142	3	13	8	214	10
11	96	10	15	147	12	4	0	3	0	14	21	116	150	150	212	128	1	15	5	192	7
6	105	8	10	189	8	3	1	1	6	14	12	119	196	89	195	74	5	11	3	181	4
6	143	9	11	153	11	3	1	4	12	11	13	157	169	103	155	95	1	5	2	148	3
4	135	10	10	132	7	1	1	3	18	11	10	149	153	97	168	81	4	12	3	154	4
3	98	10	10	108	8	2	1	3	0	5	14	111	111	87	156	74	3	10	4	143	3
6	83	7	9	198	7	2	1	4	0	14	11	95	202	79	113	68	3	6	4	103	6
4	83	6	10	180	5	1	0	1	0	6	9	92	181	94	143	88	1	6	1	137	1
4	75	10	5	144	5	3	0	3	6	6	6	89	153	68	160	54	3	11	5	144	3
2	68	7	6	120	7	3	0	3	6	14	6	76	129	47	143	34	1	12	2	129	2
8	53	6	6	108	5	2	1	2	18	3	7	67	128	68	140	61	1	4	2	135	4
4	98	9	6	99	6	1	1	3	0	8	12	111	102	74	122	68	1	6	1	116	1
5	75	6	5	108	5	3	1	2	6	5	9	86	116	63	118	54	1	6	4	108	3
6	68	9	4	135	5	2	0	2	0	3	7	82	137	96	130	88	3	6	3	122	3
4	105	13	8	126	5	4	1	3	24	8	7	121	153	81	147	68	1	10	4	134	4
5	90	11	8	135	5	1	0	2	0	3	12	106	137	88	123	81	1	5	2	116	2
5	105	13	9	116	7	2	1	5	6	11	14	123	126	99	105	88	3	7	2	96	4
6	75	16	8	126	7	6	0	4	6	18	14	97	136	101	117	88	2	11	2	105	3
10	135	14	14	105	8	9	2	6	6	23	16	160	117	117	129	105	3	4	3	122	8
8	189	18	21	116	8	11	3	8	0	24	19	215	123	187	155	171	4	8	4	143	8
13	225	19	24	95	12	9	2	7	12	30	20	257	114	330	186	317	1	8	0	179	6
11	368	20	31	116	9	7	1	8	36	35	37	398	160	346	99	336	4	6	0	93	4
10	435	21	36	144	9	11	4	8	66	27	43	466	218	406	128	389	5	8	1	120	10
19	450	27	43	162	12	16	7	13	42	38	51	496	217	542	133	527	3	7	4	123	9
20	446	36	48	189	19	19	6	15	36	41	59	501	240	473	142	450	2	10	5	128	14
13	450	24	60	243	20	21	6	16	24	20	66	487	283	452	338	432	6	12	5	321	8
24	423	28	67	284	27	20	6	19	24	35	82	475	326	453	450	428	7	16	5	429	10
46	623	47	58	473	39	42	10	35	42	69	94	715	549	642	574	599	11	14	11	549	30
47	614	52	57	494	42	41	8	32	30	69	103	713	555	692	651	648	11	18	9	624	26
42	594	34	68	662	48	43	8	30	48	53	97	670	740	649	652	648	1	1	0	651	0
48	567	40	78	536	41	41	5	43	42	71	96	654	621	670	583	615	9	17	8	558	38
53	554	40	78	494	43	43	9	42	36	71	106	647	571	685	588	630	9	21	6	561	34
31	540	34	78	480	42	39	6	26	36	60	96	605	542	622	548	567	13	18	8	522	37
46	557	25	74	360	39	29	4	34	30	87	75	627	424	713	575	660	7	20	6	549	33
42	525	28	84	273	42	48	6	28	42	62	106	595	343	746	578	696	11	25	5	549	26
43	567	38	74	330	41	45	4	29	18	48	100	648	377	789	642	738	11	17	7	618	34
61	500	32	69	330	38	41	3	17	12	69	103	593	359	779	292	735	10	21	6	266	23
55	562	34	77	338	37	46	7	19	18	83	114	651	375	834	329	783	10	21	5	303	30
69	668	45	59	360	41	39	4	30	24	98	87	781	414	813	305	768	10	14	4	287	31
60	562	45	63	444	36	42	3	32	0	89	98	667	476	768	343	720	7	18	4	321	31
70	547	44	58	465	38	41	3	36	12	78	91	660	513	800	325	754	7	18	5	302	29
54	530	41	73	441	30	36	4	30	54	74	97	625	525	757	508	708	6	23	4	482	26
48	458	38	57	494	22	30	4	30	96	95	68	543	619	591	523	540	11	20	6	497	31
60	443	36	60	557	29	23	2	33	42	110	66	539	631	545	487	504	10	15	6	466	26
65	390	41	54	660	29	36	5	39	36	105	73	496	735	487	573	450	5	18	4	551	19
66	396	42	61	525	24	29	12	34	84	159	67	504	643	466	634	420	7	20	5	609	26
48	408	41	63	651	30	28	4	39	102	90	69	497	792	549	663	506	10	21	5	637	22
55	443	43	70	576	30	25	6	35	192	107	73	540	803	496	675	444	10	18	11	647	34
62	413	46	60	630	31	32	4	43	186	113	77	521	859	495	731	456	14	15	6	710	24
57	360	51	61	612	26	30	5	42	162	92	60	468	816	530	696	486	13	21	9	667	23
54	405	49	54	525	31	37	3	47	90	96	65	508	662	527	643	473	14	21	9	614	33
53	525	42	46	612	37	34	9	40	120	104	82	619	772	630	696	576	12	25	9	662	29
47	503	42	43	612	30	32	10	44	150	108	69	592	806	550	677	507	11	18	5	654	25
42	360	50	46	585	29	29	8	43	282	81	73	453	910	460	726	410	16	19	9	698	31

6.3. TYPE OF CLUSTER ALGORITHMS

Partitioning methods and hierarchical methods are the most widespread types of cluster algorithm.

Hierarchical clustering algorithm gives hierarchy of nested clusters and is mainly used in biological applications, like taxonomy of plants and animals. This type of clustering is usually performed on small data sets.

Partitioning algorithms produce k clusters (set of disjoint clusters) such that:

• Each group must contain at least one object

• The clusters must be mutually disjointed such that each observation falls exactly in one group (25).

K-Means clustering, a partitioning method was used in this thesis to find best-separated groups of demand states. The analysis was performed in MATLAB toolbox. The next subchapter describe the methodology.

6.4. K-MEANS CLUSTERING

The K-means clustering algorithm is commonly utilized analysis in order to partitioning the clustering. The algorithm minimizes the overall "within cluster distance" from the patterns to the centroids. Each pattern is assigned to the closest centroid to find the local minimum of the objective function (29). The objective function that has to be minimized is presented in following formula:

$$E = \sum_{j=1}^{k} \sum_{x_i = w_j} ||(x_j - m_j)||^{2}$$

$$m_j = \frac{1}{N_i} \sum_{x_i = w_j} x$$

E = error of cluster partitions (sum of squared deviations);

x_i *= i-th pattern feature vector of an element of group j;*

k = number of clusters;

w_j *= j-th group;*

m_j *=centroid feature of group j*

The primary step of the algorithm are described subsequently:

a) Definition of the number of clusters;
b) Initialize centroids of each cluster;
c) Assign a data point to the cluster with the closest centroid;
d) Calculate the new cluster centroid;
e) Repeat steps 3 and 4 in order to assign all the data points to the clusters;
f) Calculate the error for given classification;
g) Reassign the data points to minimize the error.

As the number of clusters and their centroids is unknown beforehand, the first two steps can pose some difficulty. Determination of number of the clusters is performed with Silhouette width. The first centroid values was randomly chosen from the data points. Rosseeuw introduced silhouette function for graphical representation of each cluster.

Through silhouette plot is possible to observe cluster points located within the cluster and point that lie only on the intermediate position. The plots possess the ability to compare the separation and compactness among the clusters. The best number of clusters is selected by the silhouette width.

Value s(i) is associated to each object and then plotted. An object i is taken from the data set in order to define s(i). If i belongs to cluster A, a(i) is defined as average dissimilarity of i to all other objects of cluster A. And d(i, C) is an average dissimilarity of i to all other objects of cluster C, where cluster C is different from cluster A. Subsequently, the value of d(i, C) for all cluster C different to A, the smallest value is selected and represented by b(i):

$$b(i) = \min_{C \neq A} d(i, C)$$

The cluster B, which have the value b(i), is the "neighbour" of object i. Consequently, the value of s(i) is obtained from following formula:

$$s(i) = \frac{b(i) - a(i)}{\max\{\, a(i), b(i)\,\}}$$

Therefore, silhouette function values oscillate between:

$$-1 \leq s(i) \leq 1$$

The value s(i) approaches 1 as the value of b(i), the smallest "between" dissimilarity, is much larger than the a(i), the "within" dissimilarity. A value closer oscillating near one means that i-th point is closer to the data points in its group (A) rather than any other group (B). Therefore, i-th point is well classified. While, value s(i) is nearly 0, a(i) and b(i) are almost equal, it means that i-th point is in the same distance from cluster A and B. In this case, it is unclear to which group A or B should be assigned point i. This situation presents an intermediate case of classification. When, s(i) approaches to -1, a(i) is much larger than b(i). In consequences, point i assigned to the cluster A is more close to the point of cluster B. This is the worst situation, because it means that point i has been misclassified in the group A.

A wide, box shaped, silhouette plot, which has all positive value, represents a well partition of the point to the cluster.

For all data points is calculated silhouette width for all data set, which is the average value of s(i), denoted as s'(k). This silhouette width is utilized to select the best number of clusters (k). The number of cluster is selected to give the highest value of s'(k). The maximum value of s'(k) for all values of k from 0 to n-1 (n is the number of data points) is called Silhouette coefficient (SC). The following table shows the subjective interpretation of Silhouette coefficient:

Table 4: Interpretation of Silhouette coefficient

Silhouette Coefficient	***Interpretation***
≤ 0.25	No strong structure
0.26 - 0.50	A weak structure
0.51 – 0.70	A good structure
0.71 – 1.00	Very strong structure

6.5. CLUSTER INPUT VARIABLES – SENSITIVITY ANALYSES

In the next subchapters, K-means clustering was used for finding the demand states of Guglielmo Marconi Street. Moreover, this thesis was also focused on sensitivity analyses, which include creating clusters using different input variables to Matlab toolbox in order to cluster the traffic volumes with the maximum efficiency.

Three types of clustering analysis, depending on input data, were examined:

1. Flow Clustering of all movements together;
2. Flow Clustering of main arterial flows and cross street movements separately;
3. Clustering of normalized traffic flow.

6.5.1. FLOW CLUSTERING OF ALL MOVEMENTS TOGETHER

In this method, flows of all movements from three intersection were clustered together. Those flows were not be normalized; in this case, the main arterial flows are much higher than flows on crossing streets. The table below represents the clustered levels of traffic flows with their centroids for every movement. After clustering analysis is possible to observe five levels of traffic flow.

Table 5: Clustered levels of traffic flow by all movements together

	11019							11013												11022					
LEVEL	NBL	NBT	SBT	SBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR	EBL	EBT	EBR	WBL	WBT	WBR	NBT	NBR	SBU	SBL	SBT	WBR
1	48	595	434	19	71	24	16	51	572	40	69	421	39	40	6	31	30	73	95	686	10	20	7	466	31
2	45	699	580	21	154	23	41	56	596	42	53	666	36	35	6	44	114	86	69	618	16	32	13	721	48
3	31	423	247	13	64	16	7	27	372	33	50	264	20	24	5	19	41	40	55	377	6	13	5	242	20
4	46	471	474	24	116	23	22	52	415	42	55	565	30	34	6	38	115	90	69	449	14	23	9	589	36
5	12	169	107	9	35	6	3	9	121	14	16	166	11	8	1	7	15	16	17	125	4	12	5	193	9

Figures below point out the optimal number of Clusters and Silhouette function, which represents compactness and separation among the clusters. It is possible to observe that the optimal number of Cluster is two, but for the purpose to design the traffic signal, it is recommended to choose a higher number of Clusters, taking into account high Silhouette value. That is why, the formation of one, two, three or more than eight clusters will be ignored. Less than clusters will not allow enough timing plans to capture the changing traffic conditions during a day and the signal timing plans higher than 8 will provoke a switching from one plan to another too often. Second figure shows Silhouette function, which has negative value indicating misclassification of the points to the assigned clusters.

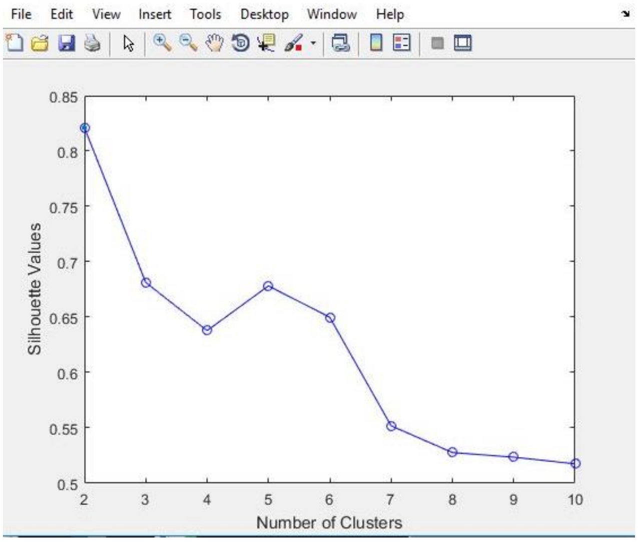


Figure 60: Optimal number of clusters

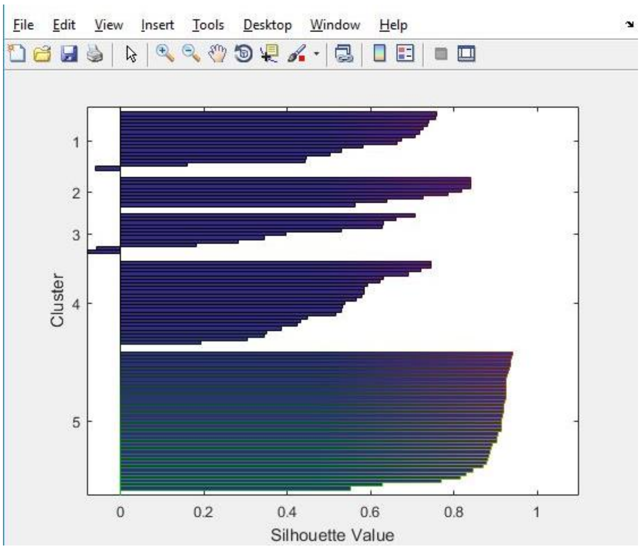


Figure 61: Silhouette values

The disadvantage of this method is that Cluster analysis does not work correctly when values of input data are very different among themselves. In result, data points are classified to the inadequate clusters. For this purpose, it is necessary the normalization of the input data entered to Matlab.

