DOCTORAL DISSERTATION

博 士 論 文

A STUDY ON UNCONVENTIONAL ARTERIAL INTERSECTION DESIGNS (UAIDs) APPLICABILITY BY DEVELOPING COORDINATION ALGORITHMS UNDER THE HETEROGENEOUS TRAFFIC CONDITIONS

混合交通環境における交錯除去交差点の適用可能性とその系統制御アルゴリズム開発に関する研究

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March 2019
2019 年 3 月

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by

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A Dissertation Submitted to the Graduate School of Urban Innovation, Yokohama National University in Partial Fulfillment of the Requirement for the Degree of

Doctor of Philosophy in Engineering

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March 2019
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ABSTRACT

The primary aim of this research is to provide transportation experts; and traffic analysts with an objective assessment on the possibility and feasibility of implementing Unconventional Arterial Intersections (UAIDs) as proposed alternative schemes to alleviate the corresponding congestions at signalized intersections in the developing, semi-industrialized and industrializing cities around the world where the heterogeneous traffic conditions are dominant. Despite the deployments of UAIDs which have been advocated as innovative treatments for the last decade to address congestions at conventional intersections in the developed world where the less complex traffic operations exist, these schemes have never been estimated under the heterogeneous traffic. Hence, this research investigates the applicability of such alternative intersections under heterogeneous traffic complexities as prevailing and dominant conditions in developing nations around the world to gain a better understanding of the performance, various functions, benefits, vulnerabilities and limitations of such designs. These complexities are characterized by the diversity of some static and dynamic properties of vehicles, aggressive driving behaviour and the non-lane based traffic system. In this research context, two UAIDs schemes namely; the Displaced Left-Turn (DLT) intersection and the Restricted Crossing U-turn (RCUT) intersection are proposed and evaluated. In order to provide credible and reliable results, three consecutive conventional signalized intersections located in an arterial corridor of downtown Cairo, the capital city of Egypt were selected to represent a realistic case study. The selected intersections currently suffer from significant delays, long queues and low average speeds as an indications of a chronic traffic dilemma.

Seeking a comprehensive assessment of such designs, this study is built based on two main pillars; an operational performance evaluation and an economic analysis. The operational performance evaluation aims to assess the different traffic operational functionality measure of effectiveness indices such as intersection throughput, total travel time, average delay per vehicle, average stopped delay per vehicle, average speed, queue length and the number of stops of each vehicle along the studied corridor. To accomplish such a comprehensive operational evaluation, a before-and-after study follows to examine the effectiveness and practicability of the different proposed alternatives. In this study, the methodology enhancement is achieved by employing VISSIM, as a powerful microscopic simulator-based platform to simulate the different configurations as closely as possible to the reality. Based on psychophysical car-following models, VISSIM has been broadly used by many practitioners

and researchers as one of the best-suited tools, as well as a cost and time effective analytical approach to model, analyze and evaluate numerous traffic schemes before the field deployments. Moreover, based on the lane-by-lane development road networks facility, VISSIM allows the construction of UAIDs exactly as they would appear in real life. In addition, as a stochastic, time-step and behaviour-based model, many practitioners and researchers have used and recommend VISSIM to fulfil the heterogeneous traffic complexity needs, especially, the aggressive driving behaviour and the non-lane based system.

Furthermore, this research develops coordination algorithms for the proposed UAIDs, particularly for the DLT intersection, depending on the two common coordination techniques: the bandwidth maximization and the delay minimization technique. The bandwidth maximization is employed as a pre-timed (fixed-time) coordination approach, while the delay minimization approach is utilized to develop a real-time demand-responsive signal control algorithm on the solid foundation of the dynamic optimization principles. This entire demand-responsive algorithm is built based on driving a mathematical model that is interpreted by MATLAB as a multi-paradigm numerical computing environment. A MATLAB programming script is coded to develop the real-time demand responsive signal control algorithm to examine the DLT intersections as a coordinated corridor. Utilizing VISSIM-COM interface as an inter-process communication allows for manipulating the attributes of most of the internal objects dynamically. Although academic in nature, the presented algorithm in this context is adaptive through a real-world practical application.

The main findings of this study emphasized the possibility of implementing the proposed UAIDs under such heterogeneous traffic conditions in an urban area where the same right-of-way is usually limited. However, special attention must be paid to a required modification in both geometric and signal controller designs to fulfil the heterogeneous traffic needs. Based on the simulation results, the superiority of the DLT and RCUT designs proposed in this study was recognized having overwhelmed their conventional counterparts. All performance indices experienced a significant improvement referring to an obvious enhancement at the level of service (LOS) at each intersection along the studied corridor. It was found that the proposed UAIDs schemes reduced the overall delay and the total travel time while the average speed was increased significantly. The outputs revealed that DLTs consistently experienced better results and overcame over the existing conventional intersections as well as the RCUT design for the three intersections studied. However, it also can be concluded that the heterogeneous traffic complexities (i.e. the diverse dynamic and static properties of mixed traffic compositions, the non-lane based system, the aggressive driving behaviour…etc.) obviously influenced the proposed UAIDs geometric design as well as the operational efficiency. Likewise, the simulation findings emphasized that the RCUT design is not appropriate for high traffic levels, similar to the previous studies findings.

The Cost-and-Benefit Analysis (CBA) as an economic assessment approach is utilized to conduct a feasibility economic analysis in which the different costs and benefits' components associated with the implantation of the proposed UAIDs intersections are defined and formulated in order to produce enough information of the proposed alternative schemes and to ascertain whether these designs should be undertaken as cost-effective treatments. From purely a financial point of view, the CBA helps the policy-makers to assess the value of a transaction or a decision aiming to estimate the strengths and weaknesses of different alternative projects in order to decide the best proposal and which alternative should be considered to accrue the most beneficial sense. Accordingly, a final selection that offers the greatest overall benefits for incurred costs is considered. Hence, the costs and benefits components are estimated for each proposed treatment including: the conventional at-grade intersection as base-case and the grade-separated intersection (selected as overpass flyover for this study), the DLT and the RCUT intersections as proposed treatments. The cost considered in this study is classified into three major components: the construction cost, the running cost and the maintenance cost. On the other hand, the benefits that redound to a reduction in a user's trip cost as a result of the traffic condition improvement along the studied corridor are considered as users' benefits. Hence, the users' estimated direct benefits result from savings in operation costs; the travel delay cost savings and the fuel consumption cost savings due to the implantation of the proposed treatments comparing to the base-case. The indirect benefits, however, include the CO_2 emissions cost savings due to the excess fuel consumption as an environmental impact.

It can be concluded, based on the CBA results, that the UAIDs are economically effective compared to the grade-separated intersections as recommended alternatives to alleviate the congestions at the conventional at-grade intersections. The results emphasized significant expected cost savings as potential benefits due to the implementation of the different proposed alternatives. The Benefit Cost Ratio (BCR) revealed the efficiency of the two proposed UAIDs intersections; DLT and RCUT, over the grade-separated intersection.

ACKNOWLEDGMENT

First and Foremost, I would like to express my sincere gratitude and deepest thanks to *THE GOVERNMENT OF JAPAN*, particularly, *THE MINISTRY OF EDUCATION, CULTURE, SPORTS, SCIENCE AND TECHNOLOGY* for granting me this scholarship, which enabled me to accomplish my doctorate degree at the Graduate School of Urban Innovation, Yokohama National University, Japan.

My greatest appreciation goes to my academic supervisor *Dr. Shinji TANAKA* for his excellent guidance, appreciated patience, continuous motivation, enthusiasm and immense knowledge. Sincerely, I owe my deepest gratitude and express my cordial appreciation to Dr. *TANAKA* for his exceptional experiences that always inspire and enrich my growth as his student.

I am pleased to acknowledge and express my warm and sincere thanks to Dr. Mohamed El Esawey and Eng. Nada Ibrahim, the department of civil Engineering, Ain Shams University, Egypt for offering the detailed traffic counting data which it was impossible to accomplish this work without their help.

Despite the distance, I would like to record my warmest gratitude and hearty thanks to my parents for their love, unflinching encouragement and continuous support that they have always shown to me. They were always there supporting, praying and encouraging me with their best wishes for more than 25 years during my education journey.

Indeed, words cannot truly express my appreciation and gratitude to my wife *Mona M. Sadek*. A special thanks and appreciated greetings should go for my wife as she was always there supporting me, cheering me up and sharing the good and bad times with me. I am really indebted to her more than she might know.

Finally, by finishing these three years of postgraduate higher education, I consider getting this degree is just a milestone towards a greater future. I obviously realized that the more knowledge someone may get, the more shortening the one may realize. Hence, by the end of this challenging journey, I humbly confirmed that there is still plenty I have to know more.

Last but not the least, because it is impossible to mention all those who should be gratitude, therefore I would like express my gratitude to everyone who guided, supported and helped me even by a word to accomplish this research. Also, kindly I would like to ask those who have not been mentioned here to forgive me and to accept my genuinely praise.

THANK YOU ALL

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1

INTRODUCTION

1.1 Background

As a result of the rapid urbanization and motorization growth, the urban communities around the world have been facing by continuously and dramatically traffic demands with corresponding congestions at existing signalized intersections. The supply of highway facilities in many different countries around the world has not kept pace with automobile growth. According to a report published in 2014 by the Asian Development Bank, the vehicle population were doubled every 5 to 7 years in Asian urban areas. Also, in the USA the average level of motorization reached 300 vehicles per 1000 people in the middle of the 20^{th} century (Reinis Kivlins et al., 2011). As a result, the existing limited urban traffic infrastructure has not been able to accommodate the explosive growth in vehicular volumes. Accordingly, the generated traffic congestions have contributed to worsen operation and safety problems with high fuel consumption. Constrained by the limited resources, transportation agencies and experts have been challenged by a number of transportation challenges as a result of the rapid and continuous travel needs. The high traffic volumes pose resultant congestions at signalized intersections with primary side effects include increased driver stress levels and greater economic losses in terms of wasted times. Similarly, in Africa, the New Partnership for Africa's Development (NEPAD) referred in its study that in 2006 there were about 20 million road vehicles both public and private, of which Northern Africa for 9.0%, Central Africa accounted for 2.0 %, Eastern Africa for 11.0 %, Southern Africa for 58.0% and 21.0% in the Western Africa (United Nations Economic and Social Council: Economic Commission for Africa, 2009). As a result, the total travel time of the urban networks was influenced dramatically. It costs around 2.0-5.0% of Gross Domestic Product (GDP) due to wasted time and higher transport costs (Asian Development Bank, 2014), In the United States, the congestion cost per traveller has been raised from 290.0 USD in 1980 to 757.0 USD in 2007.

Likewise, the delay has increased more than 160% over the past 25 years (Texas Transportation Institute, 2010; Suh, W., and Hunter, M. P, 2014). Furthermore, the intersections' Level of Service (LOS) indicators such as the overall delays, as well as the intersections throughputs, experienced adverse considerable impacts. As traffic volumes grow and congestion worsens, road users: motorists, pedestrians, and cyclists confront greater risks at intersections. Moreover, in another report by the Asian Development Bank, it was expected that around 1.1 billion more people would be living in urban areas by 2030 than in 2005 (Asian Development Bank, 2008). Apparently, it's clearly translating to a higher demand for roads. Thus, in order to address the ever-increasing traffic demands, and to increase the capacity of existing infrastructure, widening the existing roads or building additional pathways was proposed as a traditional treatment and usual approach. However, these treatments require high capital costs that it may not be easily available, particularly, in the developing countries, as a targeting category of this research context. In the United States, for adding new lanes, the Texas A & M Transportation Institute emphasized that it costs between 2-10 million USD per lane-mile of a freeway and 750,000 USD per lane-mile of surface-street (Texas A &M Transportation Institute, 2013). In the new and developing urban areas, the good coordination between adjacent land owners and urban developers, particularly, the transportation planners may provide a long-term solution for the traffic challenges. For the already developed areas, however, this coordination cannot provide a sufficient, fast and relief solutions for short-term periods (Reinis Kivlins et al., 2011). Hence, improving the operational and safety performance of the existing intersections has become a considerable interest worldwide. Therefore, the need to find a novel solution that leads to higher capacity, lower delays and fewer crashes for an efficient enhance of the operational and safety performance of the conventional intersections, has become highly required.

Aiming to cope with the ever-increasing traffic demands, transportation experts and analysts had to develop new approaches that can be generally categorized as traditional, grade-separation and unconventional approaches to alleviate the corresponding congestions at at-grade intersections. The traditional or conventional treatments are usually proposed based on improving the signal system such as using actuated signals, optimizing signal timing, or adding exclusive left-turn phases. In addition, other traditional schemes target increasing approach capacity by adding more through lanes or even exclusive turning pockets. However, under the rapid and continuous growth in the traffic demand, these traditional schemes are no longer able to considerably alleviate the corresponding congestions at at-grade intersections (Hummer 1998a; Hummer and Reid 1999; Dhatrak et al.2010). On the other hand, through a vertical separation of roadways, grade-separation intersections, principally, have been widely deployed to eliminate the crossing conflicts at intersections. Despite the higher capacities offered by these designs comparing to the at-grade counterparts, grade-separation alternative is considered a costly and aesthetically unpleasing (Goldbatt et al. 1994).

Recently, not only the less developed or developing countries but also the developed nations are facing a number of strategic challenges. As a result of the instability in the political consequences in many countries around the world, particularly after 2011 uprising, a combination of domestic challenges, together with instability in many countries, specially, in the Middle East and North Africa region has influenced the international economy in a significant way (EU, A stable Egypt for a stable region: Socio-economic challenges and prospects, 2018). Accordingly, the budget assigned to building and maintenance of transportation infrastructure becomes inadequate globally.

Over the last decade, in the United States, the Federal Highway Administration proposed several alternative schemes that are known as the Unconventional Arterial Intersection Designs (UAIDs) as a feasible vision for relieving arterials' congestions. The UAIDs have been advocated as innovative and balanced solutions that could emphasize a smooth traffic flow and improve mobility for all users. These alternative designs have been presented as innovative at-grade and grade separated intersection treatments to emphasize the vehicular LOS with comparable traffic volumes along the corridor. Typically, these proposed schemes share two fundamental concepts: facilitating the through traffic movements along the corridors, and reducing the conflicts between left-turn movements and the opposing through traffic by eliminating or re-routing one or more movements inside the intersection as an unusual movement structure (Hummer 1998a; Hummer and Reid 1999; M. El Esawey and T. Sayed 2013).

1.2 Problem Statement

Traffic congestion is an ongoing issue among the transportation industry in many cities around the world, particularly, in the developing cities where the limited resources. Although the conventional intersection designs have been in use for many years, there is a need to alleviate the traffic jams with the added congestion on the roadways. Therefore, the need for new roadways and innovative intersections has become a high priority. However, the main finds of the additional lane efficiency at signalized intersections revealed that conventional methods of improving intersections by adding new lanes have diminishing results. Adding one additional through movement lane may extend the effective operational lifespan of an intersection by 15 years, while this lifespan might be decreased for the second additional lane to 10 years and 6 years for the third additional lane (Hummer 1998a; Reinis Kivlins et al., 2011).

The urban sprawl in the Greater Cairo Metropolitan Region (GCMR), as a selected case study of this research, is growing rapidly as shown in Figure 1.1. Accordingly, the region is suffering from a significant growth of rapid motorization. The Central Agency for Public Mobilization and Statistics (CAPMAS), the official statistical agency of Egypt, emphasized in its report that the total number of vehicles has been dramatically increased as shown in Figure 1.2. With a 44.4% increased rate of the total nation's vehicles population, the total number of vehicles recorded 3.5 million vehicles in 2013 (Central Agency for Public Mobilization and Statistics (CAMPS), 2013). Accordingly, the corresponding travel needs and the total daily trips increased obviously. However, the limited existing urban networks could not accommodate the huge corresponding traffic demands and results in a chronic traffic congestion dilemma. The World Bank published report concluded that the intersections in Egypt are suffering from a lack of intersection management. The operational performance indices,

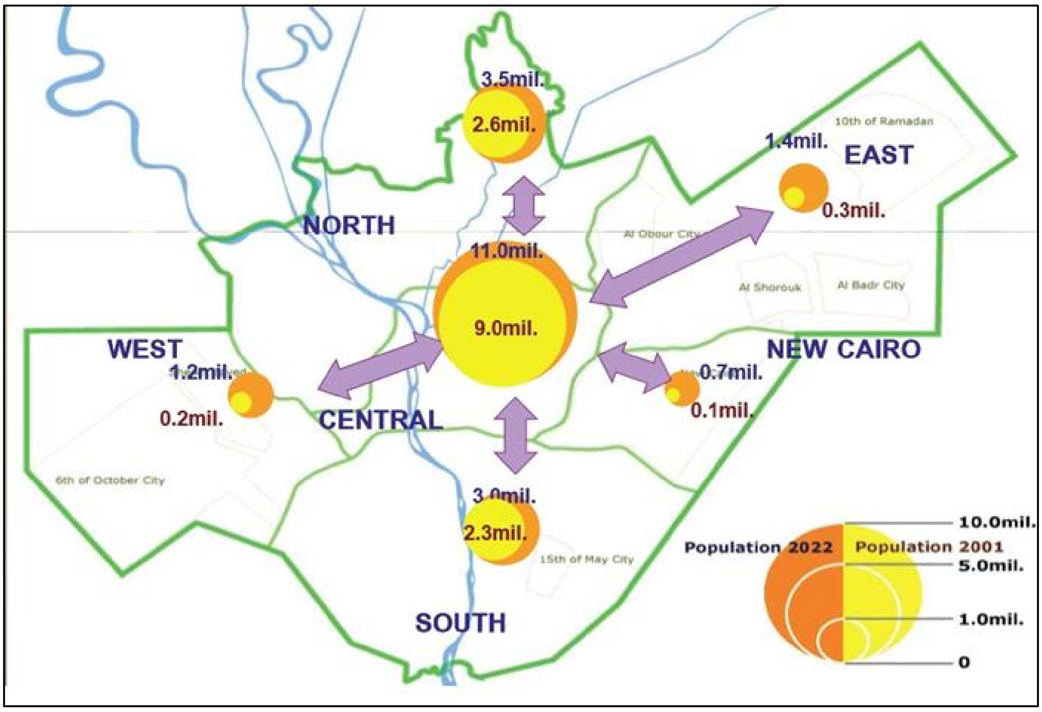

Figure 1.1 The Urban sprawl in GCMR (JICA, 2001)

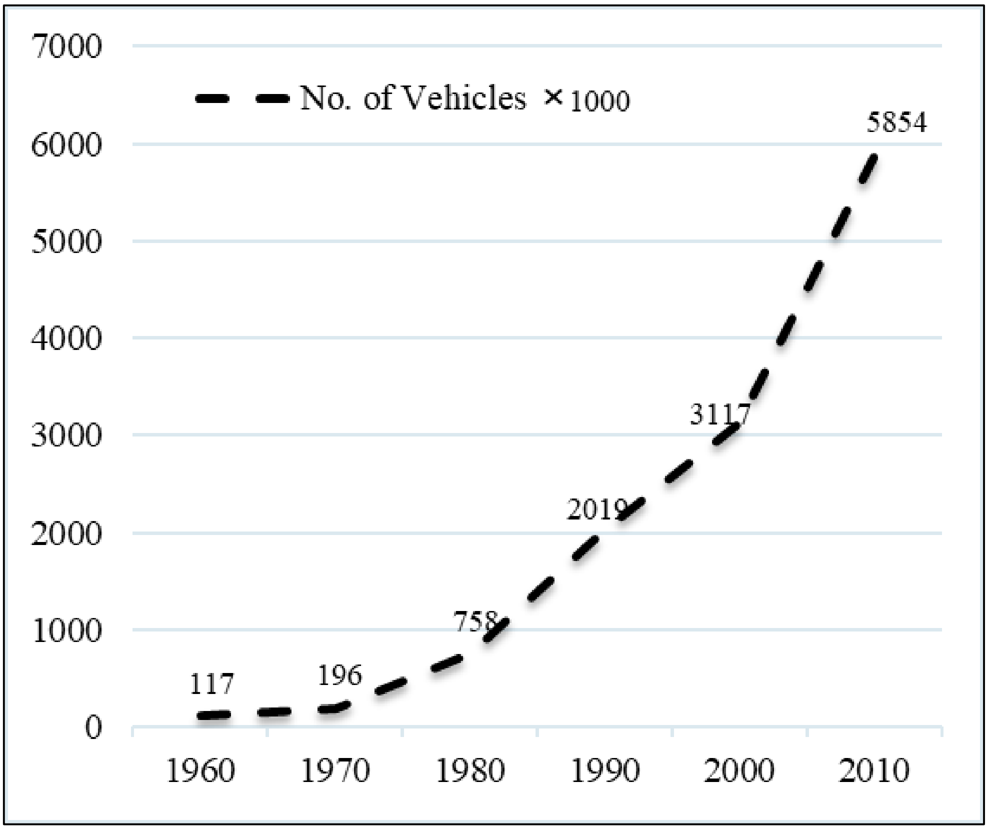

Figure 1.2 The rapid motorization in Egypt (CAMPS, 2013)

as well as the vehicular LOS indices such as the saturation flow rates, the start-up lost time and clearance lost time, were adversely influenced in central Cairo. For instance, the saturation flow rates of the existing intersections also were influenced. Downtown Cairo, the previous findings emphasized the saturation flow rates as 1617 PCU/h/ln, however, the average range of saturation flow rates within a range between 1700 and 2080 PCU/h/ln worldwide (High Capacity Manual 2000, High Capacity Manual 2010). In addition, the average speeds were dropped by at least half and recorded 15 to 40km/h of the normally expected speeds 60 to 80 km/h (D. Branston and H. van Zuylen; Siddharth et al. 2013; Muhan et al. 2013). As the most widely adopted treatment, the Restricted Lefts/Through U-turns have been applied in Egypt over the last decade. The main reason of proposing such possible replacement is the excessive delays encountered at most signalized intersections due to the poor design, the operation of signal controls and consequently lack of drivers' compliance (Elazzony et al., 2011).

On the other hand, this serious dilemma directly affects the nation like the impact on the safety conditions, or indirect way as the nation's economic productivity. The World Health Organization, in its published report, affirmed that Egypt loses about 12,000 lives annually from road traffic accidents, and more than 154,000 injuries occurred as a result of road accidents (World Health Organization, Milestones in International Road Safety Report 2005; World Health Organization, Global Status Report 2009; World Health Organization, EGYPT: A National Decade of Action for Road Safety 2011). Similarly, the World Bank reports confirmed that Egypt loses about 8 billion USD/year, up to 4% of its GDP as a yearly economic cost of traffic congestion dilemma (World Bank, Cairo Urban Transport Note, 2000; World Bank, Cairo Traffic Congestion Study Phase 1, 2010).

In spite of the deployments of several grade-separated intersections and flyovers over the last decades in many locations around the GCMR aiming to alleviate the traffic dilemma as an effective treatment for higher capacities and better safety conditions, it became undesirable alternative because of the current economic consequences in Egypt, especially, after the 2011 revolution. Accordingly, the budget assigned to transportation infrastructure becomes inadequate with a raise in annual inflation rate that recorded 34.2 % in 2017, with transportation costs rising by 36.7 % as the currency lost more than half its value because of the Egyptian pound floating (European Union, A stable Egypt for a stable region: Socio-economic challenges and prospects, 2018).

1.3 The Heterogeneous Traffic Conditions

In some parts of the world, practically, in the developing, semi-industrialized and industrializing cities, the traffic systems are dominated and influenced by the heterogeneous conditions. The uniqueness of such systems results in a complex movement of travelling vehicles. The terminology of traffic heterogeneity in this study refers to the complexity of the traffic system that occurs due to the wide variations in the operational performance characteristics of such systems compared to the less complex homogenous traffic conditions in the developed cities around the world. These variations include the big variety in vehicle types or traffic compositions (i.e. normal vehicle, motorcycles, trucks…etc.), the diverse vehicles static (i.e. length, width, heights…etc.) properties with corresponding numerous dynamic (i.e. desired speed, acceleration and deceleration…etc.) characteristics. Furthermore, as a distinguishing, prevailing and a dominant feature of such conditions, the aggressive driving behaviour is playing a considerable role in the heterogeneous traffic performance. Under such behaviour, a unique decision-making process is taken, especially, for overtaking and passing movements when a fast-moving vehicle follows a slow-moving one (Khan and Maini 1999). In addition, in the case of the lane markings absence, the aggressive behaviour, as well as the lane disciplinary behaviour, result in creating a non-lane based traffic system (Khan and Maini 1999; Mathew and Radhakrishnan 2010). Unlike the lane based system, where a driver takes the lane changing decision only if it is possible to perform a complete maneuver in one attempt, under the non-lane system drivers could change lanes freely depending on the available opportunity. In the non-lane based system, because of the non-segregation lanes by neither vehicle types nor directional flow, all vehicles travel in the same right of way simultaneously. Thus, vehicles may occupy any position across the road based on the available space without any restriction on positioning at any place across the link width (Khan and Maini 1999; Mathew and Radhakrishnan 2010). Also, during the signal red time, vehicles queue at intersections based on optimum road space utilization following no rules. Meanwhile, the smaller vehicles (i.e. motorcycles, scooters, bicycles…etc.) maximize the inter-vehicle space to reach the head of the queue during the red time, while the heavy vehicles affect the operational performance of the existing intersections (Kaur and Varmora 2015).

Several studies main findings referred to the dramatic adverse impacts on the operational performance, and safety conditions due to the operating performance characteristics of the heterogeneous traffic system comparing to its counterparts in the developed world where the less complex homogenous traffic conditions (Khan and Maini 1999; Mathew and Radhakrishnan 2010; Kaur and Varmora 2015). As a result of the diverse abovementioned characteristics of such conditions, the intersections' different vehicular LOS indices such as the saturation flow rates, the start-up lost time and clearance lost time is impacted adversely.

1.4 Unconventional Arterial Intersection Designs (UAIDs)

As a result of the intense exponential growth of the traffic volumes and the limited capacity of the road intersections, there has been a considerable interest in alternative measures to improve the intersections performance. According to the Federal Highway Administration (FHWA) information guide for signalized intersections, the alternative intersection design schemes are categorized as: intersections reconfiguration and realignment treatments and indirect left-turn treatments. A dozen unique designs that make up the unconventional designs, UAIDs were proposed as more efficient than the conventional intersections and less expensive than road interchanges to mitigate the corresponding congestions in the conventional intersections. These designs are focused on enhancing the arterial roadways to carry large traffic volumes over a greater distance with a limited direct access to adjacent development. UAIDs scheme is deployed by minor geometric modifications in the intersections. Applying minor design modifications to the existing intersection by either lane marking, physical barriers or/and constructing additional movements bays, the configuration changes of UAIDs are achieved.

In order to provide the creativity needed to find innovations that will enhance the traffic flow, untimely leading to alleviation of congestion, UAIDs were proposed on three primary design principles, including:

1. Emphasizing the through-traffic movements along the arterials by increasing the green time allotment to arterial through movements;
2. Reducing the total cycle length by decreasing the number of signal phases at major approaches;
3. Reducing the total number of conflict points at an intersection and separate conflict points that remain.

Most of the UAIDs are designed based on removing the major conflicts between left-turning and opposing through movements by eliminating or re-routing one or more movements inside an intersection as an unusual movement seeking a fewer number of signal phasing to operate the intersection. Reducing the intersection signal phases to simple two-phase operations results in reducing the total lost time needed for clearance, which theoretically allows the signal time at the primary intersection to be more fully utilized. A reduction in signal control phases typically leads to decreasing the total cycle length, which leads to better through progression on the arterials. As a result, the operational performance indices such as the stopped delays would be minimized and the roadway operational functionality would be enhanced with corresponding higher capacities.

The UAIDs can be divided into two different types based on the elevation level as at-grade intersections and the grade-separated interchanges. The at-grade is designed and implemented on the same level of the existing networks to fit the urban road networks, while the grade-separated interchanges are constructed by elevating the cross-section on the upper level as in the high and expressways. Due to the consideration given to the urban network, especially, the applicability under limited networks, this research considers only the at-grade intersection. Despite the consideration of only at-grade intersections for this study, in the next subsections, some examples of both at-grade intersections and grade-separated interchanges are illustrated and discussed.

1.4.1 At-grade Alternative Intersection Designs

Based on the above-mentioned design principles and the various configuration changes provided, different types of at-grade intersections were created. Although these alternative designs share the fundamental concept of facilitating the through traffic flow and reducing the total cycle length, the treatment ways to reduce the major conflicts between left-turning and opposing through movements are different. Considering the indirect left-turn treatments, Median U-turn (MUT), Upstream Crossover (USC), Continuous Flow Intersection (CFI) that also known as Displaced Left-turn (DLT) intersection, Quadrant (QR) intersection, and Super-Street (SSM) intersection also known as Restricted Crossing U-turn (RCUT) were designed. These new alternative schemes share the same mechanism as providing unusual movement and re-route one or more movements inside an intersection. The following sub-section presents some UAIDs designs and describes the geometric and operational performance of such schemes.

As one of the most common used intersections, Median U-turn (MUT) intersection was designed as one of UAIDs to improve the traffic efficiency inside the conventional signalized intersections. Based on treatments of the left-turn movements, MUT designs are created as conventional MUT and unconventional MUT. In the conventional MUT, as shown in Figure1.3, the left-turn movements are moved to median crossovers downstream the primary intersection as partially or fully prohibition (Hummer 1998a). The left-turn movements partial prohibition allows only left-turns from the minor approach at the primary intersection. On the other hand, the full prohibition prevents all the left-turns at the primary intersections no matter from major or minor approach (El Esawey 2013). In the unconventional MUT, a non-traversable median is utilized to prohibit the minor traffic include the through and left-turns, from crossing the primary intersection as shown in Figure1.4. The through and

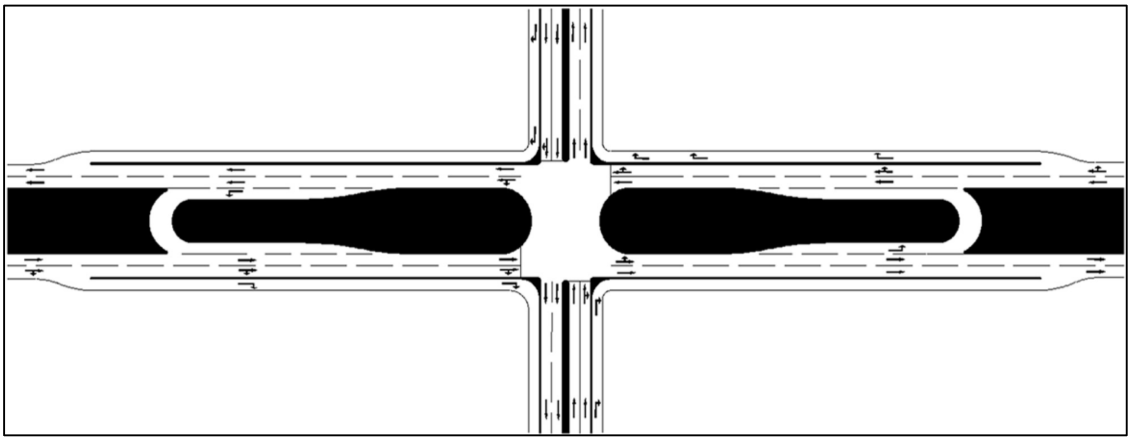

Figure 1.3 Conventional MUT Intersection (M. El Esawey and T. Sayed, 2013)

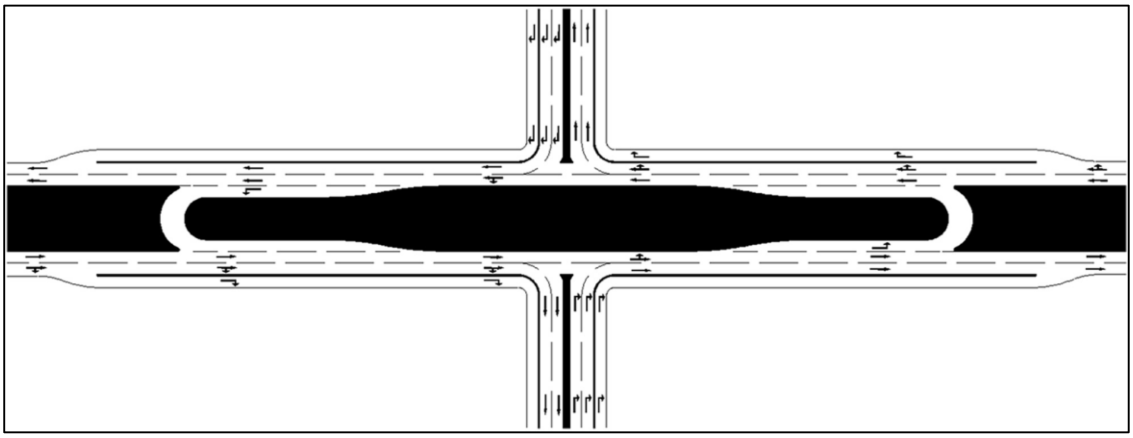

Figure 1.4 Unconventional MUT Intersection (M. El Esawey and T. Sayed, 2013)

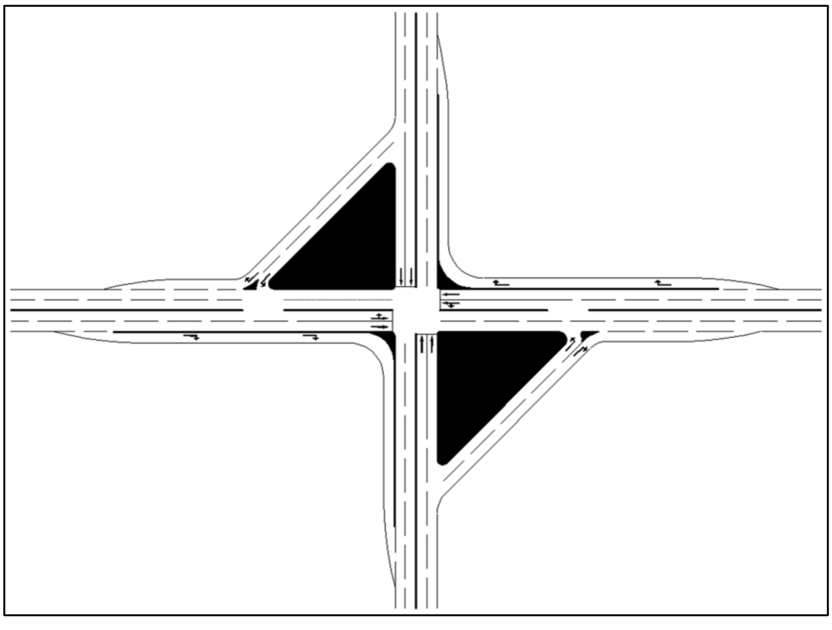

Figure 1.5 Jughandle Intersection (M. El Esawey and T. Sayed, 2013)

left-turning flow coming off the minor street are re-routed through indirect left-turns by right turning followed by U-turns crossovers located on the major streets downstream the primary intersections. As a common treatment for signalized intersections, the unconventional MUT has been presented for about 15 years in Cairo, Egypt (Elazzony et al. 2011, El Esawey and Sayed 2011b), also it has been used in Iran since 2008 (Shahi and Choupani 2009). Depending on the geometric design, and the available median width, the U-turn crossovers can be located either on the major approaches or the minor streets or even both (Reid and Hummer 1999). Also, relying on the traffic volumes, especially, the left-turn ratios, these crossovers can be a signalized or unsignlaized (Hummer and Reid 1999).

The Jughandle intersection is designed based on diverging the minor street left-turns by utilizing two one-way ramps diverging from the right side of two quadrants of the arterial as shown in Figure1.5. In order to accommodate all turning movements coming off the major street, the jughandle ramps start a hundred feet upstream the primary intersection and are connected back to the cross-street a few hundred feet downstream the primary intersection as shown in Figure1.3 (Hummer 1998b, Hummer and Reid 1999). Usually, both right and left- turning traffic at the ramp terminus (i.e. junctions between the ramp and cross-street) are stopped-controlled. However, the right-turning flow might be yield-controlled depends on the right-turn flow ratios (Rodegreds et al. 2004, Jagannathan et al. 2006).

The Upstream Crossover (USC) intersection is a four-leg unconventional design where both through and left movements are re-rotated to the left side of the road upstream the primary intersection in both major and minor approach as shown in Figure1.6. Through this unusual re-rotating / crossing movements, the left-turn conflicts with the opposing traffic are eliminated (Sayed et al. 2006, Tabernero and Sayed 2006). These crossings resulted in creating four additional crossovers as secondary intersections prior to the main intersection. Although the simple two-phase signal-timing scheme provided, a coordination of all five signals (the primary intersection and the four secondary crossovers) is needed for an efficient operation of the USC design.

Similar to the USC intersection, two more designs namely DXI and DLT are presented but with some differences. The main difference between these intersections, that in the USC both through and left movements are crossed to the left side of the road upstream the primary intersection in both major and minor approach, while in the DXI intersection both through and left movements are crossed to the left side of the road upstream the primary intersection only in major approach as shown in Figure1.7, so it can be considered as a half USC (Bared et al. 2005, Autey et al. 2012, El Esawey 2013). On the other hand, in the DLT only left-turns are crossed to the left side of the road upstream the primary intersection in both major and minor approach as shown in Figure1.8 By building an additional roadway section in one intersection quadrant, another unique alternative design was presented to remove all left-turns at an intersection. The operational mechanism of this design is to diverge the left-turns of the major and minor approach by providing signalized secondary intersections upstream the primary intersection as shown in Figure1.9. As a result, two additional intersections representing the junctions between the QR and the arterial, and another junction between the QR and the cross street from the other side are created. In this design, the primary intersection is controlled by a two-phase signal, while as the secondary intersections are controlled by a three-phase signal system (Ried 2000, Hughes et al. 2010).