6.5.2. FLOW CLUSTERING OF MAIN ARTERIAL AND CROSS STREET MOVEMENTS SEPARATELY

In this method, flows of Marconi Street movements and Cross Street movements were clustered separately. Those flows were not normalized, because the flows on the main street show similarity to each other, same as flows on crosses streets. The table below represents the clustered levels of traffic flows with their centroids for every movement.

Table 6: Clustered levels of traffic flow for Main Street and Cross Streets movements separately

Crosses Street Flows (without recalculation of centroids)										
	11019			11013						11022
LEVEL	WBL	WBT	WBR	EBL	EBT	EBR	WBL	WBT	WBR	WBR
1	193	23	45	36	6	38	104	81	67	48
2	83	24	15	39	6	31	37	72	85	30
3	36	6	4	9	2	8	18	19	23	10
4	93	23	19	32	6	40	135	93	68	37

Flows of Marconi Street (with recalculation of centroids)						
	11019		11013		11022	
LEVEL	NB	SB	NB	SB	NB	SB
1	180	113	133	169	137	194
2	519	316	472	319	555	290
3	543	493	486	549	526	606

Crosses Street Flows (with recalculation of centroids)										
	11019			11013						11022
LEVEL	WBL	WBT	WBR	EBL	EBT	EBR	WBL	WBT	WBR	WBR
1	196	23	48	35	6	39	98	83	65	48
2	85	24	15	38	6	33	48	77	81	31
3	36	6	4	9	2	8	18	19	23	10
4	99	23	20	33	7	40	158	89	70	38

It is possible to observe that in case of similar flows, cluster analysis works better. Moreover, subsequent recalculation of centroids shows that their position is slightly changed and it means that also thresholds of the traffic levels is changed.

The first two figures below show the proper numbers of clusters represented high value of Silhouette function, which in the case of the main street results in two or three levels, while for cross streets indicates four as its optimum.

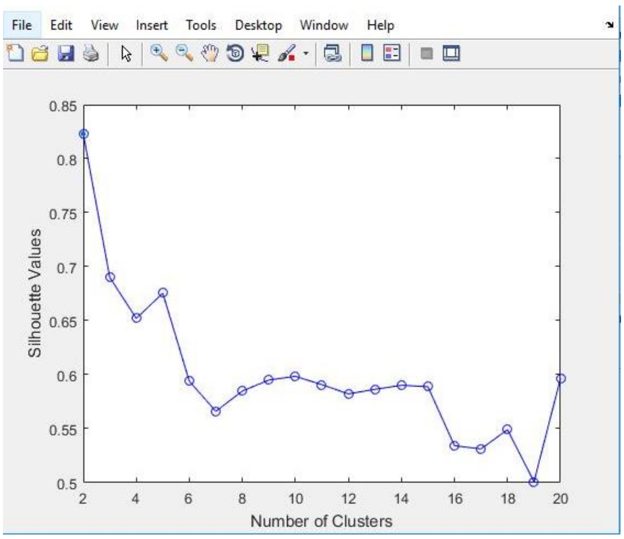


Figure 62: Optimal number of clusters for Main Street

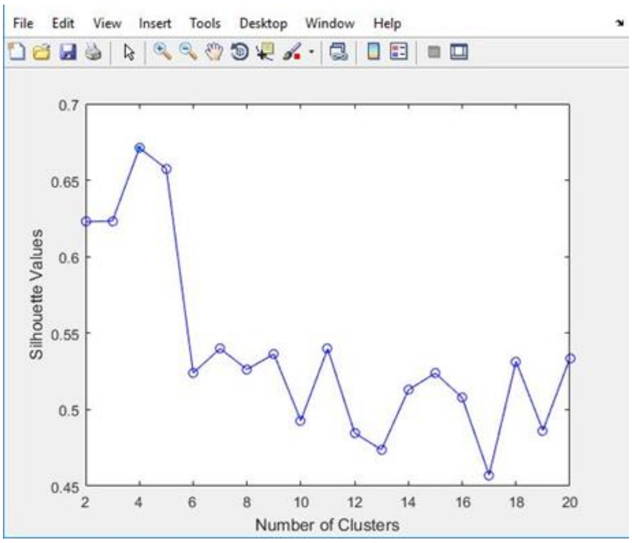


Figure 63: Optimal number of clusters for Cross Streets

Silhouette functions represent high values for both Clusters, however cross street flow levels do not present good separation one from another.

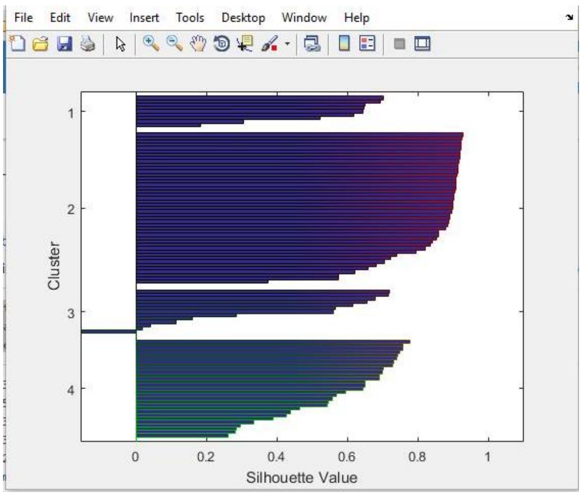


Figure 64: Silhouette values for Cross Street

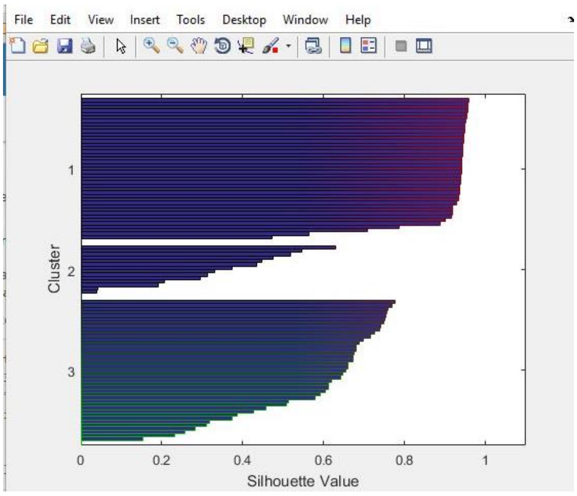


Figure 65: Silhouette values for Main Street

Following this method is possible to obtain twelve different timing plans. However, it is not necessary to introduce signal timing plan for each of twelve traffic levels, because traffic volume on the crosses streets changes simultaneously, in a similar proportion, with the flow on the Marconi Street. The creation of many timing plans causes that most of them will not be used. Moreover, many timing plans with short thresholds provoke misclassification and frequent plan changes and in results delays due to transition time.

This subchapter also proved that recalculation of the centroids for each point is very significant in order to get better separation and classification into the clusters.

6.5.3. CLUSTERING OF NORMALIZED TRAFFIC FLOW

From the previous subchapters can be conclude that collected data need to be standardized to avoid the dependence on the scale of measurement. Moreover, it is necessary recalculation of centroids for every data point.

In this method, in order to avoid dependence of clustering on the choice of measurement (vehicle/hour/lane or vehicle/hour) and scaling, the traffic streams of all movements were normalized by corresponding flow ratio. Flow ratio is a ratio of volume on an approach and the saturation flow of the approach. Saturation flow is a maximum number of vehicles from a lane group, which can cross the junction in one hour under the predominant roadway and traffic conditions if the lane group has continuous green signal for that hour. In other words, flow ratio depicts the fraction of approach capacity being used. Usually, the saturation flow volume of a single lane varies from 1200 to 2200 vehicles/hour of green. The methodology for calculating the saturation flow is provided in highway capacity manual. Therefore, traffic data for each lane group was divided by its saturation flow in order to obtain the flow ratio. This transformation is intended to normalize the collected data for clustering analysis and it is represented in this subchapter.

The table below represents the centroids of six traffic levels arising from clustering analysis. Centroids were brought back to their original form represented as values of traffic flow in the lane groups. Through clustering of standardized traffic flows it is possible to get six levels of traffic stream as the best grouping, which is possible to observe in the second figure.

Table 7: Normalized and clustered traffic levels

	11019				11013									11022			
LEVEL	NBL	NBT	SBT+R	WB	NBL	NBT+R	SBL	SB T+R	EBL	EB T+R	WBL	WBT	WBR	NBT+R	SBU+L	SBT	WBR
1	48	595	452	111	101	407	69	459	40	36	30	73	95	696	26	466	31
2	45	699	600	217	113	426	53	701	35	49	115	86	68	634	44	721	48
3	29	415	242	80	47	269	49	269	22	22	40	38	53	382	17	235	18
4	47	486	472	175	108	297	55	550	34	41	75	171	134	453	31	539	36
5	12	169	116	44	19	90	16	176	8	7	15	32	34	129	17	192	9
6	46	447	534	136	97	315	55	607	33	46	179	184	142	472	34	647	34

Furthermore, four figures depicting silhouette values for different numbers of clusters have been presented. In case of three, four and five traffic levels, the function has negative values, which results in inadequate classification of the data points to the clusters. Six traffic levels present the best separation.