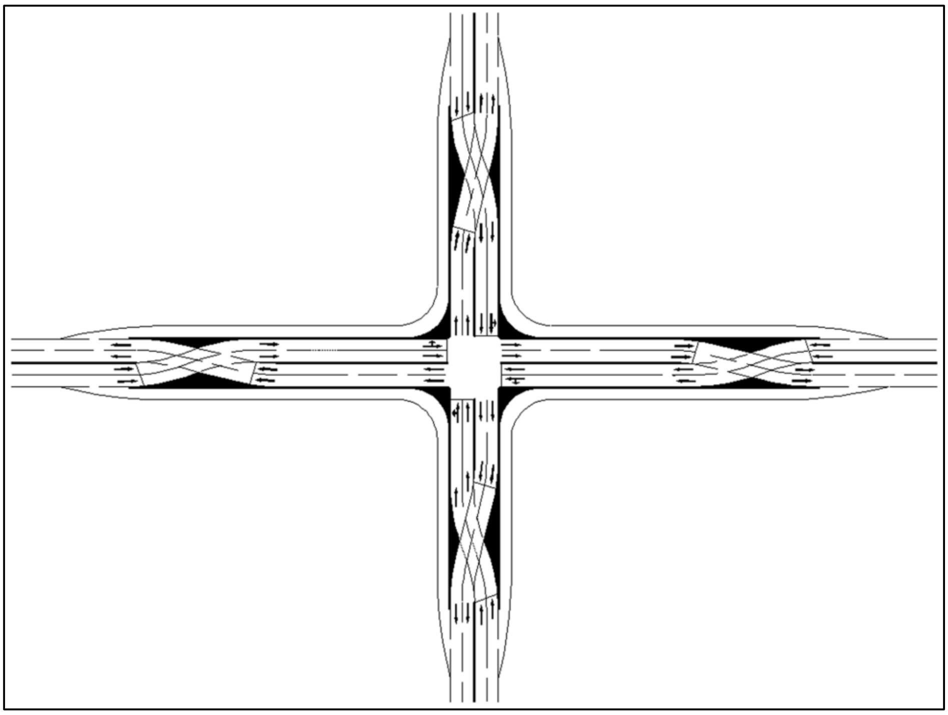

Figure 1.6 Upstream Crossover (USC) Intersection (Autey J., et al 2012)

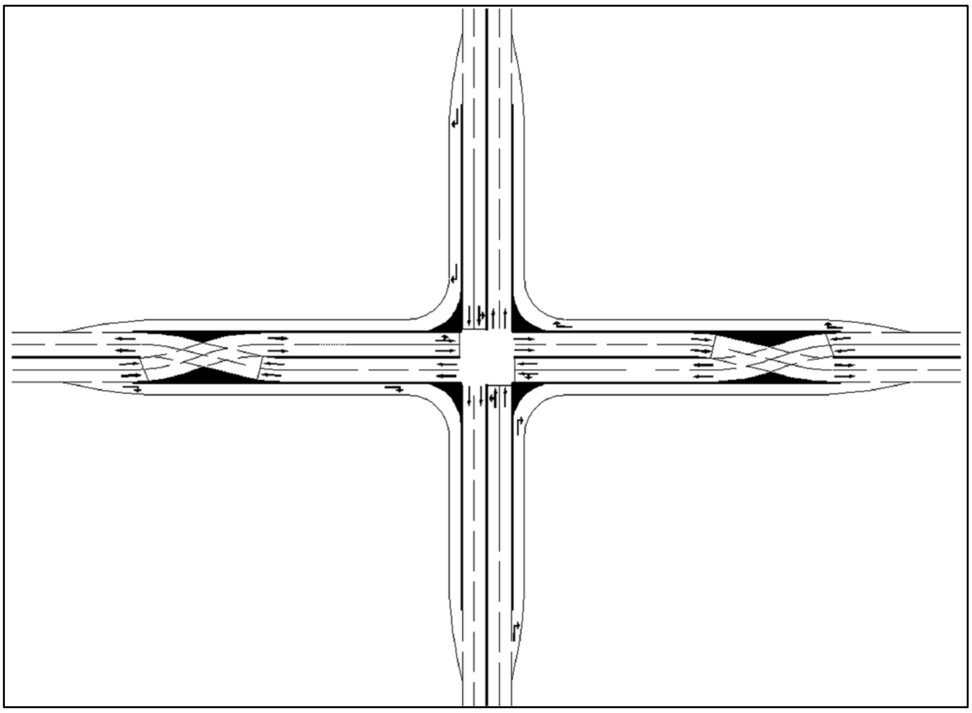

Figure 1.7 DXI Intersection (FHWA, 2014)

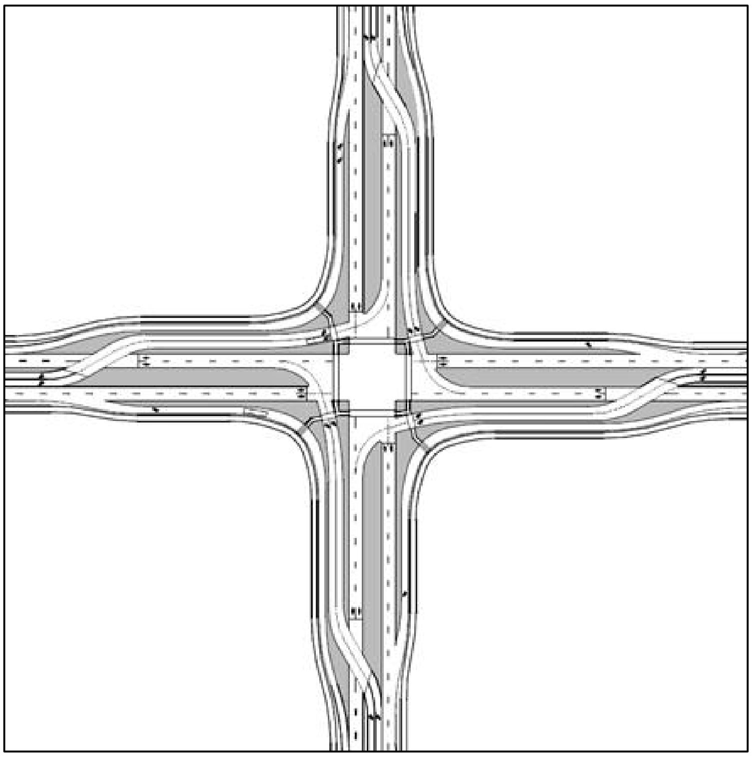

Figure 1.8 Displaced Left-turn Crossover (DLT) Intersection (FHWA, 2014)

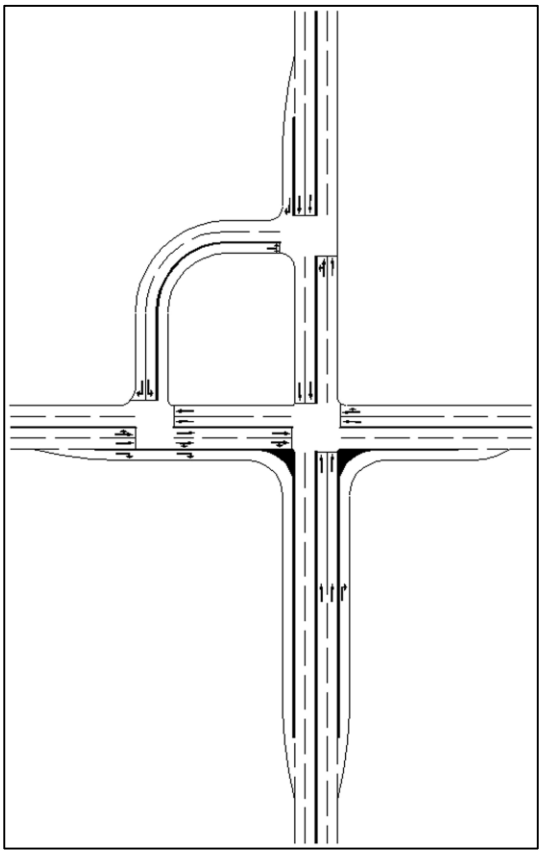

Figure 1.9 The QR Intersection Design (M. El Esawey and T. Sayed, 2013)

1.4.2 Grade-Separated Alternative Interchange Designs

Several justifications have given engineers cause to consider grade-separation of major intersecting roadways including desired intersection capacity, concerns with safety and increasing traffic volumes on many arterial and thoroughfare roadways. By elevating the through-movements on one roadway over the crossing roadway, several unconventional grade-separated intersection designs were proposed. Utilizing ramps is necessary for such designs to handle turning movements, forming secondary signalized or unsignalized intersections on one or both of the roadways. Although such designs are not common for arterials because of the higher construction costs and the surrounding land use adjustment sensitivity issue, they may be the only method to provide sufficient capacity at certain high-volume intersection locations. However, these schemes can only be applied to arterial intersections where there are no free-flow movements, which is typical in other conventional freeway intersection designs. In the next sub-section some grade-separation designs are presented, the geometric and operational performance is described, the limitations and demerits of such schemes are highlighted. The Diverging Diamond Interchange (DDI) is considered the most common grade-separation design in the US, because of its simplicity. The main difference between this new design and a conventional diamond interchange is in the treatment way of both left and through movements through the navigation between the cross street intersections with ramps. According to the FHWA, in some cases a DDI can cost less as much as 75 percent less than an equivalent conventional diamond or single point urban interchange (FHWA,2014). As some at-grade alternative intersection designs, also DDIs are designed as crossover intersections. However, these crossovers transit or diverge traffic from the right side of the road to the left side of the road and then back again as Figure1.10 depicts. All left turns occur without having to cross opposing traffic, because traffic is on the left hand side between the crossovers. Although this design works efficiently under low to moderate volumes on the cross street, the proximity of the two-signals at either ramp termini can result in inadequate left-turn storage. Also, it may cause a difficult progression in both directions on the cross street.

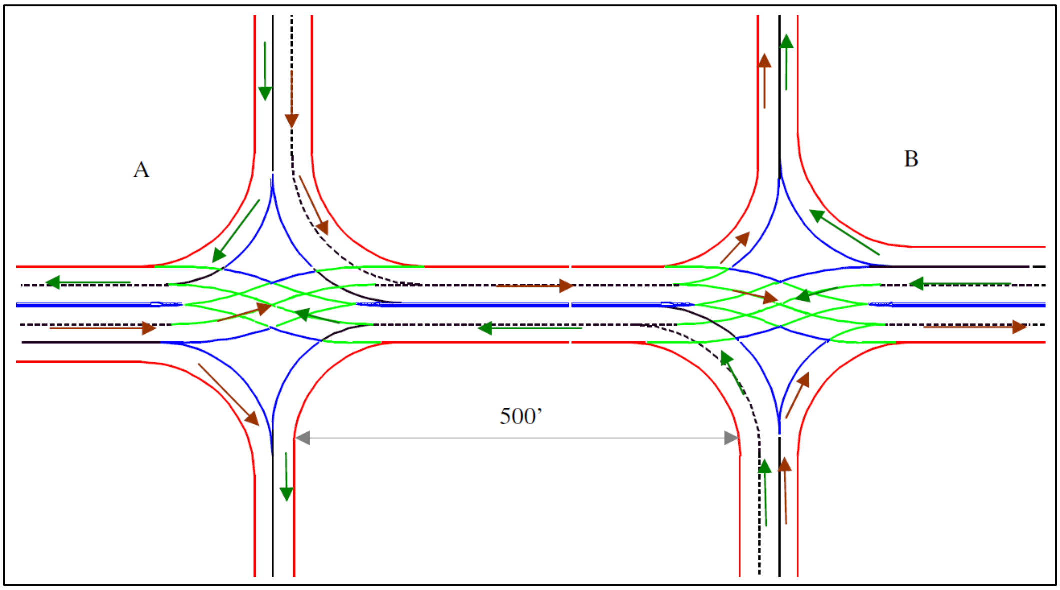

Figure 1.10 The Diverging Diamond Interchange Design (Bared J., et al 2005)

1.5 Research Objectives and Significance

The main objective of this research is to study the applicability of the UAIDs under the heterogeneous traffic environment, as a driving force to find a novel solution that leads to higher capacity, lower delays of the conventional intersections. Also, this research aims to propose different approaches by considering the coordination of the UAIDs intersections as a coordinated corridor. Therefore, in order to accomplish this research, the following five sub-objectives were formed:

1. To assess the potential operational capability of the entire existing conventional intersections under the heterogeneous traffic conditions;
2. To evaluate the operational efficiency of two proposed UAIDs schemes namely; the Displaced Left-Turn (DLT) intersection and the Restricted Crossing U-Turn (RCUT) intersection under the heterogeneous traffic conditions and limited geometric designs;
3. To compare the operational performance of the existing intersections to the newly proposed schemes as well as to compare the operational performance of the two proposed UAIDs schemes;

4. To develop a coordination system of the proposed UAIDs based on a real-time demand responsive signal control algorithm as a coordinated corridor;
5. To conduct the Cost-and-Benefit Analysis (CBA) of the existing conventional signalized intersections with two alternative schemes.

1.6 Scope and Limitations

The outcome of the research is to provide traffic analysts and the policymakers with an objective assessment on the possibility of implementing the UAIDs as a proposed alternative scheme in the developing cities around the world where the heterogeneous traffic conditions are dominant. Furthermore, this research aims to develop a coordination system of the proposed UAIDs based on a real-time demand responsive signal control algorithm. Also, the branch-and-bound algorithm is utilized as a common used algorithm for bandwidth maximization approach seeking the UAIDs coordination. Finally, based on the main findings of this research, practically the CBA results, this research will help to produce enough information about the UAIDs to ascertain whether these it should be undertaken as a cost-effective proposed alternative scheme. The scope of this research covers only two proposed UAIDs schemes namely; the Displaced Left-Turn (DLT) intersection and the Restricted Crossing U-turn (RCUT) intersection. However, freeway interchanges as grade-separation alternatives are beyond the scope of this study. Seeking a reliability in the real world and aiming to a credible evaluation, this research is built upon actual realistic data that were made available for existing intersections located in Cairo, the capital city of Egypt.

Considering the coordination methods of UAIDs proposed in this study, though this research includes the bandwidth maximization approach and the delay minimization approach, other evolutionary algorithms such as Genetic Algorithm, Monte Carlo…etc. with different optimizers should be examined to enhance the optimization problem outcomes. Likewise, the optimization of this study is formulated individually at each intersection, while the optimization methods where all intersections are simultaneously considered are beyond the focus of this research. On the other hand, the coefficient that defining the relative importance of major and minor approach delay for estimating the intersection delay were selected taking into the consideration the previous works. Therefore, a sensitivity analysis to estimate the optimal values for these coefficients needs more investigation.

In conclusion, the scope of this research is:

1. Considering the heterogeneous traffic complexities in evaluating the operational performance of two proposed at-grade UAIDs schemes namely; the Displaced Left-Turn (DLT) intersection and the Restricted Crossing U-turn (RCUT) intersection;
2. Investigating the coordination of UAIDs as a coordinated corridor;
3. Conducting the Cost-and-Benefit Analysis (CBA) for the proposed schemes;
4. Estimating the optimization problem is formulated at each intersection individually.

However, the study is constrained by the following limitations:

1. The grade-separation UAIDs are beyond the scope;
2. The safety performance and environmental impacts are research areas that need more examination and evaluation;
3. The optimization methods where all intersections are simultaneously considered are beyond the focus of this research;
4. The coefficient that defining the relative importance of major and minor approach delay for estimating the intersection delay were selected taking into the consideration the previous works.
5. The improvements in the safety conditions as indirect benefits for the Cost-and-Benefit analysis for the UAIDs are beyond the scope of this research.

1.7 Research Contributions

Despite the wave of valuable research works and the numerous efforts done by researchers focused on studying, analyzing and evaluating the different types of UAIDs, most of these studies were proposed in the developed world where the less complex homogenous traffic conditions and ideal traffic environments are existing. Therefore, the driving force of this study is to investigate the UAIDs applicability under the heterogeneous traffic complexities as a significant traffic feature in developing, semi-industrialized and industrializing countries around the world. These complexities are characterized by the diversity of some static and dynamic properties of vehicles, and the aggressive driving behaviour as well as the non-lane based traffic system. The numerous considerable efforts proposed the UAIDs as isolated intersections and a little research work has been directed to placing a series of UAIDs on a coordinated corridor. Despite these attempts that tried to study the UAIDs as a coordinated corridor, no study found to investigate the DLTs coordination applicability. Thus the novelty of this study is considering the DLTs coordination by utilizing both approaches: the bandwidth maximization progression as well as the delay minimization. Even though, the previous research used commercial signal timing software to optimize the network signal timing. Therefore, the uniqueness of this study is to develop a coordination system of the proposed UAIDs based on a real-time demand responsive signal control algorithm as a coordinated corridor.

The previous studies recommended that the cost-and-benefit assessments as one of the research areas that need more examination. Therefore, this study considers the cost-and-benefits analysis to produce enough information about the performance of the alternatives scheme and to ascertain whether these alternatives should be undertaken. By identifying, valuing and comparing the private and external costs and benefits of the proposed alternatives, a clearer idea and a better understanding is gained for the assessments required.

On the other hand, the primary researches emphasized the UAIDs as a more adequate design in rural areas, as most of UAIDs require a larger footprint (i.e. the right of way) than conventional counterparts (M. El Esawey and T. Sayed 2013). This research investigates the UAIDs in an urban area while still keeping approximately the same right of way with a similar number of lanes of the conventional measures.

In conclusion, the academic contributions of this research are:

1. Evaluating the operational performance of two proposed at-grade UAIDs schemes namely; the Displaced Left-Turn (DLT) intersection and the Restricted Crossing U-turn (RCUT) intersection under the heterogeneous traffic complexities;
2. Studying the coordination of DLTs in a coordinated corridor based on the bandwidth maximization technique as a fixed pre-time control;
3. Develop a real-time demand responsive signal control algorithm based on delay minimization for displaced left-turn coordination;
4. Conducting the Cost-and-Benefit Analysis (CBA) for the proposed schemes.

1.8 Dissertation Outlines

This dissertation consists of eight chapters. This chapter presents, the introduction and background, research motivation, the problem statement and the research objectives. Also, this chapter describes the scope and the limitations of the research as well as the main contributions of this study. Nevertheless, a brief explanation of the heterogeneous traffic conditions and provides the basic concept of the UAIDs within this chapter. The remaining chapters are arranged as follows:

Chapter 2 presents a review of the numerous efforts done by researchers including both qualitative and quantitative studies that focused on studying, analyzing and evaluating the different types of UAIDs. Also, the literature review goes over the existing researches related to the present study on the traditional treatments of improving signalized intersections as well as the primary studies that investigated the heterogeneous traffic conditions impacts. Moreover, the limitations of the conventional treatments are discussed and gaps in the literature are identified. Lastly, the main remarks and a discussion of the reviewed works are summarized.

Chapter 3 discusses the methodological framework of this research. The first part of this chapter discusses the microsimulation modelling platform as an approach utilized in this context. The heterogeneous traffic modelling including driving behaviour, non-lane based traffic system modelling as well as the model calibration and validation procedures are discussed in details. The second part of this chapter discusses the case study, the site description and the data collection are introduced. Likewise, the recent operational performance of the existing intersections is analyzed and evaluated.

Chapter 4 highlights the Displaced Left-turn (DLT) intersection including the operational mechanism, the previous research main findings as well as its representation under the heterogeneous traffic conditions. This chapter also provides the coordination possibility of a series of DLTs intersections as a coordinated corridor. A detailed explanation is given about the coordination approaches followed in this study context, the bandwidth maximization and the delay minimization.

Chapter 5 investigates the Restricted Crossing U-turn (RCUT) intersection: the operational mechanism, the previous research main findings and how this scheme was represented under the heterogeneous traffic conditions. In addition, this chapter explains the coordination approach for a series of RCUT intersections.

Chapter 6 compares the both provided schemes in this study namely; DLT and RCUT. The comparison includes the operational performance, signal phasing and timing plans as well as geometric layouts.

Chapter 7 estimates the Cost-and-Benefit of the entire existing conventional signalized intersections and the two UAIDs proposed in this context. In this chapter, the different costs and benefits' components associated with the implantation of the proposed UAIDs intersections are defined and compared with their counterparts of the grade-separated intersections in order to produce enough information of the proposed alternative schemes and to ascertain whether UAIDs should be undertaken as cost-effective treatments.

Chapter 8 concludes the key findings, the main recommendations and the important implications that can guide the transportation agencies and traffic experts to ascertain whether these UIDs alternative schemes should be undertaken in developing countries. It also postulates the possible prospects and defines areas of further research.

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CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Owing to explore the basic fundamentals and principles that have a considerable relevance in formulating the conceptual framework of the present study, this literature is reviewed. This chapter endeavors to introduce a meaningful overview of previous studies and general guidelines relevant to this study context including the heterogeneous traffic condition influences. Also, this chapter presents a review of different treatments used to improve the regular signalized intersections, that known as conventional intersections and identifies related issues in order to gain a preliminary understanding of the previous trials, principally, that targeted the operational performance enhancement. Likewise, to gain a better understanding of the UAIDs functions, operations and fundamentals, the qualitative and quantitative studies on UAIDs, some related works published within the last few years are reviewed. Despite, the reviewed literature regarding the different UAIDs schemes, because this study is mainly focusing on evaluating the DLT and RCUT intersections, therefore, they are discussed in details later in the next chapters.

Several references, basically, including books, journal articles, conference papers, academic reports, and international manuals, a meaningful and a concrete background is drawn. Based on the valuable research work reviewed, the limitations of the conventional treatment schemes are highlighted, the research gaps are identified and the potential research directions are suggested. Finally, a summary and some concluding remarks on the main points of this chapter are provided.

2.2 The Heterogeneous Traffic Impacts

Over the last few years, several studies were carried out to estimate the heterogeneous traffic influences on the traffic conditions as a dominant in the developing cities around the world. The main findings of earliest literature emphasized that the traffic behaviour in mixed or heterogeneous condition (i.e. in developing countries) is different from that non-mixed or homogeneous traffic (i.e. in developed nations) (Budi Yulianto and Setiono, 2012). As a result, considerable impacts on the operational and safety performance caused by the complexities of such conditions. As mentioned earlier in the first chapter of this dissertation, the wide variations in the operating and performance characteristics of vehicles resulted in complex operating systems comparing to the homogeneous one (Maini, P., Khan, S. 2000). Accordingly, different performance indices such as saturation flow rates were dramatically influenced under the heterogeneous traffic operation. For instance, the ideal saturation flow rate is 1,900 PCU/h/lane according to the HCM. The western studies estimated this value within a range between 1,700 and 2,080 PCU/h/lane of green time. For example, this value was estimated at 1,800 and 1,710 PCU/h/lane in the United Kingdom and Australia, respectively. However, under the heterogeneous conditions such as in those developing, semi developing nations, the saturation flow rate values were observed 1,617 in Egypt, 1,945 in Malaysia, and 1,232 in India (M. Hossain, 2001: J. Bonneson et al., 2005). Based on this brief comparison, it is noteworthy that the clear impact of such conditions on the saturation flow rates, which indicate the operational performance influence. At Stellenbosch in Western Cape, South Africa, a comparative study was carried out by C. J. Bester et.al. in 2007. The author emphasized that the saturation flow rate values in Stellenbosch are much higher compared to other countries as a result of a driver aggressiveness behaviour, particularly the speed limits. This study referred to the inverse relationship between an intersection gradients and the saturation flow rates. Nevertheless, it was found that the speed limit, gradient, and the number of through lanes have much greater impacts on the saturation flow rates in South Africa than in the USA (C. J. Bester and W. L. Meyers, 2007).

As a result of the highly varying speeds of vehicles under such conditions, which vary, from 5.0 to over 100.0 km/h, the vehicles do not follow lane discipline and move freely as a non-lane based traffic system. Within this unique system, a vehicle does not have one leader but it may have several on the front-left, the front-straight and the front-right. Hence, it is not appropriate to use lane-based vehicle interaction models, because vehicles are used to traverse in both the traverse and the lateral directions (Maini, P., Khan, S. 2000). Under the non-lane based system, the lane concept as well as the expression of flow values, based on lane width become invalid. Under this traffic system, the non-lane based system, vehicles do not follow each other within lanes. Therefore, the concept of relating headways and linear densities (such as vehicles per kilometer) is meaningless. Thus, under the heterogeneous conditions, the headways are defined based on the arrival times of vehicles that are moving on the whole width of roadway considered at a time (V. Thamizh Arasan and Rebbu Zachariah Koshy, 2005).

On the other hand, the common measures of uninterrupted traffic stream conditions such as speed, flow and density were influenced obviously under the heterogeneous operating systems. Although the initial efforts applied to develop traffic flow models for roadways by considering the heterogeneous traffic by converting heterogeneous traffic to equivalent passenger-car units, these efforts have produced different results so far. The earlier research investigated the speed-flow relationship under mixed or heterogeneous conditions based on the passenger-car equivalency factor ranging from 0.2 to 8 to include cars, buses, trucks, scooters, motor-cycles and bicycles. The observed capacity was observed as 900, 1900 and 1800 equivalent passenger cars per hour per lane for two-lane undivided roads, four and six-lane divided roads respectively (Sarna, A et.al 1989).

Indeed, the diverse physical properties of vehicles in the heterogeneous condition could also influence the operational performance, especially, inside the intersections. The two-wheeled smaller vehicles (i.e. motorcycles, scooters, bicycles…etc.) maximize the inter-vehicle space to reach and occupy the front of the stopped queues during the red time, while the heavy vehicles affect the operational performance, particularly, the start-up lost time, clearance lost time and discharge rates inside an intersection. As a result, the traffic operational functionality such as queues discharging, merging and diverging phenomena is obviously influenced (Kaur and Varmora 2015).

2.3 Conventional Treatments to Improve Signalized Intersections

Initial efforts have been done by analysts aiming to develop approaches to mitigate the corresponding congestions, enhance the traffic flow efficiency and to improve the safety level for the different road users (i.e. car users, pedestrians, cyclists…etc.) at signalized intersections. Various initiatives, generally, focused on physical modifications in the intersection layout such as reducing, extending the curb-radius or even provision of advanced stop lines are applied to increase the road space, whereas the other treatments involved minor modifications to the signal control operation by improving the signal controller designs. In the next subsections, the different treatments include both levels at-grade and grade-separated levels are discussed in the following subsections.

2.3.1 At-grade Intersection Treatments

The developed approaches can be categorized based on the purpose or the targeted road users beyond the conducted modification. Hence, some treatments target pedestrians, while other solutions focus on cyclists, motorists or transits treatments. The applied traditional measures in at-grade intersections, generally, aim to increase the approach capacity by changing the geometric layouts and structural elements by adding through lanes and exclusive turning pockets. Likewise, both the reduction of curb-radius as well as provision of curb-extension are also applied as proposed treatments. Another design that is known as the Exit lane for Left-turn (EFL) was proposed by allocating some opposing-through lanes as mixed-use lanes that can be used by left-turning vehicles. This design aims to increasing the number of discharge lanes, particularly, with high left-turning movements rates (Zhao et al., 2013). Furthermore, other traditional treatments focused on the signal control decisions improvements either through the use of actuated signals, modifying the inter-green time, signal timing optimization or by providing solutions with auxiliary signals (Xuan et al., 2011). Tandem Sorting Strategy (TTS) is one these alternative designs that built based on installing upstream auxiliary signals to improve intersection performance by dealing with turning movements, especially, when the discharge lanes are fully utilized, albeit in different ways. TSS design is one of these alternatives that use auxiliary signals (or pre-signals) to separate the vehicles belonging to different movements (i.e., through and right-turn) upstream of the intersection. As a result, the different traffic streams approach the main intersection in separate bunches and utilize as many possible lanes for discharging. Hence, TSS is considered an appropriate solution, particularly, for high left-turn rates (Gaspay 2016).

Considering pedestrians' treatments, readjustments in geometric, structural and signal control designs have been applied in order to enhance the safety conditions and facilitate the pedestrian movements inside an intersection. In terms of structural treatment, lots of modifications in the geometric layouts have been proposed such as reduction of curb-radius, provision of curb-extensions, provision of advanced stop lines or even by grade-separation of pedestrian movements by providing pedestrian decks. In such cases, where high rates of pedestrian-vehicle conflicts, significant number of children crossing (e.g. school zones), high speed turning vehicles or/and inadequate sight distance, the grade-separation pedestrian movements (i.e. pedestrian overpass or underpass) is a recommended solution. The reduction of curb-radius is applicable where the high rate of crashes between right-turning vehicles and pedestrians, the high-speed turning movements and the case of poor sight-distance between pedestrians and motorists. This treatment improves the safety conditions inside an intersection by reducing the left-turning vehicles' speed and the collision severity (Huang, H. and Zegeer, C., 2000) with shorter pedestrian crossing distances and shorter green times. On the contrary, the left-turn movements' capacity experiences a reduction. Likewise, it is a possible risk of rear-end collisions involving left-turn and through movements in case of shared discharge lanes and the large vehicles also practice a difficulty in turning maneuvers. On the other hand, the provision of curb-extension is proposed as an appropriate solution in case of high volume of pedestrian collisions as well as high incidence of the following aggressive vehicle behaviours in high pedestrian crosswalks which are not yielding to pedestrians, high-speed turns, invasion of parking lanes. Although this proposal improves the visibility of pedestrians by vehicles and reduces turning speeds that may lead to reducing the collisions with pedestrians as well as producing a shorter pedestrian crossing distance with a corresponding shorter green time, some liabilities are highlighted. Similar to the reduction of curb-radius treatment, these liabilities are such as capacity reduction for left-turn movements, a possible risk of rear-end collisions involving left-turn and through movements in case of shared discharge lanes and a turning difficulty for large vehicles. Moreover, a possible traffic diversion to other roads without curb extensions may also occur (FHWA, 2008; National Association of City Transportation Officials, 2014). The provision of advanced stop lines was also provided as a preferable solution for the high incidence of right-turn-on-red collisions between vehicles and pedestrians. This proposal may reduce the risk of collisions between pedestrians and right-turning vehicles. Furthermore, an easier turning maneuver is provided for large vehicles. Conversely, this treatment may result in increasing the intersection clearance time and higher lost time (Huang and Zegeer, 2000; Smith et al., 2005). The signal displays (i.e., WALK/DONT WALK signals, countdown displays, animated eyes display) may also be used to improve pedestrian movements. These displays lead to assistance to visually impaired pedestrians and improve pedestrian awareness with a higher percentage of successful crossings (Hughes et al., 2006b). The signal phasing readjustment such as leading and lagging intervals as well as the exclusive pedestrian phase can also be utilized to enhance the pedestrian flow.

On the other hand, lots of modifications including both geometric changes as well as signal control readjustments have been proposed as solutions that target cyclists, motorists or/and transits aiming to improve the operational and safety performance at an intersection. Most of the geometric treatments focused on providing through lanes and exclusive turning pockets in order to increase the road capacity and to ensure a smooth traffic along the corridor (FHWA, 2008; National Association of City Transportation Officials, 2014). Moreover, some particular treatments are considered specifically as transit treatments such as relocating the transit stops, exclusive bus lanes and providing the transit priority signal system in which the transit system has the priority over the other vehicles' type to ensure a smooth travel along the corridors. Meanwhile, the traffic signal systems are comprehensively considered in order to save the control delays and to increase the control flexibility inside an intersection (Webster, 1958; Allsop, 1971a; Yagar, 1977). Some trails focused on improving the inter-green time which is the duration provided between phases to guarantee the clearance of travelled vehicles during the current phase before vehicles in the succeeding phase are released at an intersection. As a result, the "dilemma zone" which results in right-angle crashes in case of inadequately time, therefore, the inter-green time modification ensures that a vehicle can proceed safely through the intersection (Zhang et. al., 2014). Meanwhile, the traffic signal optimization approach was primarily investigated to find the optimal signal parameters that minimize the delay and maximize the capacity to improve the efficiency at an intersection (Bell, 1992). These new optimal parameters provide the required green waves that ensure a smooth travel along the corridor with a minimum number of stops. The earlier researches relevant to the signal optimization approaches as well as the different optimization techniques and the main findings of the previous studies are discussed in details in chapter 4 of this dissertation.

2.3.2 Grade-Separated Intersections

Aiming to improve the conventional intersections' operational performance, getting higher capacities, lower delays and fewer crashes, the grade-separated intersections have been widely applied over the last decade as a common used treatment. While at-grade intersections control the traffic movements via signal control or/and separating the traffic flow, the grade-separated intersections, however, separate the traffic movements to vertical separation of roadways, higher or lower levels than the existing level. The provided grade-separated intersections result in eliminating all grade crossing conflicts and accommodating another merging, diverging and weaving maneuvers with less hazard and delay than at-grade intersections (Tom V. Mathew, 2017). The grade-separated intersections flexible designs included either underpass such as tunnels or overpass intersections such as flyovers by elevating the intersections to a separated higher level by the constructed ramps. The previous findings revealed that these grade-separated intersections exhibited a considerable enhancement in the traffic performance considering the different existing geometric designs. In spite of the higher capacity provided by such designs, they constrain vehicles' movements and the flexible access to roadside facilities, which may affect the land utility. Moreover, this design is considered a costly and aesthetically unpleasing treatment (Goldblatt et al. 1994; El Esawey, M., Sayed, T. 2013). Furthermore, this solution entails a long process that involves public consultations, budget requests, massive construction planning and requires road right-of-way which is usually not a simple process especially in urban areas.

2.4 Limitations of The Conventional Treatments

In spite of the achieved improvements in both safety and operational efficiency at an intersection due to the above-mentioned applied adaptations, the proposed treatments cannot resist against the rapid growth in motorization and the significant increasing in the number of road users, the conventional treatments reached the maximum possible benefit that can be gained. Although, the capacity extension obtained by the different structural modifications, however, these solutions acquire huge construction costs and more footprints or right-of-way requirements which may interrupt the adjacent land use. On the other hand, the signal control readjustments are considered as a good enough treatment to address the capacity issue and implement control measures that avoid spillover. However, in intersections where a significant number of turning movements, the protected turning phase may improve the intersection safety and operational efficiency but adding additional phases will lead to increasing the cycle length causing additional lost time and more delays (Kell and Fullerton, 1991). Even if the adaptive signal controller can provide more green to the more critical oversaturated links, however, limitations in the detection system may result in inaccurate signal settings (Gasoay et al., 2013). Meanwhile, due to the variability of directional traffic ratios, especially, in case of highly variant traffic, the lane assignments which are designed based on average traffic demands during peak hours, may not be entirely flexible. For instance, in the case of a four-lane approach, usually one lane each is allocated to the left and right-turn movements, while the other two lanes are assigned for the through flow.

Although the adaptive control gives a higher (but limited) green time for the higher turning movements, the number of discharged vehicles becomes limited by the number of assigned lanes and the given green time. As a result, the turning movements' queue may or may not be dissipated (Gaspay 2016). Therefore, theoretically, to address the issue of high turning-movements demand, additional turning lanes becomes necessary. However, when additional lanes cannot be provided, the original problem where turning vehicles are not given adequate green times will be raised. Furthermore, the drawback of underutilized lanes may appear in case of assigning one lane to a single or pair of movements. For instance, when a single approach is assigned two phases (e.g., one phase assigned for through and left-turners, the other phase for right movements), during the green time, not all lanes are utilized for discharging vehicles.

For the abovementioned reasons, many researchers implied that these traditional measures are exhausted and are no longer able to address the congestions at signalized intersections. Hence, recent researchers have identified innovative ways such as the UAIDs as adequate and a novel solution that can efficiently enhance both operational and safety conditions inside the signalized intersections. Although the additional capacities will be given by these innovative approaches, they require small to almost no additional space. (Hummer 1998a; Reid and Hummer 1999; Dhatrak, A., et al. 2010; El Esawey, M., Sayed, T. 2013).

2.5 Qualitative and Quantitative Studies on UAIDs

Lots of considerable, numerous efforts and valuable research works have been presented in the direction of the UAIDs implementation technologies to identify the basic principles of the analysis of such schemes. Therefore, the present literature mainly considers both qualitative and quantitative studies with an emphasis on quantitative analysis. The qualitative studies usually target describing designs and operational mechanism, introducing the possible gained benefits as well as suggesting the best operating traffic conditions, while the quantitative studies focus on evaluating and analyzing the operational performance and the safety conditions. Seeking a meaningful and concrete background of the different applied UAIDs schemes, the existing studies that give a good overview and general guidelines relevant to this study which are reviewed. To highlight the benefits and report the drawbacks of such innovative designs, almost all of the previous literature targeted a comparative performance analysis of a particular unconventional design and its conventional counterpart or even another unconventional design (El Esawey, M., Sayed, T. 2013). Because the operational mechanism of such unconventional designs was discussed in the previous chapter in details, therefore, this portion deals with reviewing the quantitative studies which highlighting the evaluations of operational performance and safety conditions of different UAIDs schemes. Because this study is mainly focusing on evaluating the DLT and RCUT intersections, therefore, the relevant studies of these two designs are discussed in details later.

Using SimTraffic as a simulation tool, while optimizing signal phases and splits using Synchro, Chlewicki compared the performance of a similar conventional intersection to the Double Crossover (DXI) intersection that also known as the Synchronized Split-phasing (SSP) intersection. His results emphasized the superiority of the SSP over the conventional one (Chlewicki 2003). Bared et al., used VISSIM to compare the DXI to a four-leg conventional counterpart under four different volume scenarios. The results showed that the two designs performed similarly under low volume levels, while the DXI outperformed its conventional intersection under heavy volume levels and heavy left-turn scenarios (Bared et al., 2005). Following a different approach without employing a micro-simulation platform, Asokan et al. used the critical lane volumes to estimate the capacity of isolated conventional and some types of UAIDs (Asokan et al 2010). However, the authors did not discuss the validation process followed for their proposed methodology (El Esawey, M., Sayed, T. 2013). Zhou et al. evaluated the operational performance of Right-turn plus U-turn (RTUT) as an alternative to Direct Left-turn. As a function of major and minor road traffic flow rates, the authors developed regression models to compute the delay and travel time of DLT and (RTUT) (Zhou et al 2002). In a further study, using field data to relate average weaving speed of RTUT and the waving length, a linear regression model was developed to determine the optimal location to place U-turn openings required RTUT (Zhou et al 2003).

Over the last decade, the MUT design has been rising as the most widely new alternative as a viable access management strategy used to relieve arterial congestion. Therefore, several studies target evaluating its operational efficiency and safety conditions. For instance, brief guidelines for the MUT implantation were provided by Stamadias et al. when they examined the safety and operational performance associated with allowing U-turns at a signalized intersection during an exclusive left-turn phase (Stamadias et al. 2004). Based on CORSIM results, Bared and Kaiser reported a significant overall travel time reduction of the MUT design comparing to a typical conventional one under balanced volumes (Bared and Kaiser 2002). Interestingly, MUT design was put into service in Iran as an indication of this design effectiveness in the developing nations. Shahi and Choupani analyzed the MUT design for five locations in Iran by developing the regression models to estimate the travel time of left-turning, the minor street through movements' travel time, weaving time, the speed of U-turning vehicles and speed of non-weaving flow by using the collected field data (Shahi and Choupani 2009). Other studies dealt with the unsignalized MUTs to estimate the capacity and/or delays at the U-turn crossovers using gap acceptance models (Al-Masaeid 1999; Liu et al. 2008). On the contrary, several negative impacts have been attributed to such appealing treatment as a result of the absence of clear criteria or guidelines that regulate the usage of this alternative design in Cairo, Egypt. Although, the fertile testing environment for the effectiveness of such design due to a large number of implemented U-turns at various urban intersections with various physical and operational characteristics in Cairo, the U-turn intersection did not show a superiority. In their study, El Azzony et al., compared three different treatments: 1) Restricted Lefts/Through U-turns; 2) Restricted Left U-turns with two-phase signal control for through traffic, and 3) no restriction of direct movements at an intersection with full signal control. Based on simulation-based assessments, the authors emphasized that the superiority of the four-phase signal control was recognized (El Azzony et al. 2011). Likewise, Shokry and Tanaka studied the operational efficiency of the signalized intersections involving U-turns in Aswan, Egypt. The authors revealed the significant adverse impact on the operational performance and emphasized the traffic flow turbulence as a result of placing the U-turning movements that share the same assigned signal phases allocated to the direct left-turning vehicles inside a signalized intersection (Shokry and Tanaka 2015).

On the other hand, several empirical studies revealed the reduction in accidents rates in MUTs comparing to making direct turns at an intersection (Catronovo et al., 1995; Xu, 2001; Jagannathan, 2007). The MUT design reduced the most common type of accidents in junctions with direct left-turns and angle crashes that result in more severe injuries. However, more side-swipe crashes are more likely to occur as a result of increasing the weaving vehicles rates (Ruihua and Heng 2009). Potts analyzed crash data from 125 median openings from 7 states in the USA. In his study, it was found that most accidents occur between the U-turning vehicles that attempt to merge onto the main approach (Potts 2009). Meanwhile, it was reported that the three most common types of collisions associated to MUT design are because of the rear-end, side-swipe, and angle crashes (Liu et al., 2007; Xu 2001).

Regarding the coordination of these unconventional designs, little research has been directed to placing and investigate a series of unconventional intersections on a coordinated corridor, while most of the previous work on these proposed schemes as isolated intersections (El Esawey, M., Sayed, T. 2013). Based on CORISM as a microsimulation-based assessment, Reid and Hummer compared traffic operations for the conventional Two-way Left Turn Lane (TWLTL) and MUT and RCUT as two alternative unconventional designs along with an arterial that has five signalized intersections. The authors concluded that the MUT and RCUT geometric designs resulted in improving both system travel time and average speed compared to the TWLTL design, especially, during the peak hours. However, during the off-peak hours, the MUT and RCUT design operated similarly to the TWLTL intersection (Reid and Hummer 1999). Also, El Esawey and Sayed investigated the benefits that may merge when deploying a series of USC intersections on a major urban corridor in Doha, Qatar. They emphasized that most of examined travel time measurement sections experienced lower delays in case of USC coordinated corridor comparing to the conventional corridor. The reductions in the average control delay for the USC intersections ranged between 7.6 and 22.9% as an indication of the corridor performance improvement. Meanwhile, the total travel time was enhanced by 19.4, 14.8 and 13.6% for the AM peak, Midday peak and PM peak respectively (El Esawey, M., Sayed, T. 2010).

2.6 Gaps in Literature

Based on the reviewed literature, it can be concluded that in spite of a great deal of literature, as well as the wave of valuable research works, has investigated the qualitative and quantitative benefits of the different UAIDs, little research focused on assessment of such innovative designs considering the other prevailing conditions under the heterogeneous traffic as a significant and a dominant feature in developing, semi-industrialized and industrializing countries. Also, based on the highlighted literature the primary recommendation emphasized that the different UAIDs as more adequate in rural areas, as most of these designs require a larger footprint (i.e. the right-of-way) than conventional counterparts. Therefore, it is useful to study the applicability of implementing the UAIDs in an urban area where the right-of-way is usually limited. Furthermore, several studies employed software optimizer to derive the signal control parameters. Hence, no further details on relevant signal setting parameters such as phasing plans, split times, offsets among different signal groups are provided in the previous research works. Utilizing such software optimizers without adequate knowledge and theoretical basic foundations may lead engineers and traffic analysts to inaccurate conclusions. On the other hand, in most cases, the microsimulation platform was employed to evaluate the performance of the different designs. However, no study was found to give a clear procedure of representing such schemes considering the different simulation parameters. Also, the needed process of calibrating and validating of these models, which proves the credibility of the presented models, have never been discussed.

It should be noted that most of the previous research dealt only with the UAIDs individually as isolated intersections, however, few studies were directed to placing a series of UAIDs on a coordinated corridor. Although a little research work tried to investigate some of the UAIDs schemes in a coordinated corridor, no study was found to consider the DLTs coordination possibility. Hence, it can be summarized that the signal coordination of the different UAID designs is rarely studied. Meanwhile, most of these trials that considered the coordination approach, generally, did not take into consideration the impacts of the heterogeneous traffic conditions on the optimizing the intersections' delays along corridors. On the other hand, no study was found to evaluate the cost-and-benefit assessments as one of the research areas that need more examination. Therefore, a clearer idea and a better understanding of the cost-and-benefit analysis of the alternatives scheme to ascertain whether these alternatives should be undertaken.

2.7 Conclusion and Discussion

This chapter reviewed the early literature and previous works that give a meaningful and a concrete background relevant to this study. Several previous studies which were carried out to estimate the heterogeneous traffic influences on the traffic conditions as a dominant in the developing cities around the world were highlighted. The main findings of most of these efforts emphasized the considerable impacts on the operational performance and safety conditions due to the complexities of such conditions. Also, both conventional such as at-grade including geometric design modification, signal controller readjustments and grade-separated treatments that were proposed to improve the conventional intersections, were discussed. Likewise, to gain a better understanding of the UAIDs functions, operations and fundamentals, the qualitative and quantitative studies on different types of UAIDs were reviewed. To highlight the benefits and report the drawbacks of such innovative designs, a comparative performance analysis of some unconventional designs and their conventional counterpart or even other unconventional alternatives were also introduced.

In conclusion, based on the reviewed literature, it can be summarized that in spite of the wave of valuable research works that investigated the qualitative and quantitative benefits of the different UAIDs, little research focused on assessment of such innovative designs considering the other prevailing conditions under the heterogeneous traffic as a significant and a dominant feature in developing, semi-industrialized and industrializing countries. Also, most of the implemented UAIDs were constructed in rural areas, as most of these designs require a larger footprint (i.e. the right-of-way) than conventional counterparts. Therefore, this study investigates the possibility of implementing these alternatives in an urban area where the same right-of-way is usually limited. Meanwhile, it was found that most of the previous studies used software optimizer to derive the signal control parameters. In other words, no further details on relevant signal setting parameters such as phasing plans, split times, offsets among different signal groups were provided in the previous research works. Also, no study was found to give a clear procedure of representing such schemes considering the different simulation parameters. The needed process of calibrating and validating of these models, which proves the credibility of the presented models, have never been discussed. Furthermore, it should be noted that most of the previous research dealt only with the UAIDs individually as isolated intersections, however, few studies were directed to placing a series of UAIDs on a coordinated corridor. Although a little research work tried to investigate some of the UAIDs schemes in a coordinated corridor, no study was found to consider the DLTs coordination possibility. Although these little trials considered the coordination approach, generally, they did not take into consideration the impacts of the heterogeneous traffic conditions on the optimizing the intersections' delays along corridors. On the other hand, no study was found to evaluate the cost-and-benefit assessments as one of the research areas that need more examination.

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CHAPTER 3

RESEARCH METHODOLOGY AND STUDY AREA

3.1 Methodological Framework

The methodological framework of this study was built by taking into consideration the novel nature of UAIDs schemes, as well as the pre-deployments attempts of such schemes. Hence, this study methodology is developed relying on two main pillars to achieve the main goals and to accomplish the research objectives. The first pillar is evaluating the operational performance, while the other one concerns with estimating the economic assessment. Herein, the overall process that had been done is described in Figure 3.1. Also, a brief explanation of the research structure is as follows:

First, it is essential to represent the existing intersections for effective analysis and accurate evaluation of the operational performance for the current situation. However, a special attention should be paid to fulfil the heterogeneous traffic needs as the key factor of this research. Seeking a credible representation close to the real world, the model calibration and validation process is necessary. Hence, a conditional match of the simulated parameter values with observed traffic field data is needed in order to represent and validate the effectiveness and practicability of the simulated models as closely as possible to the reality before providing credible results.

Second, the two proposed UAIDs schemes namely; the Displaced Left-Turn (DLT) intersection and the Restricted Crossing U-turn (RCUT) intersection are simulated and evaluated under the heterogeneous traffic conditions and limited geometric designs. Based on the simulation outcomes, a fair comparison between the operational performance of each of the entire existing conventional intersections and its countermeasure of two proposed UAIDs schemes is conducted. A comprehensive before- and-after analysis of simulation outcomes has been carried out in terms of total travel time, intersections average delay, overall capacity, queue length and the number of stops of each vehicle to evaluate the potential implementation of the proposed schemes.

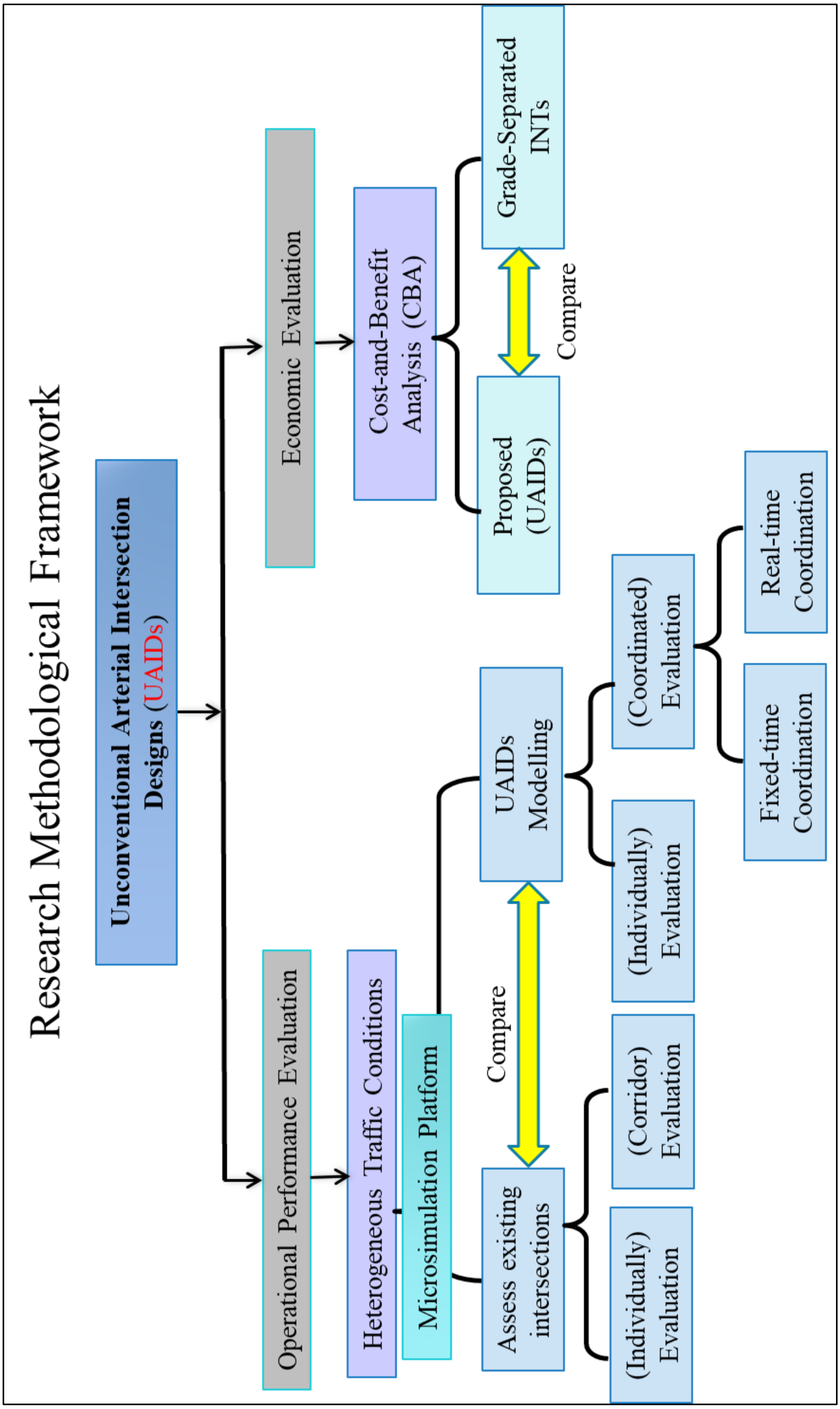

Figure 3.1 The methodological framework of this research

Third, considering the different coordination schemes (i.e. bandwidth maximization and delay minimization) a coordination system of each of the proposed UAIDs in this study is developed. Nevertheless, by utilizing a real-time demand responsive signal control algorithm, a MATLAB script was coded in order to represent the DLT intersections as a coordinated corridor.

Finally, aiming to estimate the cost-and-benefits assessments, a comparison of the three entire existing conventional signalized intersections with the alternatives proposed was done. By utilizing the CBA approach, enough information can be provided about the alternatives schemes to ascertain whether these alternatives should be undertaken.

3.2 Micro-simulation Approach

As the best-suited tool, cost and time effective and crucial analytical approach; the micro-simulation platform has become an increasingly important means for solving real-world problems. In order to model, analyze and evaluate different traffic schemes before field deployments, this approach has been broadly used worldwide in the last few years (El Esawey, M., Sayed, T. 2013; Mathew, T., Radhakrishnan, P. 2010; Ni et al., 2004). Therefore, many commercially available micro-simulation packages are developed such as VISSIM, CORSIM, AIMSUN, SimTraffic, PARAMICS and INTEGRATION. By utilizing the capability of modelling particular geometric designs, traffic system physical components replications such as road network, traffic control systems and driving behaviours are presented as in the real-world traffic conditions. Additionally, different configurations, scenarios and strategies are evaluated effectively. Hence, it is essential to consider some factors before select the micro-simulation package which include, but not limited to: the ease of coding, simulation running time, appropriateness of using default values of simulation parameters, visualization capabilities and the accuracy of performance measures reported in the output. Considering the above-mentioned factors, the methodology followed in this context is illustrated through employing VISSIM, as a powerful simulation-based assessment approach as well as a widely psychophysical car-following model, highly recommended to analyze the operational performance of UAIDs (El Esawey, M., Sayed, T. 2013).

Nevertheless, it allows the construction of UAIDs exactly like they would appear in the real life, based on the lane-by-lane development road networks facility (El Esawey, M., Sayed, T. 2013). Moreover, as a stochastic, time-step and behaviour-based model, many practitioners and researchers have used VISSIM to fulfil the heterogeneous traffic complexity needs, especially, the aggressive driving behaviour and the non-lane based system. The Wiedemann-74 model as a psychophysical car-following model was employed to simulate the different models of this study context. Based on the provided combination of both psychophysical aspects of physiological restrictions of the driver's perception, the aggressive driving behaviour under the heterogeneous conditions was represented effectively (VISSIM 5.4 Manual 2012). On other hand, in order to represent the non-lane based conditions of the heterogeneous traffic as close as the reality, the road network is created through utilizing the space-oriented feature which allows vehicles to move anywhere in the road without lane restrictions. The space-oriented feature allows any number of vehicle types can be created, it also allows the overtaking from right and left sides. Regarding UAIDs previous studies, out of 27 studies VISSIM was utilized in 15 different studies, CORSIM was used in 5 studies, AIMSUN and PARAMICS are used in one study, SimTraffic was employed in three studies and TRAF-NETSIM was utilized twice. Apparently, due to the extensive use of VISSIM in many studies, it is noteworthy that VISSIM is an effective, accurate and convenient micro-simulation platform for UAIDs analysis.

3.2.1 Heterogeneous Traffic Modelling

Despite the outperformance of VISSIM as one of the best-suited tools, as well as a cost and time effective micro-simulation approach to model, analyse and evaluate the different UAIDs schemes, VISSIM has been developed in the developed world where the heterogeneous conditions complexities are ignored. Accordingly, inexact, unrealistic and incredible models might be presented when the heterogeneous conditions complexities are applied using the default simulation parameters (Mathew, T., Radhakrishnan, P. 2010). Therefore, in order to overcome such discrepancy that leads to incorrect results, the calibration and validation process is highly necessary. Seeking a reliable simulator-based test, a considerable attention should be turned to reset and customize the different simulation parameters settings. By bridging a conditional match of the simulated parameter values with observed traffic field data, the effectiveness and practicability of the simulated models can be represented as close as possible to the reality. Nevertheless, a special readjustment should be considered to address the unique characteristics of heterogeneous traffic such as the complex manoeuvres under such traffic. The readjustment process includes the geometric configurations, the diverse vehicles' static and dynamic properties, manoeuvrability and the driving behaviour parameters such as lateral and lane change behaviour as well as manoeuvrability (Mathew, T., Radhakrishnan, P. 2010; VISSIM 5.4 Manual 2012). In this research, the given flexible VISSIM capability is utilized, so that the model-specific parameters' needed adjustments were carried out to fulfil the heterogeneous traffic requirements as the following three phases.

3.2.1.1 Driving behaviour modelling

This phase attempts to represent the different driving behaviour traits as a key factor, the main pillar, the most common and unique characteristics of the heterogeneous traffic. These unique characteristics include but not limited to; the manoeuvrability of the smaller vehicles (i.e., motorcycles, scooters), lane discipline and the aggressive driving manner. On the basis of the car following model Wiedemann 74 that mainly suitable for urban traffic and merging and weaving area, the different driving behaviour altitudes are represented by tuning and resetting the VISSIM different available driving behaviour parameters. The field observation, the traffic data analyzed, and the several previous works associated with modelling heterogeneous traffic flow was highly considered to replicate these traffic characteristics. The field observation done for this research emphasized that the two-wheels vehicles had a continuous stimulus to occupy the front queues by maximizing the interspace between the other travelled vehicles. Therefore, in order to model this manoeuvrability attitude, two signal heads are installed as two stop lines. The first signal head is configured to control all vehicles types movements except the two-wheels vehicles. The second signal head, which is 2.0 m ahead of the first one is assigned to control the smaller vehicles movement as shown in Figure.3.2. Moreover, the queuing forming style was checked as the diamond queueing shape for a better representation of the observed queues under such traffic conditions. The diamond shaped queueing is a given feature in VISSIM that allows for staggered queues (e.g. for cyclists, motorcyclists) to behave according to the realistic shape of vehicles in the real world (VISSIM 5.4 Manual 2012).

On the other hand, the field observation also referred to the side-by-side stacking of vehicles across the roadway. In such conditions, the travelled vehicles were observed to move across the road width within small lateral distances due to the lanes lines' absence as well as the poor lane discipline. So that, the lateral driving behaviour is needed to be changed to meet the obtained realistic data. Following the previous efforts done in the several previous studies as a guideline for this study context, the minimum lateral distances for different vehicle types were readjusted based on the vehicle type and the speed as illustrated in Table 3.1. Also, smooth close-up behaviour parameter is also activated for a reliable representation as close as the reality. By activating this option, vehicles slow down more evenly when approaching a standing obstacle.

In Addition, other simulation parameters values such as emergency stopping distance, the number of observed preceding vehicles, look ahead and back distance, minimum headways and average standstill distance, were changed by giving appropriate values that fulfil the field observation as shown in Table 3.2. The Emergency stop option is used to model the lane change behaviour for vehicles that following their Routes. This Emergency stop distance defines the last possible position for a vehicle to change lanes. The minimum emergency stop distance is 5.0 m measured upstream from the start of the connector, which connects two different lanes. Hence, an additional 5.0 m is added for each additional lane, when a vehicle needs to change more than one lane (VISSIM 5.4 Manual 2012). Also, the look-ahead and look back distance values were changed to fulfil the driving complexities under such heterogeneous traffic. The look ahead distance defines the distance that a vehicle can see forward in order to react to other vehicles either in front or to the side of it within the same approach,

Table 3.1 Min. lateral distances for different vehicle types

Vehicle type	Min. Lateral Distance (m)	
	0 km/h	50 km/h
Two-wheeled Vehicles	0.3	0.7
Normal Vehicle	0.5	0.9
Microbus	0.5	0.9
Minibus	0.6	0.9
Bus	0.6	1.0

Table 3.2 Wiedemann 74 model parameters tuned variables

Parameters	Tuned Variables	
Look ahead distance	Min. 5.0 m	Max. 100.0 m
Look back distance	Min. 30.0 m	Max. 150.0 m
Avg. standstill distance	0.5 m	
No. of observed Vehicles	2 veh	

while the back distance defines the distance that a vehicle can see backwards to react to other vehicles behind (VISSIM 5.4 Manual 2012). The number of observed vehicles affects how well vehicles in the network can predict other vehicles' movements and react accordingly. Therefore, it might be useful to increase this value, especially, if there are several cross sections of the network within a short distance. Moreover, the average standstill distance that defines the average desired distance between stopped cars was modified to meet the field observation obtained data. According to VISSIM, this distance has a variation between -1.0 m and +1.0 m which is normally distributed around 0.0 m with a standard deviation of 0.3 m. If the option of standstill distance for static obstacles is checked, the vehicles using this parameter set will use the given value (default: 0.5 m) as standstill distance (VISSIM 5.4 Manual 2012). Finally, the min. headway values were reset to define the minimum distance to the vehicle in front that must be available for a lane change in standstill condition. The min. headway value describes the minimum time headway towards the next vehicle on the slow lane so that a vehicle on the fast lane changes to the slower lane.

3.2.1.2 Vehicle modelling

Depending on readjusting and modifying the default VISSIM vehicles standard models, this phase aims to replicate as accurate models as it is existing in the real world. The diverse static and dynamic characteristics of the existing vehicles are represented based on the field observation obtained data. The static properties include refers to the different dimensions (i.e. length and width) for each vehicle type, whereas the dynamic characteristics referred to change acceleration, deceleration and desired speed-distribution. The field observation done for this research indicated to four vehicles types observed with different properties as shown in Table 3.3. The two-wheeled vehicles are represented by motorcycles, while the normal car represents the private car and taxis. On the other hand, the microbuses are a representative of the 14 or / and 15 passenger public vehicles. Although these microbuses are a kind of a public transport, they do not have certain stopping locations, they act similarly to the private cars and taxis. In this research the buses represent the public and private buses with 40 passengers or more which belongs to public transportation authorities, schools or companies, while the minibuses represent the smaller busses with 30 passengers or more. Similar to the microbuses, the field observation also indicated that minibuses they do not have certain stopping locations and they used to stope frequently upon the passengers' needs. In addition, it was observed that these vehicles can stop even in the second lane from the curb. These repeated stops generated cause merging and diverging conflicts with the through direct flows. As a result, the stopped vehicles caused an inverse impact as they increase in both the total trip time and the delay time. Likewise, the start-up and headways times were adversely affected, and bottleneck areas were created from these repeated stops.

In most of the developing cities around the world, practically, in Cairo, Egypt as a case study in this research, the insufficient public transportation system plays a real role in the existing traffic

Table 3.3 Vehicle static and dynamic characteristics

Vehicle type	Length (m)	Width (m)	Desired Speed (Km/h)	Acceleration		Deceleration	
				Max.	Desired	Max.	Desired
Motorcycle	1.8	0.6	40	2.5	1.7	1.7	1.2
Car	4.0	1.6	50	1.5	1.2	1.2	1.0
Microbus	5.0	1.9	50	1.5	1.2	1.2	1.0
Minibus	8.0	2.0	30	0.8	0.7	1.2	0.6
Bus	10.3	2.5	30	1.3	0.8	1.4	0.6

operational performance. A lack of mass transit modes forces commuters to use individually owned vehicles. In addition, private and national companies, as well as governmental authorities, have produced their own bus systems for their employees. However, these private units provided have resulted in an increase in traffic demand.

3.2.1.3 Non-lane based traffic modelling

The non-lane based traffic operational system is another salient characteristic of such traffic complexity of the traffic stream of less developing countries. Within this unique system, the drivers accustomed to use the whole width of road and exhibit lack of discipline following lane in particular. The non-lane based traffic system allows the vehicles to occupy any part of the road by change their location laterally based on the road availability and a suitable time gap (headway) in the destination flow without any rules. This gap size is dependent on the speed both of the lane changer and the vehicle that comes from behind on that lane where the lane changer aims to change. Although a part of this phenomenon is due to the driving behaviour, the other part is caused by the absence of the lane lines. In many of the less developed countries where the limited budget for infrastructure and some time the absence of the traffic lane lines importance, the lane lines are also not painted at all or hardly seen in case of its existence. Therefore, for an efficient replication of this operational of an unstructured traffic system where no restrictions to change lanes exists, the road network is replicated by utilizing the space-oriented feature given in VISSIM that allows vehicles to move anywhere in the road without lane restrictions. The default lateral lane-change behaviour parameters in VISSIM, however, are needed to be taken into account for an accurate representation. First, by activating the both side -left and right sides- overtaking and free-lane selection which allows the drivers to change lane only based on the desired safety distance of the trailing vehicle on the new lane. This safety distance depends on its speed and the speed of the vehicle that wants to change to that lane. Second, by changing the lateral lane behaviour to uncheck the cooperative lane change to represent the inconsiderate driving behaviour. By unchecking this option, the following vehicle does not change cooperatively to a lane which is less suited for its own route, and it does not change lanes cooperatively if the leading vehicle speed is more than 10.80 km/h (3 m/s) faster or if the collision time would exceed 10 seconds with the speed of following vehicle increased by 10.80 km/h (VISSIM 5.4 Manual 2012). Third, the desired decision at free flow was checked to be permitted at any position and the same was done to allow diamond shaped queuing. Fourth, the waiting time before diffusion was changed to 180.0 s instead of 60.0 s as a default value. The waiting time before diffusion defines the maximum time that a vehicle can wait at the emergency stop position waiting for a gap to change lanes in order to stay on its route. So, when this time is reached the given value, the vehicle is taken out of the network (diffusion) and a message will be written to the error file denoting the time and location of the removal (VISSIM 5.4 Manual 2012). Finally, the free desired position at free flow option is selected.

3.2.2 Model Calibration

The model calibration is a process in which the different input parameters of the default simulation sets are reset and tuned until the model accurately replicates the field conditions (Siddharth and Ramadurai 2013). By bridging a conditional match of the simulated parameter values with observed traffic field data, the effectiveness and practicability of the simulated models can be represented as close as possible to the reality. As a result of the data limitation, the difficulties related to the field data collection or/and the lack knowledge of the appropriate, readily and available procedures to calibrate and traffic simulation models, most of the previously conducted analysis was done relying on values of default parameters (H. Naghawi, AlSoud, and Alhadidi 2018). Accordingly, skeptics always consider simulation platforms as inexact, unrealistic at best and unreliable black-box technology at worst (Hellinga B.R. 1998). Realizing its importance, earlier researchers (Hellinga B.R. 1998; Park and Schneeberger 2003; Cohen, S. L. 2004; Mathew, T., Radhakrishnan, P. 2010) suggested general guidelines, methodologies and techniques of calibration. Hence, to avoid unrealistic expectations of the capabilities of simulation models, it is essential for the models to be well calibrated and validated to minimize a discrepancy before providing credible results. Therefore, any model created in VISSIM needs to be calibrated so as to sufficiently represent the real world conditions, particularly, when the heterogeneous conditions are considered.

3.2.2.1 Previous works in model calibration

To address the unique characteristics of such conditions, special procedures are required. It is necessary to screen the significant parameters which influence the output of the model in a significant way, especially, the complex manoeuvres under such heterogeneous traffic including the diverse vehicles' static and dynamic properties, manoeuvrability and the driving behaviour parameters. Various calibration methodologies including single and multi-parameter calibration were employed. These methodologies attempt to comprise different optimization algorithms using different Measure of Effectiveness (MOEs) to prescribe the consistency between field and the corresponding simulated data. It is noteworthy that the perception of the majority of studies for the calibration of different simulators done till now, a high attention was paid for sensitivity analysis to predict the significant parameters that may affect models accuracy. Sensitivity analysis is considered one of those significant ways that can be used to figure out these significant parameters. The sensitivity analysis is the study of how the uncertainty in the output of a mathematical model or system (numerical or otherwise) can be apportioned to different sources of uncertainty. In other words, it is the process of recalculating the outcomes under alternative assumptions to determine the impact(s) of such a variable(s). By incrementing the values of the candidate parameters by small units, the sensitivity analysis is assessed. Consequently, to confirm the significant consistency of the candidate sensitive parameter(s) on the MOEs indices, a statistical test is needed (Mathew, T., Radhakrishnan, P. 2010).

Most of early literature of micro simulators calibration were categorized either by the optimization methodology, the MOE of fitness function and by the analysis way of parameter consistency. First, the parameter optimization is one of the important techniques for calibration, so that several optimization algorithms are utilized to select the candidate parameters for calibration. In order to generate random sets for these parameters within specified bounds, the heuristic optimization techniques (i.e. Genetic Algorithm (GA), Monte Carlo, PSO...etc.) are widely used. Through the studies reviewed, it was realized that GA is used as one of heuristic search methods in many of previous studies to determine a suitable combination of parameters values for different simulators (i.e. VISSIM, PARAMICS, CORSIM, TRANSSIMS…etc.). GA underlines on the principle of best individual survival from the population through many iterations (Rrecaj, A., and M.Bombol, K. 2015). Lee et al (2001) used a GA-based approach to calibrate a PARAMICS traffic model (Lee, D. H., Xu, Y., 2001). Also, Kim and Rilett illustrated a GA-technique for CORSIM and TRANSSIMS models for two freeway segments in Houston, Texas Using ITS data (Kim S., Kim W. and Rillet L, 2005) while Menenni et al. used GA to calibrate VISSIM parameters based on speed-flow data (Menneni, S., Sun, C., Vortisch, P. 2008). Taking into consideration the heterogeneous conditions, Mathew and Radhakrishnan (2010) used GA and found the sensitive parameters by increasing each parameter by 10% while keeping other parameter values constant (Mathew, T., Radhakrishnan, P. 2010). The authors concluded that the driving behaviour parameters, particularly, lateral and longitudinal parameters which reflect the heterogeneous complexities, were found as significant sensitive parameters. Meanwhile, to reduce the huge number of scenarios of parameter combinations, the Latin Hypercube technique was used in several studies (Manjunath, P., and Mathew, T. 2013; Siddharth and Ramadurai 2013; Rrecaj, A., and M.Bombol, K. 2015). Park and Qi used the Latin hypercube experimental design to reduce an overall number of 200 scenarios with 5 seeded simulations with a total of 1000 runs for the eight calibrated parameters (Park, B., and Schneeberger, J. 2003; Rrecaj, A., and M.Bombol, K. 2015). Similarly, Mathew et al. (2013) used Latin hypercube method to generate appropriate scenarios and then the solution parameter set was determined by the use of GA (Manjunath, P., and Mathew, T. 2013). On the other hand, other optimization methods such as Elementary Effects (EE) method were also used to find out the important parameters that affect the model accuracy. Based on Quasi-optimized trajectory as an improvement to EE method, Ge and Menendez (2014) used this approach in a case study involving a network in City of Zurich to reduce the computation time needed for sensitivity analysis (Ge Q., and Menendez, 2014). Likewise, Park and Schneebergern (2003) utilized Monte Carlo Algorithm to calibrate a model for a larger network in Virginia, USA of 12 intersections with coordinated and actuated signals (Park, B., and Schneeberger, J. 2003). Fellendorf and Vortisch (2001) developed a calibration method for VISSIM based time-step technique by setting two values (1.0 and 0.1 s) (Fellendorf, M., and Vortisch, P. 2001). Likewise, Fuzzy Logic algorithm was calibrated based on roadway simulation model using distance divergence and desired speed as simulation parameters (Wu, J., Brackstone, M., and McDonald, M. 2003). Gundaliya et al. calibrated a heterogeneous traffic flow VISSIM model based on cellular automata for midblock by using driver behaviour probability as a parameter and solved using a complete enumeration (Gundaliya, P. J., Mathew, T. V., Dhingra, S. L. 2008).

Second, various MOE of fitness function are often used to estimate the calibration efficiency including travel time, queue length, maximum flow rate, capacity and delay. These different MOEs are used to prescribe the consistency between field observed and the corresponding simulated data. The average travel time was evaluated in many of previous studies (Park, B., and Schneeberger, J. 2003; Kim S., Kim W. and Rillet L, 2005; Kim S. 2006). For instance, in Park and Qi (2003), the initial evaluation of the left-turn movements average travel time was chosen as MOE as it was considered that it directly reflects the level of service (Park, B., and Schneeberger, J. 2003). Kim J.

and Rilett examined the travel time to represent observed distributions obtained from the field and not the mean or central tendency mean of the parameters (Kim S., Kim W. and Rillet L, 2005). Using a bi-level calibration framework to develop a methodology in which the Origin-Destination (OD) matrix is calibrated simultaneously with model behaviour parameter using In his dissertation, Seung-Jun Kim (2006) emphasized that traffic conditions have large variability. Therefore, he indicated that the aggregated MOE such as the average travel time may not be the most appropriate MOE. On the other hand, since it is considered as higher-level measurement and a lower level defined parameter, capacity also was used to calibrate a VISSIM model by a multi-parameter sensitivity analysis (Lownes, N. E., and Machemehl, R. B. 2006). The speed-flow graphs also were used to develop a method of pattern recognition which serves as a methodology to study the match of the speed-flow graphs from simulation and field observations. The dissimilarity of two speed-flow graphs can be measured by calculating the amount of uncovered area by the other shape. Consequently, a generic objective function was developed based on minimizing the measured dissimilarity (Menneni, S., Sun, C., Vortisch, P. 2008). On the contrary, unlike the above-reviewed papers, some researches selected the candidate parameters as well as The minimum and maximum values were based on the engineering judgment of the authors (Kim S., Kim W. and Rillet L, 2005).

Third, the previous studies can also be distinguished based on the analysis way of parameter consistency to figure out and define the most significant factors that influence the model efficiency. For this purpose, either non-parametric (i.e. Moses, Wilkoxon...etc.) statistical based tests or parametric (i.e. ANOVA, F-test, T-test...etc.) statistical based tests are employed to find the sensitive parameters that influence the model efficiency (Rrecaj, A., and M.Bombol, K.2015). Among the literature reviewed, it is noticeable the Analysis of Variance (ANOVA) test, as a statistical technique is being widely used for obtaining the optimal set of parameters by drawing inferences about population means 4),23),26). By drawing inferences about population means, ANOVA is used to come up with conclusions whether the particular studied factors influence the response variable (Siddharth and Ramadurai 2013). ANOVA can indicate whether the parameter is important for the results, by measuring the mean difference and the sum of squares (SSR) between different groups at either the 0.5 or the 0.05 level of confidence (Rrecaj, A., and M.Bombol, K.2015). Lots of considerable studies used ANOVA test to perform the sensitivity analysis with different evaluation indices. Depending on evaluating the maximum queue length and travel time, ANOVA was used to perform a variance analysis of eight model parameters (Muhan N., et al. 2013), while Manjunatha et al. used the delay values of both globally and locally calibrated models (Manjunath, P., and Mathew, T. 2013). On the other hand, to fulfill the sensitivity analysis for calibrating the ranges of the parameter values, F-test and T-test are also used to measure the mean difference between groups (Rrecaj, A., and M.Bombol, K.2015). In another study done by Park and Qi (2005) it was found the minimum gap and the desired speed distribution were two parameters important to the results. By utilizing F-test to measure the mean difference between groups, the authors concluded that the travel time became higher when the minimum gap increased or the mean desired speed decreased Park and Qi (2005). Based on the above reviewed literature, it can be concluded that the driving behaviour parameters are found as sensitive parameters which influence the microscopic models in significantly, especially, under such heterogeneous traffic conditions.

3.2.2.2 Calibration process of this study

The methodology proposed in this study context includes representation of vehicles, geometry and traffic, followed by identification of calibration parameters, settings their ranges heuristically until the simulated models are represented as close as possible to the reality. Despite the enormous simulation parameters provided in VISSIM that can be reset and refined during the calibration, only some parameters may have a significant effect on the models. Lots of literature emphasized that the driving behaviour parameters that affect the model accuracy significantly (Siddharth and Ramadurai 2013; Manjunath, P., and Mathew, T. 2013; Rrecaj, A., and M.Bombol, K. 2015). These parameters include but not limited to desired speed, acceleration and clearance distance (Asamer, van Zuylen, and

Heilmann 2018), minimum headway, standstill distance, lane change distance, emergency stopping distance and waiting time before diffusion (Park and Schneeberger 2003).

In the present study, accordingly, the driving behaviour parameters are also considered for the calibration process. Hence, the sensitivity analysis is used to find the optimal values of the most significant driving behaviour candidate parameters that influence the models efficiency. The calibration proposed in this research is underlined on four steps. The first step is to define the network elements in VISSIM by representing the geometric configuration, vehicles properties, traffic control system and driving behaviour initially with the default setting (pre-calibrated) values to ascertain the need of calibration. The second step is to change the default parameters until the absolute error between the field and simulated MOE is less than the threshold values. The travel time between the consecutive intersections in both directions, west and eastbound along the studied corridor is selected as MOE. In this study, the acceptable variation threshold is 17.0 % or less. By incrementing the values of the candidate parameters by small units, the sensitivity analysis is assessed. Consequently, to confirm the significant consistency of the candidate sensitive parameter(s) on the MOEs indices, a statistical test is needed The last step is to represent the model by the new values given to the parameters until the absolute error is insignificant. In this study, the estimation of the maximum and minimum of parameter values were based on the relevant previous research, as well as the engineering judgment. Similarly, to the first review study ANOVA test, as a statistical technique was utilized for measuring the closeness of the observed and simulated travel time. The outcomes of the ANOVA single factor test indicated the significant consistency between the simulation models efficiency and the different simulation parameters. The travel time between the consecutive studied intersections as MOE for the different calibration trials as shown in Table 3.4. The ANOVA results are illustrated in Table 3.5, where (SS) is the sum of squares, (df) is the degree of freedom, (MS) is the mean square deviation, (F) is the test statistics, (P-value) is the probability value under the appropriate F, (F-crit) is the critical value of F (5,30) distribution under 5% significance level. The variance analysis shows the F-value is bigger than

F-crit with a small P-value which emphasizes that the null hypothesis is rejected and the readjusted simulation parameters have a significant impact on the efficiency of models. The calibration results referred to the significant impact of the different vehicles' dynamic characteristics, as well as the driving behaviour parameters on the model efficiency. As mentioned in the previous sections, in this research the candidate driving behaviour parameters are the minimum lateral distances for different vehicle types, the number of observed preceding vehicles, the average standstill distance and look ahead and back distances.

Table 3.4 Travel time difference in (s) between the consecutive intersections for the different calibration trials

	YA to AT		AT to AA		AA to ME		ME to AA		AA to AT		AT to YA	
Observed	65		223		174		202		593		117	
1st trial	85.4	31.4%	286.5	28.5%	255.3	46.7%	206.1	2.0%	401.9	-32.2%	78.6	-32.8%
2nd	71.9	10.6%	259.7	16.4%	275.4	58.3%	206.9	2.4%	564.2	-4.9%	79.9	-31.7%
3rd	72.4	11.4%	291.6	30.7%	294.1	69.0%	158.9	-21.4%	657.5	10.9%	94.8	-19.0%
4th	93.7	44.1%	327.1	46.7%	237.2	36.3%	206.3	2.1%	604.2	1.9%	328.5	180.7%
5th	102.2	57.2%	282.1	26.5%	153.0	-12.1%	162.1	-19.8%	226.1	-61.9%	70.3	-39.9%
6th	57.4	- 11.7%	239.4	7.4%	155.4	-10.7%	157.7	-21.9%	224.8	-62.1%	42.4	-63.8%
7th	52.9	- 18.6%	309.5	38.8%	160.4	-7.8%	157.2	-22.2%	225.7	-61.9%	40.5	-65.4%
8th	53.8	- 17.2%	246.9	10.7%	150.1	-13.7%	157.1	-22.2%	226.3	-61.8%	39.1	-66.6%
9th	62.4	- 4.0%	247.4	10.9%	153.0	-12.1%	364.0	80.2%	274.7	-53.7%	168.9	44.4%
10th	66.0	1.5%	221.0	-0.9%	195.0	12.1%	206.0	2.0%	594.0	0.2%	98.0	-16.2%

Table 3.5 ANOVA test results of travel times by different simulation trials

ANOVA: Single Factor						
Source of Variation	SS	df	MS	F	P-value	F crit
Between Groups	3.160872	9	0.3512	2.351	0.0026	2.073
Within Groups	7.466882	50	0.1493			
Total	10.62775	59				

3.2.3 Model Validation

The validation such a process required to check the extent to which the simulated model is representing the reality. The validation is neither about selecting the best model among several alternatives nor about testing the goodness of fit between two random samples. However, the validation is about comparing two models of the simulated and the observed (Ni et al., 2004). So that, in this study context, the model validation process is achieved based upon the statistical validation method to test the goodness of fit and the confidence intervals to quantify the similarity between field observed and simulated values (Toledo and Koutsopoulos 2004; Joseph E. Hummer and Jagannathan 2008; H. H. Naghawi and Idewu 2014). Therefore, a comparison between the VISSIM generated traffic volumes and the corresponding observed volumes was conducted. For this purpose, the most popular goodness of fit measures: GEH empirical test, as well as the Root Mean Square Percent Error (RMSPE) were utilized to accomplish the validation process as shown in Table 3.6. The GEH empirical static test was designed as a modified Chi-square static test, while the RMSPE is used to replicate the error as a percentile rate (Toledo and Koutsopoulos 2004; H. Naghawi, AlSoud, and Alhadidi 2018). Regarding the GEH test, according to Design Manual for Roads and Bridge (DMRB), the model can be used confidently when the variance (the difference between the observed and simulated counterparts) of 85.0% of the total population is less than 5.0 (Oketch and Carrick 2005; Feldman 2012). On the other hand, the RMSPE acceptable threshold should be within a range of 15.0% or less (Ni et al. 2004; Hourdakis, Michalopoulos, and Kottommannil 2003). Based on Equation 3.1 and Equation 3.2 the GEH and RMSPE test was calculated. The results indicate that the model replicates reality with high accuracy.

$$GEH = \sqrt{\left(\frac{(m-c)^2}{0.5*(m+c)}\right)} \tag{3.1}$$

Where:

m is the output traffic volume from the simulation model (veh/h);

c is the input traffic volume (veh/h)

$$RMSPE = \sqrt{\frac{1}{N}\sum_{1}^{N}\left(\frac{Y_{Sim} - Y_{Observed}}{Y_{Observed}}\right)^2} \qquad (3.2)$$

N is the number of simulation Runs (veh/h);

$Y_{Observed}$ is the simulation run throughput volume (veh/h);

Y_{Sim} is the simulation run throughput volume (veh/h).