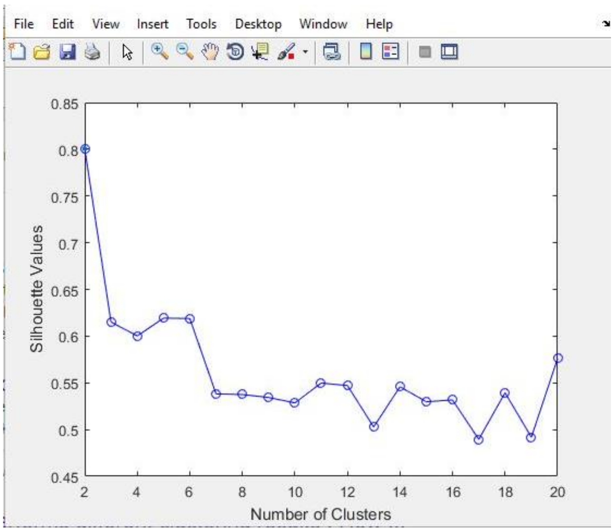


Figure 66: Optimal number of clusters

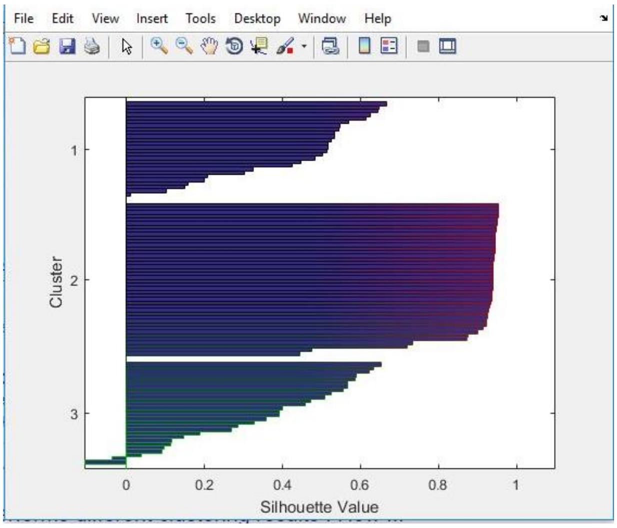


Figure 67: Silhouette value for three clusters

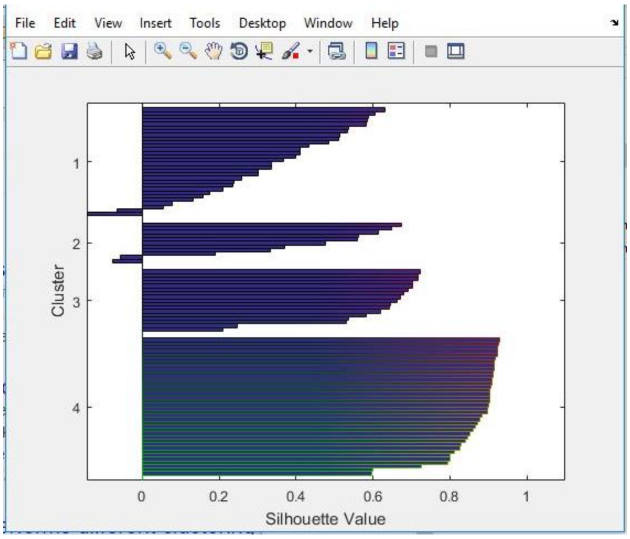


Figure 68: Silhouette values for four clusters

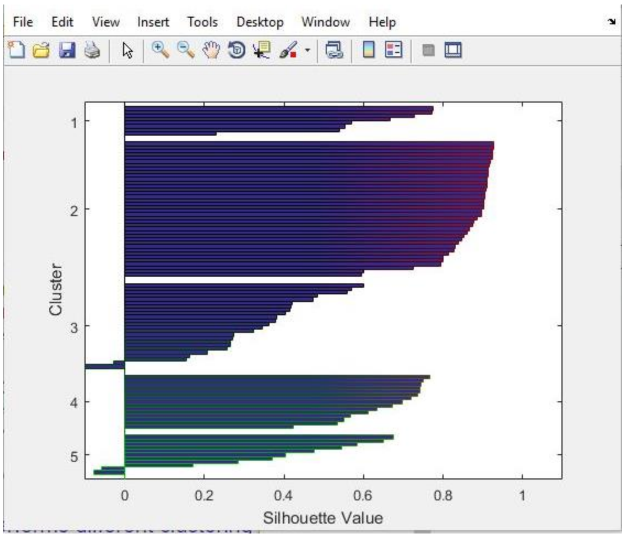


Figure 69: Silhouette values for five clusters

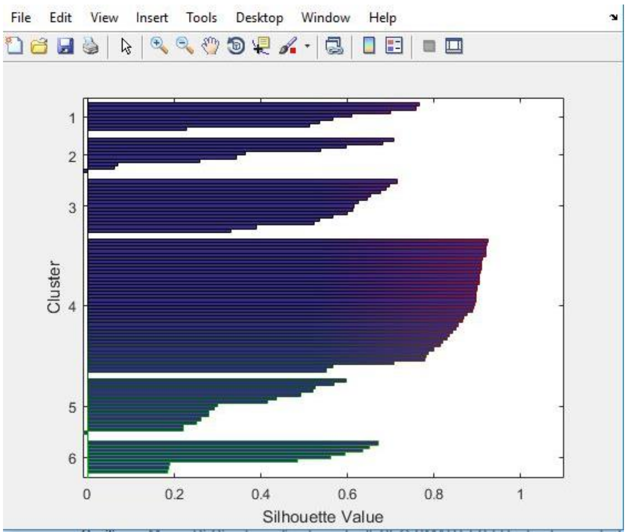


Figure 70: Figure 67: Silhouette values for six clusters

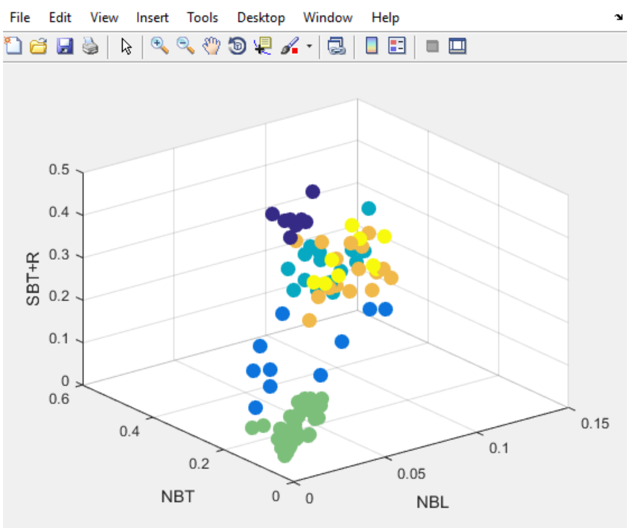


Figure 71: Example of 3D space flow distribution

The last figure illustrates the distribution of different clusters in 3D space with normalized flows of NBT, NBL and SBT+R movement as example.

This method seems to be optimal, because all movements after standardization have similar values and in result, it is possible to obtain well-separated clustering. This method was used for further analysis and it was a base for creation of new six signal timing plans.

Chapter 7

TRAFFIC SIGNAL PLAN ASSIGNMENT

7.1.CHANGES IN SIGNAL PLAN DURING A DAY

From the previous chapter it was possible to obtain six different traffic levels through cluster analysis. Each of 15-minutes detected traffic flow was assigned to the one of this traffic states. The following figures represent distribution of current and clustered traffic states over the examined day.

The first figure presented below shows the division of actual timing plans during a day. There are five different signal timing plans currently operated using Time of Day mode. This plan control the entire network, depending on the specific time of day.

Through cluster analysis, it was possible to observe that the optimal number of plans are six. Moreover, trend of traffic flows is irregular during the day; therefore, it is necessary to change the timing plans more frequently in order to satisfy the traffic demand. The transition from one signal plan to another cannot be too short, because of delays associated with it. Second column of the table 2 and 3 shows the number of cluster associated to every 15-minutes volumes. Second figure presented below illustrates it graphically.

Moreover, it is possible to observe in the second figure, some changes in timing plans that last only 15 minutes and after come back to the previous plan. In order to avoid misclassification of values and transition delays connected with it, is suggested to make further research how to improve the classification accuracy. Following this observation some correction in transition of timing plan was implemented, which are presented below in third figure. This proposal of collocation of signal timing plans was subsequently used in simulation and evaluation.

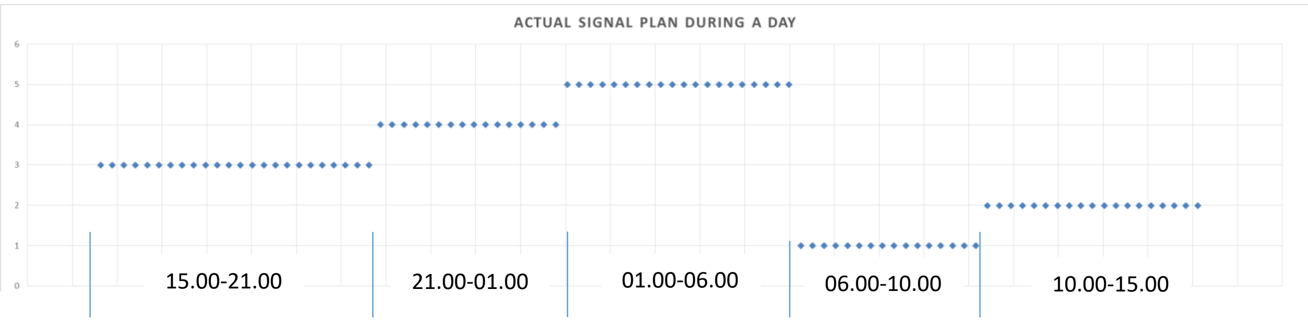


Figure 72: Changes in actual signal timing plans during the day

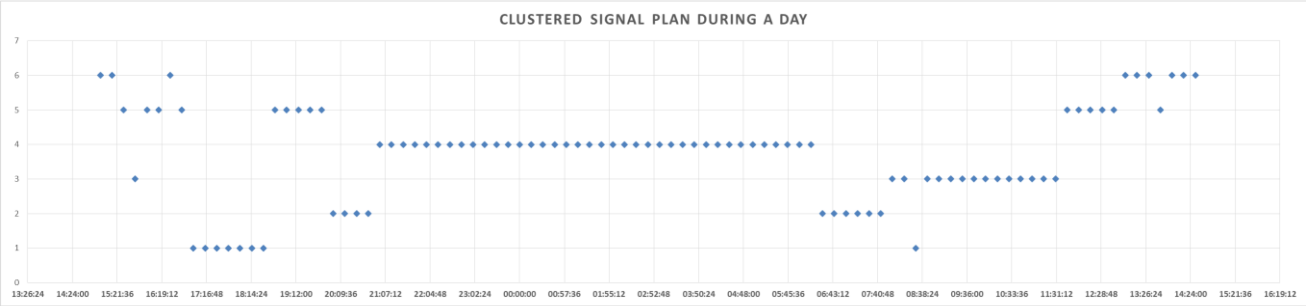


Figure 73: Changes in clustered traffic plans during the day

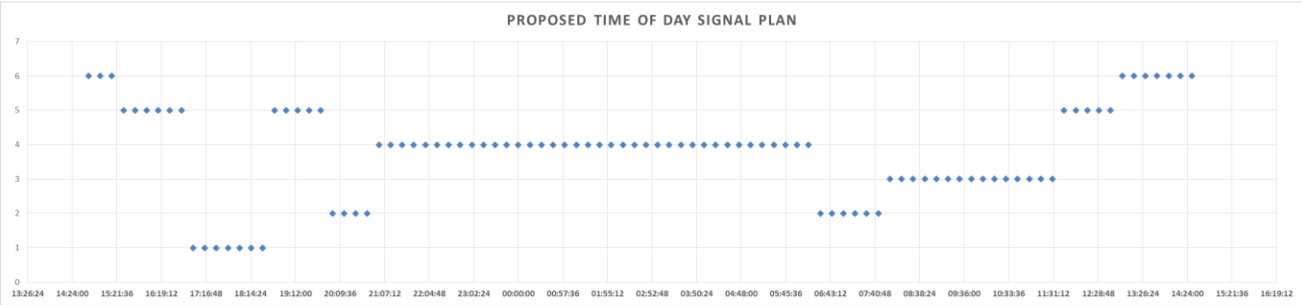


Figure 74: Changes in clustered traffic plans during the day after modifications

7.2. DEVELOPMENT OF TRAFFIC SIGNAL PLAN

The traffic signal plan needs optimally serve a demand state. Calculation of best-suited signal timing plan for given demand on the network can be performed with many software programs, such us PASSER II, SYNCHRO or TRANSYT-7F. Due to scope of this thesis, signal-timing plans were generated using SYNCHRO 8. The model built in SYNCHRO must be as accurately as possible with respect to signal timing parameters, roadway geometry and volume data.

Following figures illustrate models of all intersections designed in Synchro and considered in this thesis:

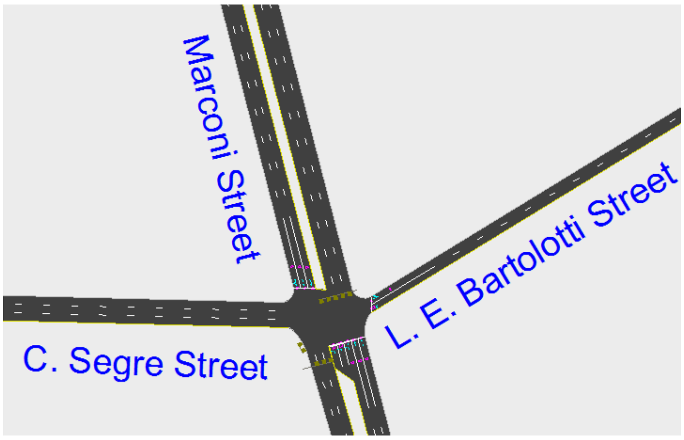
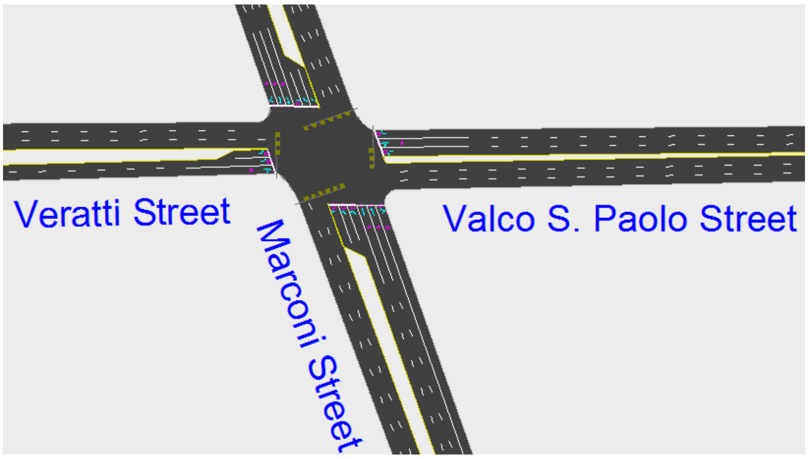


Figure 75: Synchro model of intersection 11019

Figure 76: Synchro model of intersection 11056



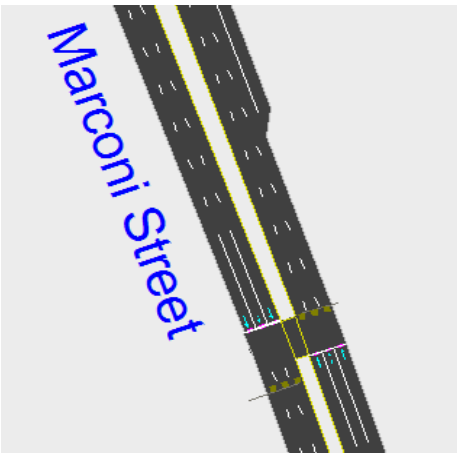


Figure 77: Synchro model of intersection 11013

Figure 78: Synchro model of intersection 11090

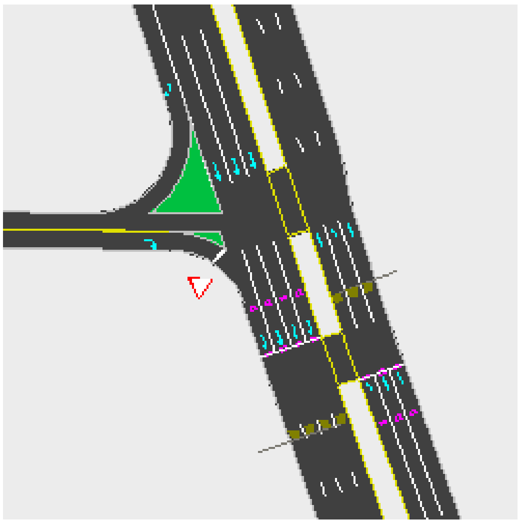


Figure 79: Synchro model of intersection 12044

Figure 80: Synchro model of intersection 11022

The 85-percentile approach volumes for each state was chosen in order to develop signal-timing plans. Basing on the engineering judgment 85-percentile of demand was chosen as the design volume. The reason for this choice is the fact that the optimal cycle length results in minimum delay. If a shorter cycle length than the optimum cycle length is implemented, it is possible to observe high rate of delay grow. On the other hand, delay increases at a slower rate while higher cycle length is assigned. Moreover, for a given volume cluster a higher demand need a longer cycle length. It can also be noticed that cycle length corresponding to the maximum volume in the demand group will result in least delay for entire group. Therefore, it is most suitable to choose a higher volume on each approach. Is recommended to use the 85-percentile value for each approach volume in a given state as design demand for a given state, because it is rarely possible that all the approaches will simultaneously reach their maximum. The cycle length also cannot be too high due to certain outliers in a given state, because it can cause excessive delays for cross street traffic. Based on design volumes, optimized timing plans were created in SYNCHRO 8 and assigned to each of traffic state.