Table 3.6 Model validation by GEH and RMSPE values

Intersection	Movement	Observed volumes (veh/h)	Simulated volumes (veh/h)	GEH variance value	RMSPE error value (%)
AT	W-E	2270	2346	1.58	1.4
	E-W	3040	2995	0.82	1.5
	S-N	1640	1673	0.81	2.0
	N-S	1434	1394	1.06	2.8
AA	W-E	1905	1978	1.66	3.8
	E-W	3578	3294	4.84	7.9
	S-N	1596	1498	2.49	6.1
	N-S	1716	1699	0.41	0.99
ME	W-E	2245	2142	2.13	4.59
	E-W	2574	2466	2.15	4.19
	S-N	1385	1321	1.74	4.62
	N-S	1244	1266	0.62	1.77

Another comparison between the simulated and observed travel time between consecutive intersections was used for the model validation. The results indicated the accuracy of the model as shown in Table 3.7. Although the differential ratio between the observed and simulated travel time fluctuated between 0.9% and 1.98, it recorded 12.07% and 16.23% for the travel time from AA to ME and from travel time from AT to YA respectively as a result of the non-lane based system impacts.

Table 3.7 The travel time comparison for validating the model

Average Travel Time	Simulated (s)	Observed (s)	% Error
YA - AT	66	65	1.54
AT - YA	98	117	16.23
AT- AA	221	223	0.9
AA - AT	564	593	1.18
AA - ME	195	174	12.07
ME - AA	206	202	1.98

3.3 Site Description

In order to provide credible and reliable results, realistic data for this research, three consecutive existing conventional signalized intersections located in an arterial corridor of downtown Cairo, the capital city of Egypt were selected to represent a realistic case study. This arterial is considered one of the most significant corridors in Cairo as Figure.3.2 depicts. This corridor connects the Central Business District (CBD) with new urban residential communities located to the east of the capital retail. Maximizing the advantage of its high traffic volume and connectivity, many commercial and industrial facilities are located along the road. In addition, a largely residential area is also located adjacently next to the studied intersections. The three intersections namely Al Tayran (AT), Abbass Al-Akkad (AA) and Makram Ebid (ME), are suffering from an adverse operational performance with a low LOS (Elazzony et al. 2011; El Esawey and Sayed 2011b; Shokry et. al 2017). The studied intersections are classified as four-leg intersections. The intersected major and minor approaches are a three-lane divided road with a posted speed of 50 km/h. However, as a non-lane based traffic system as a salient property of the heterogeneous traffic characteristics due to the driver aggressiveness and the absence of the lane markings, drivers used to perform as four lanes per approach flow on the main studied corridor as shown in Figure.3.3 and Figure.3.4. The surrounding environment was observed and considered as a one of the key factors that affect the traffic performance of the studied intersections. The field observation reveals the aggressive driving behaviour which can be recognized obviously

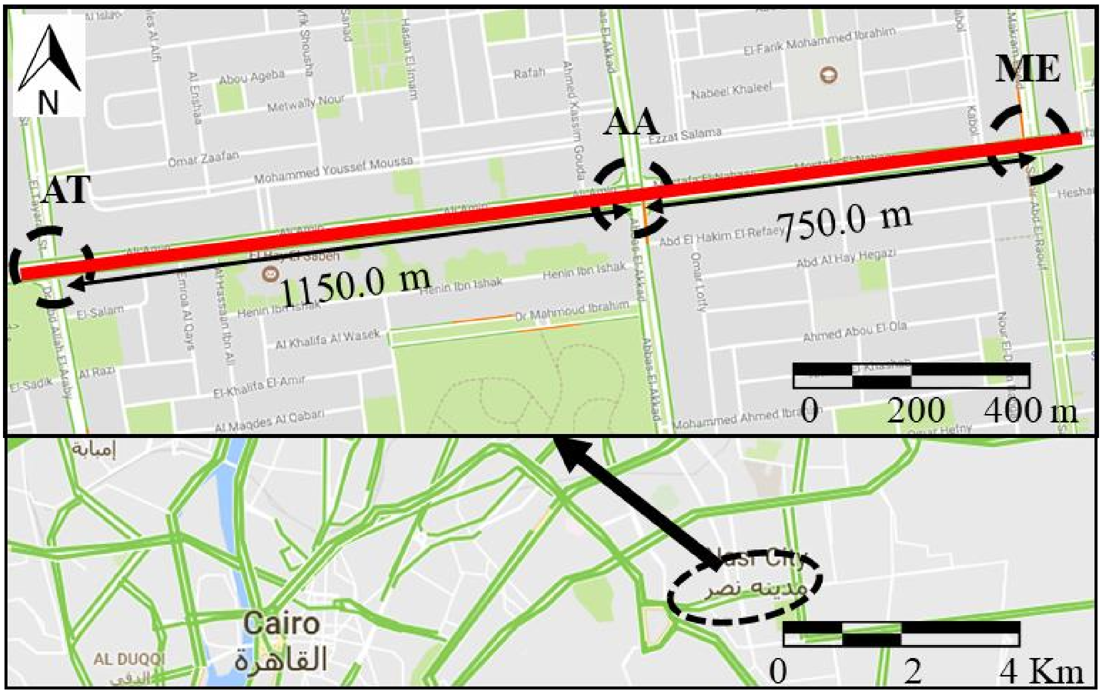

Figure 3.2 A Google map of the case study

Figure 3.3 The non-lane based traffic system in the three-lane studied corridor

Figure 3.4 Non-lane based traffic system in the three-lane studied corridor

through the overtaking from both sides, undisciplined lane behaviour and the small lateral and head distance between different travelled vehicles. Also, the frequent stops of different public transportation modes (i.e. microbuses, minibuses, shuttle buses…etc) were observed along the studied corridor. Due to the absence of certain stopping locations for these public vehicles, they used to stop frequently upon the passengers' needs even sometimes in the second lane from the curb. Accordingly, merging and diverging conflicts with the through direct flows were occurred as an inverse impact on the entire intersections. Likewise, the absence of the priority rules along the studied corridor was clearly revealed from the field observation, particularly, inside the unsignalised U-turns as shown in Figure.3.4.

Regarding the buses movements inside the intersections, despite the exclusive bus lane existence, it is not permitted for shuttle buses, and other private buses such as school buses to use those exclusive lanes. These bus lanes are exclusively only for the city public buses that are belonging to the public authority. Alternatively, other buses have to use the ordinary lanes and share the same right of way with the other vehicles' types making that mixed composition as shown in Figure.3.5 and Figure.3.6.

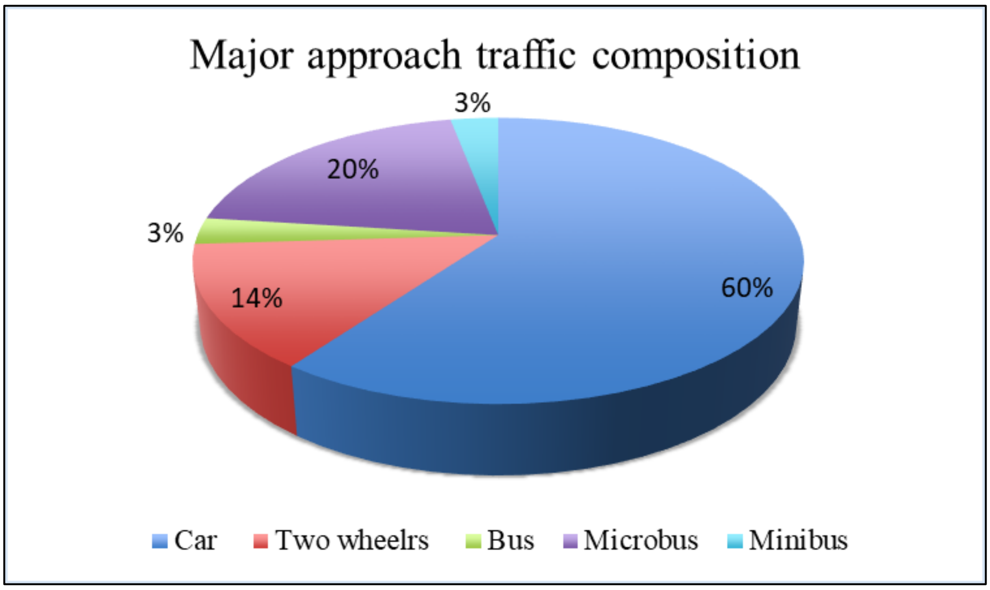

Figure 3.5 Traffic compositions of the major street of the studied corridor

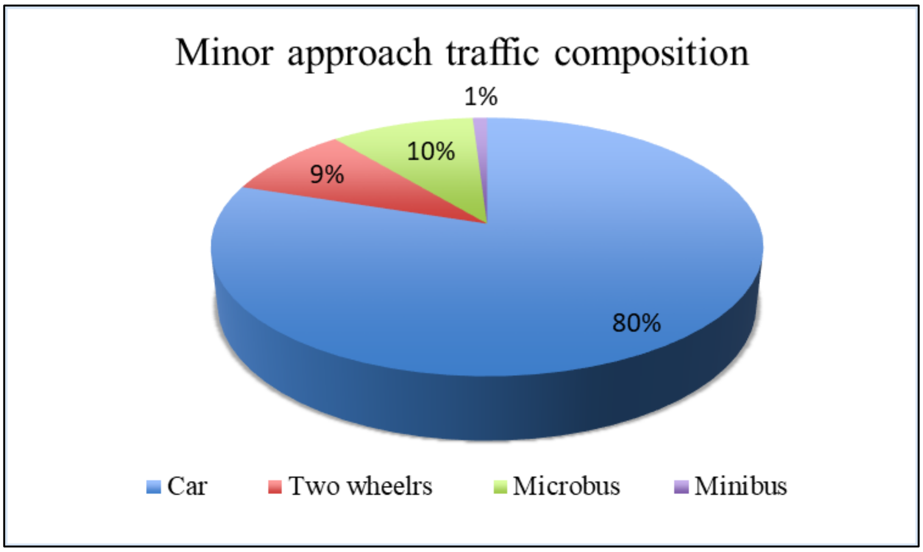

Figure 3.6 Traffic compositions of the minor intersected streets of the studied corridor

3.4 Data Collection

In this research, actual realistic data were made available by the Department of Civil Engineering, Ain Shams University, Egypt. The data collection could be accomplished based on full motion video observation technique. The required details of the three studied intersections include the relevant data needed for the heterogeneous traffic flow, as long as the surrounding environment are collected from video observation. The conducted survey also includes the traffic composition of each turning movement from the different approaches periods including morning, afternoon, peak and off-peak periods. The recorded observation includes the fifteen-minute volumes in the morning between (8:00 to 9:00) and (10:00 to 11:00), while the evening observation between (16:00 to 17:00). Within these recorded videos, traffic flows, bottleneck areas, and the operating system could be efficiently observed. These video shots were used in estimating the major factors that have an impact on the intersection performance for analyzing the current situation. Therefore, a comprehensive traffic analysis is conducted to estimate these factors including the saturation headway rather than the flow rates. Thus, the saturation flow rates for straight, U-turning, and left-turning streams are estimated during the analysis process.

3.4.1Traffic Environment

The traffic volumes including the directional flow ratios were collected at the three entire intersections as shown in Figure.3.7. It can be realized that the arterial corridor experience high through traffic volumes, while the minor approaches suffer from high left-turning volumes. The through traffic movement was the highest among the studied intersections between 38.0 % to 79.0 %, while the left-turns ratios were ranged between 6.5 % to 65.0 % and the right-turn free flow traffic has oscillated between 2.5 % to 23.0 % as shown in Figure.3.8. The maximum observed traffic volumes were 8384 veh/h, 8795 veh/h and 7448 veh/h for AT, AA and ME intersection respectively as shown in Figure.3.7. The total traffic volume and the directional flow ratio of each approach for each intersection are illustrated in Figure.3.8. The observed traffic composition consisted of 75.0% of normal vehicles in, 10.0% heavy vehicles (including buses, minibuses, and small trucks) and 15.0% of motorcycles. Also, the traffic composition of each turning movements and its complexities was obtained and analyzed. The traffic flow characteristics as well as signals' time plans, the intersections are described as follow:

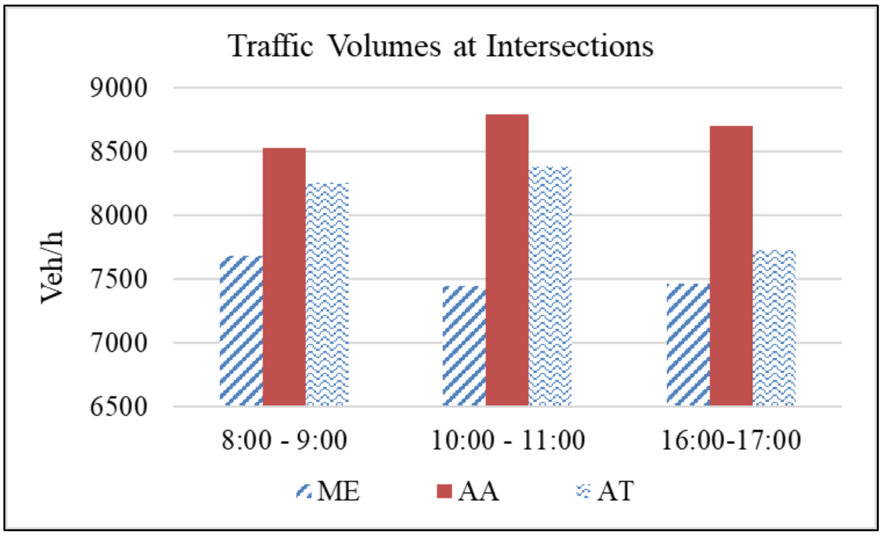

Figure 3.7 The traffic volumes of the three studied intersections.

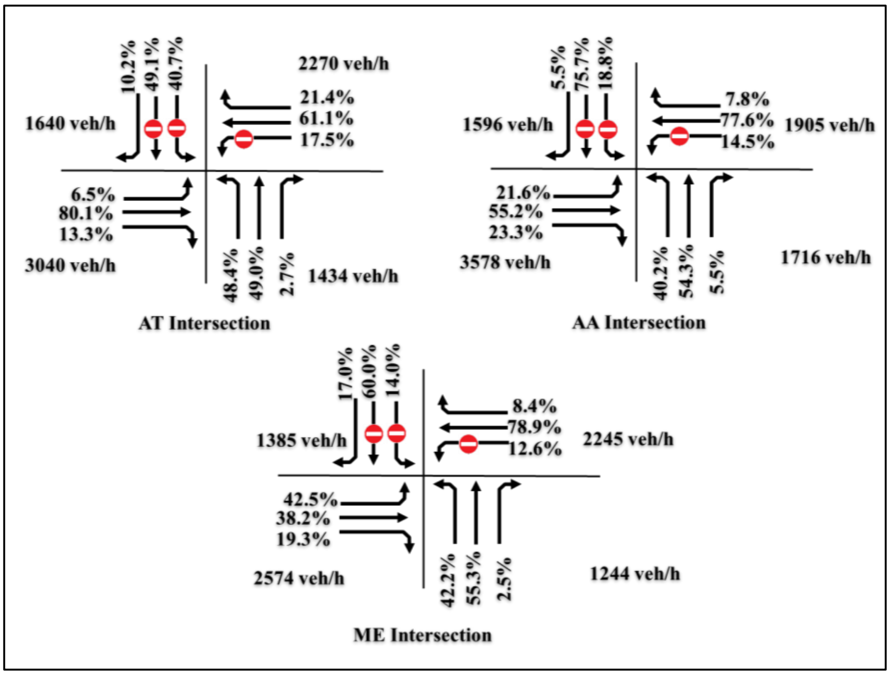

Figure 3.8 The traffic volumes of the three studied intersections.

1. All the intersections studied were operated by two protected fixed cycle signal groups with a conventional phase (red –green-amber) for both major and minor stream as shown in Figure 3.9;

2. The major approach signal group controlled only the through traffic flows (west and eastbound), while the minor phases controlled only the northbound through and left-turning flows in one protected phase;

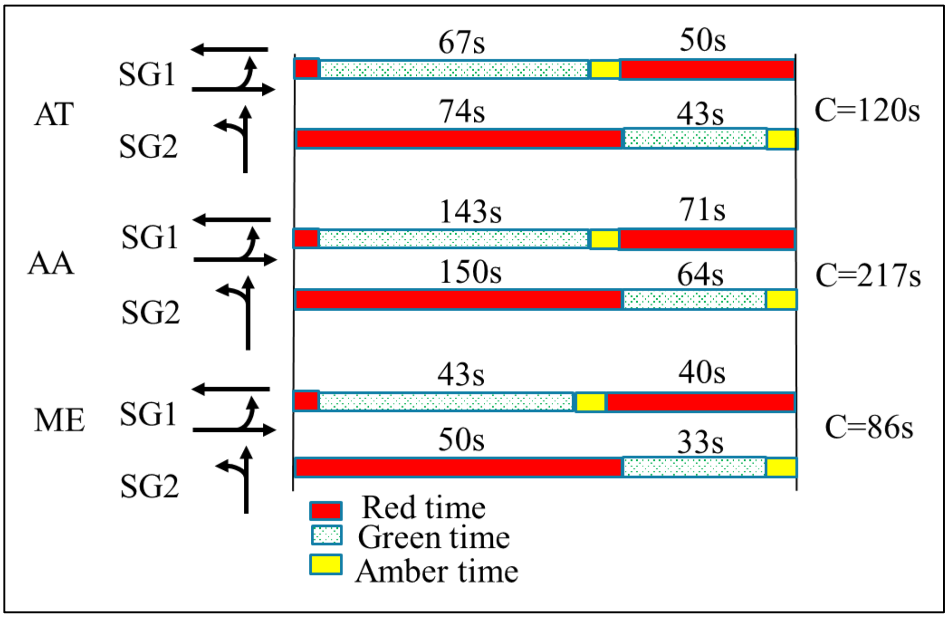

Figure 3.9 The signal time plans inside the intersections

3. Because of the prohibition of the major stream left-turning flows inside the intersections, the eastbound left-turning flow is enforced to make indirect left-turning movement through the U-turns provided in the east of the intersections, while the westbound left flows had to use the U-turns in the west of the main intersections;

4. Similarly, the unsignalized southbound left-turning flow is enforced to use the U-turns in the west of the main intersections then continue traveling in the major eastbound stream as it is depicted in Figure.3.10.

Based on the observation, the heterogamous conditions influence on the traffic operational functionality was clearly revealed. Driver aggressiveness was observed through, lane changing behaviour as well as the manoeuvrability of small vehicles and stop line violation. As a result of the lane marking absence, drivers used to minimize the lateral distance and four vehicles were observed in the three lanes divided corridor. Also, the two- wheeled vehicles usually had a continuous stimulus to sneak and occupy the front queues through the interspace between the other bigger vehicles. During the red time, drivers used to exhibit a greedy behaviour by occupying reserved for left turning traffic.

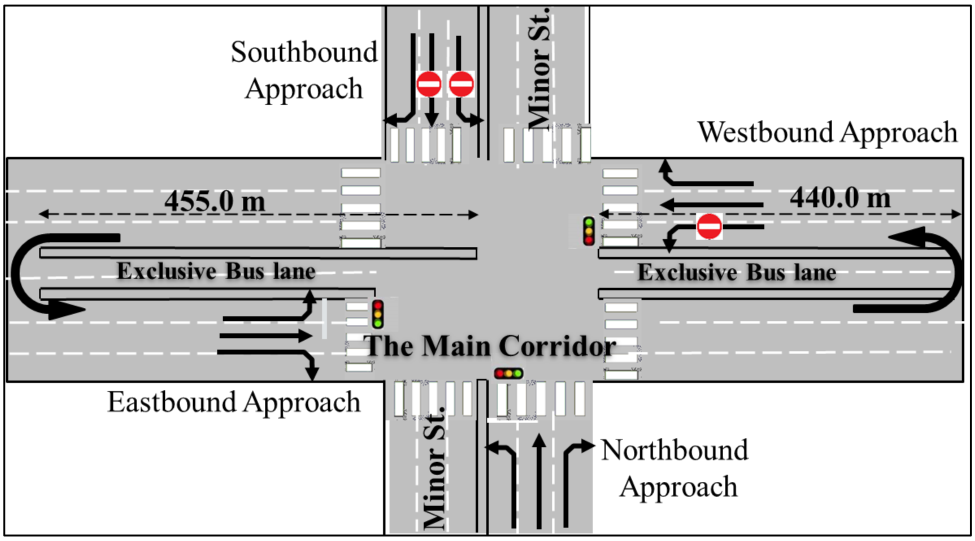

Figure 3.10 A typical geometric layout of the case study

A free channelized lane is provided as Right on Red (ROR) operation for both major and minor right-turns. The intersections are controlled by a pre-timed traffic signal with a cycle length of 120.0, 217.0 and 86.0 seconds for AT, AA and ME intersection respectively as shown in Figure.3.9. Each signal comprises of two signal groups with a conventional phase. The first signal group to control the arterial west-east flow, while the second group is assigned for the minor approach north- flows. The southbound through and left-turns are not permitted to perform at the intersections. These movements must perform in-direct left-turn by using the U-turns provided to the east of each intersection as it is depicted in Figure.3.10.

3.4.2 Geometric Data

The geometric characteristics of the existing entire intersections were obtained by utilizing Google Earth. These characteristics include the lane widths of a different approach, the distances between the studied intersection and other adjacent intersections as well as the distance of indirect U-turns up and downstream of the existing entire intersections.

The analyzed intersections had the following geometric criteria:

1. All intersections were classified as four-leg intersections;

2. Each Intersection consisted of three lanes of 3.5 m width / lane for both the major and minor approaches.

3. An exclusive bus lane per direction of a 3.5 m width / lane was provided in the main corridor for the public buses per direction, excluding the other three lanes per direction.

4. A free channelized right-turn lane was also provided on the studied intersections for both major and minor streets.

The distance between AT and AA intersection is 1150.0 m, while it was 750.0 m between AA and ME intersection as shown in Figure.3.2. Meanwhile the distances between the indirect U-turns up and downstream of the existing entire intersections are depicted in Figure.3.11, Figure.3.12 and Figure.3.13 for intersection AT, AA and ME respectively.

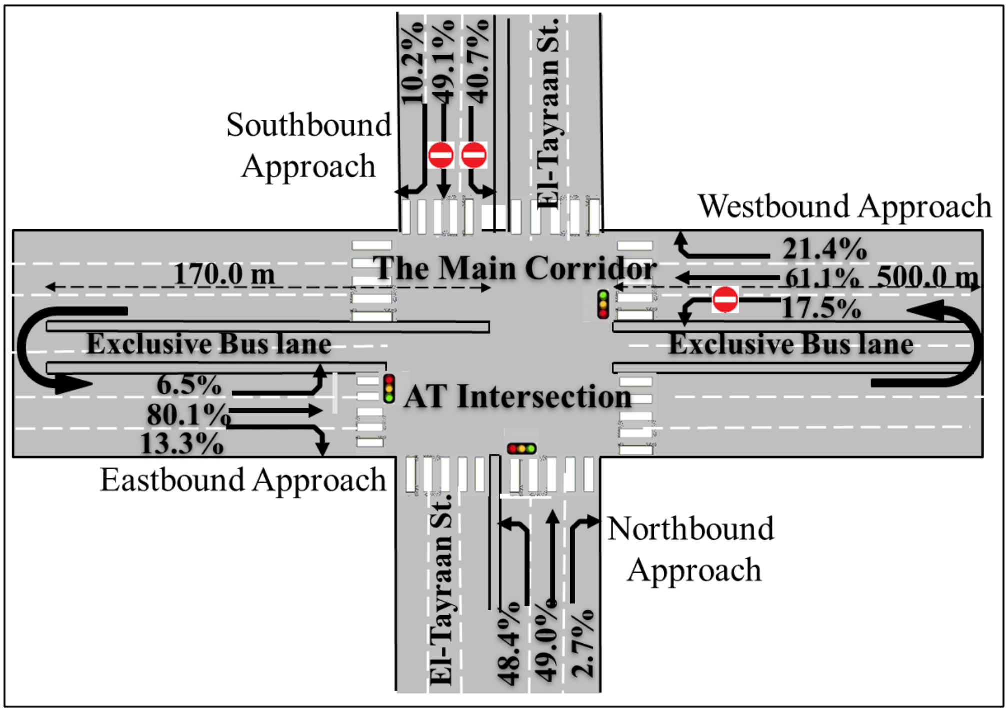

Figure 3.11 A typical geometric layout of AT intersection

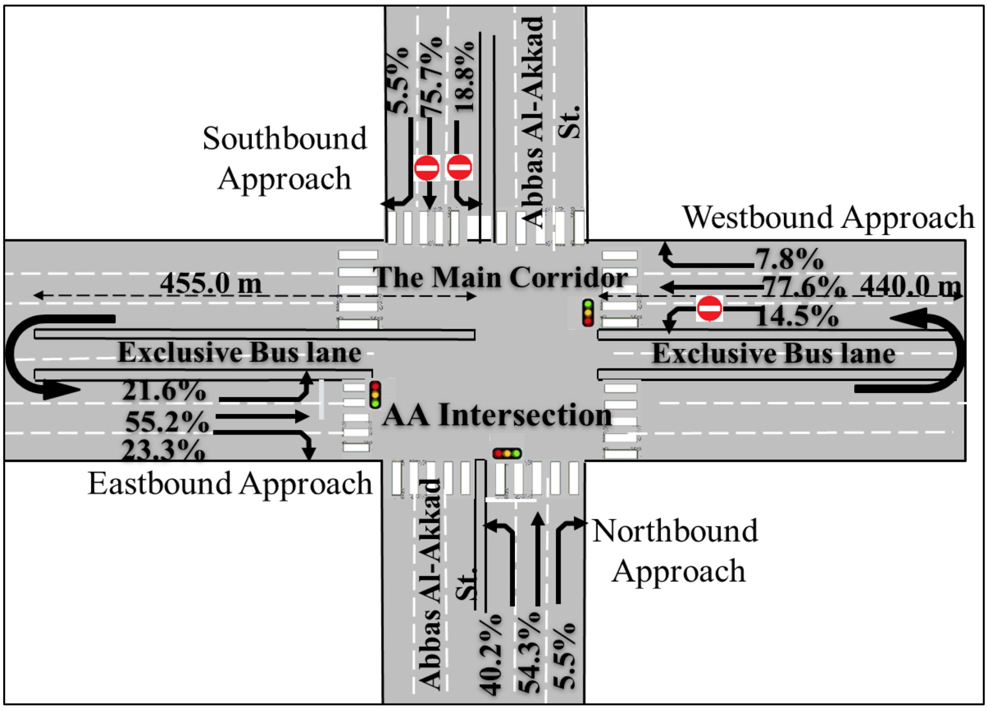

Figure 3.12 A typical geometric layout of AA intersection

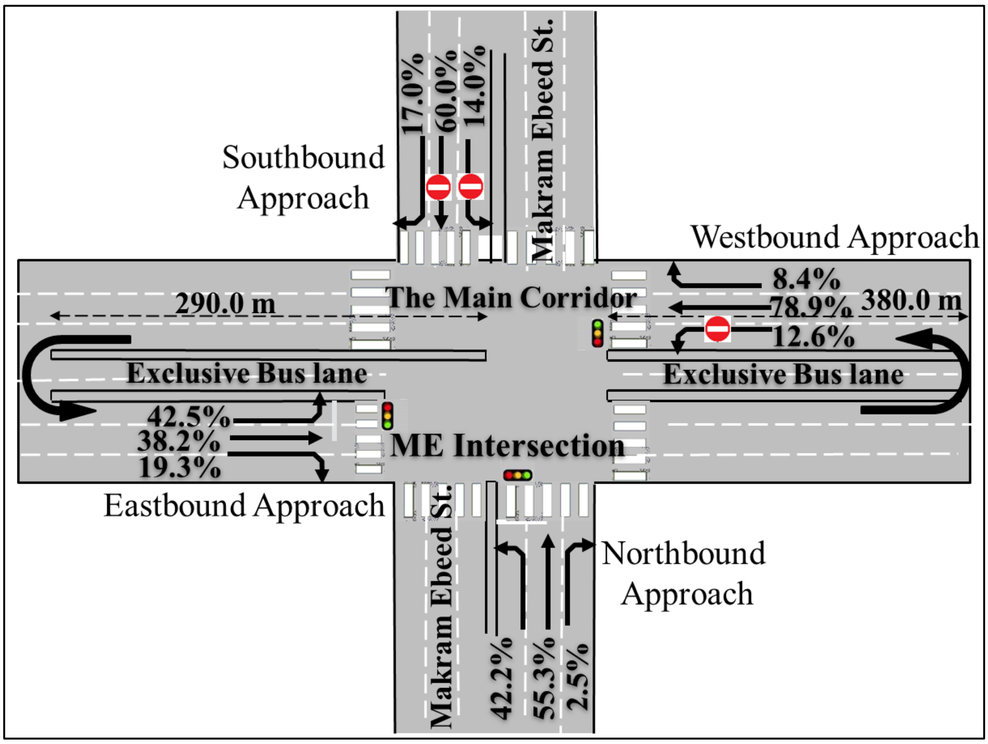

Figure 3.13 A typical geometric layout of ME intersection

3.5 Evaluating the current operational performance

Seeking reliable results and enough fair evaluation of the entire existing intersections operational performance, more than 50 iterative trials were executed with one hour of simulation time for each run. Based on the simulation outputs, several performance indices such as intersection throughputs, total travel time, average delay as well as average stopped delay per vehicle, average speeds along the corridor, queue length and the number of stops of each vehicle were detected. Pursuing accurate measurements, 25 travel time sections were defined, 14 queue counters were allocated for each travel approach along the studied corridor. Meanwhile 77 data collection point were assigned for each lane of the analyzed intersections.

The simulation outputs revealed that operational performances studied corridor got dramatically impacted because of the abovementioned traffic conditions, especially, during the peak periods under heavy traffic conditions. Based on the results, it was emphasized that the non-lane based traffic system, the aggressive driving behaviour and the various dynamic properties are the most influential parameters affecting the intersections performance inversely. Also, the diverse dynamic properties of the existing traffic compositions could obviously influence the queues discharging rates. Thus, the traffic demand along the arterial corridor experienced an obvious long travel time, particularly, during peak hours. Accordingly, the results indicated to the exceeding travel time of all directions inside the all studied intersections as shown in Figure.3.14. As a result of prohibition the direct operation of the southbound flow, the southbound travel time as well as their queue length were experienced a significant adverse impact as shown in Figure.3.14 and Figure.3.15. So that, the westbound travel time experienced longer travel time and extra delay comparing to the eastbound direction as shown in Figure.3.14.

On the other hand, the average and maximum queue lengths of all directions, particularly, the eastbound flow were significantly grown as shown in Figure 3.15. Moreover, all of the studied intersections could not discharge the existing traffic demands. In other words, it can be recognized that the intersections capacities were obviously lower than the demand. Cumulatively, during the peak hours, queue spillbacks of adjacent intersections affect and disrupt the adjacent intersections along the corridor.

Additionally, the non-signalized median U-turns along the corridor failed to accommodate the existing traffic during peaks, especially, due to the absence of the priority rules and could not operate efficiently under the heavy approach volumes. Therefore, the MUTs resulted in bottlenecks because of the merging traffic ahead along the studied corridor. Thus, the westbound and eastbound travel times as well as the average delay along the subject corridor were obviously increased. Consequently, the whole corridor operation was inversely impacted as shown in Figure.3.14. Entirely, the level of service inside the intersections experienced significant reduction. The estimated average speed, the average stopped delay per vehicle, and the average number of stops per vehicle were certainly affected and revealed the poor operational performance of the studied intersections as shown in Table 3.8.

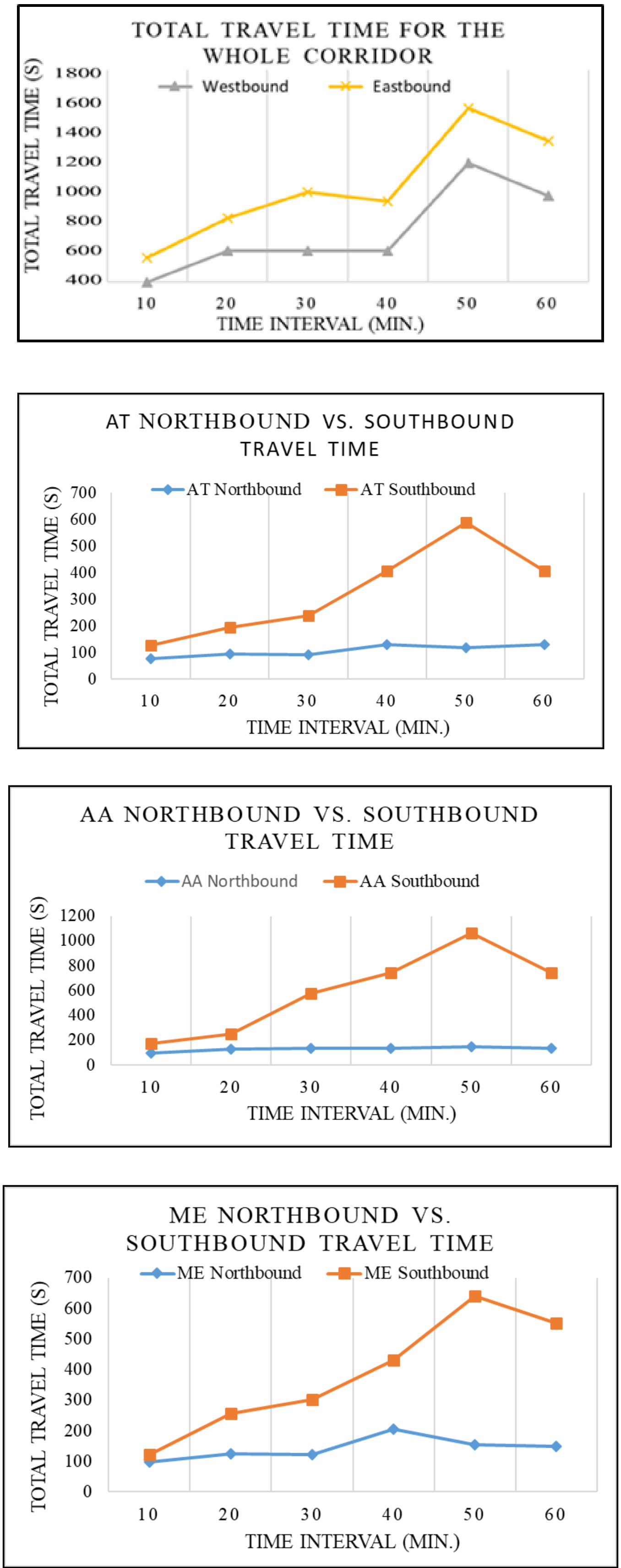

Figure 3.14 The total travel time of the entire analyzed intersections

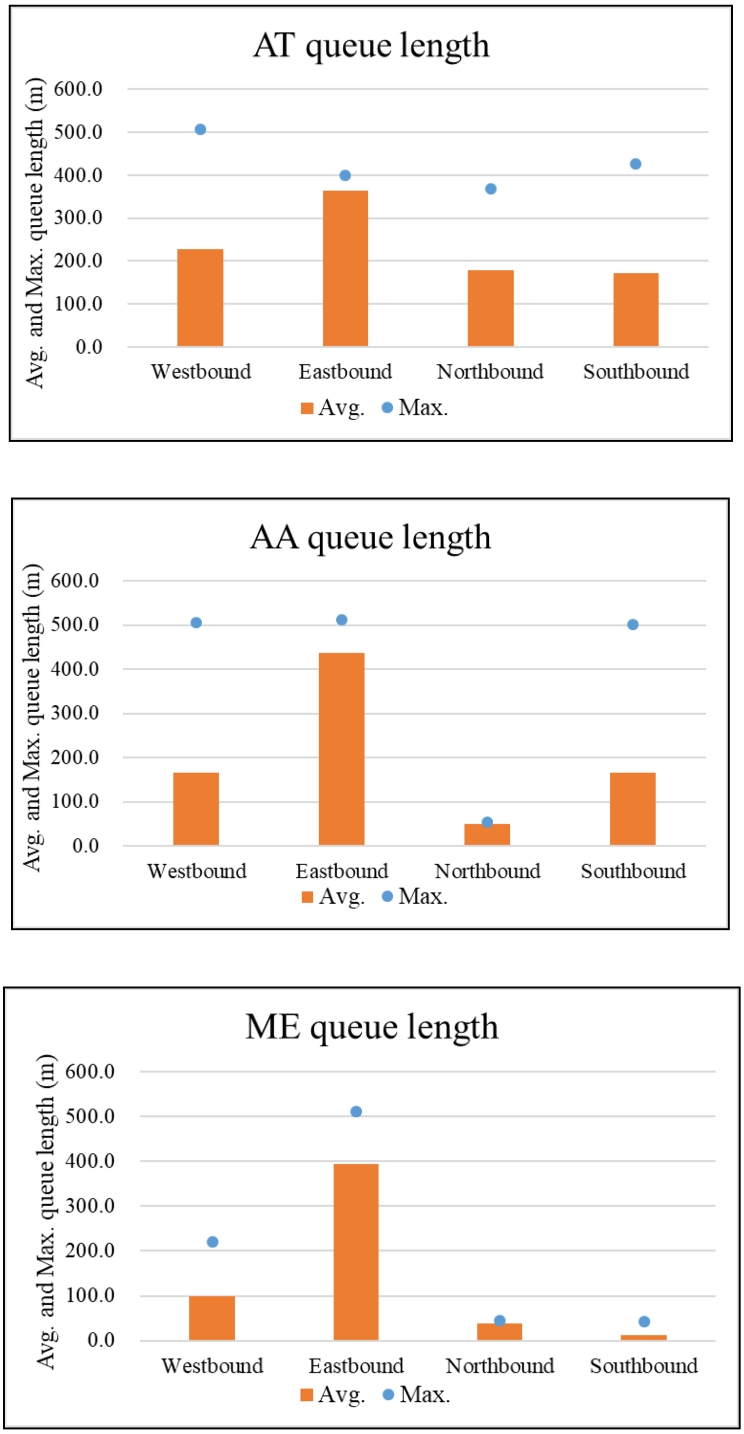

Figure 3.15 Queue length inside the studied intersections.