For each intersection, it was necessary to optimize cycle lengths and the intersection splits. Subsequently, network-wide cycle length was optimized in order to have one common cycle length throughout the entire corridor. The last step was the optimization of network offsets. The following table shows optimized cycle lengths for the corridor and splits for cross-streets.

Table 8: Signal Timing Plans

Signal Timing Plans									
STATE	INTERSECTION CODE	CYCLE LENGTH (sec.)	SPLIT (sec.)						
			NBL	NBT+R	SBL+U	SBT+R	EB	WB	OFFSET (sec.)
1	11019	119	16	79	-	63	-	40	28
	11056		-	69	-	85	34	-	28
	11013		-	79	-	79	40	40	21
	11090		-	73	-	73	-	-	35
	12044		-	73	-	73	-	-	93
	11022		-	78	-	78	-	41	89
2	11019	131	15	67	-	82	-	49	22
	11056		-	73	-	88	43	-	22
	11013		-	87	-	87	44	44	20
	11090		-	91	-	91	-	-	52
	12044		-	91	-	91	-	-	80
	11022		-	89	-	89	-	40	88
3	11019	140	13	98	-	85	-	43	134
	11056		-	91	-	104	37	-	134
	11013		-	100	-	100	40	40	5
	11090		-	116	-	116	-	-	21
	12044		-	100	-	100	-	-	116
	11022		-	100	-	100	-	40	125
4	11019	115	7	72	-	65	-	43	93
	11056		-	71	-	78	37	-	93
	11013		-	61	-	61	54	54	76
	11090		-	91	-	91	-	-	42
	12044		-	75	-	75	-	-	9
	11022		-	75	-	75	-	40	14
5	11019	115	13	72	-	59	-	43	78
	11056		-	65	-	78	37	-	78
	11013		-	75	-	75			84
	11090		-	75	-	75	-	-	106
	12044		-	75	-	75	-	-	12
	11022		-	75	-	75	-	40	13
6	11019	110	12	68	-	56	-	42	92
	11056		-	62	-	74	36	-	92
	11013		-	69	-	41	41	41	90
	11090		-	84	-	48	-	-	0
	12044		-	70	-	70	-	-	51
	11022		-	70	-	70	-	40	42

For each of traffic plan, an adequate offset was developed by SYNCHRO and subsequently adjusted manually. The offset needs to minimize delays of the corridor and possess a green band as constant as possible along the itinerary. Figure below presents a software time-space diagram of new developed timing plans.

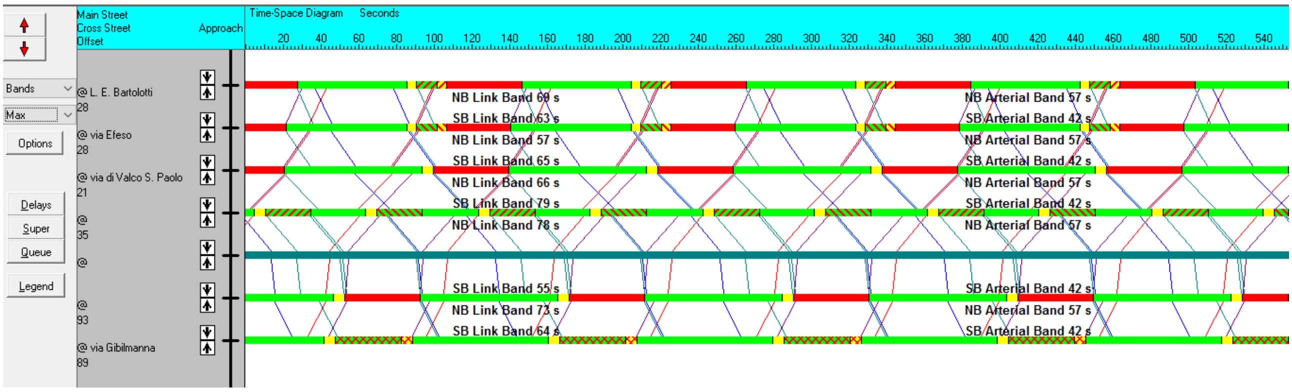


Figure 81: Time-Space diagram of Timing Plan 1 during afternoon volumes

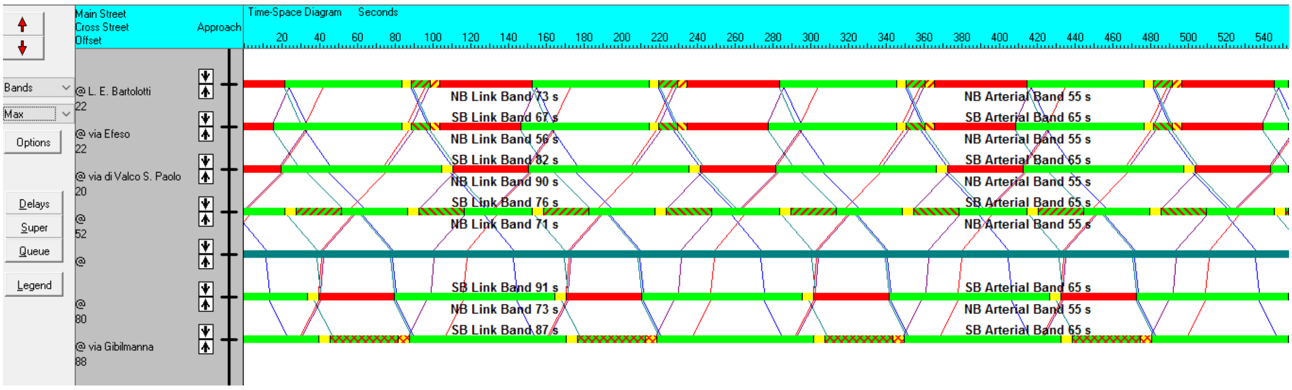


Figure 82: Time-Space diagram of Timing Plan 2 during afternoon volumes

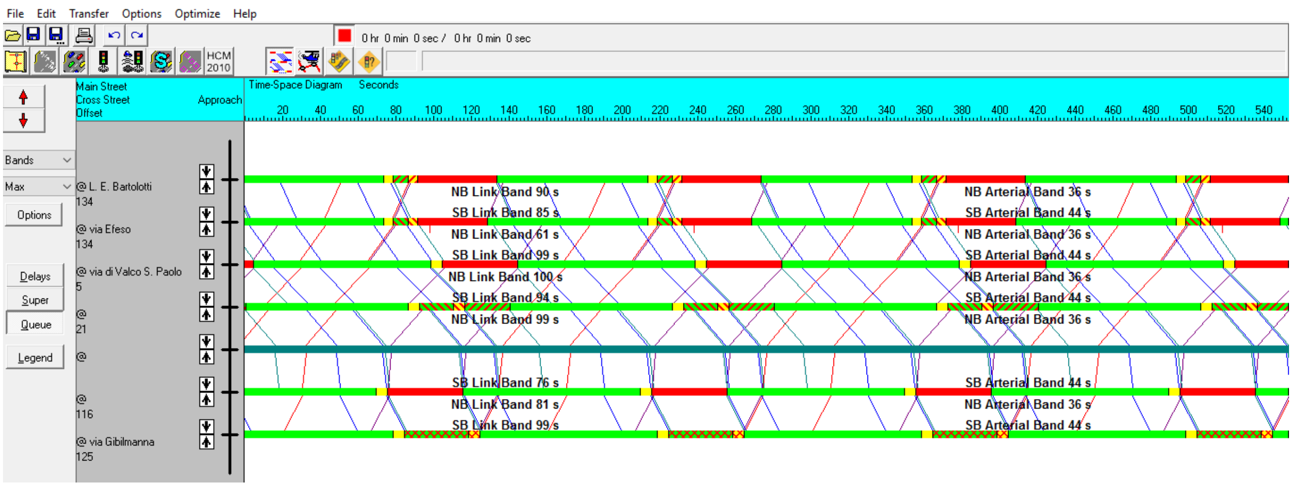


Figure 83: Time-Space diagram of Timing Plan 3 during afternoon volumes

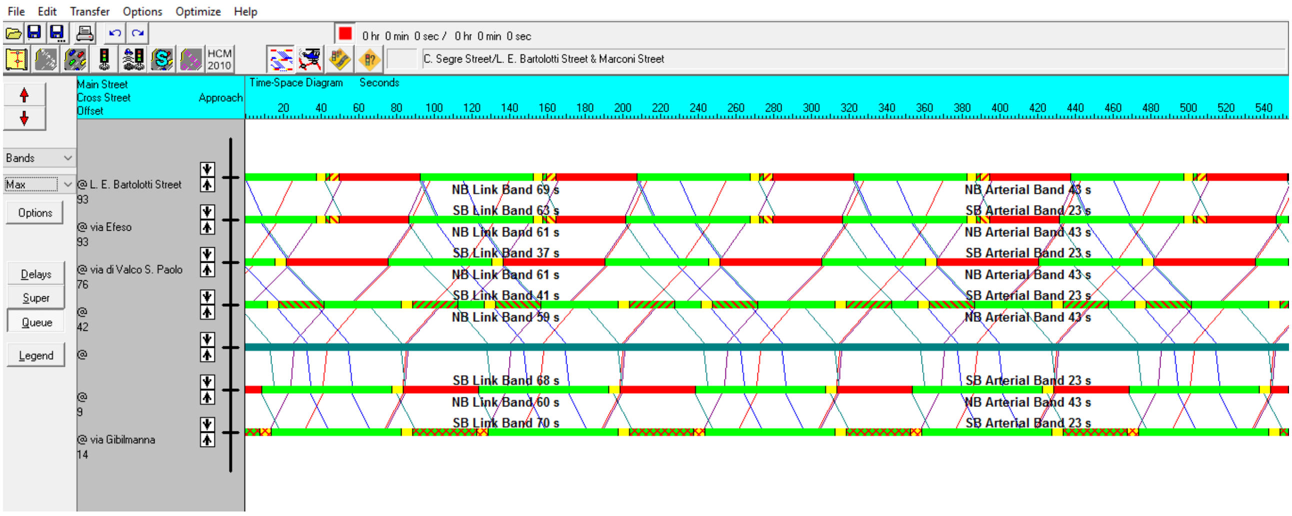


Figure 84: Time-Space diagram of Timing Plan 4 during afternoon volumes

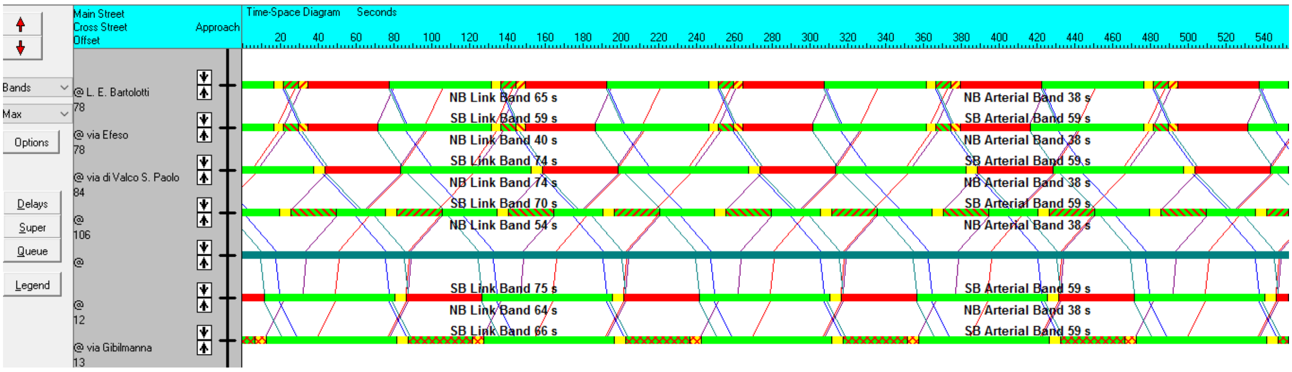


Figure 85: Time-Space diagram of Timing Plan 5 during afternoon volumes

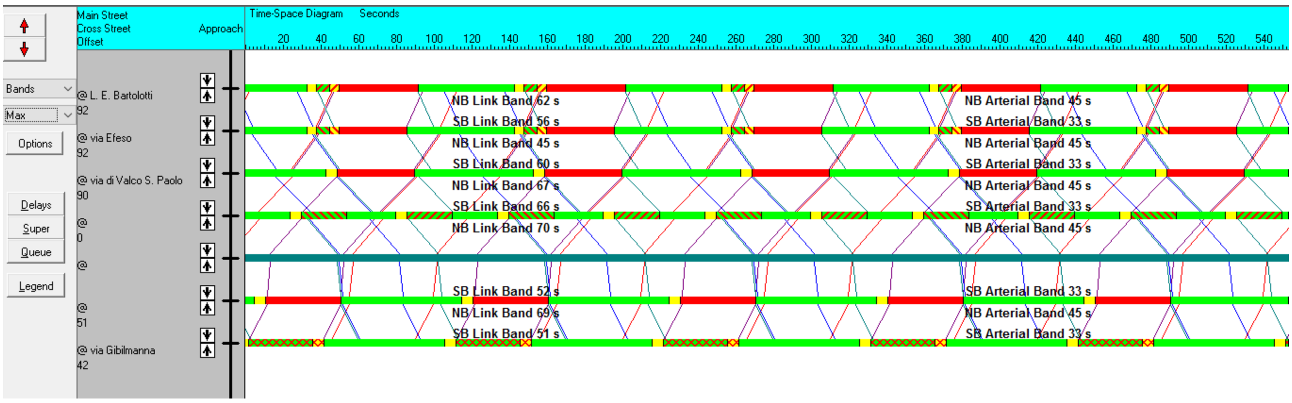


Figure 86: Time-Space diagram of Timing Plan 6 during afternoon volumes

7.3. PATTERN MATCHING MECHANISM

Traffic responsive plan selection mode provides a mechanism by which is possible to change timing plans in real time in response to variation in traffic demand. Traffic controller chooses and implements optimal timing plan to actual traffic conditions. Pattern matching mechanism is a method for implementation of traffic responsive mode in any network. Its algorithms in general have more potential to differentiate between different traffic patterns. In order to classify the current traffic demand to adequate state K Nearest Neighbour Classification was performed.