Table 3.8 The simulation outputs

	ME	AA	AT
Throughputs (veh/h)	5444	3977	4863
Avg. delay/veh. (s)	199.0	786.67	333.13
Avg. speed (km/h)	19.94	2.22	13.58
Avg. No. of stops/veh.	6.05	61.82	16.17
Avg. stopped delay/veh. (s)	43.07	1967.03	141.09

In conclusion, the results emphasized that under heavy traffic conditions during the peak periods with the limited geometric designs, the studied intersections could not accommodate the high traffic demands. The mixed traffic salient features such as aggressive driving behaviour, non-lane based driving and various traffic compositions influenced the operation of intersections. Moreover, the indirect left-turns as well as long cycle length resulted in long queues at the intersections. During the peak hours, the spillback queues in both the intersections and U-turns interrupted the other adjacent intersections performance along the corridor. As a result, the LOS of intersections was significantly decreased.

CHAPTER 4

OPERATIONAL PERFORMANCE EVALUATION OF DISPLACED LEFT-TURN (DLT) CROSSOVERS DESIGN

4.1 Introduction

As a result of the continues growing demand on the transportation systems, the corresponding resources to address these demands are becoming scarce. At conventional at-grade intersections, all traffic participants suffer from the significant increase in their travel time as a result of a large number of longer signal phases, large volumes of left-turn movements as well as the growing difference in traffic intensity on different lanes (Reinis Kivlins et al., 2011). Most of the previous studies emphasized that much of the vehicle delay incurred at conventional arterial intersections is caused by the high left-turn demand (Reid and Hummer 1999; El Esawey, M., Sayed, T. 2013). Therefore, in order to alleviate the traffic congestion at signalized intersections, Displaced Left-turn Crossover (DLT) design that is also known as CFI (continuous flow intersection), has been presented as an UAID to address the traffic congestion at conventional signalized intersections (El Esawey, M., Sayed, T. 2013). The novel solution provided by DLTs results in higher capacities, lower delays and fewer crashes. This novelty allows left turn flows to cross the opposing traffic lanes upstream of the main intersection and move simultaneously with the through traffic within the same signal phase. This design has become a prevalent treatment over the past decade in some developed cities around the world, particularly in north America. Most of the extensive preliminary studies focused on studying the applicability of this design under the ideal traffic operations where less complex conditions. Although this new scheme was shown to outperform over conventional signalized counterparts in terms of operational performance, DLTs have been not estimated under heterogeneous traffic as a dominant environment in developing, semi-industrialized and industrializing countries. Hence, in order to fulfill the heterogeneous condition needs, the driving force of this research is to investigate the operational performance of DLTs under heterogeneous traffic where the complexities of such traffic compositions including the diverse static and dynamic properties of vehicles, the aggressive drivers' behaviour and the lack of lane discipline are existing. To give a full understanding of the proposed design, the previous research main findings are highlighted and the operational mechanism is explained in details. Likewise, the DLT representation including the geometric layouts, the signal timing plans under such complexities of the heterogeneous traffic is also presented.

On the other hand, despite the wide and prevalent deployments of DLTs, most of the previous research work investigated UAIDs, particularly DLTs, as isolated intersections. Among the considerable sparse works highlighted the DLT design, little research focused on the operational performance of DLTs as a corridor by considering the coordination approach. Therefore, in this chapter the coordination of consecutive DLT intersections is investigated by utilizing the most common coordination techniques; bandwidth maximization and delay minimization. The bandwidth maximization is employed as a pre-timed (fixed-time) coordination approach, while the delay minimization approach is utilized to develop a real-time demand-responsive signal control system on the solid foundation of the dynamic optimization principles. This entire demand-responsive algorithm was built based on developing mathematical model and simulations utilizing PTV-VISSIM as a simulator-based approach and MATLAB as a multi-paradigm numerical computing environment. Commissioning the VISSIM-COM interface and MATLAB, an inter-process communication and dynamic object creation was provided. Although academic in nature, the presented algorithm in this context could be evolved through a real-world practical application. As a realistic study case, the same obtained data from three signalized intersections that were analyzed in the previous chapter was also used to examine the efficiency of the proposed DLTs.

4.2 Displaced Left-turn (DLT) Intersection Mechanism

In this section, the operational mechanism of the DLT intersection is discussed and highlighted. This design basically aims to reduce the delay of through streams as well as to reduce the number of conflict points. By eliminating the left-turn conflicts through displacing the left-turn lane to the opposing direction the main concept could be achieved (Hummer, J. E., D.Reid, J. 1998). This intersection's innovation is the allowance of the operation of through movements and left-turns at the same time as shown in Figure 4.1. The traffic flow is facilitated by emphasizing a simultaneous proceeding of left-turn traffic and through traffic within the same signal group. Both of through and left-turn traffic could move simultaneously at the main intersections using a two-phase signal as shown in Figure 4.2. Therefore, the design could also refer to the continuous flow intersection (CFI) (Jagannathan and Bared 2004). Utilizing two-phase signal timing plans instead of the typical four phases given for the other conventional signalized intersections as shown in Figure 4.3, leads to a reduction in the number of signal phases. In view of that, the intersection operational performance indices could be enhanced significantly. Moreover, reducing the number of conflict points inside the main intersections results in improving the safety conditions (Jagannathan and Bared 2005). Through this unique design, the left-turn flow is permitted to cross laterally to the edge of the other side of the road a few hundred meters upstream of the main intersection as shown in Figure 4.1. By rerouting the left-turn movements, drivers cross over to the left of the road into an exclusive left-turn lane (Joseph E. Hummer 1998). As drivers enter the primary intersection, the left-turns proceed unopposed. Thus, left-turns are allowed to move simultaneously with through traffic, resulting in significant operational efficiency. The left-turn displacements, consequently, create four additional secondary intersections in major and minor approaches upstream the primary intersection as Figure 4.1 depicts. Depending on the geometric configuration as well as the traffic volumes, the DLT crossover could be implemented either in only major approach or both major and minor approaches.

This design is described as a system of two-phase intersection for the primary and the four created additional crossovers (Reid and Hummer 2001). As a result, a significant reduction in the total cycle length could be done. By reducing the cycle times, leading to shorter average queues as well as shorter storage bays with an overall significant improvement (Joseph E. Hummer 1998; J. E. Hummer and D.Reid 1998; Jagannathan and Bared 2004). Meanwhile, the individual signals within the DLT

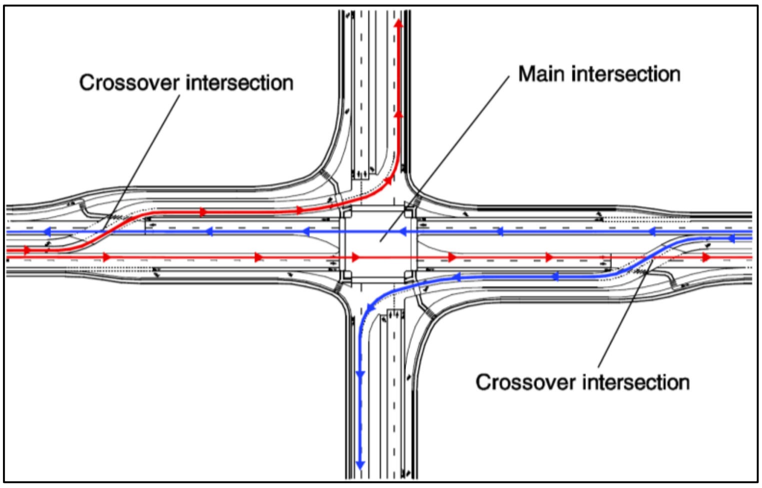

Figure 4.1 The DLT operational mechanism (FHWA, 2014)

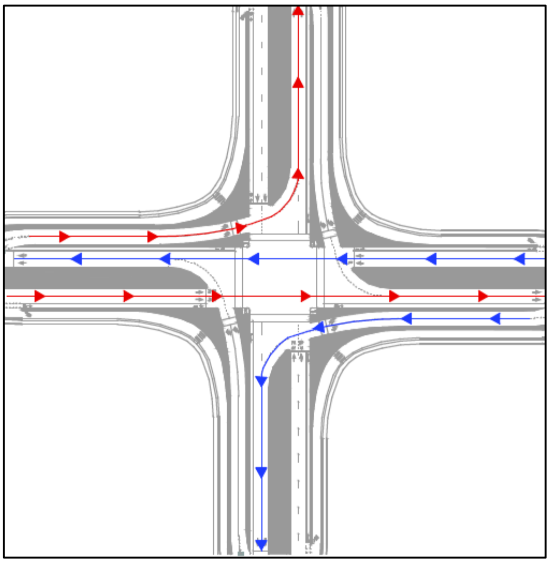

Figure 4.2 DLT east/west phases (FHWA, 2014)

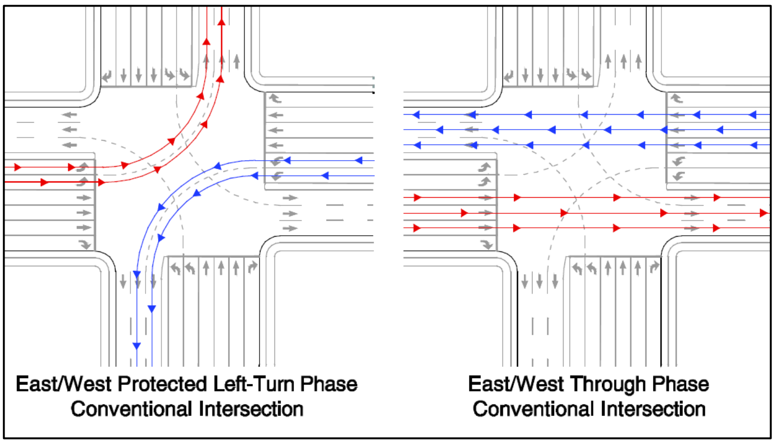

Figure 4.3 Displaced Left-Turn Intersection (FHWA, 2014)

are easy to coordinate with each other. By considering this coordination, most of the drivers would only have to stop once and a huge difference in travel time is achieved. Consequently, more traffic flow is processed efficiency, an obvious progression along the corridor is enhanced with shorter travel times on the main roadway (El Esawey and Sayed, 2013). As a result, the DLT design succeeds to keep and maintain traffic moving as its name implies.

4.3 Previous Findings

Owing to investigate the operational performance of the DLT as an applicable scheme, lots of considerable and valuable research works that provided guidelines relevant to this study are reviewed to identify the basic principles of the analysis of DLT. Most of these attempts aimed to highlight the performance of the DLT scheme by conducting a comparison between DLT and either conventional intersections or other UAID designs. By analyzing one or more of the UAIDs, general documentation of the results and/or potential implementation may give valuable cues on the implications of using a particular design(s) (El Esawey and Sayed, 2013). Taking into consideration the novel nature of UAIDs schemes, and bearing in mind the fact that field implementations of such designs are rare, most of the previous works have been conducted based on the micro-simulation platform. The previous research concluded that the conventional designs never produced the lowest average total time, and at least one unconventional scheme would outperform its conventional counterpart in at least one volume scenario (Joseph E. Hummer 1998; J. E. Hummer and D.Reid 1998; M. E. Esawey and Sayed 2013; Joseph E. Hummer and Jagannathan 2008). Although the quadrant roadway and MUT designs vied for the lowest average total time, the CFI always recorded the highest move-to-time ratio for all designs (J. E. Hummer and D.Reid 1998). Also, the previous findings emphasized that in locations where the availability of additional right-of-way and driveway access is not a major concern, the DLT scheme is a cost-effective and timesaving option compared with the grade-separated interchange designs (Dhatrak, A., et al. 2010).

Autey J. et al. used VISSIM to compare the operational performance of four unconventional intersections: DLT, USC, DXI and MUT under balanced and unbalanced volume scenarios. The authors emphasized that DLT is always superior to other conventional intersections in almost all volume conditions in terms of average intersection delay and the overall intersection capacity. The DLT intersection experienced a significant growth of capacity by 99% higher than that of the conventional one, whereas the USC and DXI capacities were about 50% higher than the conventional counterparts, the DLT constantly exhibited the lowest delay among the all compared counterparts (Autey, Sayed, and Esawey 2010).

Likewise, in another comparative study, considerable savings in average control delays and average queue lengths comparing three different DLT configurations to their similar conventional designs under low, moderate and high traffic volumes. Under the DLT scheme, a significant saving in the average control delay occurred as following: from 48.0% to 85.0% for low traffic, 58.0% to 71.0% for moderate traffic and 19.0% to 90.0% for high traffic volumes. Consequently, an obvious enhancement in the average number of stops could also be enhanced. A corresponding reduction in the average number of stops was raised from 15.0% to 30.0% under saturated traffic flows and from 85.0% to 95.0% for saturated traffic conditions. The analysis also showed a significant increase in intersection capacity for the three studied DLTs over the conventional ones (El Esawey, M., Sayed, T. 2011).

In order to obtain a fair travel time comparison, Hummer, J. E., D.Reid, J used CORSIM to compare a conventional intersection and seven UAIDs (the quadrant roadway intersection, MUT, RCUT, bowtie, Jughandle, split intersection and DLT). The authors concluded, based on the simulation results, the conventional designs never produced the lowest average total time, and at least one unconventional scheme would outperform its conventional counterpart in at least one volume scenario. Although the results showed that quadrant roadway and median U-turn designs vied for the lowest average total time, the continuous flow intersection always had the highest move-to-time ratio for all designs. The results of average travel times, vehicle miles and the number of trips indicated that one or more UAIDs had lower travel times than the conventional design in every studied site. Based on the moving-to-total time ratio, it was found that the DLT design was always keeping traffic moving as its name-continuous traffic flow- implies (Hummer, J. E., D.Reid, J. 1998).

Dhatrak, A., et al. compared the operational performance of the DLT design with another non-traditional at-grade design called the Parallel Flow Intersection (PFI) that also operates with the same number of signal phases as DLT. The authors used the maximum through and left-turn movement throughputs for three different high-volume scenarios as a comparison index. Based on VISSIM outputs, both designs produced similar results as a result of a two-phase signal system in both designs. The results showed that the PFI maximum through movement throughputs are very close to their counterparts in the DLT. However, the maximum throughputs of the left-turn movement were found to be lower than those in the DLT intersection, due to a greater number of stops, on average, in PFI than they would in a DLT. As a result, in the two study cases, the DLT design was able to process 180 to 80 more vehicles per hour per lane than the PFI (Dhatrak, A., et al. 2010).

Echoing findings similar to those of the research summarized above when Cheong et al. examined the operational performance of DLT and compared it with the PFI and USC designs. Also, it was emphasized the superiority of the DLT over the other two designs, PFI and USC for most traffic conditions. However, the average delays in through movements were found to be lower for the PFI than the DLT and USC at low traffic volume levels. At high volume levels, it was shown that the average delays in the left-turn movement were similar for the PFI and DLT intersections (Cheong et al. 2008).

As an important design consideration, El Esawey and Sayed investigated the spacing between primary and secondary intersections on the performance of DLT and USC designs. The results showed a consistency for both designs and this spacing distance. It was emphasized that both designs experienced an increase in capacity with a corresponding increase in the spacing between intersections.

The outperformance of the DLT over the USC design in all study scenarios, particularly, under high-volume scenarios (El Esawey, M., Sayed, T. 2007).

Unlike the above-reviewed articles when almost in all of them the vehicle traffic was homogenous or less complex conditions, another study addressed some specific aspects of heterogeneous conditions where the complexities are evidential. Considering the complexities of such conditions in Cairo, Egypt, the overall attained results referred to the superior of DLTs over the conventional intersections. However, the mixed traffic conditions influenced DLTs performance improvement rates comparing to the previous studies where hypothetical traffic data was used. The driving behaviour, as well as the diverse dynamic properties, could affect the discharge rate in both main intersections as well as left-turn crossovers (SHOKRY et al. 2017).

Based on the presented preceding review, it can be concluded that although the several, considerable and valuable research, little research attempts to investigate some UAIDs designs, considering the other prevailing conditions under the heterogeneous traffic as a significant and a dominant feature in developing, semi-industrialized and industrializing countries. Hence, in this chapter, much attention is given to the applicability of the UAIDs under heterogeneous traffic conditions as common characteristics of developing cities around the world. In addition, it can be obviously realized that the DLT design has never been estimated as a coordinated corridor. Therefore, the coordination of consecutive DLT intersections is investigated in this chapter.

4.4 DLTs Representation Under Heterogeneous Conditions

Aiming to obtain a reliable and accurate representation of the DLT design considering the different complexities of such heterogeneous conditions, several factors should be taken into account. Following DLTs design principles and considering the earlier works, guide manuals as well as the pre-deployments of the scheme proposed, DLTs configuration could be simulated accurately. To ensure a fair comparative performance analysis between the DLT and their conventional counterparts, equitable settings should be considered. As a key factor, the simulation parameters, all the different simulation parameters settings that were calibrated and validated including driving behaviour, vehicle properties, non-lane based system representation...etc., are kept as constant as for the existing intersection models as is illustrated in details in chapter 3. The other settings are illustrated as follow:

4.4.1 Geometric Layout Representation

First and foremost, the similar geometric design elements should be kept fixed despite the differences in movement configurations and right of way requirements (i.e. same footprint / land area) regardless the number of lanes (El Esawey, M., Sayed, T. 2013). Researches followed two different approaches to achieve this concept. The first approach is built upon keeping the same right of way to all comparable designs regardless the number of lanes as in Bared et al. (2005) to analyze the DXI intersection (Jagannathan and Bared 2005). On the contrary, the second approach has been extensively employed in most of the previous works (Reid and Hummer 1999; Reid 2000; Sayed et al. 2006; Kim et al. 2007; Autey et al. 2011). This approach sets the same number of lanes per approach constant for each alternative design, and the right of way is regarded as one constraint that controls potential construction (El Esawey, M., Sayed, T. 2007).

In this study, a full DLT design with crossovers in both major and minor approaches is proposed. Depending on the traffic volumes obtained from the real-world field observation and keeping the same right of way as for the existing intersections, different DLT geometric configuration was configured and assigned for each studied intersection. For the minor approach, the same geometric layout was assigned for all studied intersections. A pair of 3.5m width lanes was dedicated exclusively for through-movement traffic for each direction, while the other two lanes were assigned for crossover left-turning movements lanes with a channelized free right-turning lane as shown in Figure.4.4, Figure 4.5 and Figure 4.6. However, the geometric layout was different for the major approach in the studied intersections based on the traffic volumes and the directional ratios at each intersection. Three lanes of 3.5m width were employed for the through-traffic westbound flows of AT DLT because of the high traffic demand, while only one lane of 3.5m width was employed for the left-turn crossover movements west and eastbound as shown in Figure.4.4. Also, due to the high traffic demand in both east and west through flows, three lanes were allocated for both AA west and east through traffic. Accordingly, the right channeled lane of the westbound as well as the southbound was eliminated and one lane was allocated for left-turn west and eastbound as shown in Figure.4.5. For ME intersection, two exclusive lanes of 3.5m width were assigned for each direction for through traffic flows, while the other two lanes of 3.5m were installed as crossover lanes as shown in Figure 4.4. Moreover, the two existing bus exclusive lanes were kept on the main street to utilize the same right of way given to the existing intersections. A channelized right-turn lane of 3.5m for each direction was provided for all intersections as shown in Figure.4.4, Figure 4.5 and Figure 4.6 except the eliminated ones in AA DLT.

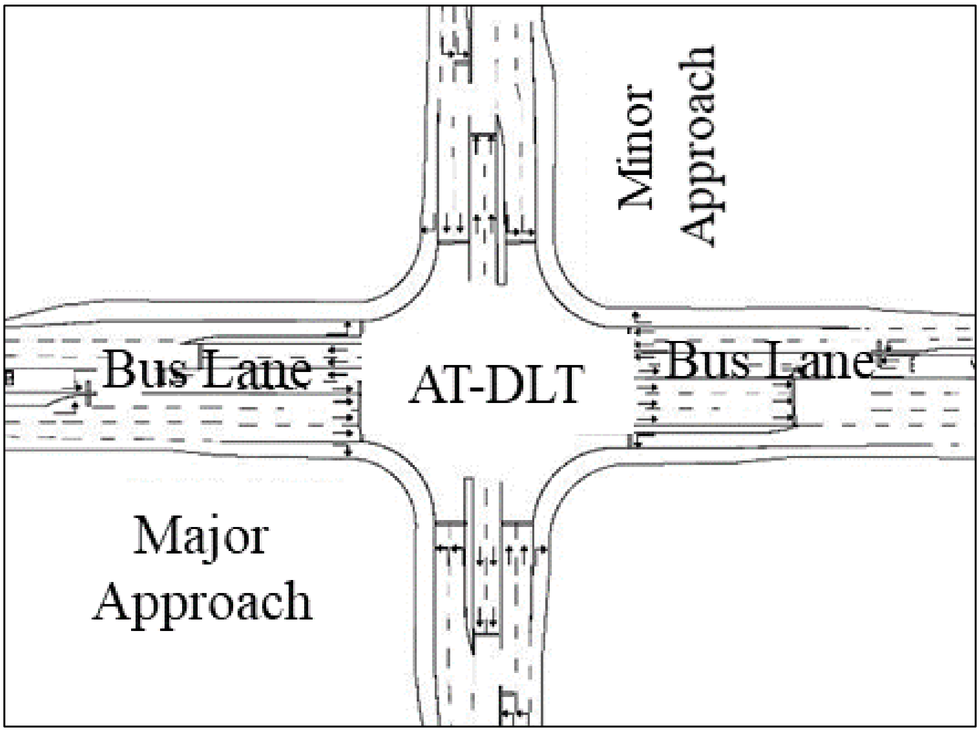

Figure 4.4 The geometric layout of AT-DLT intersection

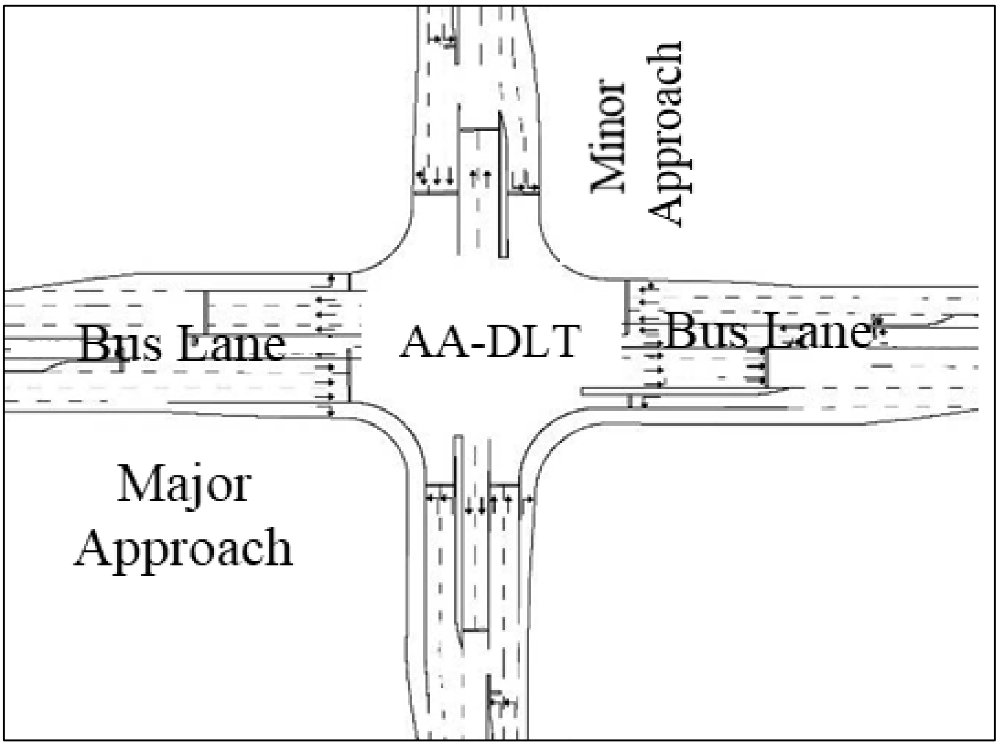

Figure 4.5 The geometric layout of AA-DLT intersection

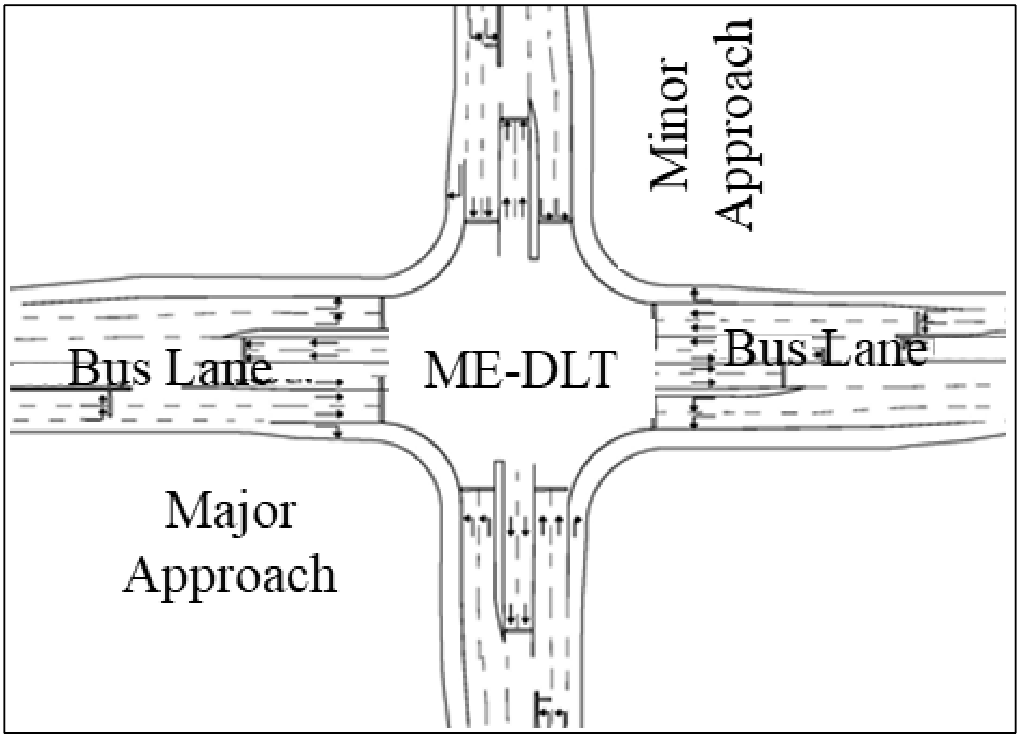

Figure 4.6 The geometric layout of ME-DLT intersection

4.4.2 Signal timing plans calculation

The signal timing plans calculation method is an important factor should be taken into account is before running the simulation and initiate the evaluation. Generally, the cycle length is calculated based on either theoretical/mathematical concepts or by trial and error method. The theoretical cycle length is favorable when the traffic volumes are uniformly distributed on the four approaches of the entire intersection, while the trial and error approach is recommended in some practical circumstances (El Esawey, M., Sayed, T. 2007). The majority of UAIDs signal optimization studies used signal optimization software such as Synchro and TRANSYT-7F as the most widely used optimization tools. The theoretical approaches such as the critical lane volumes (Goldblatt et al. 1994, Reid and Hummer 2001) or the ratio between arterial and cross-street traffic volumes (Dorothy et al. 1997) were also used to optimize the cycle length of UAIDs.

In this research, the signal timing plans are calculated based on a mathematical approach, namely Webster model. This model was developed to estimate the optimal delays based on the critical flow inside the intersection. Following the DLT design principle, to design the signal control system as a two signal phase scheme, the signal timing plans of different DLT proposed intersections in this study were estimated. The proposed system was designed to facilitate the through traffic movements by reducing the total cycle length time. In addition, a needed integration between the main intersection and the primary left-turn crossovers created was taken into account. For this purpose, Webster's (1966) method was utilized to estimate the total cycle and green times of the fixed-time signals of the intersections selected. According to Webster's method, the overall delay for the vehicles inside the intersection was minimized by optimizing the green time based on the flow ratio of each phase (Indonesian Highway Capacity Manual, Part 1 Urban roads, Urban and semi-urban Traffic Facilities 1993; Shokry, S., and Tanaka, S. 2015). First, the cycle time (C) of a two-phase system is calculated based on the primary intersection critical flow ratios as illustrated in Equation 4.1. Then the green time of each phase (gi) of the primary intersection as well as the other additional upstream left-turn crossovers of major and minor approaches is determined as in Equation 4.2. For each DLT intersection, a group of six signal phases are needed to control the main intersection and the four-formed secondary crossovers as shown in Figure 4.7. The signal offsets between each phase are efficiently considered to ensure a smooth traffic and to achieve the continuous flow concept. The used parameters were estimated upon the data obtained from the field observations as well as analyses conducted as shown in Table 4.1 with a lost time of 5.0 seconds for each phase. Although, the saturation flow rates could not be estimated from the field observations because of the disruption occurred in some signal heads during the observation time, it was assumed as the standard value of 1800 veh/h/ln.

$$C = (1.5 \times LT + 5) / (1 - \Sigma FR) \tag{4.1}$$

$$gi = (C - LT) \times Max.FR / (\sum Max.FR) \tag{4.2}$$

Where,

C : optimal signal cycle time (s);

LTI : total lost time per cycle (s)

FR : critical flow ratio for each phase (Q/S);

FRcrit : Max. value of FR value among the signal group for the same phase in the approaches being discharged during a signal phase.

Table 4.1 Primary intersection signal time plan

	Design parameters	Main Intersection		Major App. Crossover.		Minor App. Crossover.	
		Phase (1)	Phase (2)	Phase (3)	Phase (4)	Phase (5)	Phase (6)
AT	Max. flow	2435	805	397	2435	694	805
	Max. flow/L	811.67	402.5	397	811.67	347	402.5
	(Q/S)	0.427	0.21	0.209	0.427	0.182	0.21
	ΣFRcrit	0.637		0.636		0.394	
AA	Max. flow	1975	1208	773	1975	690	1208
	Max. flow/L	658.3	604	773	658.3	345	604
	(Q/S)	0.346	0.32	0.407	0.346	0.18	0.32
	ΣFRcrit	0.666		0.753		0.5	
ME	Max. flow	1771	957	1094	1771	525	956
	Max. flow/L	885	478	547	885	263	478
	(Q/S)	0.466	0.252	0.288	0.466	0.138	0.252
	ΣFRcrit	0.718		0.754		0.39	

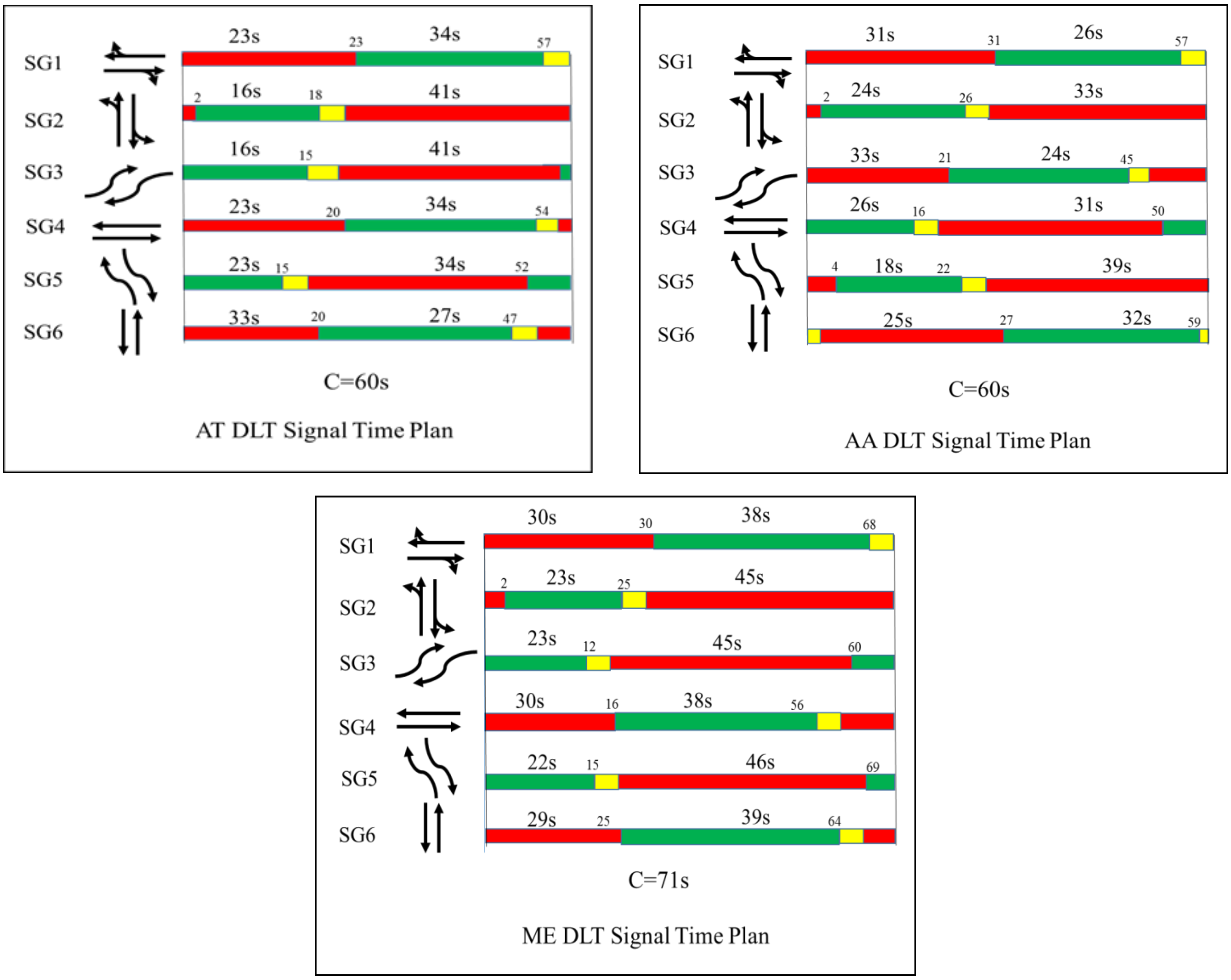

Figure 4.7 The signal-timing plan of the proposed DLT intersections

4.5 DLTs Evaluation as Isolated Intersections

In this section, each DLT design was proposed and compared with its existing conventional counterpart. In order to obtain reliable results and seeking an equitable comparison, the simulation results were obtained and analyzed based on fifty trials of one-hour simulation for each. For each trial, the total travel time, the overall delay, the intersection capacity, the average and maximum queue length, the average speeds, and the average number of stops were estimated for each studied intersections as evaluation indices.

The simulation results indicated to an undoubted improvement for all performance evaluation indices, especially, under the heavy traffic volume levels as a result of DLTs outperformance over the conventional intersections. Due to the continuous flow functionality provided by the DLT scheme, the results revealed that DLT proposed designs at AT, AA and ME intersections overcame the operational performance of the existing intersections. An obvious enhancement in intersections throughputs, the average total delay time per vehicle, the average speeds and the average stopped delay per vehicle as shown in Table 4.2. Similarly, the proposed DLT exhibited considerable savings in both overall delays, as well as total travel time for all the analyzed intersections as shown in Figure 4.8, Figure 4.9 and Figure 4.10. However, due to emphasizing the major approach traffic demands over the minor ones, the northbound through as well as northbound left-turn traffic movements did not experience an obvious enhancement. Conversely, the northbound through and left-turn flow experienced more travel time and delay at ME-DLT design, due to the relatively longer red time as shown in Figure 4.10. Accordingly, the average and maximum queue lengths for all approaches could be shorter than recent ones apparently. It also could be realized that the left-turning queues have totally vanished as shown in Figure 4.11, Figure 4.12 and Figure 4.13.

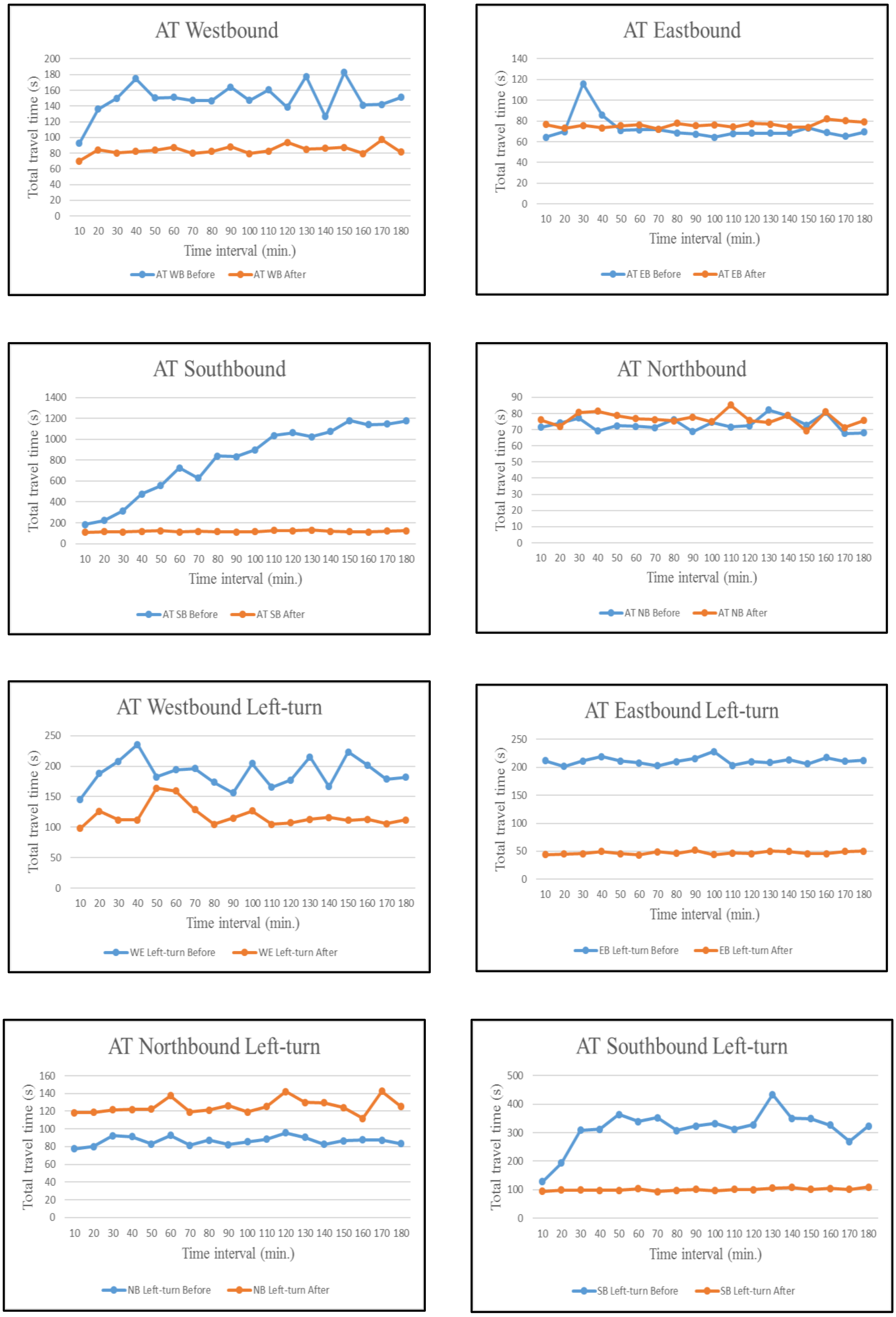

Figure 4.8 Total travel time of AT conventional intersection vs. DLT design

Table 4.2 A comparison between before and after applying DLTs

Parameter	ME		AA		AT	
	Conven.	DLT	Conven.	DLT	Conven.	DLT
Throughputs (veh/h)	5444	7092	3977	6433	4863	6537
Total travel time (h)	708	537	4075	839.67	1107	596
Avg. delay/veh. (s)	199.0	132.13	786.67	218.63	333.13	124.63
Avg. speed (km/h)	19.94	23.56	2.22	17.36	13.58	24.71
Avg. No. of stops/veh.	6.05	2.08	61.82	6.34	16.17	1.45
Avg. stopped delay/veh. (s)	43.07	24.34	1967.03	73.22	141.09	16.87

Although, the significant enhancement of the analyzed intersections because of DLTs, the improvement indices are lower than their counterparts under the ideal condition shown in the previous studies. Due to the heterogeneous conditions considered in this study (i.e the diverse dynamic and static properties of mixed traffic compositions, the non-lane based system, the aggressive driving behaviour…etc) were obviously influenced the operational performance improvement rates of the proposed DLT designs. Apparently, the diverse dynamic properties of such conditions as well as the aggressive driving behaviour influenced the queues discharge rates in both main intersections and major approach crossovers as well as clearance time. Therefore, in order to optimize the DLTs performance the different signal group offsets as a key design factor of DLTs should efficiently be studied and set.