The K Nearest Neighbour method (KNN) is one of the algorithms for predicting the class of the new vector multidimensional data entered into the system. The KNN algorithm is simple but work very well in practice. When new vector data is a real number, the most common distance function is Euclidean distance.

The nearest neighbour decision rule assigns to an unclassified sample vector the classification of the nearest of a set of previously classified points. This rule is independent of the underlying joint distribution on the sample points and their classifications, and hence the probability of error such a rule must be at least as great as the Bayes probability of error. That is why nearest-neighbour method introduce a significant guarantee. The Bayes error rate for a classification problem is the minimum achievable error rate, which will be nonzero if the classes overlap. Bayes error rate is the average over the space of all examples of the minimum error probability for each example. The optimal prediction for any example x is the label with the highest probability given x. The error probability for this example is then one minus the probability of this label. Formally, the Bayes error rate is presented as follow

$$E = \int_{x \in X} p(x)[1 - maxp(i|x)]$$

where the maximum is over the c possible labels i = 1 to i = c. As the size of data set approaches infinity, the one nearest neighbour classifier guarantees an error rate of no worse than twice the Bayes error rate.

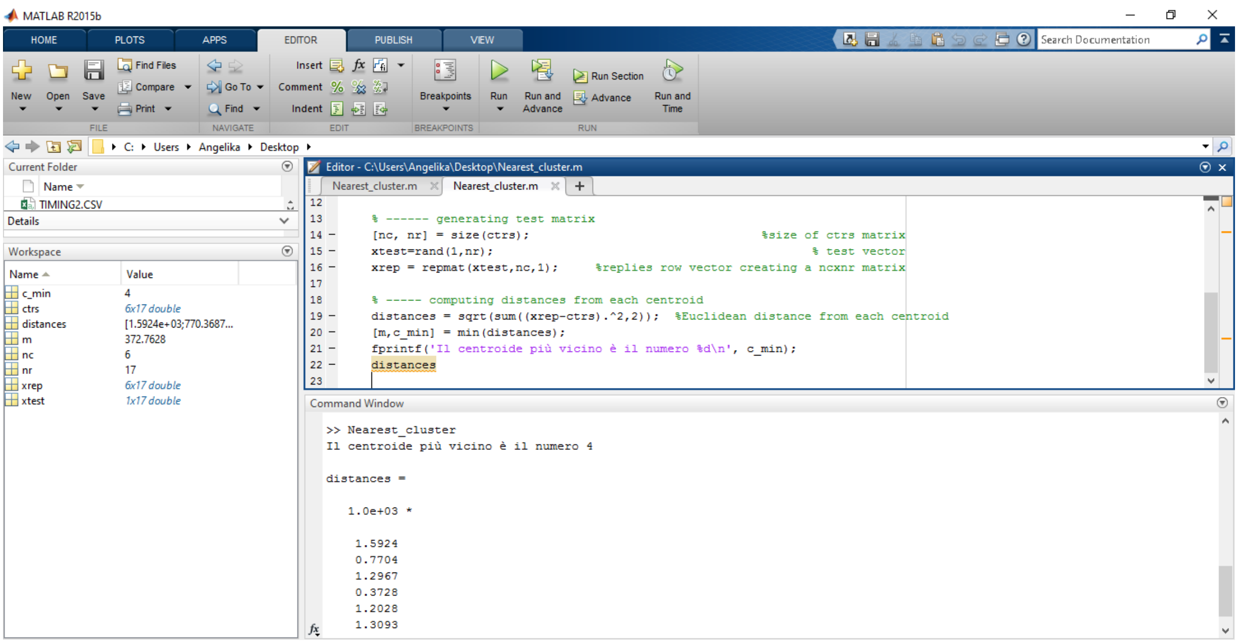
It is possible to decide how many number of classes (K) is required. In the case of implementation of TRPS mode, it is necessary to get only classification, then algorithm is simply called the nearest neighbour algorithm.

The table below illustrate six centroids with theirs design volumes. These centroids are stored and subsequently the KNN algorithm match the new volume vector to the one of them.

Table 9: Design volumes of centroids

Intersection	Cluster / Timing Plans						
	Approach	1	2	3	4	5	6
11019	NBL	45	29	48	12	47	46
	NBT	699	415	595	169	486	447
	SBT+R	600	242	452	116	472	534
	WB	217	80	111	44	175	136
11013	NBL	113	47	101	19	108	97
	NBT+R	426	269	407	90	297	315
	SBL	53	49	69	16	55	55
	SBT+R	701	269	459	176	550	607
	EBL	35	22	40	8	34	33
	EBT+R	49	22	36	7	41	46
	WBL	115	40	30	15	75	179
	WBT	86	38	73	32	171	184
11022	WBR	68	53	95	34	134	142
	NBT+R	634	382	696	129	453	472
	SBU+L	44	17	26	17	31	34
	SBT	721	235	466	192	539	647
	WBR	48	18	31	9	36	34

The K Nearest Neighbour algorithm was performed with MATLAB software. This research was tested and assigned new volume data to the closest centroid belonging to proper timing plan. The figure below illustrates the MATLAB codes used to computing distances from each centroid and the result with the nearest cluster. In order to accelerate the calculation of distance from input vector to each centroid, six identical input vector data were entered. Each of these input vector calculate distance to one of the centroid. This operation made possible to obtain the distance result six times faster.



One major drawback of K Nearest Neighbor method is in calculating distance measures directly from the training set is in the case where variables have different scale of measure or there is a mixture of numerical and categorical variables. In order to overcome this drawback is important to normalize the detector output before run the algorithm.

Chapter 8

SIMULATION AND EVALUATION

8.1.SIMULATION

Detected flows during the specific hours of the day have been modelled in Synchro with current signal timing plans, and after, with new created and optimized signal-timing plans.

In order to better understand how traffic flow will respond to our new developed signal timing plans before the field application, it was applied microscopic simulation model implemented in SimTraffic software. This thesis also test behaviour of vehicles through second software DYNASMART-P that contains mesoscopic simulation model.

Moreover, to compare the functionality of traffic responsive mode and current time of day mode, simulation of flows during both modes of control was done. For the scope of this thesis, simulation and following comparison was effectuated only on one parts of the day, from 3 pm until 9 pm, where is presented very variable trend of the flow with the high traffic volumes during afternoon peak hours. How it is possible to observe in this period of day, the current timing mode is operating with one signal timing plan. While, new proposed approach based on cluster analysis works under traffic responsive mode, which implements four changes in signal control system, which is presented in table below:

Table 10: Differences of timing plans implementation in TRPS and TOD mode

Time	Traffic responsive mode	Time of day mode
	Selected signal timing plan	
15.00-15.30	6	3
15.30-17.00	5	
17.00-18.45	1	
18.45-20.00	5	
20.00-21.00	2	

During the evaluation of traffic responsive mode, it was also considered the delay regards to transition between different timing plans.

8.1.1. SIMTRAFFIC SIMULATION

The simulation outputs are the main support of the effectiveness of the proposed procedure; therefore, it is relevant to consider how accurately these results represent actual traffic states. SimTraffic accounts for conditions such as driver behaviour characteristics, road type and grade, vehicle type etc. Repetitively runs using alternate random number seeds for can representing a dynamic simulation. Furthermore, SimTraffic is also capable of simulating during periods of transition between timing plans to account for transition effects. Calibration of the simulation tool would also provide a better-supported representation of actual traffic conditions.

To determine the network improvements, a calibrated SimTraffic model was used to measure travel times and delays during the six hours period discussed earlier. The Synchro file is an input model to SimTraffic, it seeds the network with vehicles, and computes results throughout the entire network based on simulated travel-time/delay runs. Simulations were run for each of the actual situations, which equate to the TOD operation of the signals and for the "new" situation, which have the same volumes, but use the optimized timing plans for afternoon time period simulation.

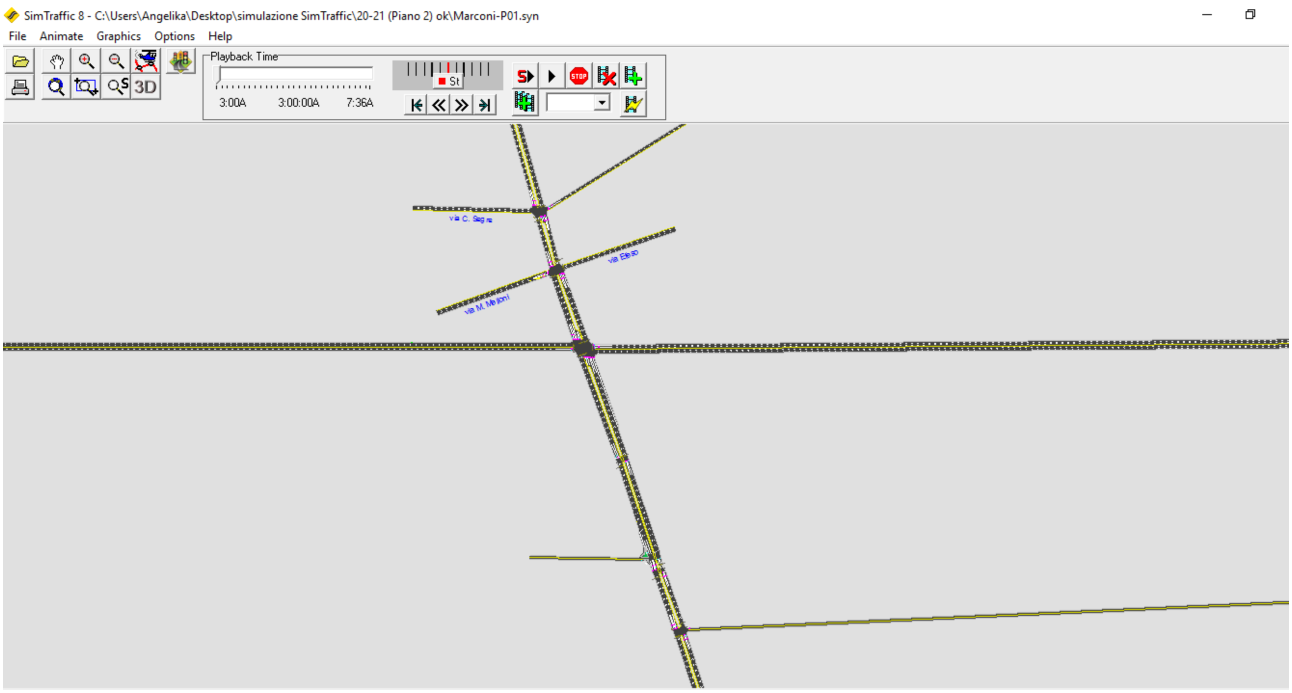


Figure 87: SimTraffic network

Several measures of effectiveness (MOE) were used to quantify the differences between the sets of plans:

- Total Delay (hr)
- The total delay per vehicle (control delay, queue delay and total delay) multiplied by the number of vehicles in the network.
- Fuel Consumption

- Emissions (g) – Calculated based on the travel time.
- Arterial Travel Time (hr) –
- Stops/Vehicle – The number of stops per hour per vehicle.
- Hourly summary of vehicle travel time through the network.

SimTraffic was used to run simulations for this research primarily because it is the tool implemented in Synchro, which saves the time of creation of completely new project. Another benefit to SimTraffic is that the number of intersection allowable to model by the software is extremely large (> 100). However, SimTraffic only allows 19 intervals (15-minute intervals) to be simulated at a time, resulting in a tedious process. Moreover, SimTraffic model, was run and recalibrated several times, but could not represent real traffic conditions on Marconi artery. The model was more congested than in realty, also after recalibration of its parameters. Furthermore, simulation process need many hours, every six hours run longs 7-9 hours of simulation. Moreover, several simulations were run because of continuous need of recalibration of the model, which did not represent real traffic conditions. For these reason only simulations for each interval separately was performed. However, in this case the simulation does not take into account transition times. The results of simulations with correspondent TOD and TRPS plans for every interval of time are presented in the tables below. Comparison of total travel time, total delay, number of total stops and quantity of used fuel are illustrated.

Total travel time (h)				Improvement [%]
Mode		TOD	TRPS	
Cycle Length		Plan 3	Different P	
Time	**15.00-15.30**	724,4	591,1	18,00%
	15.30-17.00	2852	2124,3	26,00%
	17.00-18.45	2923	1885	32,00%
	18.45-20.00	2286	1776	36,00%
	20.00-21.00	764	610	20,00%

Figure 88: Comparison of total travel time

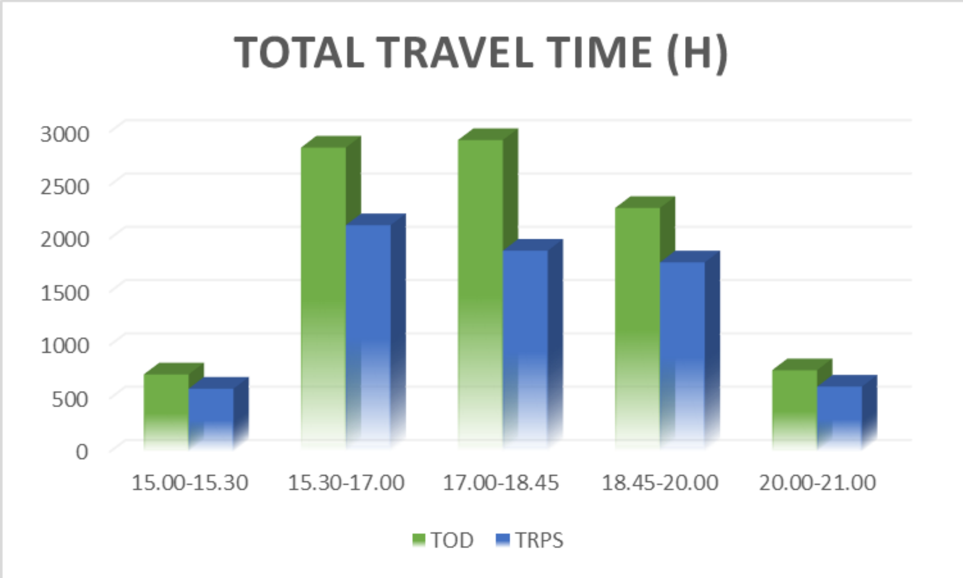


Figure 89: Total travel time representation

Total delay (h)				Improvement [%]
Mode		TOD	TRPS	
Cycle Length		Plan 3	Different Plans	
Time	**15.00-15.30**	163,3	158,2	3
	15.30-17.00	1072	828	27
	17.00-18.45	1453	1061,3	23
	18.45-20.00	1266	987	22
	20.00-21.00	131	111	15

Figure 90: Comparison of total delay

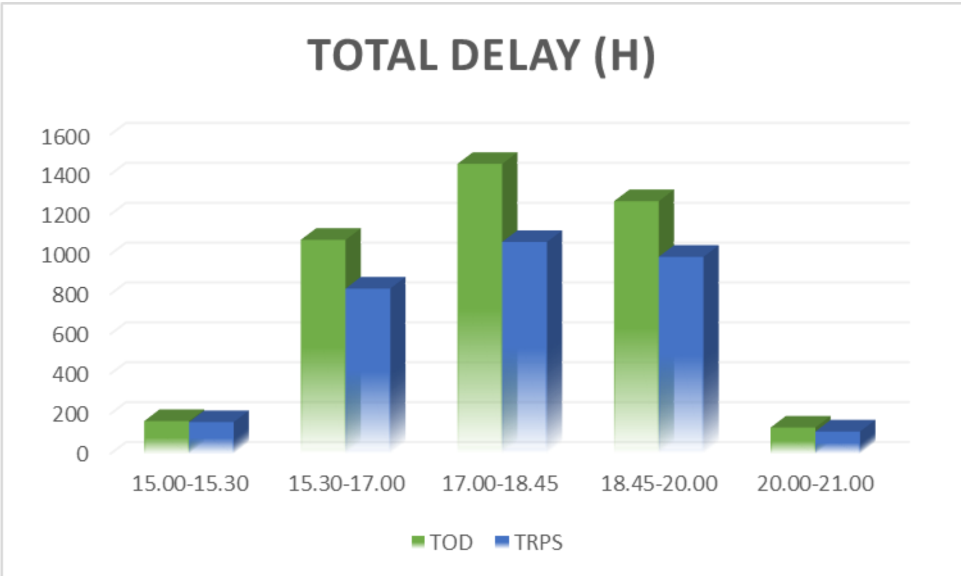


Figure 91: Total delay representation

Total stops				Improvement [%]
Mode		TOD	TRPS	
Cycle Length		Plan 3	Different P	
Time	15.00-15.30	8080	7187	11
	15.30-17.00	35985	24813	14
	17.00-18.45	47718	41081	31
	18.45-20.00	31919	22356	30
	20.00-21.00	7508	6585	12

Figure 92: Comparison of total stops

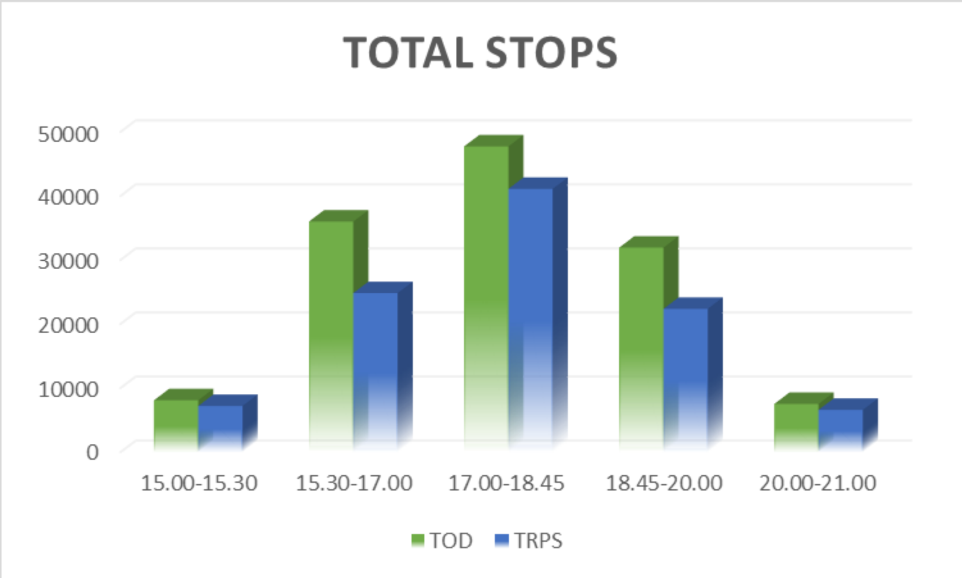


Figure 93: Total stops representation

Fuel used (l)				Improvement [%]
Mode		TOD	TRPS	
Cycle Length		Plan 3	Different Plans	
Time	15.00-15.30	2023	1615	20
	15.30-17.00	5066	4719	15
	17.00-18.45	5985	5075	7
	18.45-20.00	4626	3618	22
	20.00-21.00	2256	1813	20

Figure 94: Comparison of used fuel

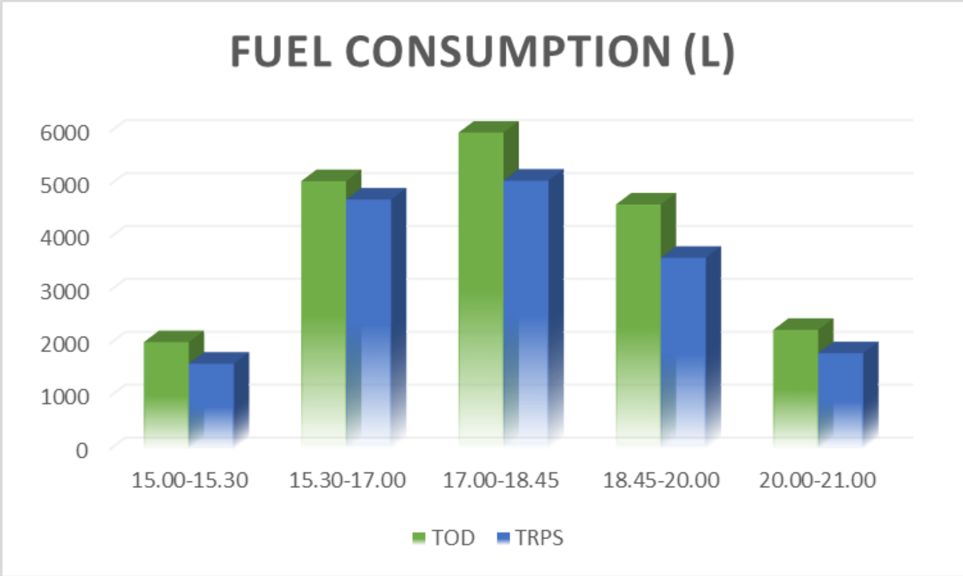


Figure 95: Fuel consumption representation

For the analysis periods, the SimTraffic simulations show benefits in total delay and fuel consumption and reductions in hazardous emissions.

In order to better evaluate the effectiveness of the real conditions, it was experimented the simulation with other dynamic software, DYNASMART-P.

8.1.2. DYNASMART-P SIMULATION

Before testing the new timing plans in DYNASMART-P model, the network had to be accurately constructed. The main steps of design was creation of links and nodes, definition of the intersection movements and assignment of signal control to every intersection. The figure below represents designed network in DYNASMART.

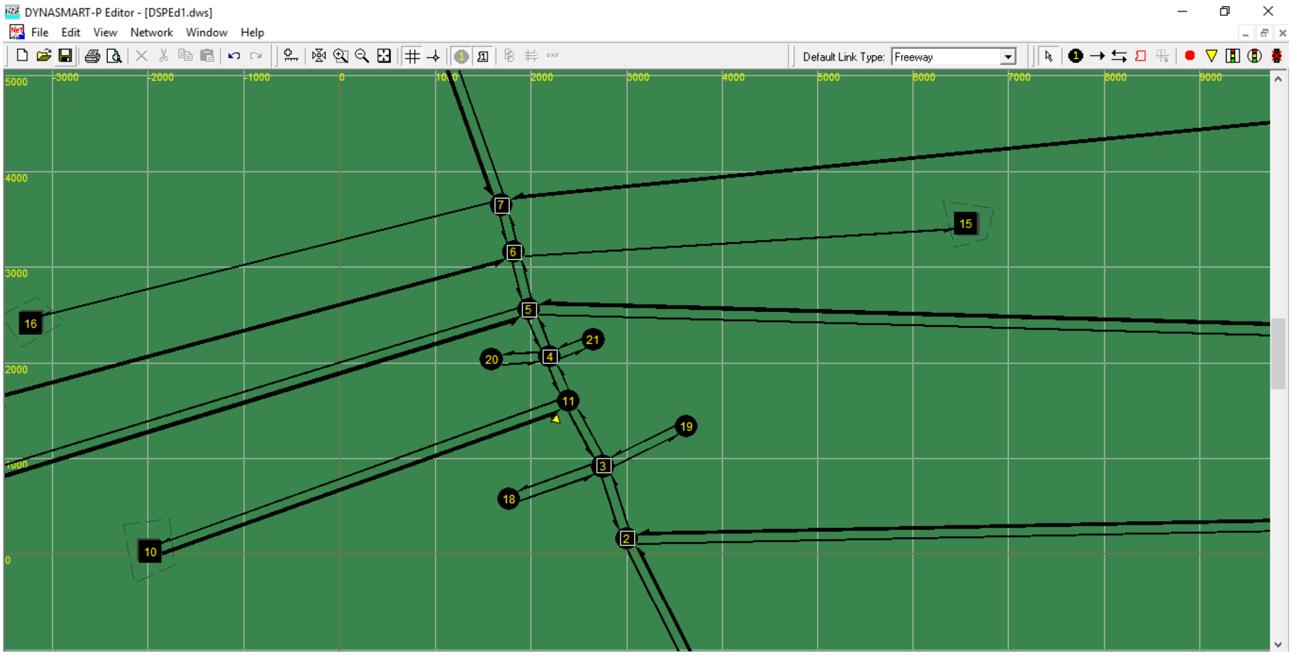


Figure 96: DYNASMART network

As the traffic network in exam is very congested, in order to make possible the entrance of all vehicles, the links with heavy traffic volume needed be extended as in following figure.

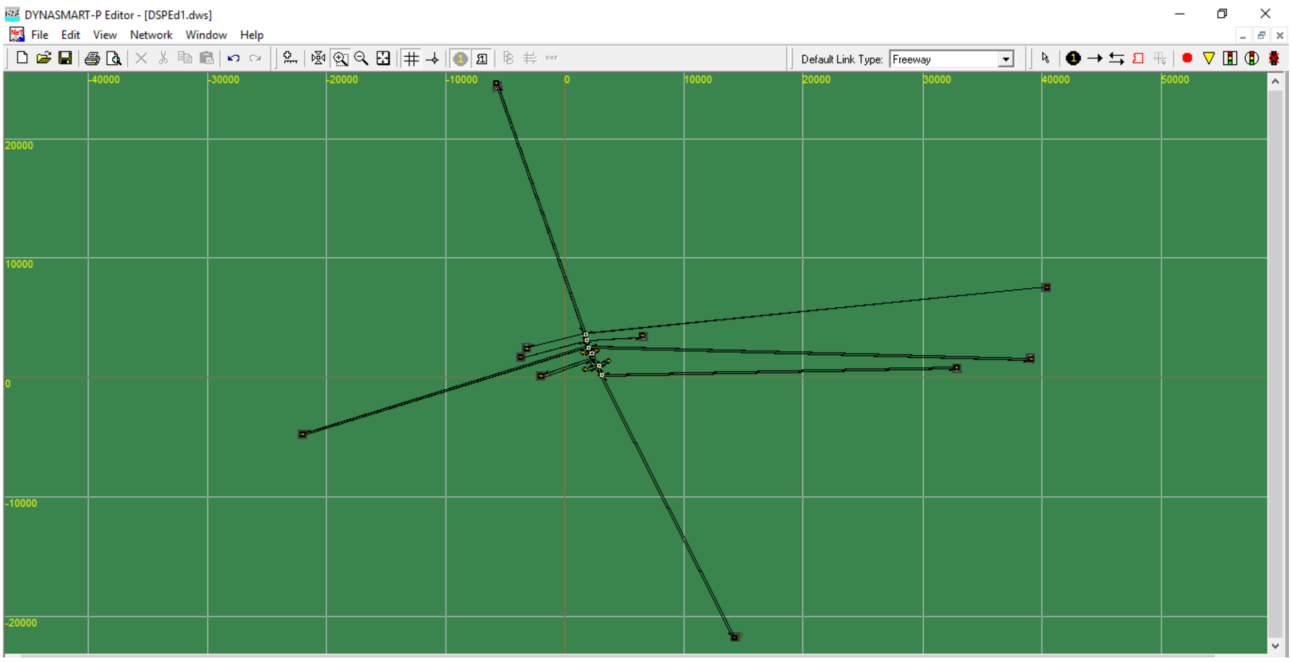


Figure 97: DYNASMART network with extended links

The next step was a creation of Origin Destination matrix for every 15-minutes vehicle volume counts. One of this OD tables for the volume measured from 16.45 until 17.00 is presented below. The volume of every origin was distributed with proper percentages to every destination.