On the major approach, the spacing distances between the primary intersections and crossovers were influenced due to the traffic demand and driving behaviour along the studied corridor. According to the federal highway agency, the recommended distance is 91.4 m to 152.4 m (FHWA: Displaced Left-Turn Intersection Informational Guide, 2014), however in this study, the spacing distances were allocated as 200.0 m, 105.0 m and for 85.0 m ME, AT and AA respectively as an indication of the heterogeneous traffic impacts.

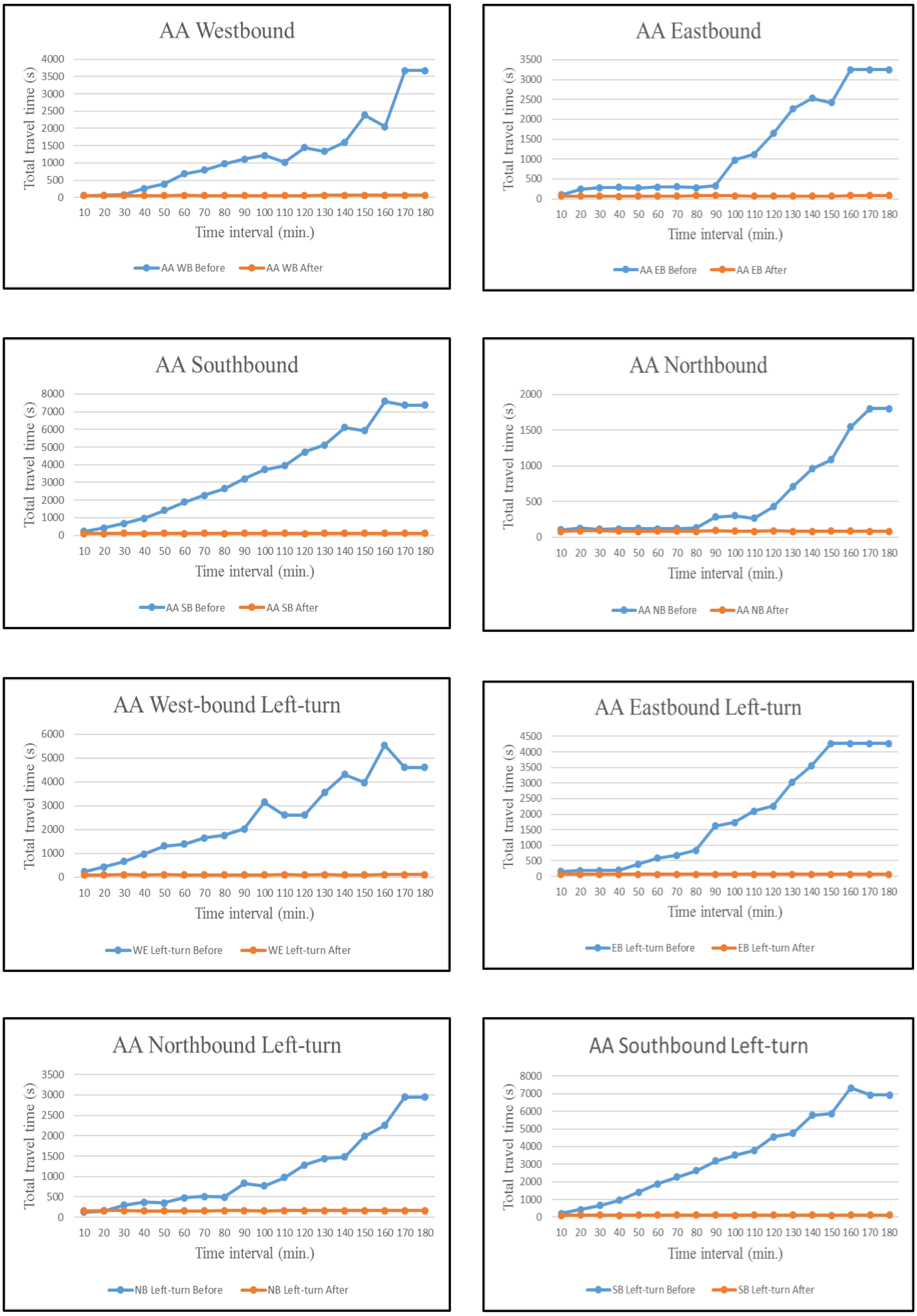

Figure 4.9 Total travel time of AA conventional intersection vs. DLT design

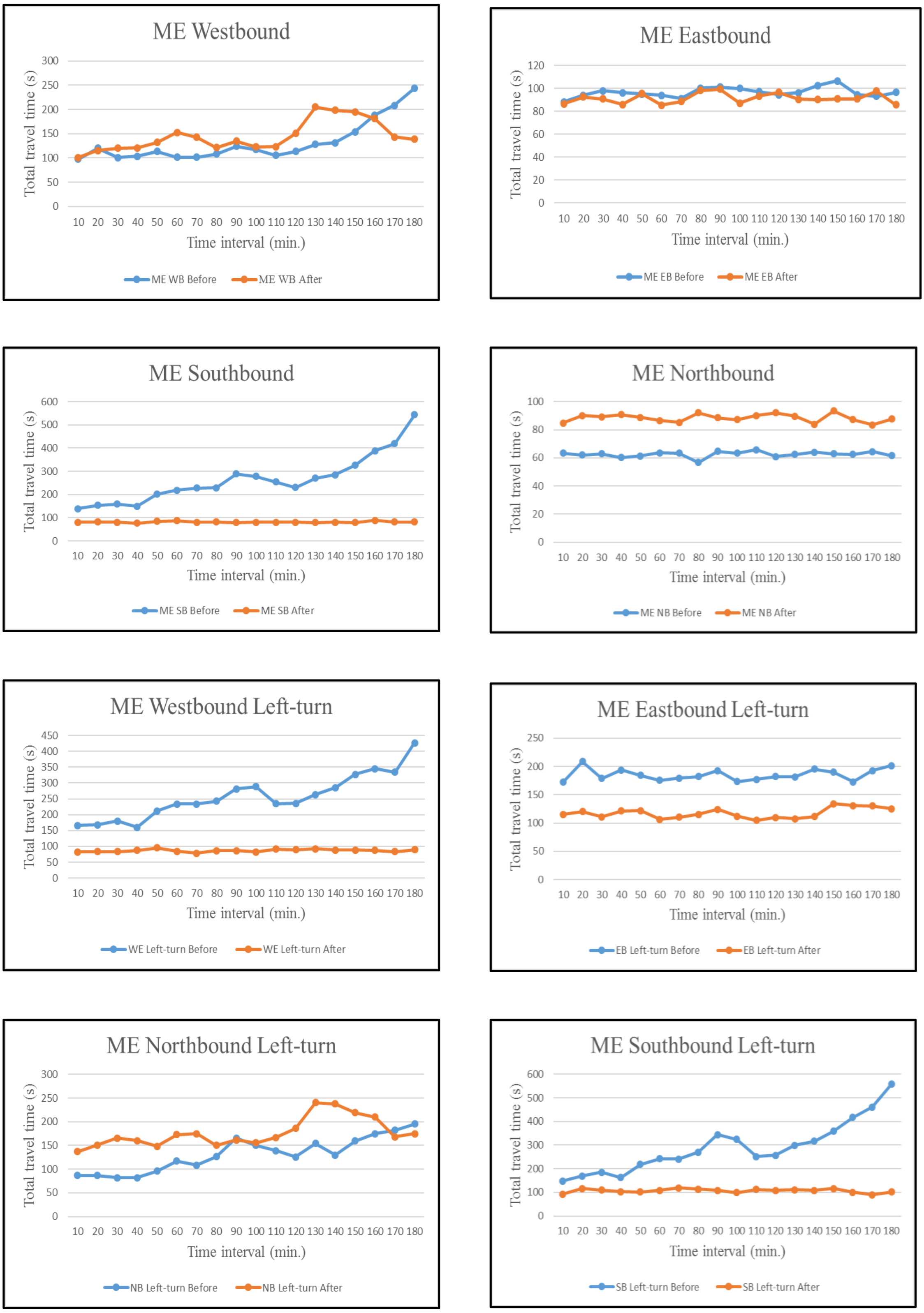

Figure 4.10 Total travel time of ME conventional intersection vs. DLT design

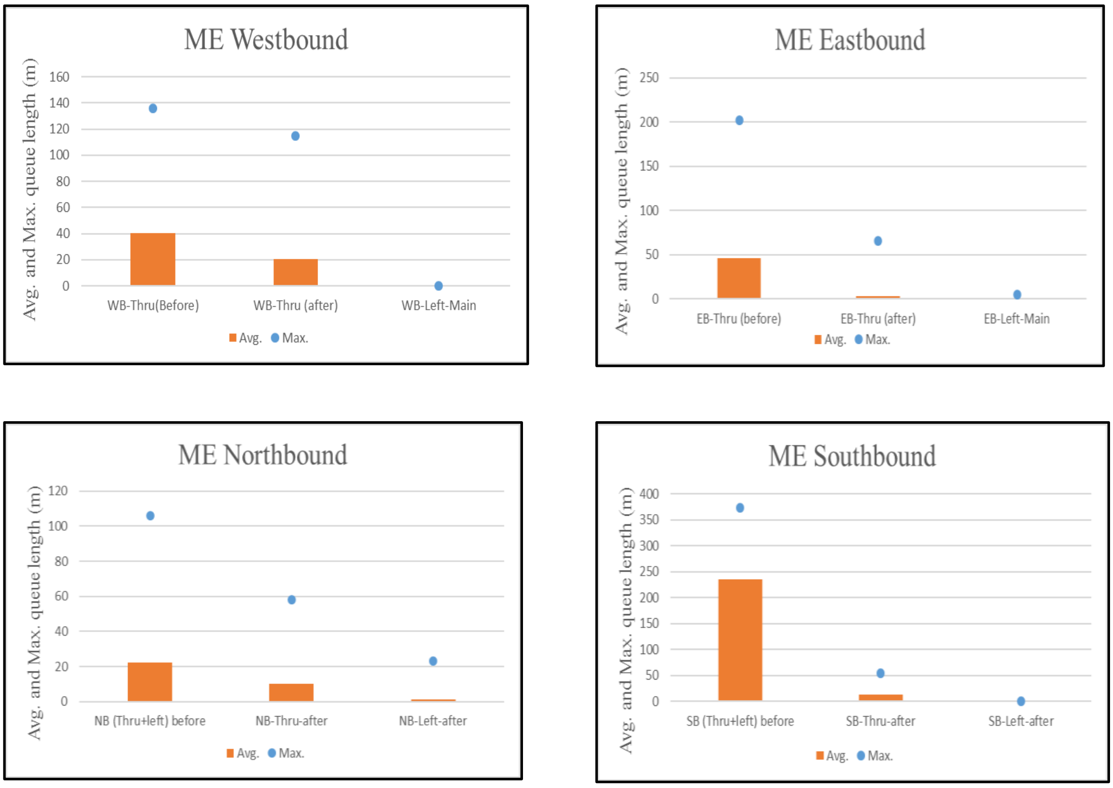

Figure 4.11 Max. and Avg. queue length of ME conventional intersection vs. DLT

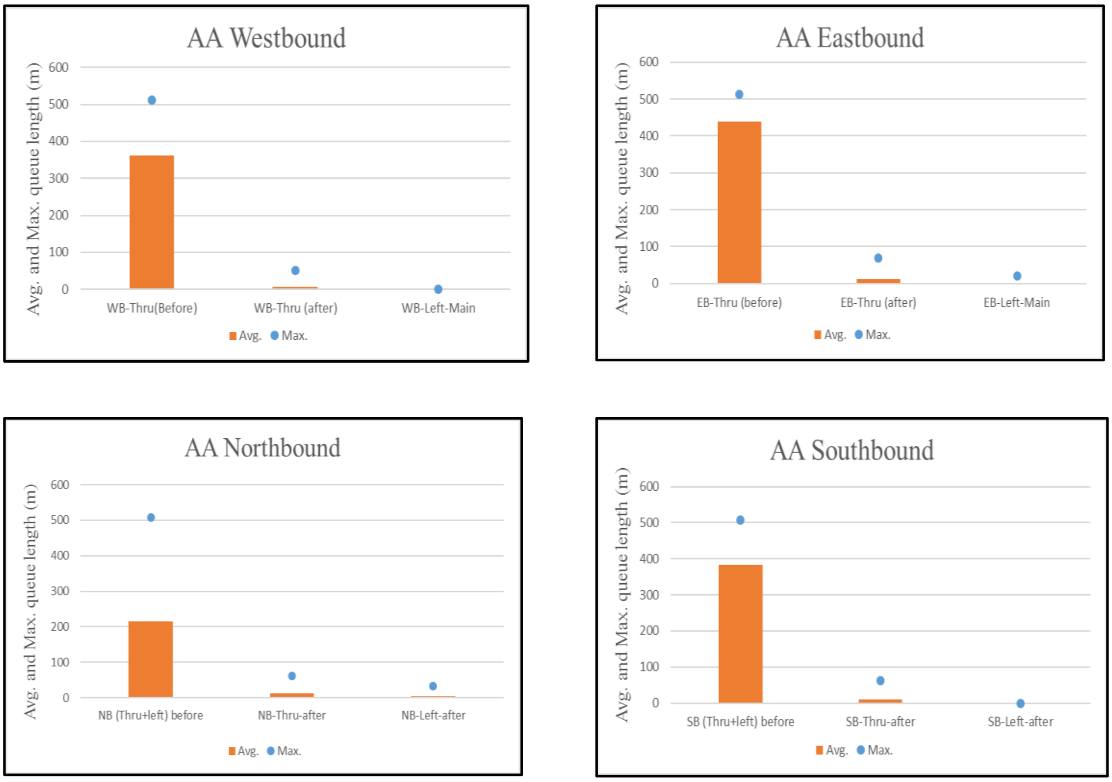

Figure 4.12 Max. and Avg. queue length of AA conventional intersection vs. DLT

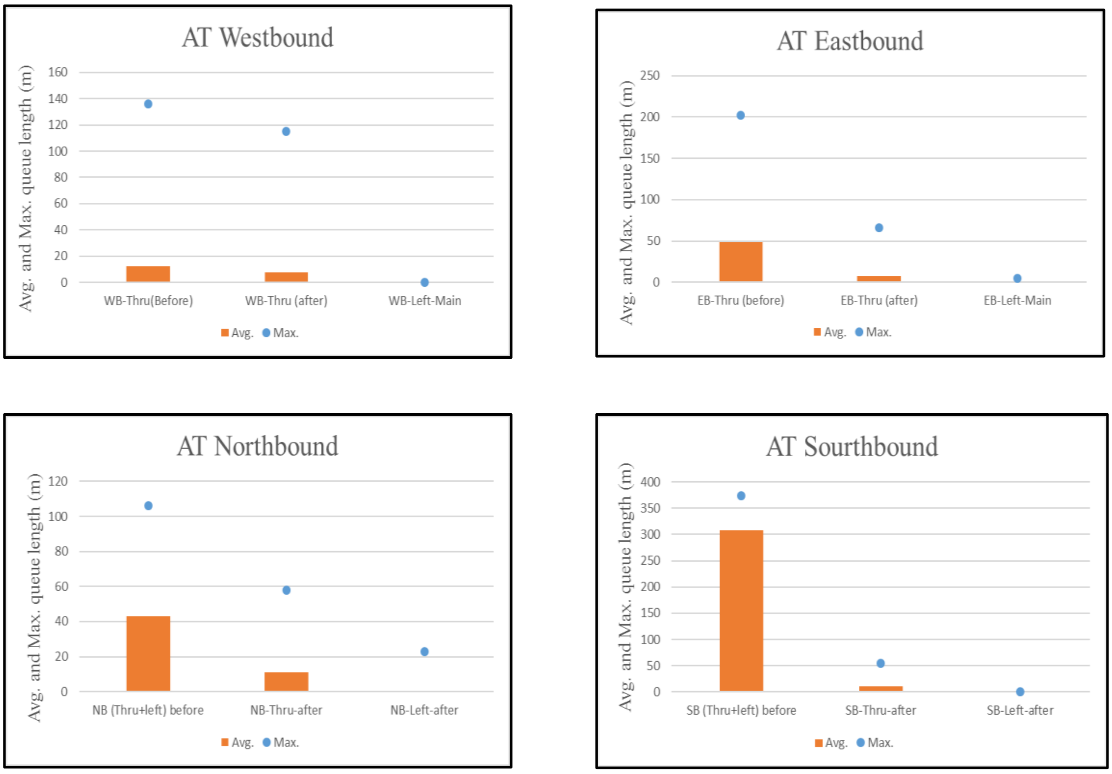

Figure 4.13 Max. and Avg. queue length of conventional intersection vs. DLT

4.6 DLTs Coordination

The signal coordination is the ability to synchronize multiple intersections to enhance the operation of one or more movements in a system as shown in Figure 4.14. This process requires adjacent the coordinated signals to operate at the same cycle length or at a multiple of the cycle length with pre-determined offset and coordination points (FHWA, 2008). In order to accomplish the coordination for the proposed DLT designs in this research, the fixed-time coordination is highlighted as a static system, while a real-time coordination is representing a dynamic system. The optimization is that mathematical technique of finding the point that minimizes or maximizes a function to get the optimal solution as shown in Figure 4.15. Although the considerable attention paid to the UAIDs implementation technologies, DLTs have never been estimated as a coordinated corridor, (El Esawey, M., Sayed, T. 2013, Shokry et al. 2017).

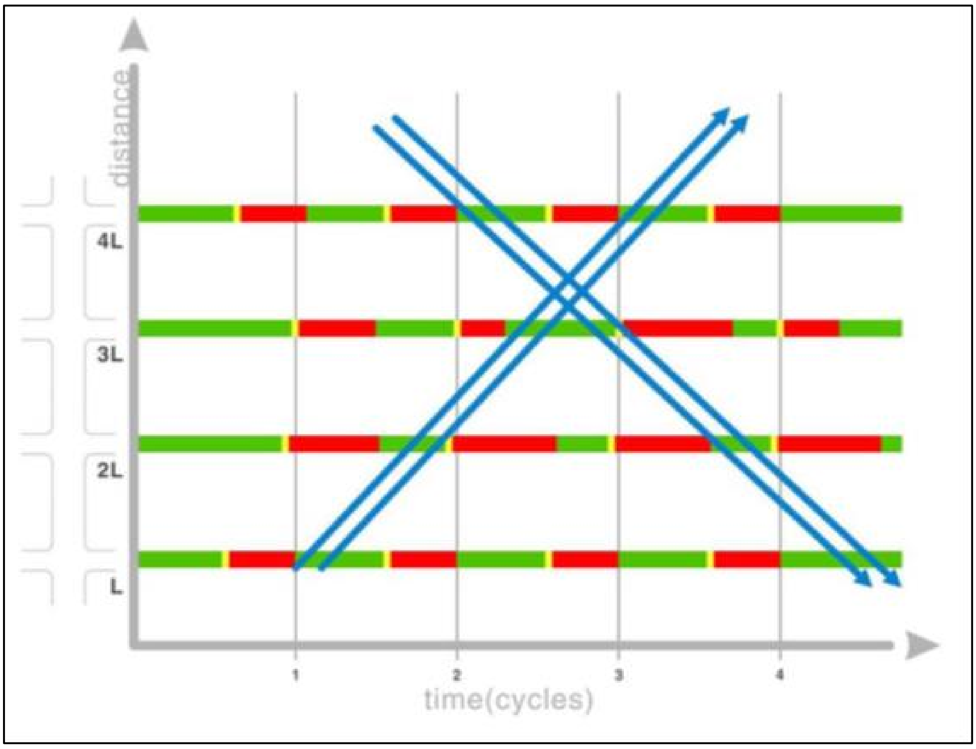

Figure 4.14 Time-Space Diagram of a Coordinated Timing Plan (FHWA, 2008)

A great deal of literatures highlighted the qualitative and quantitative benefits of DLTs design over other conventional signalized intersections, however, little research focused on methodologies dealing with the synchronization of DLTs in a corridor. Lots of literature clearly showed the extensive results regarding the arterial coordination, little research has turned attention to the DLTs coordination. On the other hand, even though most of the previous works emphasized the considerable impacts of the traffic heterogeneity, particularly, on the intersections' performance, little research investigated the signalized intersection coordination under heterogeneous conditions. A lane-based optimization model was proposed as a systematic approach in order to obtain an optimal operation of DLTs. The model tried to optimize the selection of intersection type, the length of the displaced left-turn lane, signal timing and lane markings as a multi-objective optimization problem. For the purpose of providing a synchronized DLTs and facilitating a smooth flow along the studied corridor, the proposed model is adapted to calculate the optimal offset for each pair of consecutive intersections. Therefore, the coordination was broken down into two stages, the coordination between the intersection AT and AA

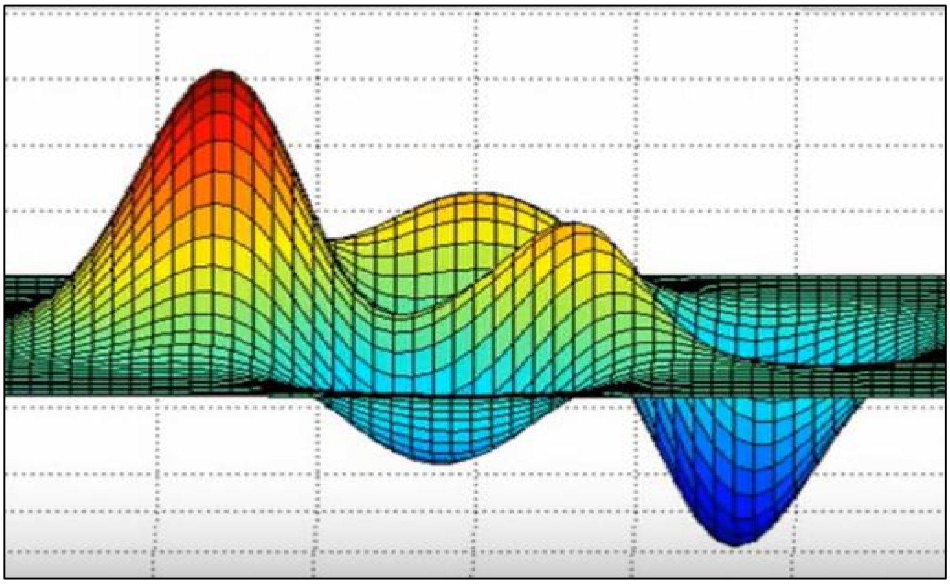

Figure 4.15 Optimization technique (Optimization Toolbox User's Guide, 2014)

is the first stage, while the coordination between the intersection AA and ME is the second one. As a series of mixed-integer non-linear models, the optimization problem was formulated and solved by thestandard branch-and-bound technique. The extensive numerical and simulation analyses revealed that the optimal designs and performance improvement could vary under different traffic demand types and different geometric configurations (Zhao, J. et al. 2015). Hence, this research investigates the coordination of consecutive DLTs, particularly, under the heterogeneous traffic conditions considering a real case in Cairo, Egypt.

4.6.1Bandwidth Maximization Approach

Aiming an optimal offset for a set of coordinated intersections, the optimization technique is widely used as a mathematical method to maximize or minimize a function subject to certain constraints. For this purpose, two main common methods are normally used, the bandwidth maximization and delay minimization method. Generally, the extensive research investigated various signal coordination problems, where the proposed models were built based on two main approaches; bandwidth-based models and performance-based models. In this research the fixed-time coordination approach was investigated by utilizing the bandwidth maximization method. The bandwidth is defined as the amount of time available for vehicles to travel through a system at a determined progression speed (Signal timing manual, Federal Highway Administration). Generally, the bandwidth maximization approach is usded to maximize the two-way green bandwidths of a given arterial; therefore, vehicles may have larger chances to travel without stops. However, the performance-based models attempt to optimize the signal settings to improve the performance indices such as delay, stop and queue length directly, as in (Zhang, L. et al. 2016).

4.6.1.1 Previous Works in Bandwidth Maximization

The first trial to suggest a mathematical formulation for the bandwidth maximization problem was done by Morgan and Little in 1964 (Morgan, J., and Little, J. 1964). Later on, many research works have been developed to obtain the optimal offset values. As mentioned earlier, however, a number of studies focused on investigating the operational performance of different unconventional intersection designs, the methodologies to obtain the optimal signal timings for them were not discussed coherently. The Monte Carlo algorithm and the bandwidth maximization method were used as two different optimization approaches to obtain the optimal signal timings of DLTs for two different traffic demand scenarios. The main findings referred to the flexibility of the Monte Carlo method to provide intersection-tailored solutions with intersection specific design parameters. Moreover, different optimal solutions were found for the two optimization approaches. However, the Monte Carlo simulation could provide near optimum parameter selection ranges for the given traffic demands, as in (Suh, W. et al 2014). In Jagannathan, R., and Bared, J. 2004, another optimization model was developed by using the operational research solver WINQSB instead of utilizing the commercial signal timing packages. By breaking down the DLT intersection into a group of hypothetical intersections the signal timings and offsets of three different design configurations were optimized. As a result, the DLT intersection outperformed the conventional one for the cases modeled even at low traffic volumes. The reduction in the number of phases on approaches with DLT geometries of the intersection resulted in tremendous vehicular delay savings as well as a considerable increase in capacity could be clearly emphasized. According to the results, the cycle length was emphasized to be the most dominating factor that influenced signal timing plans for all modeled cases (Jagannathan, R., and Bared, J. 2004). Based on NEMA software, a phase sequence optimization model was proposed for coordination control. Based on the developed script, it was concluded that leading and lagging left turn phasing were more likely to be involved in progression bandwidth solution of signal systems with randomly distributed spacing (Ma, N., et al 2010). Also, the genetic algorithm as an artificial intelligence technique was used to design a signal timing strategy that produces the smoothest traffic flow. The results revealed that the genetic algorithm could find balanced conditions of green phase times and a reasonable cycle length as a function of traffic demand (Foy, M. et al 1992). A coordination methodology was proposed based on a two-way bandwidth maximization model to consider the queuing process. In order to estimate and calculate the queue clearance time during the optimization of phase sequences, a queueing model was developed not only by considering the phases that provide the right of way to coordinated directions but also the phases that guarantee the right of way to uncoordinated directions. Compared with the Maxband model and the Multiband model, the simulation results have demonstrated the effectiveness of the proposed model under different demands scenarios, (Ye, B.et al 2015). Also, another two models namely MaxBandLA and MAxBandGN were developed based on Little's maximization model, as small-sized mixed-integer linear programs. The MAxBandGN was proposed to deal with the offsets optimization for all the signals in a grid network without cycle constraints, while the MaxBandLA model was developed to optimize the arterial partition and the signal coordination plans. The numerical tests of the two proposed models emphasized the potential capability to produce coordination plans compared with optimized signal plans by SYNCHRO (Zhang, L. et al. 2016). Likewise, aiming to handle oversaturated signalized intersections, a genetic algorithm-based signal optimization program was developed. Under three different demand volume levels; low, medium and high demand, the developed algorithm was examined. The results revealed the superior capability of the proposed program under the low and high demand volume cases. Statistically, in terms of queue time, it performed better than signal timing plans presented by TRANSYT-7F. However, for the medium-demand volume level, the proposed program provided a signal timing plan with statically equivalent queueing time compared with TRANSYT-7F software (Park, B. et al 1999). On the other hand, to consider stochastic variability in drivers' behaviour and vehicular inter-arrival times, a stochastic traffic signal optimization method was developed. This method consisted of the stochastic simulation model in addition to three widely-used optimization methods (i.e., genetic algorithm, simulated annealing and OptQuest engine) as an external optimizer. The results indicated the outperformance of the proposed method comparing to the existing optimization programs included in TRANSYT-7F and SYNCHRO. Also, the results emphasized that the additional controller and detector settings could improve the coordinated actuated signal control operation systems (Ilsoo, Y. et al 2006).

4.6.1.2 Branch-and-Bound Algorithm

In this study, the branch-and-bound algorithm was utilized as a recommended technique to solve the mixed-integer linear programming optimization problems. By systematic enumeration, a set of candidate solutions is formed as a rooted tree as in Figure 4.16 illustrates. As a constructive algorithm in nature, it proceeds by splitting the original problem into branches of sub-problems. By successively generating smaller and smaller subclasses, the branching process ultimately ensures that only one feasible solution, one of which will be optimal. The optimal solution is obtained by repeatedly partitioning the class of all feasible solutions into smaller and smaller subclasses. Consequently, at each partitioning, the subclasses of solutions are mutually exclusive and all-inclusive. Each node in the rooted tree is considered as a new integer linear programming optimization problem so that each feasible solution belongs to exactly one subclass. Also, based upon the upper and lower estimated bounds of the optimal solution, a bounding function is used to limit the search space and to each branch. The branch will be discarded in case of not producing a better solution than that found by the algorithm in the previous step (Clausen J. et al 1999).

4.6.1.3 The Basic Model Concept

Based on the common cycle calculation and offset optimization method, the entire control algorithm was built and developed. Employing Webster's (1958) method, the optimal total cycle length of each studied intersection was calculated as illustrated in Equation 4.1, while the split green time was calculated as in Equation 4.2. The longest cycle length was assigned for the three intersections to ensure the fundamental coordination along the whole corridor, then the split time was reassigned based on each phase flow ratios as in Equation 4.2. Each intersection requires six signal groups to control the main intersections as well as the upstream crossovers in both major and minor approach as Figure 4.13 depicts. According to Webster's delay minimization method, the overall delay inside the intersection is minimized by optimizing the green time based on the flow ratio of each phase (Shokry et al 2015).

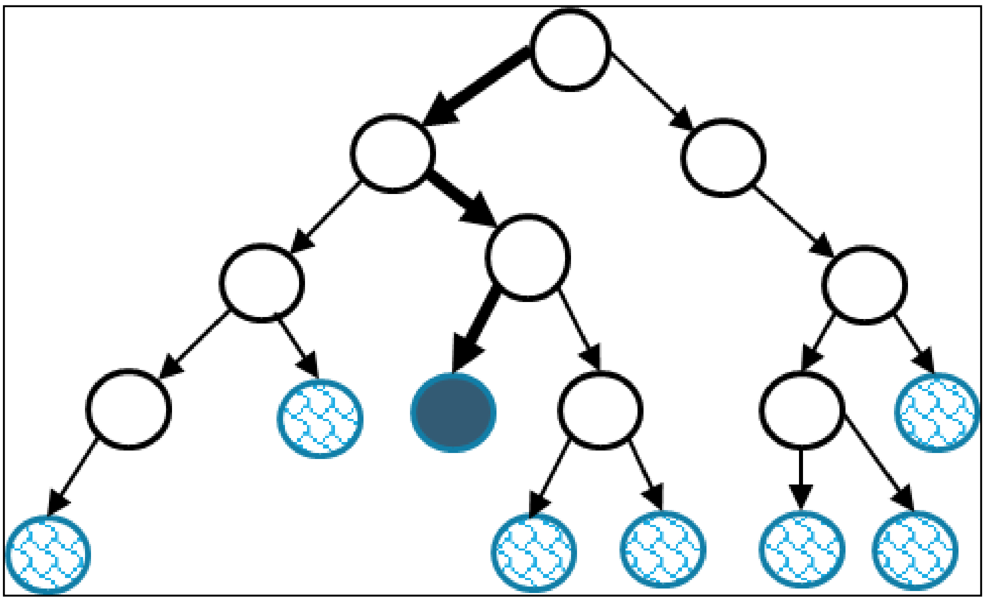

Figure 4.16 A diagram explains the main concept of the branch-and-bound algorithm.

4.6.1.4 The Objective Function

In this research, to obtain the optimal offset value which maximizes the green bandwidth of the major approach for both directions, the objective function is formulated as a linear mixed-integer problem. One objective function is used to consider the bandwidth of two directions by giving different weights of each direction bandwidth, as illustrated in Equation 4.3. In this context, however, both eastbound and westbound bandwidth has the same weight.

$$\text{Maximize: West } b + \text{Wwst } \overline{b} \tag{4.3}$$

Where:

West : the weight for eastbound bandwidth

Wwst : the weight for westbound bandwidth.

The objective function is subjected to six sets of constraints as follow:

$\overline{b} = b;$

$w_i + b \leqq 1 - r_i;$

$\overline{w}_i + \overline{b} \leqq 1 - r_i;$

$(w_1 + \overline{w}_1) - (w_i + \overline{w}_i) - m_i = (r_i - r_1) - (t_i + \overline{t}_i);$

$m_i = \text{integer};$

$b, \overline{b}, w_i, \overline{w}_i > 0.$

Where:

$b, \overline{b}$: the bandwidth of west and eastbound respectively in seconds;

$w_i, \overline{w}_I$: the total lost time per cycle in seconds;

r_i, r_1 : red time of (S_i) DLT and (S_1) DLT respectively in cycles;

$t_i, \overline{t}_I$: travel time from/to (S_i) DLT and (S_1) DLT respectively (cycles) as shown in Figure 4.17.

4.6.1.5 Simulation Outputs and Discussion

The simulation outputs revealed the significant enhancement in all operational performance indices as an obvious indication of the smooth flow at the coordinated DLT intersections along the studied corridor. Based on the proposed algorithm, the optimal offsets were successfully obtained; accordingly, the signal controllers were reset. The longest calculated cycle length of AA-DLT design the intermediate intersection along the corridor, of 71 seconds was assigned for all the intersections to ensure the coordination through the studied corridor. The optimal offset values could produce desirable green waves, which improved the overall traffic flow propagation along the studied corridor. To evaluate the coordination results, the operational performance of the coordinated models is compared with the isolated (non-coordinated) DLT simulation outputs that were discussed in the previous section of this chapter as well as the performance of the existing conventional intersections that were analyzed and evaluated in the previous chapter of

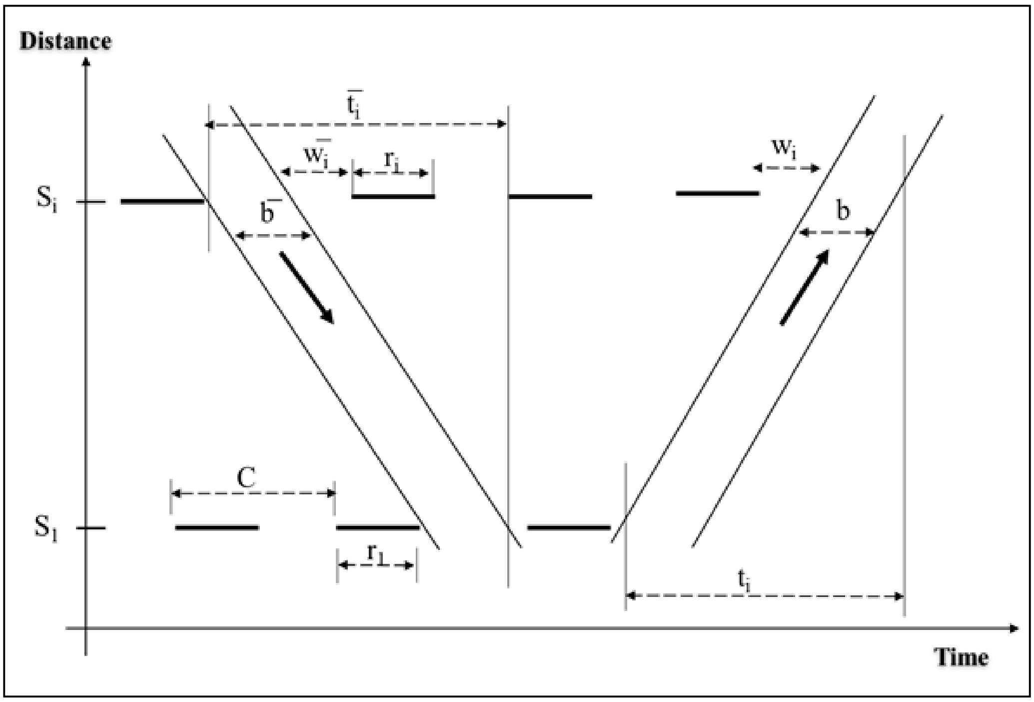

Figure 4.17 The time-space diagram shows the coordination structure.

this dissertation. Despite the reduction in the total cycle length by -38% comparing to the existing cycle length of the studied intersections, the intersections throughputs were obviously icreased for all the intersections along the corridor as shown in Figure 4.18.

All the simulation results revealed the superiority of the coordinated DLTs over the existing intersections. All performance indices such as the total travel time, the overall delay, the average speeds and so on, showed improvements as shown in Table 4.3. Also, a significant reduction was highlighted in terms of the average delay of the coordinated corridor which was reduced by -81.15% as shown in Table 4.3. Likewise, the average number of stops for the whole corridor was decreased by -88.23%. The results also showed a smooth travel along the coordinated intersections that could be illustrated by the travel time results. The results revealed the obvious enhancement of the travel time for both coordinated directions (west and eastbound) in the studied corridor as shown in Figure 4.19. The westbound average travel time was reduced by -72.36%, while the eastbound travel time was reduced by -48.61%. Although the average travel time has

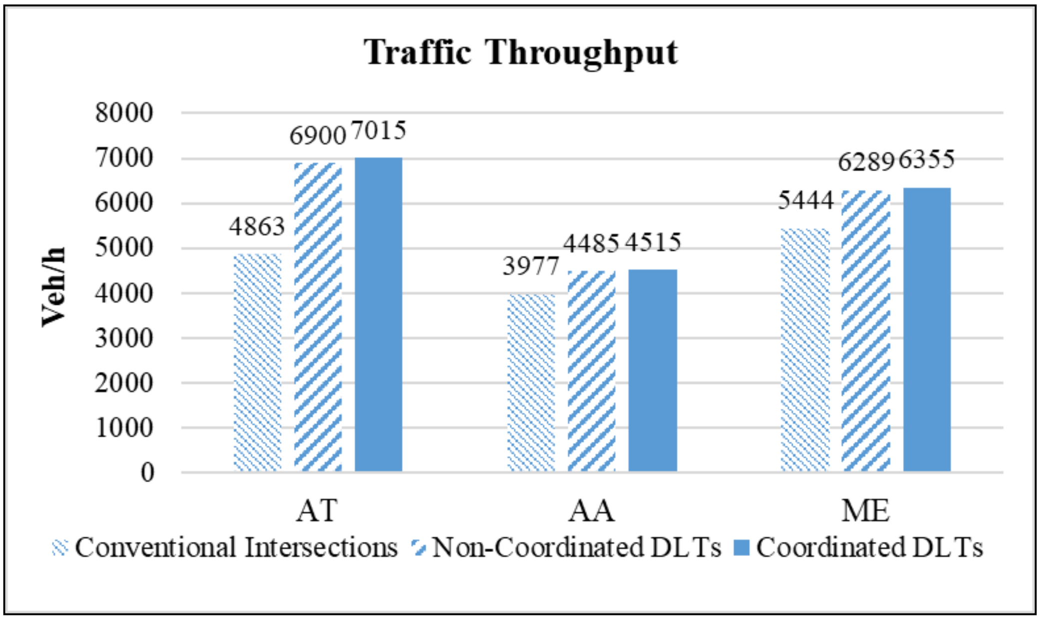

Figure 4.18 Traffic throughput comparison of coordinated and non-coordinated DLT vs. their conventional counterparts

fluctuated along the corridor, the coordination of DLTs along the corridor could provide a stable travel time as a result of the green bandwidth maximization. Furthermore, considerable savings in overall delays in all the intersections were observed. The reduction percentage of overall delay is -53.02%, while the increase percentage in average speed along the corridor is 72.65%.

Table 4.3 Corridor operational performance indices.

Performance index	Conventional intersections	Non-coordinated DLTs	Pre-timed Coordinated DLTs
Total travel time (h)	2,544.14	1,335.38	1,306.38
Avg. delay/veh. (s)	409.11	200.518	192.17
Avg. speed (km/h)	12.98	19.21	22.415
Avg. No. of stops/veh.	20.66	2.88	2.43
Avg. stopped delay/veh. (s)	166.30	36.44	30.72

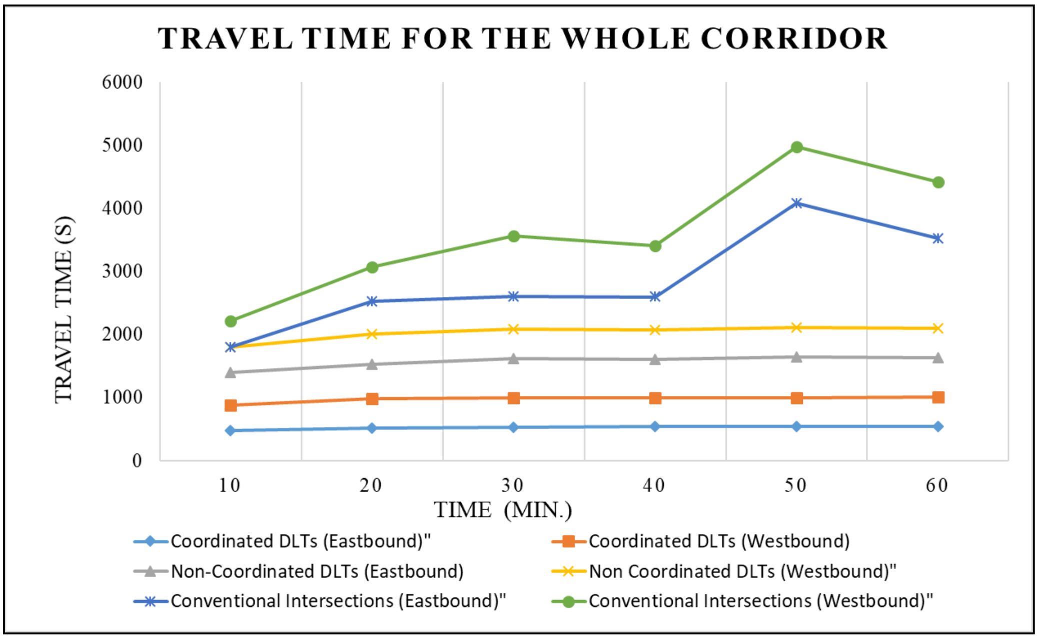

Figure 4.19 The total travel times along the studied corridor under pre-time coordination.

4.6.2 Delay Minimization Approach

The delay inside at an intersection is defined as the additional travel time caused by the sum of all components of delay for any lane group, including control delay, traffic delay, geometric delay, and incident delay as shown in Figure 4.20. According to the FHWA, the control delay is defined as a measurement of the aggregate sum of stopped vehicles for a particular time interval divided by the total entering volume for that movement (Signal timing manual, Federal Highway Administration). As a key factor influences the total delay at an intersection, lots of the previous research works attempted to minimize the control or the stopped delay. Although the bandwidth maximization approach provides larger chances by assigning more green time for vehicles to travel without stops, some sceptics criticized that this approach may not guarantee an optimal delay at an intersection. So that, the delay minimization technique is widely used to minimize the delay at an intersection subject to certain constraints as a mathematical method.

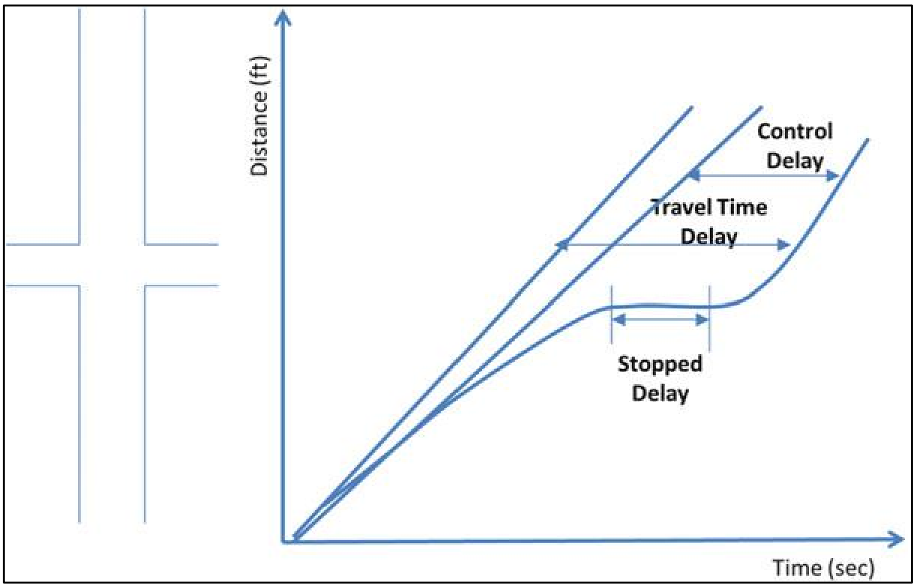

Figure 4.20 A graphical illustration of delay (FHWA, 2008).

4.6.2.1 Previous Works in Delay Minimization

In order to comprehend and establish a coherently theoretical framework in employing the delay minimization approach for this study context, this literature review is presented. Several available and considerable studies have been presented since Webster developed the traffic signal control optimization principles. Based on Webster's proposed model, other advanced models have been provided considering the stochastic conditions (Newell, G. F. 1960; Miller, A. J. 1963).

The three widely used optimization methods namely genetic algorithm, simulated annealing and OptQuest engine were employed to address the stochastic traffic signal optimization method. Utilizing a micro-simulation environment, the performance of the proposed method that consists of the stochastic simulation model and an external optimizer was compared with the existing optimization programs including TRANSTY-7F and SYNCHRO (Ilsoo, Y. and Park, B. 2006). The comparison results of the six tested networks indicated the outperformance of the provided method over the existing above-mentioned software; however, the analyzed networks were operated in an ideal traffic environment. In other words, the complexities of the heterogeneous conditions such as the non-lane based traffic and aggressive driving behaviour were not appropriately investigated. Mirchandani, P. and Head, L. developed a simple estimation procedure accounting for arrivals and estimated departures based on queue discharge rates. The RHODES system that referred to a real-time traffic-adaptive signal control system showed promising results of several CORISM models of actual transportation networks (Mirchandani, P. and Head, L. 2001).

Also, on the basis of the delay minimization technique El-Tantawy, S. et al presented an adaptive traffic signal control in Canada. Based on Reinforcement Learning (LR) as one of the efficient approaches to solving such stochastic closed loop optimal control problem, a seamless application of adaptive traffic signal control scheme was provided. The proposed LR controller could save 48% of average vehicle delay comparing to the optimized pre-timed controller and fully-actuated controller (El-Tantawy, S. et al 2014).

Taking into consideration balancing the interests of cyclists and motorized traffic, Apostola, T. developed a traffic signal optimization system relying on the delay minimization approach. In order to optimize the traffic signals of a coordinated corridor, the presented multi-objective function considered the number of stops, delay and desired speed of cyclists in addition to the delay of motorized traffic. By summing up the delay caused by the traffic signal, the delay because of the formed queues and the overflow delay, both motorized vehicle and cyclist's delay was calculated. So that, the overall delay occurs when the arrival rate exceeds the service rate at a traffic signal. The proposed system was implemented and tested by employing VISSIM-COM and FminconMultistart optimizer in MATLAB. Four different software namely; VRIGen, TRAFCod, PTV-VISSIM and MATLAB were employed to accomplish this study objective. As a traffic control generator, VRIGen was used to find the optimum structure, minimum total cycle time and green times for all traffic streams. The output generated files with the traffic program of VRIGen is the input of TRAFCod as a full-scale traffic simulator program that allows for external traffic signal controllers. The proposed system could enhance the cyclists' performance by giving speed advice to cyclists based on optimal green times, either by synchronizing the traffic signals at cyclists' speeds or by combining both strategies (Apostola, T. 2014). Ravikumar, P., & V. Mathew, T. developed a vehicle-actuated signal controller in 2011 by highlighting some strategies, particularly, for heterogeneous traffic conditions implementation. The developed controller proposed several strategies such as changing the detector configuration and the loop size, logical grouping of signal phases and the use of dummy phases. Although the implementation strategy overcame the existing pre-timed signals, the sensitivity of the thresholds for cycle time, green time, gap needs more investigation (Ravikumar, P., & V. Mathew, T. 2011). Based on estimating the sampled travel times measured between upstream and downstream locations of a signalized intersection, Ban, X. et al proposed an estimated delay algorithm. The different signal phases were estimated only based on the delay patterns, without the need to know signal timing or traffic flow information. Accordingly, the proposed model was represented on the basis of the delay characteristics. Piecewise linear curves were plotted due to the characteristics of the queue forming and discharging. For a nontrivial increase in delay after the start of the red time, detection of the start of a cycle could be obtained. The least squares–based algorithm was developed to match measured delays in each cycle by using the estimated piecewise linear curves (Ban, X. et al 2009). In another study by Wey, W., the platoon dispersion constraints directly translated into the signal control model in order to model the movement of the traffic along the streets between the intersections in a time-expanded network. The problem was formulated as a linear multi-commodity network flow problem. The results revealed the fast solution for the proposed algorithm (Wey, W. 2000). Meanwhile, based on delay minimization method, Zhou formulated a mixed-integer non-linear program. Here, cycle length and the phase sequence were also decision variables. Both approaches assumed that queues did not exceed the storage area (Zhou, Y. and Zhuang, H. 2014).

Referring to the above-mentioned highlighted literature, it can be concluded that most of the previous studies built up their architecture algorithms and analysis upon platoon arrivals fundamentals to develop arterial traffic-adaptive control systems (Mirchandani, P., & Head, L. 2001). However, the impacts of the heterogeneous traffic conditions on the optimizing the intersections' delays along corridors were not considered. Therefore, the novelty of this study is developing a real-time demand-responsive signal control system by taking into consideration the heterogeneous traffic complexities' impacts. Hence, the intersection delay is estimated depending on each individual vehicle characteristics.

4.6.2.2 Delay Estimation

Due to the fact that the cumulative delays at intersections are the major contributing factor to arterial delays so that it is essential to estimate the delay at an intersection accurately. As it has been mentioned earlier in the previous section of this chapter, a part of this research is to propose real-time demand-responsive signal control logic that adaptively controls the traffic demand. Likewise, to fulfill the needs of the dominant heterogeneous traffic conditions, the delay at an intersection is not considered for a platoon, however, it is estimated individually for each vehicle. Therefore, the cumulative delay at an intersection is calculating by summing up the delay of an individual vehicle in each approach at the intersection.

The delay of each vehicle is calculated as the sum of:

1. The traffic signal control delay;

2. The delay occurred due to the formed queues in front of an individual vehicle in each lane at an intersection.

Although, the stopped delay that is defined as the delay occurred when a vehicle is totally immobilized. In this study, a vehicle when moving at 5 km/h experiences the stopped delay. As a result of using only two phases to control each DLT primary intersection, the green time at major approach will start when the green time at the minor approach ends and vice versa. Therefore, at an intersection during the same cycle, the major and the minor approach delays are dependent. By assuming a point-queue upstream the main intersection, the speed and the position of each vehicle are used to calculate the estimated arrival time-point for each vehicle to reach the stop line as Figure 4.21 depicts. Consequently, the total number of queuing vehicles in the formed queues in front of each stopped vehicle at each lane inside an intersection is estimated as Figure 4.22 depicts. The delay of each vehicle upstream of each approach was calculated accordingly, then the total delay for each approach could be estimated by summing up the delay of all arriving vehicles at the subjected intersection as illustrated from Equation 4.4 to Equation 4.9.

$$d_j^{\,major} = S.T^{\,major} - t_{i,arr}^{\,major} + (\textstyle\sum_{J-1}^{J} N_q/S) - d.s^{major} \tag{4.4}$$

$$d_j^{\,minor} = S.T^{\,minor} - t_{i,arr}^{\,minor} + (\textstyle\sum_{J-1}^{J} N_q/S) - d.s^{minor} \tag{4.5}$$

$$S.T^{\,major} = S.T^{\,minor} + g_{i,f}^{\,minor} + g_{i,ex}^{\,minor} + t_{i,clr} \tag{4.6}$$

$$S.T^{\,minor} = S.T^{\,major} + g_{i,f}^{\,major} + g_{i,ex}^{\,major} + t_{i,clr} \tag{4.7}$$

$$D^{\,major} = \textstyle\sum_{J=1}^{J} d_j^{\,major} \tag{4.8}$$

$$D^{\,minor} = \textstyle\sum_{J=1}^{J} d_j^{\,minor} \tag{4.9}$$

Where:

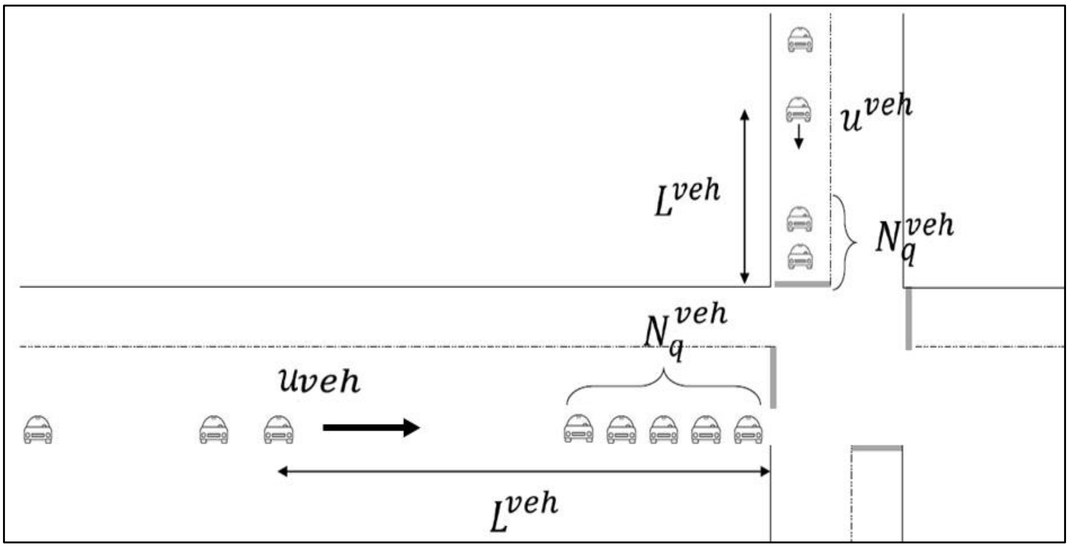

Figure 4.21 Data required for the delay estimation

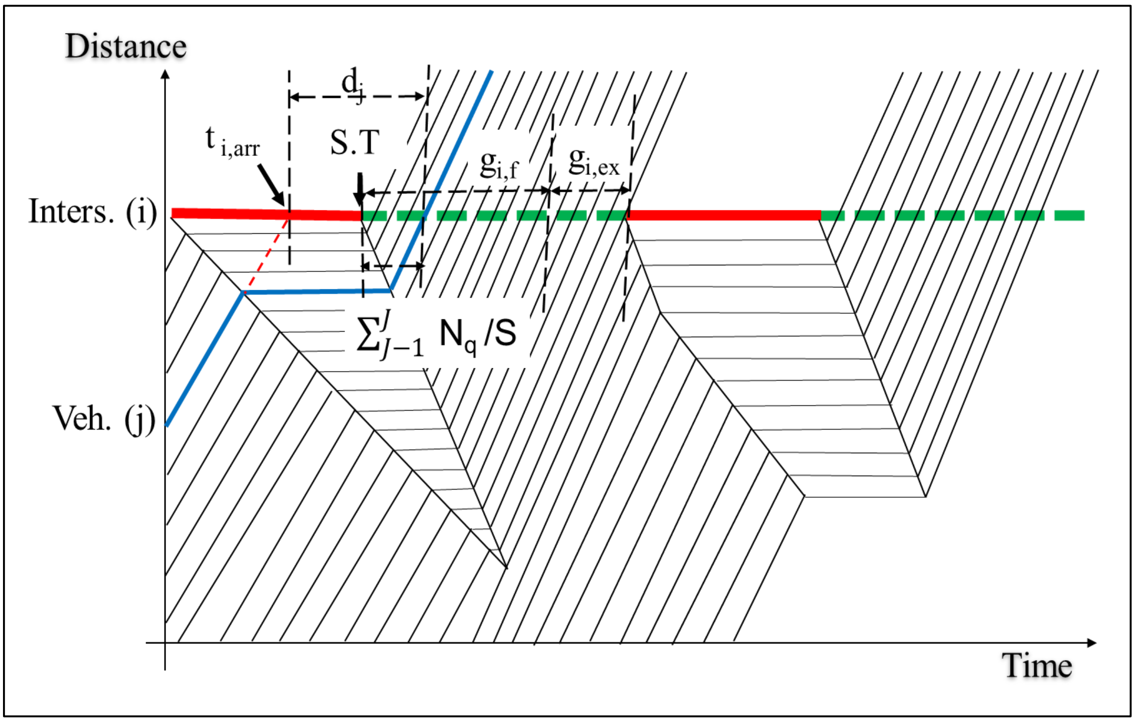

Figure 4.22 Time-space diagram

$d_j^{\ major}$, $d_j^{\ minor}$: the delay of an individual vehicle in the major and the minor approach respectively;

S.T ^{major}, S.T ^{minor}: the time point when the green time of the major and the minor starts at an intersection (i) respectively;

$t_{i,arr}^{major}$, $t_{i,arr}^{minor}$: the time point when each individual vehicle on the major and

the minor arrives at the stop line of an intersection (i) by assuming point queue respectively;

$\sum_{J-1}^{J}$ Nq: the total number of queuing vehicles in the formed queues in front of

vehicle (j) at an intersection (i);

S: the saturation flow rate at an intersection (i) (veh/ln/s);

$d.s^{major}$, $d.s^{minor}$: the delay saved because of the extension green time in the major and the minor approach at an intersection (i) respectively in seconds (s);

$g_{i,f}^{major}$, $g_{i,f}^{minor}$: the fixed green time of major and minor approach at intersection (i) respectively in seconds (s);

$g_{i,ex}^{major}$, $g_{i,ex}^{minor}$: the extension green time of major and minor approach at an intersection (i) respectively in seconds (s);

$t_{i,clr}$: the clearance time at intersection (i) in seconds (s);

D^{major}, D^{minor}: the total delay in major and minor approach at an intersection (i) respectively in seconds (s);

J: the total arriving vehicles at an intersection (i);

The delay of each vehicle consists of the stopped delay in addition to the queue discharging delay. The stopped delay is estimated as the time difference between the time point when the green time of the subjected phase starts at an intersection and the arrival time point for each vehicle to reach the stop line by assuming point queue. The standing queue discharging delay is estimated by dividing the number of formed vehicles in front of each stopped vehicle by the saturation flow rate at the subjected intersection as shown in Equation 4.4 and Equation 4.5. For this study context the time step was set as 0.1 seconds, therefore the delay is estimated for each time step iteration.

4.6.2.3 The Control Algorithm

The control algorithm in this study is developed based on the advanced promising technology in the area of vehicle detection and communication. Due to the fact the ability of adaptive signal control systems to provide control strategies in response to the real-time traffic conditions, this study algorithm was built. The adaptive traffic signal control system provides the needed potential to significantly alleviate traffic congestion as opposed to the commonly used fixed-timed and actuated control systems for isolated intersections (McShane, Roess, & Prassas,1998). The proposed system in this research takes the input detector data for real-time measurement of traffic flow, and the signal controller is adapted iteratively based on the input data. Although the hourly traffic volumes are not direct inputs, the dependent gap out time (the spent time between two passing vehicles) as well as the total number of queueing vehicles in the formed queues are the actual inputs of the proposed algorithm. To ensure a smooth flow along the coordinated corridor, the proposed control algorithm prioritized the flow continuity of the running flow. Therefore, the gap out time condition prioritized over the queue length condition statement at each intersection as Figure 4.23 depicts. During the green time of a running stream approach, the gap out time for each two successfully moved vehicles is measured at each lane upstream the primary intersection. Simultaneously, the queue length of the stopped standing queues during the red time of the other approach is also calculated by summing the lengths of stopped vehicles' and the gap-distance between each pair. To avoid any blockage or bottlenecks occurrence, and emphasize the functionality of DLTs operation performance, both standing queue length upstream the main intersection as well as upstream the crossovers should not exceed the storage length. For this end, the DLTs capability for both approaches would not be influenced. If one of the abovementioned conditions are satisfied, the original pre-timed plan would be operated based on the initial design that is individually designed for each isolated intersection. Otherwise, the optimization would be executed to find the optimal extension green time for both major and minor approach that fulfill the minimum delay at the intersection.

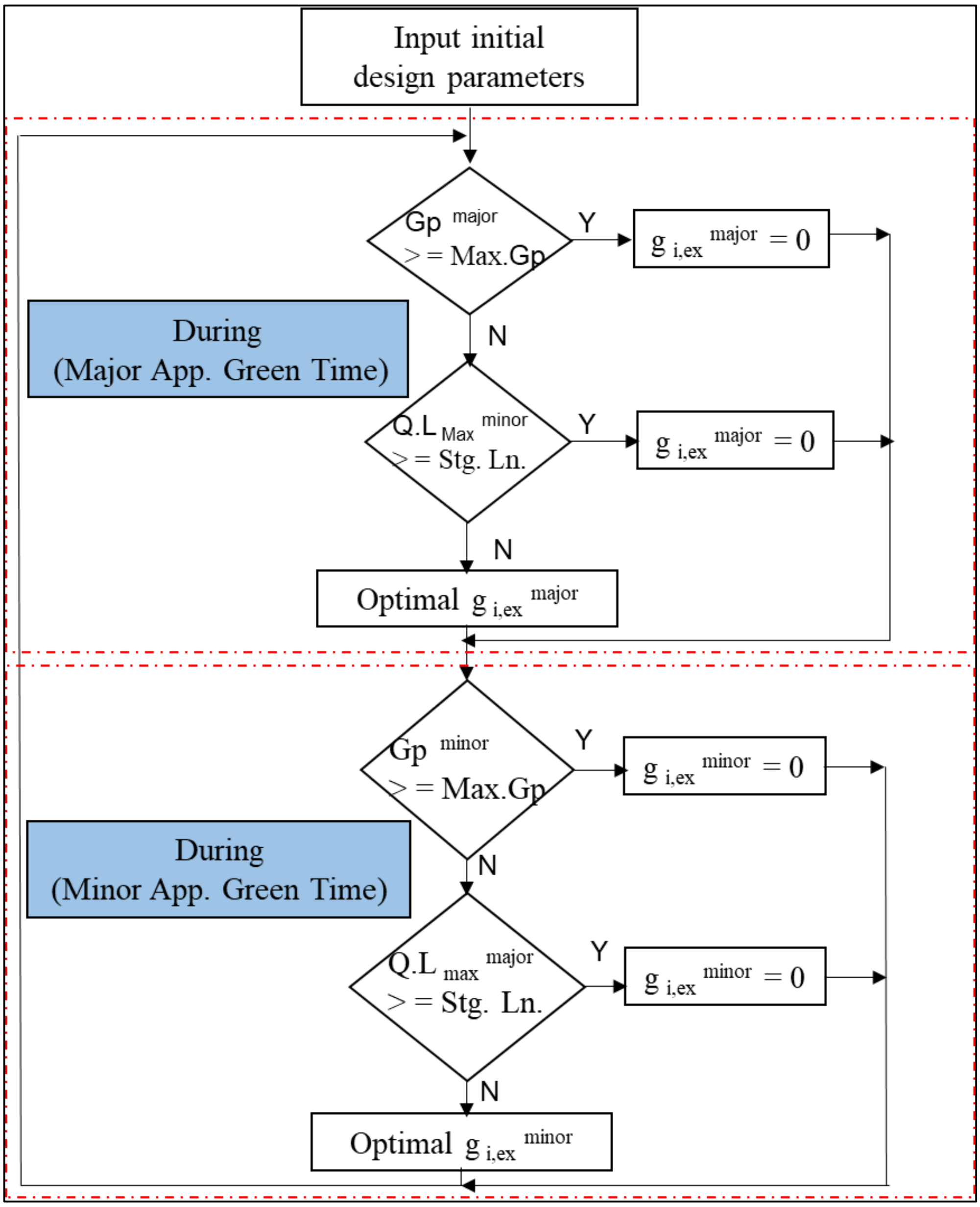

Figure 4.23 The control algorithm flowchart

The initial input design parameters of the entire proposed control algorithm were calculated based on Webster delay function (1966) as it is illustrated in the previous section of this chapter. The cycles time as well as the green splits of each phase for different signal groups of each intersection on the studied corridor were estimated as a fixed-time cycle. These variables were assigned as initial input for the proposed algorithm as Figure 4.23 illustrates.

where:

$Q.L_{max}^{major}$, $Q.L_{max}^{minor}$: Max. queue length in the major and minor approach at an intersection (i) respectively in meters (m);

Gp^{major}, Gp^{minor}: Gap-out time between two passing vehicles in major and minor approach at an intersection (i) respectively in seconds (s).

Stg. Ln.: Storage length at an intersection in major and minor approach (i) in meters (m)

4.6.2.4 Traffic Detection

As an important controller element that the proposed control algorithm relies on, the positions of the detectors used in this design was precisely determined and carefully installed upstream the main intersection along the major and minor approach. The detectors' positions were installed 10 meters ahead of the end of each lane to emphasize enough storage length for the stopped standing queues during the red time of the other approach as shown in Figure 4.24. The detectors are crucial to detecting the gap-out time between two passing vehicles as well as the total number of queueing vehicles in the formed queues to estimate the queue length as Figure 4.21 depicts. To calculate the vehicles arrival time at the stop line as well as the number of vehicles formed in front of the stopped ones, the speeds and the position of each vehicle is needed at time step.

4.6.2.5 Optimization Problem

In this research context, the optimization problem is formulated as a one objective optimization problem to minimize the control delay for both road users, minor and major approach at an intersection. The optimization problem aims to obtain an optimal extension green time, that minimizes the total intersection control delay for both road users at an intersection. The proposed optimization problem is formulated as a non-linear constraint programming problem. The designed system endeavors to minimize the total delay for consecutive DLTs traffic signals by considering the individual vehicle arrivals caused by platoon dispersion as natural stochastic variations of heterogeneous traffic

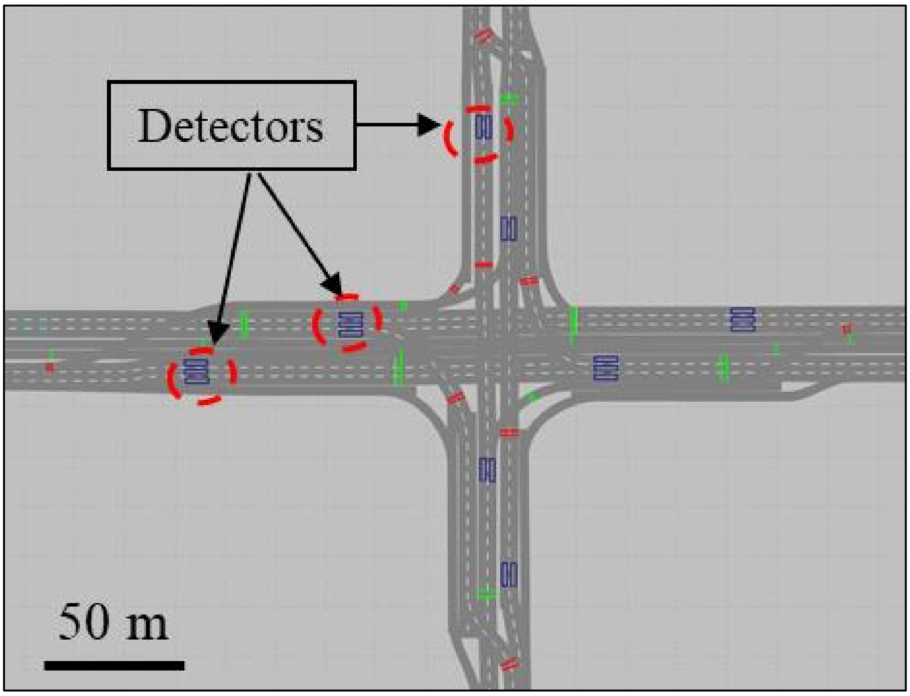

Figure 4.24 A layout shows the detectors' positions.

conditions. Hence, in this study, the optimization is individually performed for each intersection, accordingly, the coordination of the DLTs is implicitly achieved. MATLAB optimization solver is utilized for processing the obtained data to algorithmically select the optimal signal timing fast and accurate. For this purpose, the *fmincon* function is nominated in this study due to the fact that it finds the minimum of a constrained nonlinear multivariable function. This function gives the constrained local minimum point, where the function value is smaller than at nearby points, but possibly greater than other points in the search space (Optimization Toolbox User's Guide, 2014). By starting at an initial estimate, *fmincon* finds a constrained minimum of a scalar function of several variables simultaneously as constrained nonlinear optimization or nonlinear programming.

4.6.2.6 The Objective Function

The objective function was formulated to minimize the accumulative delay in both major and minor approach. By giving different weight coefficient defining the relative importance of major and minor approach delay, the contradicting objectives for both road users were considered. The relative importance of major approach delay coefficient was adjusted as double as the minor approach delay coefficient as Equation 4.10 depicts. The extension green time for both approach road users, is the optimization variable that can be changed to achieve the optimum solution. The lower and upper limit of the optimization variable- the extension green time- was selected as 0.0 and 10.0 seconds respectively. On the other hand, the optimization horizon was adjusted to one cycle length.

The objective function is illustrated in Equation 4.10:

$$f(X) = \min \{\alpha D^{major} + \beta D^{minor}\} \in g_{i,ex} \quad (4.10)$$

Where:

X: the optimal total intersection control delay for both road users, minor and major approach;

α, β: coefficient defining the relative importance of major and minor approach delay.

The relative importance coefficients (α, β) were selected by considering the main findings and recommendation of the previous works (Apostola, 2014). The previous findings revealed that the optimal (α) value is two times (β) value.

The objective function is subjected to one constraint for the extension green time as follow:

$$0 \leqq g_{i,ex}^{major}, g_{i,ex}^{minor} \leqq 10 \quad (4.11)$$

Where:

$g_{i,ex}^{major}$, $g_{i,ex}^{minor}$: the extension green time of major and minor approach at an intersection (i) respectively in seconds (s);

As indicated above in the previous equations, it can be concluded that the pre-given parameters are: a) the time point when the green time of major and minor starts at an intersection (i), b) the time point when each individual vehicle on major and minor arrives at the stop line of an intersection (i) by assuming point queue, c) the total number of queueing vehicles in the formed queues in front of vehicle (j) at an intersection (i), the saturation flow rate at an intersection (i), d) the delay saved because of the extension green time in major and minor approach at an intersection (i), and e) the fixed green time of major and minor approach at an intersection (i) respectively in seconds. Utilizing the objective function in order to estimate the extension green time for both approach road users major and minor approach which minimizes the total intersection control delay for both road users at an intersection (i).

4.6.2.7 Programming the Control Algorithm

A non-linear programming code was developed in MATLAB as a multi-paradigm numerical computing environment. Also as a fourth-generation programming language, MATLAB emphasis on matrix manipulation, numerical calculations and data optimization of functions which is needed to accomplish the real-time demand-responsive signal control system proposed in this research (Optimization Toolbox User's Guide, 2014). Through VISSIM-COM Interface, the user is able to manipulate the attributes of most of the internal objects dynamically. An inter-process communication and dynamic object creation are provided by utilizing VISSIM-COM interface and MATLAB (Apostola, 2014). As it can be seen in Figure 4.25 the way in which the proposed system is iteratively operated at each time step. First, the simulated model network is created and saved in VISSIM through a user-friendly graphical interface (GUI) offered by VISSIM. Second, the different data that required to estimate the delay at an intersection (i.e. the speeds, the position of vehicles, the gap-out time…etc.) is detected through VISSIM installed detectors. Third, the needed input data is sent from the VISSIM via COM interface to MATLAB to start the dynamic optimization process. Finally, after processing the optimization and obtain the optimal extension green time values, the taken decision returned back to VISSIM to execute the decision through the signal controller programs. The developed programming code is attached as an Appendix at the end of this dissertation.

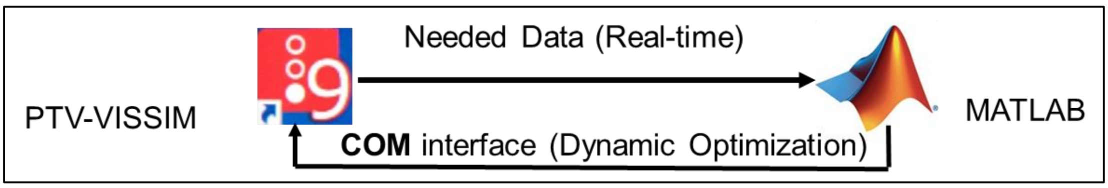

Figure 4.25 A flow chart of the system implementation environment.

4.6.2.8 Simulation Outputs and Discussion

The overall results are summarized and discussed in this section to draw up the operational performance of the consecutive DLT intersections under the real-time demand-responsive signal control system proposed in this research. To examine the efficiency of the proposed algorithm, the simulation results are compared to the entire conventional intersections, the non-coordinated DLTs and the pre-timed (fixed-time) coordinated DLTs. The simulation results emphasized the superiority of the coordinated DLTs over the existing conventional intersections, the non-coordinated DLTs and the pre-timed coordinated DLTs. All performance indices such as the total travel time, average delay, queue lengths, average stopped delay per vehicle, average speeds, and the average number of stops pointed to an undoubted improvement as illustrated in Table 4.4. As a traffic continuity indication, the average number of stops dropped significantly from 20.66 to 0.55, while the average stopped delay per vehicle decreased by -97.33% as it indicated in Table 4.4. Also, the obtained outputs revealed prominent savings in average control delays along the studied corridor as an indicator for the delay minimization at the entire DLTs intersections. The average delay per vehicle is dropped by -91.68% compared to the existing conventional intersections, while it is dropped by 50.98% and 53.03% for the non-coordinated DLTs and pre-timed coordinated DLTs respectively. The proposed algorithm provides a smooth travel along the coordinated DLT intersections, the average speeds significantly increased from 12.98 km/h for the conventional intersections to 41.74 km/h for the real-time coordinated DLTs and 22.41 km/h for the pre-timed coordinated DLTs and 19.21 for the non-coordinated DLTs as it indicated in Table 4.4.

Based on the results, the proposed system not only decreases the total travel time but also provides a stable travel time as a result of the smooth operation along the subjected corridor. The travel time indices revealed an obvious enhancement of both west and eastbound of the studied corridor as Figure 4.26 depicts. The enhancement of the travel time through the three DLT intersections along the

Table 4.4 Corridor operational performance indices.

Performance index	Conventional intersections	Non-coordinated DLTs	Pre-timed Coordinated DLTs	Real-time Coordinated DLTs
Total travel time (h)	2,544.14	1,335.38	1,306.38	1,206.30
Avg. delay/veh. (s)	409.11	200.518	192.17	34.03
Avg. speed (km/h)	12.98	19.21	22.415	41.74
Avg. No. of stops/veh.	20.66	2.88	2.43	0.55
Avg. stopped delay/veh. (s)	166.30	36.44	30.72	7.07

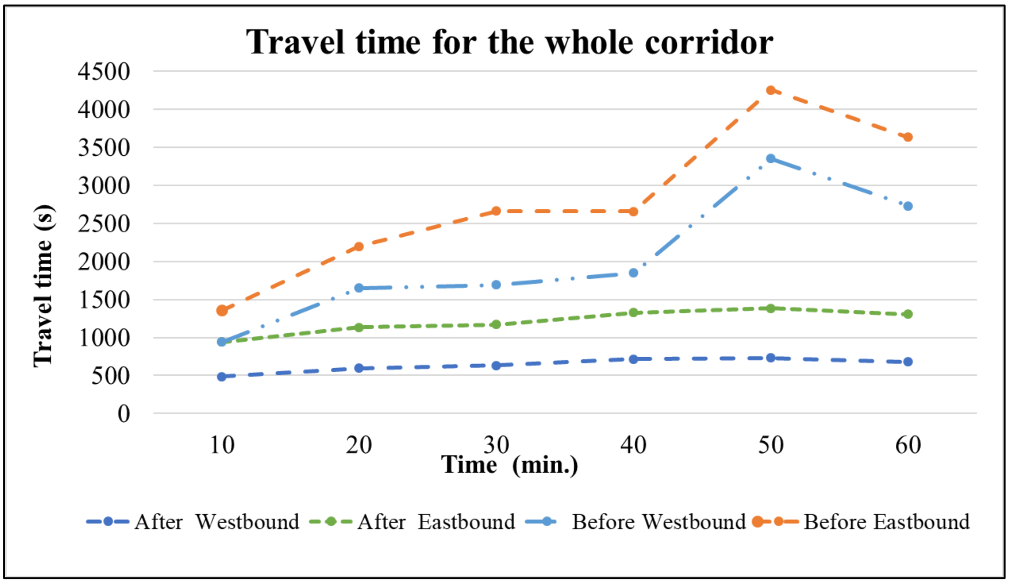

Figure 4.26 Travel time comparison of the whole corridor under real-time coordination.

corridor obviously proves the efficiency of the proposed algorithm. Additionally, the minor approach travel time, north and southbound travel time was obviously enhanced as shown in Figure 4.27. Furthermore, the minor approach performance indices also referred to a significant improvement. The average stopped delay per vehicle, the total travel time, the average delay as well as queue lengths could improve indisputably.