Table 11: Origin Destination Matrix

OD matrix	zones	1	2	3	4	5	6	7	8	9	10
zones	nodes	1	8	9	10	12	13	14	15	16	17
1	1	0	303	9	0	28	23	0	45	24	0
2	8	492	0	17	0	27	33	0	0	32	0
3	9	0	16	0	0	2	1	0	2	1	0
4	10	0	0	0	0	0	0	0	0	0	0
5	12	38	29	1	0	0	7	0	4	2	0
6	13	111	86	4	0	95	0	0	13	7	0
7	14	31	24	1	0	2	2	0	21	2	0
8	15	0	0	0	0	0	0	0	0	0	0
9	16	0	0	0	0	0	0	0	0	0	0
10	17	146	16	5	0	8	10	0	0	21	0

Using the video data heavy vehicles was counted and its fraction was added to the flow in the model. Once DYNASMART model was fully defined it was calibrated the vehicle, the roadway, and intersection parameters so that the simulation would reflect real-world measurements as closely as possible. All times periods from 15.00 until 21.00 was simulated with appropriate timing plan. DYNASMART-P provided the ability to simulate over alternate timing plans to address transition times. It allows for realistic results based on actual traffic conditions.

The calibration efforts resulted in DYNASMART-P shows the network more congested than in real-world conditions on the major cross-streets. Among the behaviours and situations, that model cannot introduce or accurately recreate are red signal, parking manoeuvres, pedestrians, bicyclists or public transport. Taken collectivity, none of the above factors reflects appropriately the real traffic flow on Marconi artery and its cross-streets.

To the model that was calibrated to be representation of reality, was loaded the new timing plans and was done several simulations to test each one. The DYNASMART-P software has the possibility to run simulation and change the timing plans at the proper period of time. The model in this situation calculate also delay regards to transition from one timing plan to another.

Using the obtained values for traffic responsive variables and actual traffic scenarios for the selected day, timing plans implemented during six hours were simulated and evaluated.

Figures below show the comparison of Time of Day plan versus TRPS plan for weekdays. These simulation runs showed stable performance and smooth transitioning. The performance measures associated with these plans are shown in this section. The first figure represent comparison of average times per vehicle for different modes in different period of time. In the last row all six hours performance are presented. The DYNASMART simulation has demonstrated the improvement of 19% with introduction of TRPS mode compared to TOD mode.

	Average travel time/ vehicle [min]			Improvement [%]
	Mode	**TOD**	**TRPS**	
	Cycle Length	**Plan 3**	**Different Plans**	
Time	**15.00-15.30**	8,59	8,54	1
	15.30-17.00	14,08	13,4	5
	17.00-18.45	14,69	12,53	15
	18.45-20.00	11,92	11,04	7
	20.00-21.00	9,35	8,98	4
	15.00-21.00 (6h)	42,18	34,15	19

Figure 98: Comparison of average travel time per vehicle in TRPS and TOD mode

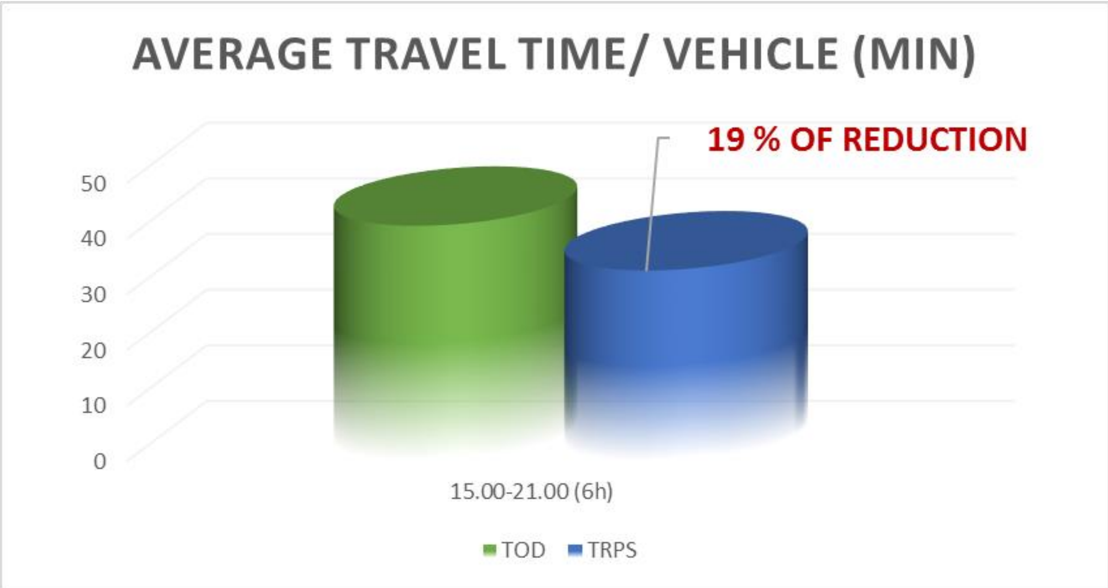


Figure 99: Benefit of TRPS mode

The next figures, directly taken from DYNASMART software, show changes in average speed and average travel time during 360-minute period. The first two represent parameters of TOD mode, on the contrary, the next ones illustrate TRPS mode.

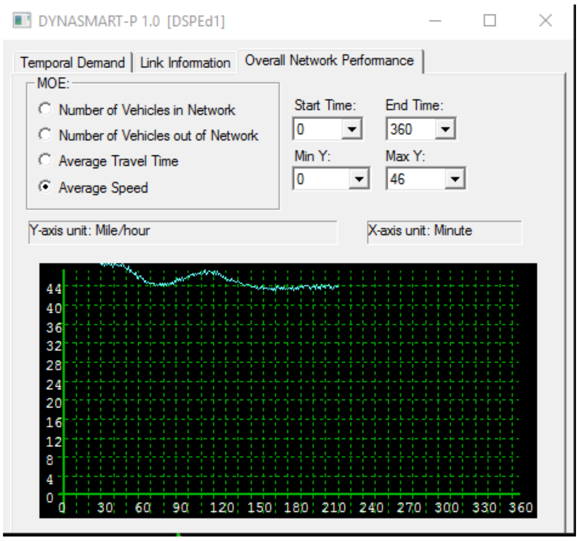


Figure 100: Average speed in TOD mode

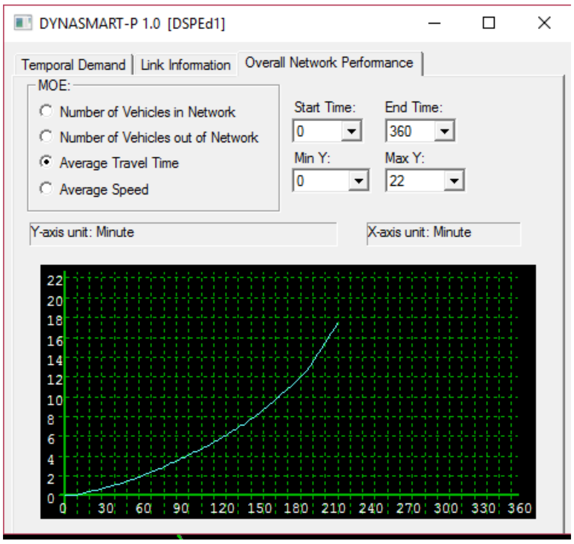


Figure 101: Average travel time in TOD mode

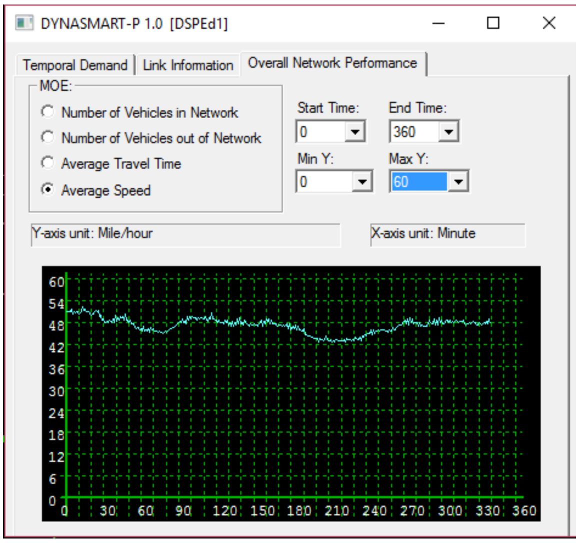


Figure 102: Average speed in TRPS mode

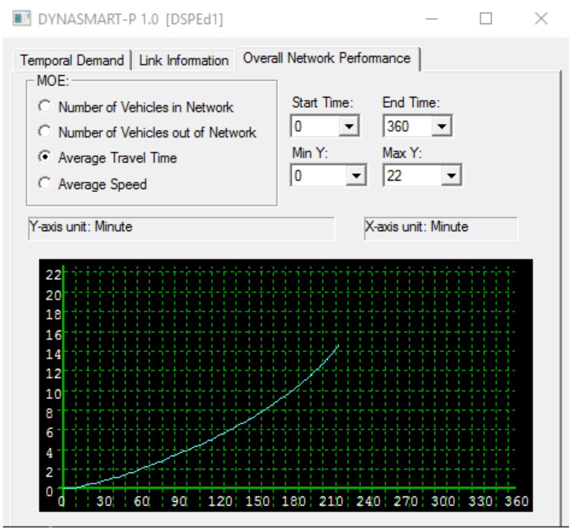


Figure 103: Average travel time in TOD mode

The followiwng figures show the vehicle presented in network during the simulation in both modes. How it is possible to observe, the newtork with optimized timing plans is less congested than in TOD mode.

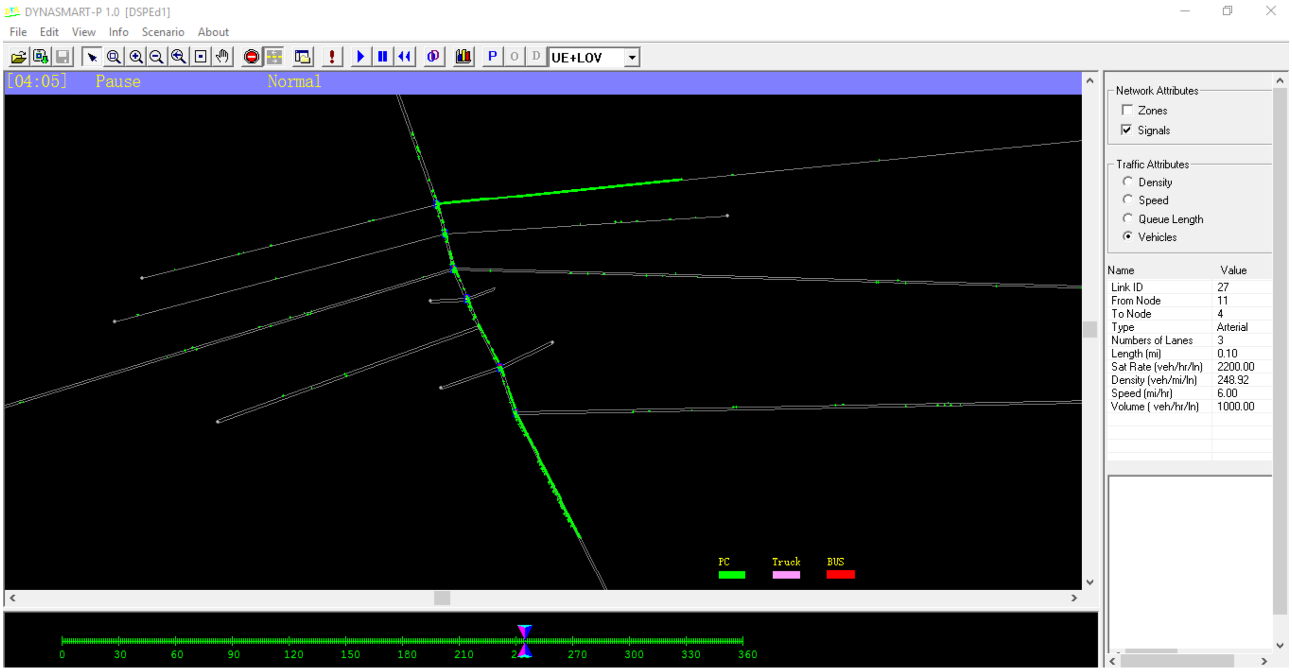


Figure 104: Vehicles in network simulated with TOD mode

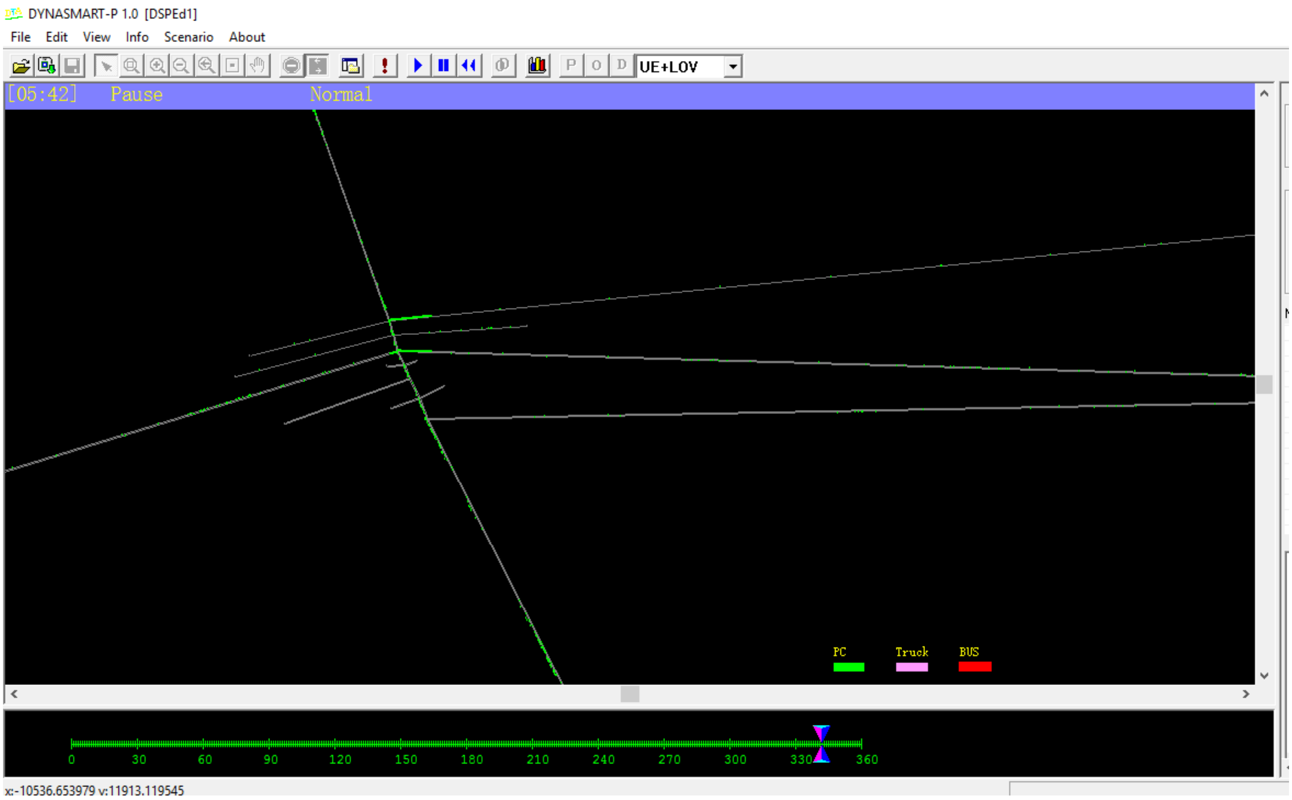


Figure 105: Vehicles in network simulated with TRPS mode

The next figures present output report from simulation with DYNASMART-P software.

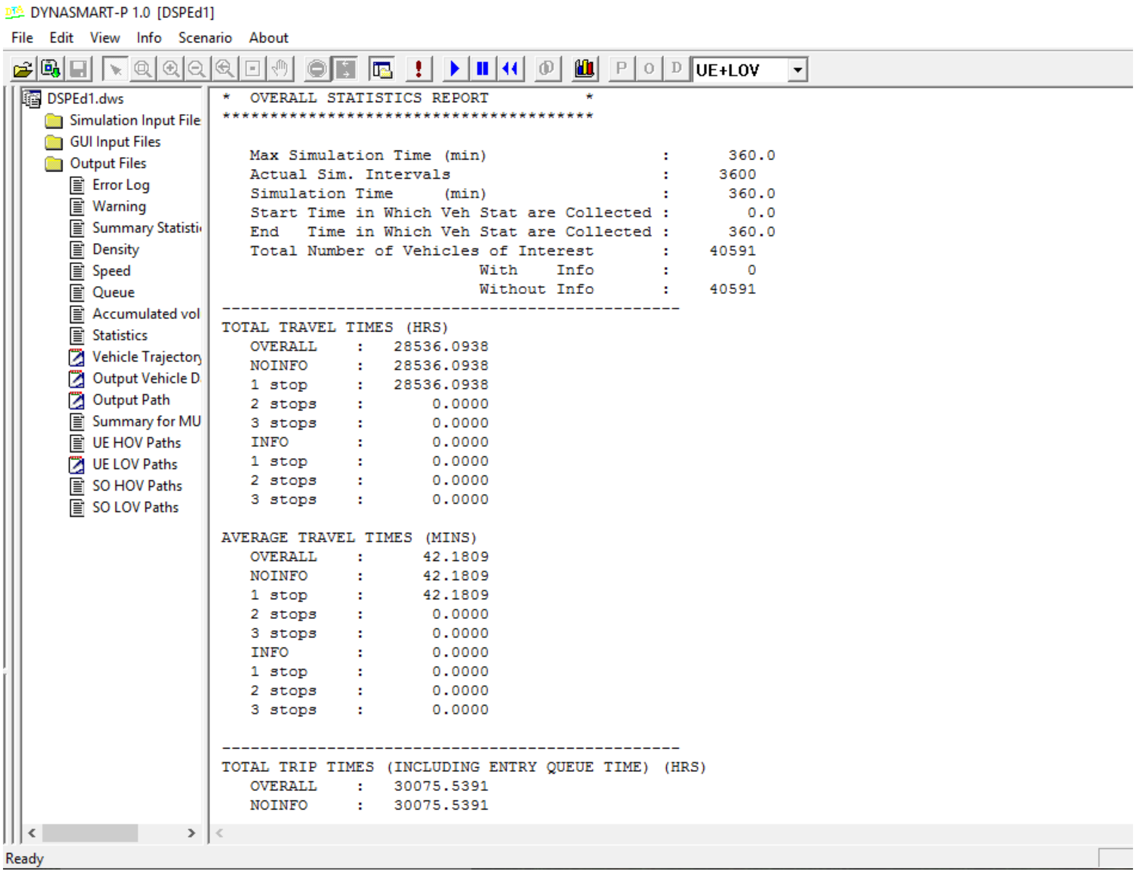


Figure 106: TOD output report

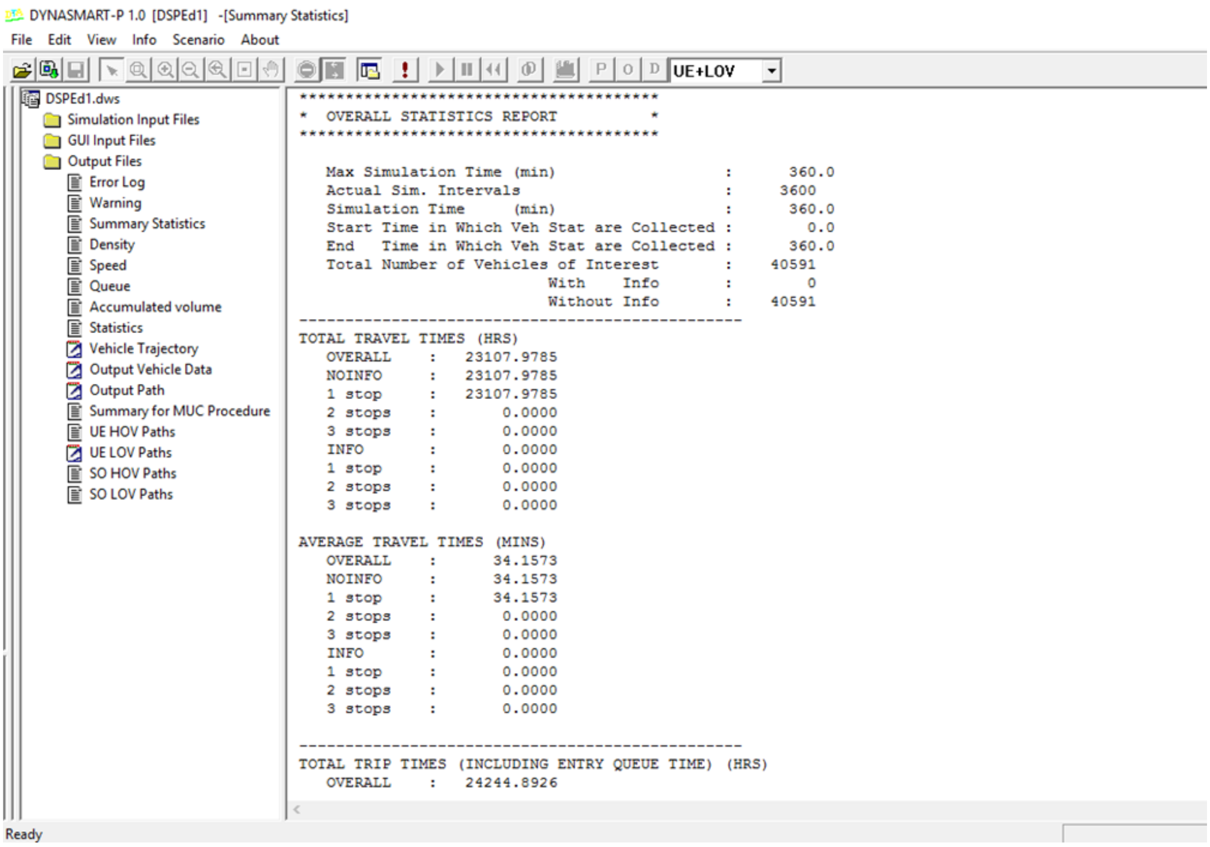


Figure 107: TRPS output rep

Chapter 9

CONCLUSIONS AND RECOMMENDATIONS

9.1. CONCLUSION

Closed-loop traffic control systems can be operated by either Time of Day mode or Traffic Responsive Control mode. TRPS has the greatest potential to provide an optimal operation if properly configured. However, there are very limited guidelines to configure a TRPS system for optimal operation.

TRPS mode provides a mechanism by which the traffic signal system is able to change timing plans in real-time in response to variations in existing traffic conditions. TRPS mode can achieve significant results due to its ability to accommodate abnormal traffic conditions such as incidents, holiday traffic and special events.

The most important benefit of TRPS mode is reduction of the need for frequent redesign/updates of signal timing plans.

A methodology for design and evaluate TRPS mode has been conducted in this thesis, by introducing a strategy to cluster and detect traffic conditions through pattern recognition techniques. For the scope of this research traffic responsive control systems was studied along Marconi Street.

A main step approach was used, namely:

1. Detection of existing traffic data

2. Identification of demand states using K-means clustering;

3. Design of optimum timing plan for each traffic scenario using SYNCHRO 8;

4. System validation by SimTraffic and DYNASMART-P simulation to compare the TOD and TRPS performance in Guglielmo Marconi arterial network.

This research illustrates design of the TRPS system for Guglielmo Marconi Street in Rome. This closed-loop system consisted of six signalized intersections in an urban setting with highly variable traffic demand levels and patterns. The Guglielmo Marconi artery was used to illustrate the optimization procedure required for selecting optimal timing plans for the overall system.

Essentially, this thesis provides a systematic description on how to set up TRPS mode after detecting and collecting 15-minutes traffic volume data. Measurements of traffic flow at the intersection was performed with SmartEye video detection system. Video detector data can be utilized to simplify the process of timing plan development, as well as to allow a means for constant feedback on performance of signal timing plans. For the scope of this thesis only two days data from system detectors has been processed to further analysis. For real application of TRPS mode, it is recommended that cluster analysis and timing plan development based on historical database of traffic volumes. With years of data available, it may be tempting to construct an overly detailed analysis. However, traffic data should not go too far back in time in order to not distort the results. Traffic varies over time and an approximate historical cut-off for data collection should be determined such that the results are not influenced by out-dated traffic trends.

The K-means cluster methodology was proposed in this thesis in order to group together similar vehicular volumes. This research investigate different types of input data for cluster analysis in order to get the maximum efficiency and the best separation of groups. The performance of these various input types shows that the traffic stream normalized by the corresponding flow ratio produce the cleanest TOD intervals from the clusters. The major advantage of relying on machine learning and clustering instead of trivial thresholding is that it is possible to gather a much more meaningful information, because several features participate to the detection of a specific scenario. Since such an approach applied to traffic control is still not so common, this should be consider as a major contribution of this thesis.

Based on sensitivity analysis illustrated in this thesis, it is recommended that a minimum number of observations should exist in each cluster and should be applied to the cluster algorithm. This produces substantial clusters that exist for sufficient times, in order to be supported by an entire timing plan. Otherwise, clusters may be developed based on one or two erroneous observations that cannot be supported by a timing plan and thus take away from the refinement of the remaining clusters during the 24-hour period.

The cluster analysis resulted in a selection of six timing plans to be used with the TRPS mode. The approach discussed in this thesis illustrate that only a few timing plans are needed for the subset of all traffic networks that share the same characteristics. Once the timing plans for certain network have been identified by Synchro 8, behaviour of TRPS mode needs to be simulated and evaluated.

TRPS mode has to ensure that the most suitable plan in the controllers' database is selected to match the existing traffic conditions. Pattern matching mechanism called Nearest Cluster and using Euclidean distance, was implemented in Matlab to classify newly detected traffic volumes to the most similar cluster and give the information with the best-suited signal timing plan to apply. The proposed approach is able to produce good system parameters, which consequently achieve good traffic responsive system.

In comparison with TOD mode, TRPS bring up the most suitable timing plans for the existing traffic condition. Instead, TOD is limited to bringing up timing plans according to a fixed time schedule regardless of the existing traffic condition. To conduct a fair comparison between the TRPS and TOD mode, the six hours simulations with actual and designed timing plans was executed. It was necessary to predict the total delay expected from implementing a TRPS mode and actual delay connected with TOD mode.

Simulations show 19% reduction of average travel time per vehicle with TRPS mode in comparison to TOD mode. Moreover, total delay, numbers of total stops and fuel consumption also decrease.

The implementation of traffic responsive control mode in the traffic network improves the overall system performance.

9.2. RECOMMENDATIONS FOR FUTURE RESEARCHES

Further researches that are believed to be a good potential for traffic responsive control mode of operation are pointed below:

- In this thesis, only pretimed control at the intersection was evaluated. However, in further study it is required to determine the effect of pedestrian on TRPS networks and possibility of introducing pedestrian calls or pedestrian phases along the artery. This can have a great effect on pedestrian safety as well as overall system performance.

- It is recommended to perform a research comparing the pattern matching mechanism and threshold mechanism. It is believed to have one system performs better than the other does. Limited studies were conducted to show which system is better.

- All the research performed studying traffic responsive control mode including the work presented in this thesis consider only arterial networks. It is required to have a research about implementation of traffic responsive control mode of operation on system networks.

- It is necessary to perform a research on the methodology to reduce misclassification of traffic state and reduce the frequent transition of timing plans.

- Development of computer software for TRPS configuration for a particular syste

FIGURES

TABLES:

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