The obvious enhancement of the corridor performance, however, the heterogeneous traffic complexities dramatically influence the performance rates. The diverse dynamic properties and the

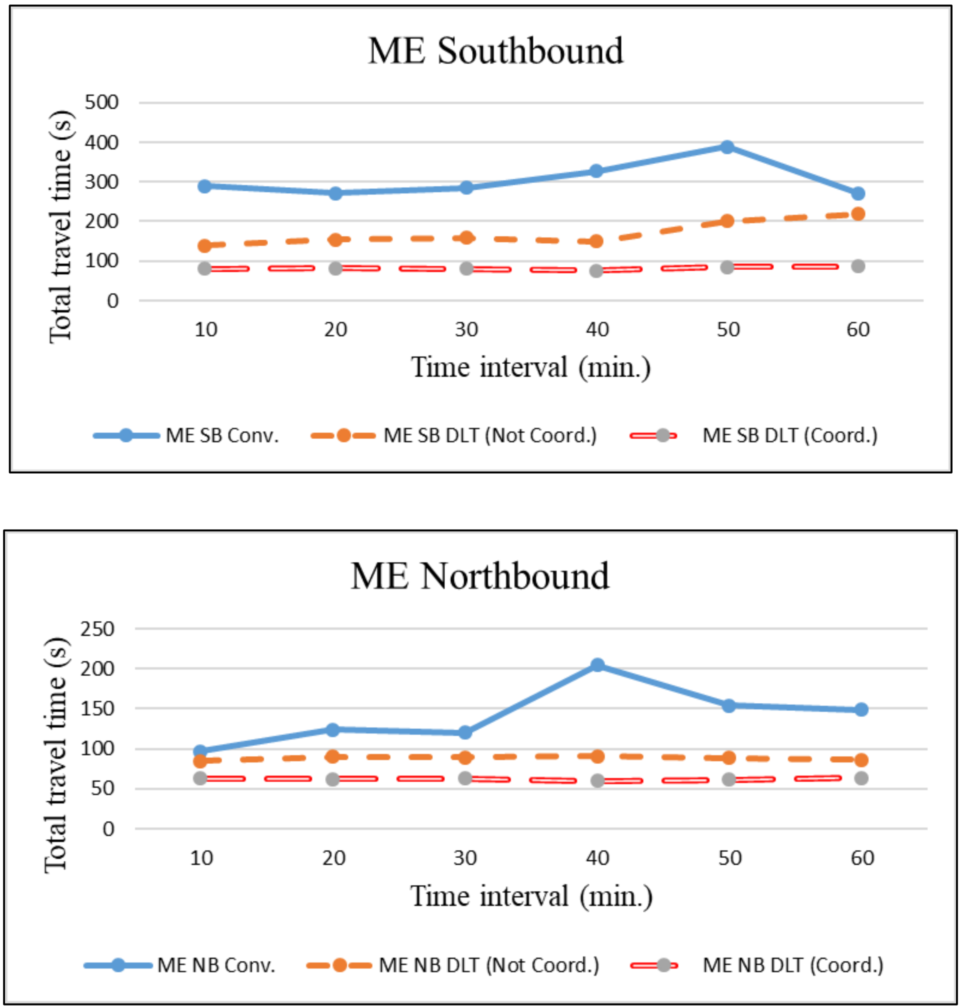

Figure 4.27 Travel time comparison of ME intersection minor approach

non-lane base phenomenon, especially, upstream the crossovers affect the gap-out time. The aggressive driving behaviour upstream the crossovers could obviously influence the flow headways upstream the detectors. As a result, the restricted statement conditions in the proposed algorithm, particularly, the storage length condition

limits the extension green times.

Taking into consideration the storage length limitation for both road users, major and minor approach, could restrict the optimization function. As a result of considering the storage length upstream the main intersections as well as upstream the crossovers, the statement conditions could be satisfied, especially, under heavy traffic conditions and cross ponding short storage lengths. The designed storage length is a key design factor to ensure the continuous functionality of the provided displaced left-turn crossovers.

4.7 Conclusion

This chapter discussed the DLT innovative scheme under the heterogeneous traffic conditions. Although academic in nature, the proposed DLT in this context was evolved through a real-world practical application for three entire signalized intersections as a realistic study. In the first part of this chapter, the DLT design was individually examined and evaluated as isolated intersections under the heterogeneous traffic complexities. The simulation results indicated to an undoubted improvement for all performance evaluation indices, particularly, under the heavy traffic volume levels as a result of DLTs outperformance over the conventional intersections. An obvious enhancement in intersections throughputs, the average total delay time per vehicle, the average speeds and the average stopped delay per vehicle was emphasized. However, the significant enhancement of the analyzed intersections under the proposed DLTs, the improvement indices are lower than their counterparts under the ideal condition showed in the previous studies. Due to the heterogeneous conditions considered in this study (i.e the diverse dynamic and static properties of mixed traffic compositions, the non-lane based system, the aggressive driving behaviour…etc) were obviously influenced the operational performance improvement rates of the proposed DLT designs.

The second part of this chapter discussed the coordination possibility of a series of DLTs intersections as a coordinated corridor under the same traffic conditions. The most two common coordination techniques; bandwidth maximization and delay minimization were employed to propose the coordination of consecutive DLT intersections. The bandwidth maximization was utilized for a pre-timed coordination approach, while the delay minimization approach was used to develop a real-time demand-responsive signal control system on the solid foundation of the dynamic optimization principles. This entire demand-responsive algorithm was built based on developing mathematical model and simulations utilizing PTV-VISSIM as a simulator-based approach and MATLAB as a multi-paradigm numerical computing environment. To accomplish the real-time demand-responsive signal control system, an inter-process communication and dynamic object creation are provided by utilizing VISSIM-COM interface and MATLAB. Through VISSIM-COM Interface, the user is able to manipulate the attributes of most of the internal objects dynamically. To examine the efficiency of the proposed coordination techniques, the simulation results are compared to the entire existing conventional intersections, the non-coordinated DLTs. The simulation results emphasized the superiority of the coordinated DLTs over the existing conventional intersections and the non-coordinated DLTs. All performance indices such as the total travel time, average delay, queue lengths, average stopped delay per vehicle, average speeds, and the average number of stops pointed to an undoubted improvement. Moreover, the real-time demand-responsive signal control system outperformance over the pre-timed coordinated DLTs.

In conclusion, the DLT scheme revealed an undoubted improvement for all performance evaluation indices of the entire existing conventional intersections as isolated intersection as well as coordinated corridor However, the heterogeneous conditions could also influence the operational performance improvement rates of the proposed DLT designs.

CHAPTER 5

OPERATIONAL PERFORMANCE EVALUATION OF RESTRICTED CROSSOVER U-TURN (RCUT) DESIGN

5.1 Introduction

The Restricted Crossing U-Turn (RCUT) intersection, that is also referred to as Superstreet Median (SSM) intersection is another alternative innovative scheme proposed in this dissertation that was provided by the FHWA since the last decade. The SSM design was provided as another extension of Median U-turn (MUT) intersection as shown in Figure 5.1. The SSM intersection, however, features an additional break for the through-moving traffic flows on the cross street which allows independent operations for opposite intersections on the arterial streets as Figure 5.2 depicts. This alternative design is proposed as a promising solution for more dominant flow on arterials and main corridors to mitigate the traffic congestion at conventional signalized intersections. The novelty of this design is the only at-grade intersection known at this time to enable each direction on a two-way arterial to operate independently. Because this design does not allow movements to cross both directions of the major street, so there is no need for both directions of the major street to receive the same signal indication at the same time. Accordingly, the flow in both directions can be progressed at any speed in different signal spacing (FHWA, Restricted Crossing U-turn Informational Guide 2014). In this intersection, the two main directions travelling flows on the main artery are separated by adding an additional break for the through-moving traffic flows. RCUT design guarantees that each side of the arterial corridor can have its own cycle length and progression speed based on the direction of operational conditions such as flow rates and directional ratios. Due to the presented two-phase signal system, two one-way streets would be provided and an efficient smooth traffic flow is emphasized. As a result, the control signal of each direction allows independent operations on the arterial streets (FHWA, Restricted Crossing U-turn Informational Guide 2014). The SSM intersection changes traffic from the minor road enters or crosses the main highway by relying on a combination of right and U-turns. Despite the early

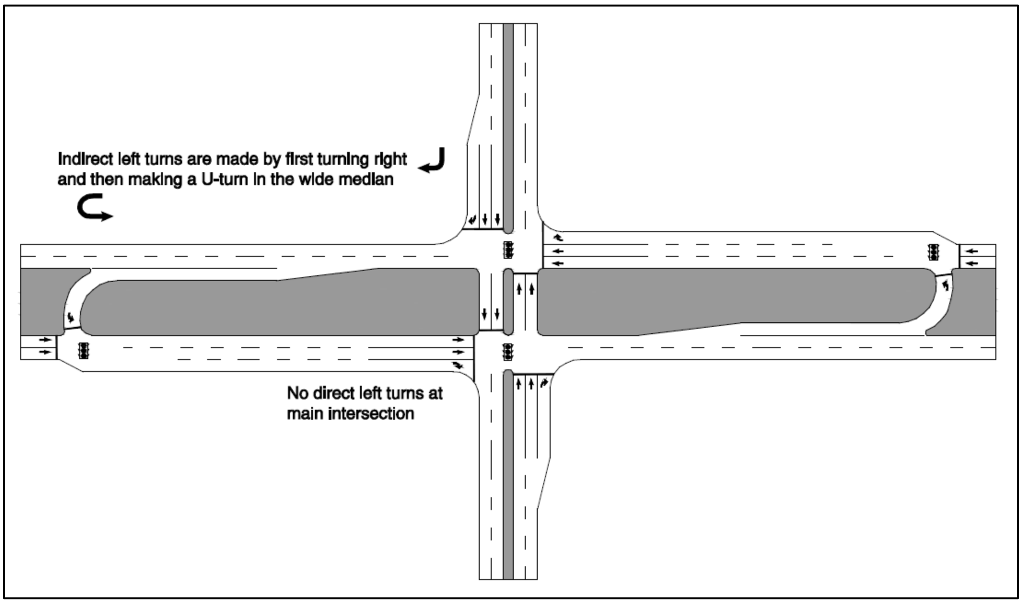

Figure 5.1 The median U-turn intersection geometric layout (FHWA, 2014).

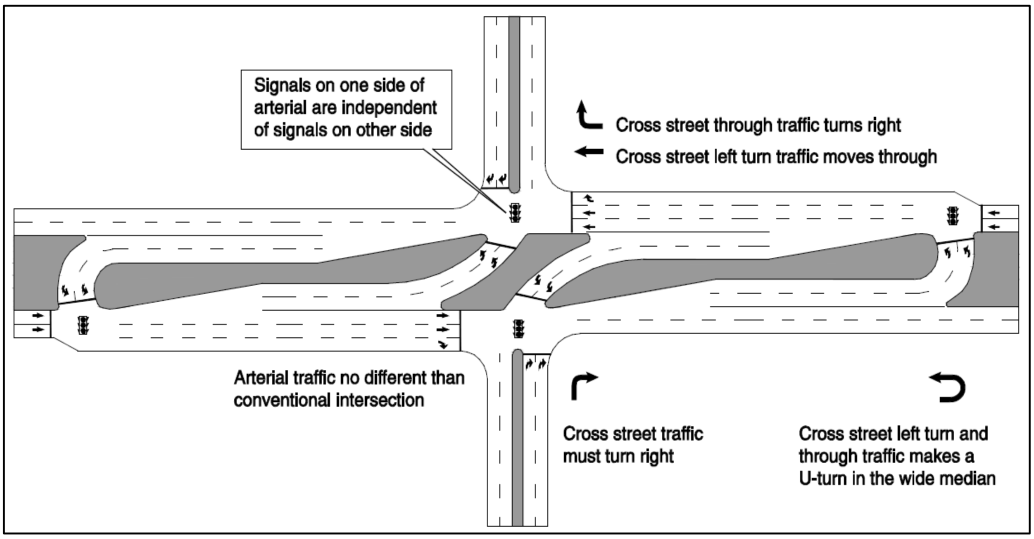

Figure 5.2 The Supestreet intersection geometric layout (FHWA, 2014).

deployment of this design, most of the initiative works have been implemented under the ideal traffic operations where less complex conditions. Recently, many RCUT intersections have been constructed in Maryland and North Carolina, USA (Hummer and Jagannathan 2008, Hughes et al. 2010, Haley et al. 2011, El Esawey and Sayed 2013). Therefore, the purpose of this research is to provide insight on the potential with an objective assessment of the possibility for implementing RCUT design under the heterogeneous traffic complexities as the uniqueness of this research. Therefore, in this chapter, it is essential first to highlight the operational mechanism including the vehicular movement, as well as the unique geometric layout. Second, it is also needed to review the previous research main findings before initiating the representation under such heterogeneous conditions. In addition, this chapter explains the coordination approach for a series of RCUT intersections.

5.2 Restricted Crossing U-Turn (RCUT) Intersection Mechanism

Generally, in the conventional intersections, the traveling flow along the arterial highway is allowed to operate left or turn right, while RCUT differs in the way in which vehicles entering or crossing the main highway from the minor road approaches. As one of UAIDs, RCUT basic design is built based on re-rotate movements away from the main intersection. Hence, the RCUT redirects the left-turn and through movements from side streets. In order to accomplish the RCUT unique design concept that allows the arterials' movements to operate independently, SSM is channelized through the major street median at specific locations as shown in Figure 5.3. Due to channelizing islands at the intersections, the minor road traffic is obligatory enforced to operate turning right and joining the main highway through traffic. Shortly, the provided median U-Turn allows the minor road traffic to proceed in the opposite direction as an equivalent of an indirect left-turn. Consequently, the minor street flow will turn right and continue traveling along their original minor route as the equivalent of crossing the major approach at a conventional intersection. Similar to the other UAIDs concept design, the cycle length is decreased obviously by separating the two main directions on the main corridor on the main corridor into two-phase signal system instead of the typical four phases assigned for the conventional countermeasure. Likewise, another salient property of such design is facilitating the traffic signal controller synchronization for the consecutive pairs of RCUTs as a result of the independent operation of each approach. Accordingly, the traffic flow is facilitated by emphasizing a the through and left-turn traffic of the major corridor within shorter cycles.

On the other hand, the safety is substantially improved by reducing the total number of conflict points by eliminating two of the highest risk movements from the main intersection and replacing them

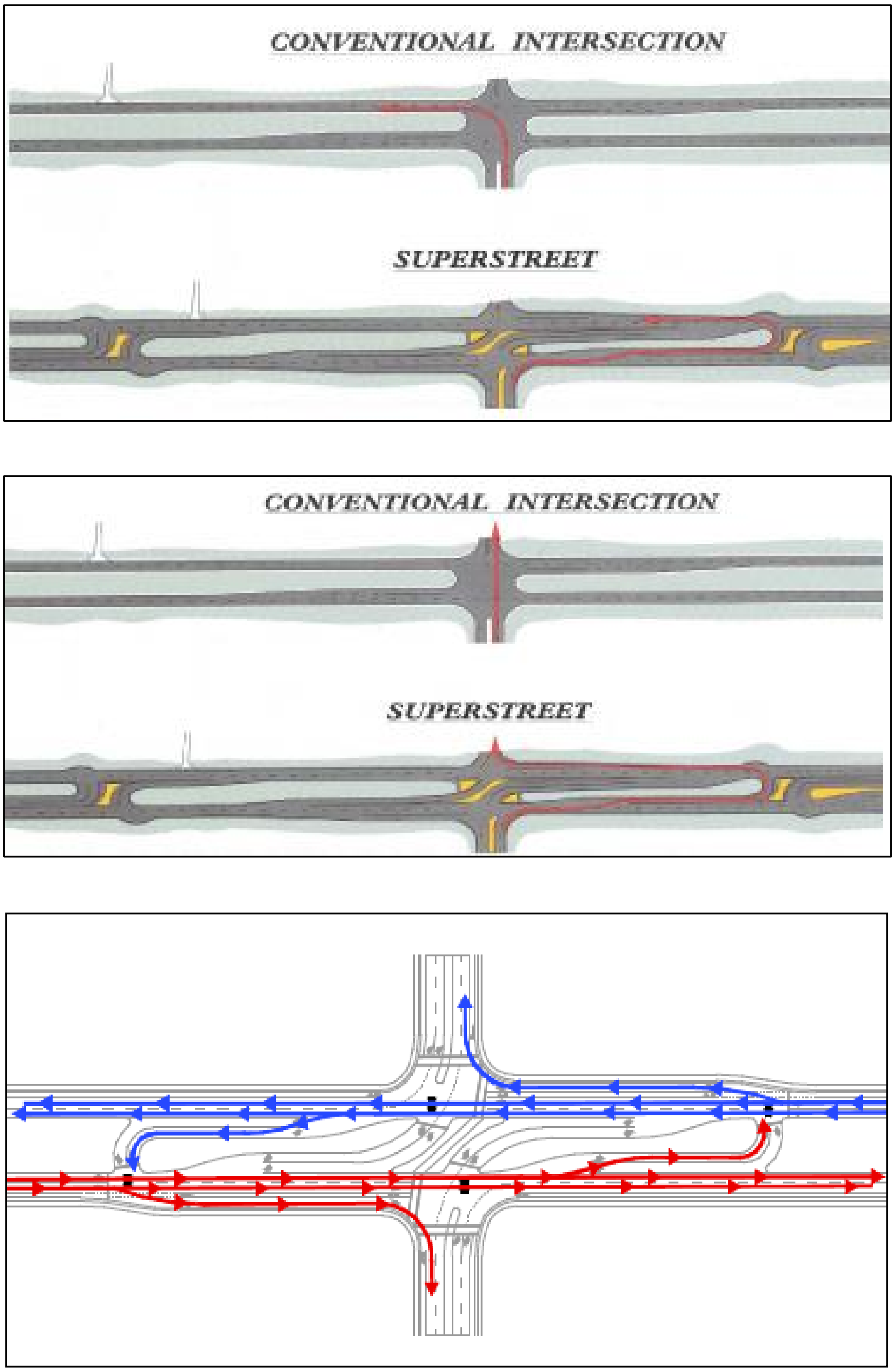

Figure 5.3 The Supestreet intersection vehicular movement (FHWA, 2014).

with indirect left-turns as U-turns followed by right turns. Although the improvement in both operational and safety conditions, one of the trade-offs with RCUT intersections is that vehicles entering from the minor road travel slightly longer distances as compared to a conventional intersection. Moreover, as another disadvantage of the RCUT design is the combined median and land width needed to accommodate large vehicles to make U-turns at crossovers may not be enough for a complete maneuver (FHWA, Restricted Crossing U-turn Informational Guide 2014). However, the average travel times may significantly be improved when RCUT are implemented along with a signalized corridor, hence the name super street.

5.3 Previous Findings

Seeking a meaningful and concrete background of the RCUT design, the general guidelines and the previous efforts relevant to this study are reviewed and discussed in this section. In order to investigate the operational performance of the RCUT as an applicable scheme under such heterogeneous traffic complexities, lots of considerable and valuable research works relevant are highlighted to identify the basic principles of the analysis of RCUT. Consistently, the previous works revealed that the RCUT design showed evidence of decreasing delay time and improving queue length when compared to the conventional design (Kim, Edara, and Bared 2007; El Esawey and Sayed, 2013; H. H. Naghawi and Idewu 2014). Despite the much efforts and attention that paid regarding the technologies of the different UAIDs, the conventional optimization procedure is still based on iterative trial-and-error approach (El Esawey and Sayed, 2013). Although many researchers have investigated the RCUT design, they basically used hypothetical data that represent different congestion levels. However, the other prevailing conditions that might exist at the intersection were ignored (H. H. Naghawi and Idewu 2018). Kim et al. used VISSIM to compare the operational and safety performance of the RCUT design to the conventional designs under three different scenarios. The three scenarios considered three geometric layouts on major approach as one left-lane and two-through lanes, one left-lane and three-through lanes and two left-lanes and three-through lanes. The authors reported that the RCUT design performed similarly the MUT design but has some additional features due to the additional properties that allow the through traffic progression on the major road in both directions by preventing the minor road traffic from crossing the major road (Kim, Edara, and Bared 2007).

To illustrate the performance of the RCUT scheme, most of the previous attempts focused on comparing the RCUT to either conventional intersections or other UAID designs based on the microsimulation platform. Hummer and Jagannathan also used VISSIM to compare the operational performance of five geometric design cases. The authors concluded that the RCUT design experienced shorter travel time than the conventional intersection under low minor road volumes compared to the total intersection volume less than 0.25 (Joseph E. Hummer and Jagannathan 2008). As a realistic case, significant improvements in traffic performance were found when the RCUT design was implemented in North Carolina, USA (FHWA, Superstreet Benefits and Capacities 2010). H. Naghawi and Idewu employed CORSIM to assess the operational efficiency of RCUT design with its conventional countermeasure. It was found that the network experienced a reasonable saving in the average delay from 27.39 to 82.26 percent, while the average network queue length was obviously decreased almost by 97.5 percent when the SSM was implemented (H. H. Naghawi and Idewu 2014). Meanwhile, Joseph E. Hummer et al. used the critical lane volume procedure to estimate the capacity of the RCUT intersection after readjusting the ideal saturation flow rate for trucks. An average saturation headway value of a 2.1 seconds was used to calculate the base critical lane capacity of 1587 pcphpl. The results emphasized that RCUT can handle major road through traffic volumes and left-turning volumes up to 2600 vph and up to 600 vph respectively, whereas it can handle volumes up to 900 vph and 400 vph for the minor road through and left-turning volumes respectively (Joseph E. Hummer et al. 2007). Similarly, the RCUT intersections in South Korea experienced fewer delays and performed better than other traditionally operated corridors during peak hour volumes (Moon et al. 2011).

Comparing to the MUT design, Kivlins et al. indicated that both MUT and RCUT designs are more convenient for the traffic conditions with a considerable traffic intensity on the arterial street and relatively small traffic intensity on the intersected minor street. Both proposed UAIDs ensured acceptable levels of service for all traffic flows and provided lager total intersection capacity than the conventional intersections. The authors concluded that MUT design is suitable for the conditions of approximately equal amounts of left-turning vehicles on the arterial, while the RCUT design is more preferable in such cases when larger amounts of left-turning vehicles on the arterial than on the minor approach (Reinis Kivlins et al., 2011). After reviewing five UAID designs, Hummer and Reid summarized that the RCUT design is recommended to be considered under any left-turn volume level on the major approach. The through and left-turn traffic volume on the minor street, however, is recommended to be low or moderate levels (Hummer and Reid 1998b). Based on the outcomes of CORISM for a typical suburban arterial corridor, Jonthan et al. revealed that MUT and RCUT intersections have the potential to improve the travel time and the average speed compared to the Two-Way Left-Turn Lane (TWLTL) design. Although the TWLTL had consistently fewer stops per vehicle than the MUT and RCUT design which require a longer path with more stop opportunities, the MUT and RCUT schemes significantly reduced the travel time and increased the average speeds during the morning and afternoon peak house. However, during the off-peak hours all the analyzed alternatives showed very similar results (Hummer and Reid 1999).

In the contrary, Naghawi et al. concluded that the RCUT simulated model did not perform as expected under heavy traffic volume in Amman, Jordan. Although the proposed design witnessed a reduction of 13.0 to 87.0 percent in the average delay and a reduction up to 97.0 percent in the maximum queue length compared to the existing conventional intersection while studying the feasibility and possibility for implementing the SSM on arterial roads and improved the LOS from F to C, the authors reported that the RCUT scheme is not appropriate for high traffic volumes. (H. Naghawi, AlSoud, and Alhadidi 2018). Likewise, the previous research indicated to some perceived disadvantages associated with the RCUT design such as the side streets higher travel time. Hummer et al. reported that the travel time savings for through movements could be mitigated by the higher travel times occurred in the side streets because of the high traffic volume (Hummer et al. 2007). The FHWA also referred to another drawback of this proposed scheme as the wide median needed to allow large vehicles to make completely U-turns and successful maneuvers (FHWA 2009).

As above-reviewed articles in which the vehicular movements are under less complex conditions and the other prevailing conditions and specific aspects of heterogeneous conditions were ignored. Therefore, the driving force in this chapter is to provide insight on the potential to implement the RCUT design under the heterogeneous traffic complexities as a significant traffic property in developing, semi-industrialized and industrializing countries around the world.

5.4 RCUTs Representation Under the Heterogeneous Conditions

Several aspects should be taken into account carefully in order to give reliable insight on the potential to implement the RCUT design as well as to obtain an accurate representation of such scheme under the different distinguishing parameters of such heterogeneous conditions. Relying on the RCUT design fundamentals and following the available guide manuals, the main findings of the previous research as well as the pre-deployments of such design in the real world, RCUT configuration is modeled as close as the reality in this study. Therefore, equitable settings are precisely considered for a fair comparative performance analysis between the proposed scheme and its conventional counterparts. The different simulation settings parameters that were calibrated and validated while the modeling of the existing conventional intersections as well as the DLT intersections including driving behaviour, vehicle properties, non-lane based system representation...etc., are kept as constant as for the existing intersection models as is illustrated in details in chapter 3 of this dissertation. The next section illustrates the modeling way of the geometric design and the signal control system is designed based on the real obtained data of such complex movements and other salient characteristics of the heterogeneous traffic.

5.4.1 Geometric Layout Representation

In this study context as mentioned earlier, a special attention is paid to the right of way requirements (i.e. same footprint/land area) regardless the number of lanes, especially, as a result of the evaluation of such alternative scheme in the urban networks where the land use issue is a considerable concern. Therefore, in order to conduct an equitable comparison to assess the efficiency of the proposed scheme, the same geometric design elements, particularly, the total width of each approach are kept fixed in spite of the differences in geometric modifications. For the major approach of the three analyzed intersections AT, AA and ME, the different RCUT geometric configurations are designed and assigned for each studied intersection based on the traffic data obtained from the real-world field observation. Based on the traffic volumes, the directional ratios and the saturation flow rate at each intersection, the signal controller system at each intersection is designed, and the geometric layout is modified accordingly. The geometric design along the major approach at the proposed RCUT intersections is assigned as three-lanes of 3.5m width are allocated for the through-traffic east and westbound flows at the primary intersection, while a pair of lanes of 3.5m width are employed for the left-turn movements at AT, AA and ME intersections as shown in Figure 5.4, Figure 5.5 and Figure 5.6. However, the lanes allocated for the U-turns are different for the analyzed intersections due to the different directional ratio of the through and left-turn movements at each intersection. Hence a couple of lanes are allocated for the U-turn crossover lanes at AT, AA and ME intersections. However, three left-turns lanes are assigned for the U-turns west of AT and AA intersection due to the high traffic demand performing U-turn movements at these specific U-turn crossovers as shown in Figure 5.4, Figure 5.5. The right channeled lanes of the east and westbound are kept as same as the existing conventional intersections for the right-turn movements along the arterial. On the other hand, the assigned exclusively bus lanes of 3.5m width for the public buses are kept as it is provided for the existing intersections to ensure an unbiased comparison and also to avoid the disturbance may occur to the public bus services along the studied corridor. For the minor approach, the same geometric layout is assigned for the all proposed designs as the existing conventional intersections without any kind of change.

Due to the high traffic demand at the U-turn crossovers, with a saturation flow rate of 1800 veh/h/ln and a midblock of three lanes per approach, the U-turn signal group failed to accommodate the assigned traffic volumes of 3040 veh/h and 3578 veh/h of AT and AA intersections respectively. As a result, and in order to keep the same road cross-section with the same right of way, the SSM proposed midblock geometric was changed as four lanes for the eastbound approach and two lanes for the westbound as shown in Figure 5.4, Figure 5.5.

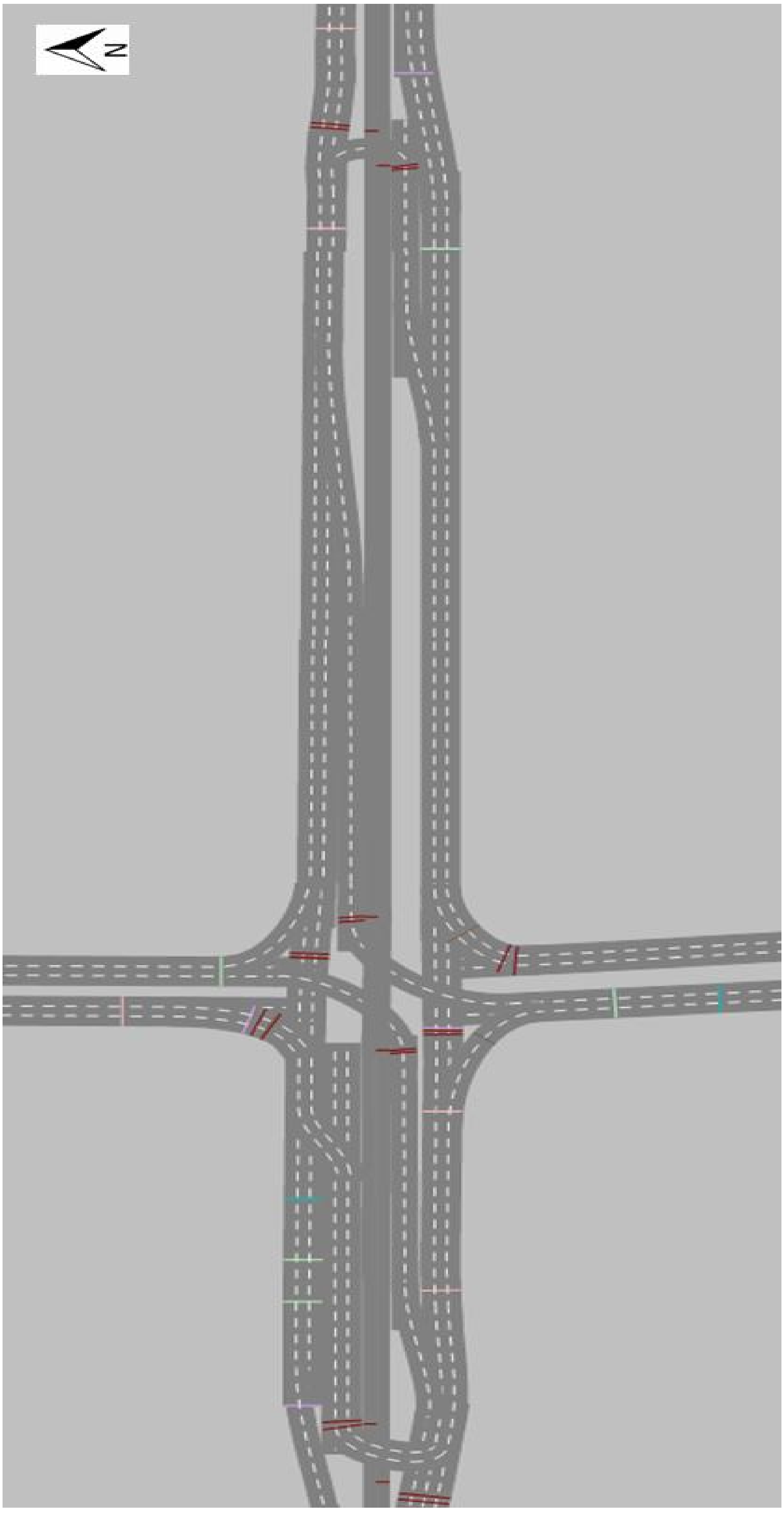

Figure 5.4 A typical geometric layout of AT-RCUT intersection

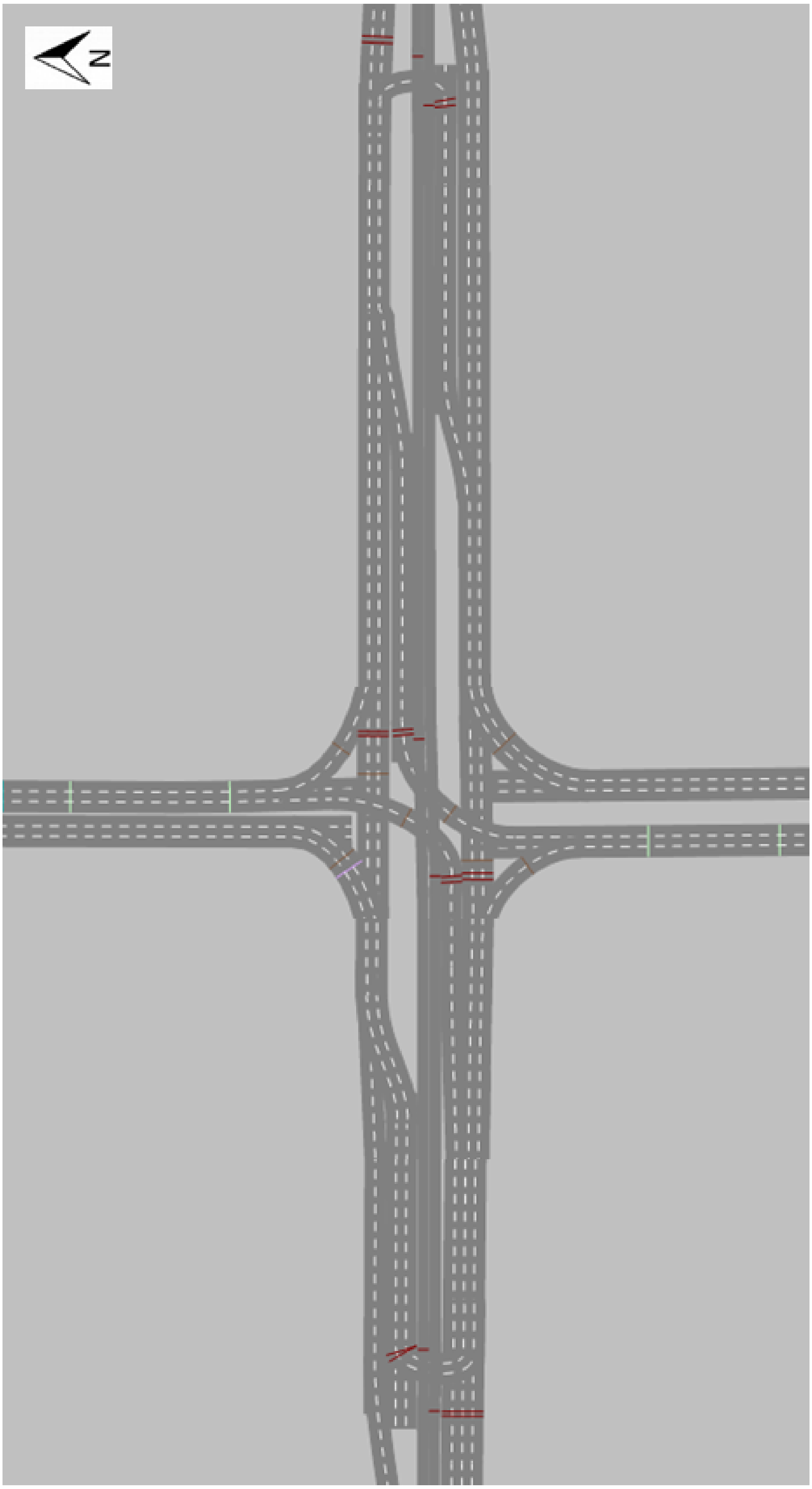

Figure 5.5 A typical geometric layout of AA-RCUT intersection

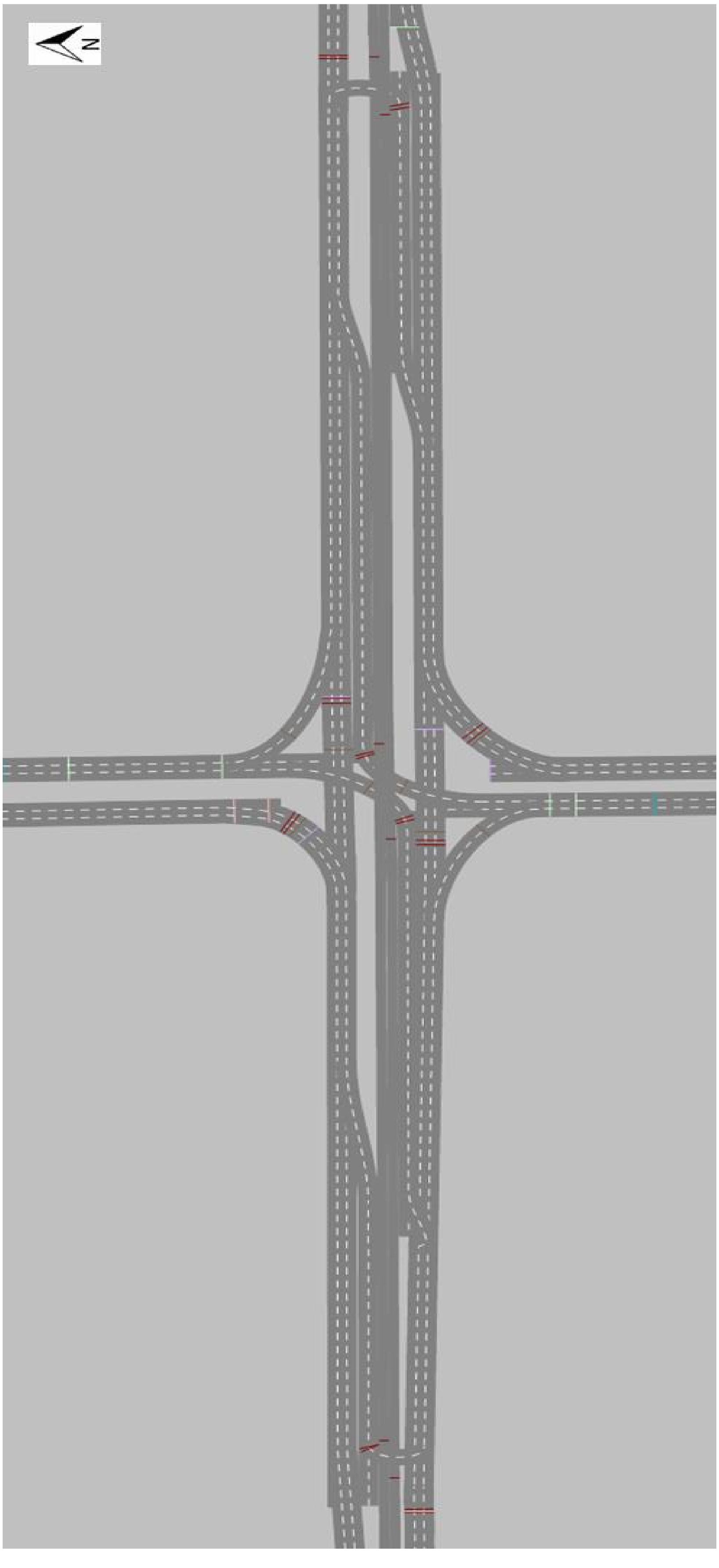

Figure 5.6 A typical geometric layout of ME-RCUT intersection

5.4.2 Signal timing plans calculation

Based on the traffic volume levels at an intersection, basically, the RCUT design can be controlled either by signal controller system as shown in Figure 5.7, or by stop signs as shown in Figure 5.8. This research, however, considers a signal control system to control the proposed RCUT design to control such heterogeneous traffic complexities. Moreover, depends on it volume levels, the minor approach through-movement and left-turn flow may also be controlled either by signal controller or may merge directly with the major approach stream without stop signal. However, in this study context, the minor approaches for all analyzed intersections are controlled by signal controller system due to the high traffic volume for these approaches as illustrated in Table 5.1 as well as to full control such traffic conditions, principally, the aggressive driving behaviour.

Table 5.1 Traffic demand of each direction at each intersection

Intersection	Directional flow	Observed volumes (veh/h)
AT	W-E	2270
	E-W	3040
	S-N	1640
	N-S	1434
AA	W-E	1905
	E-W	3578
	S-N	1596
	N-S	1716
ME	W-E	2245
	E-W	2574
	S-N	1385
	N-S	1244

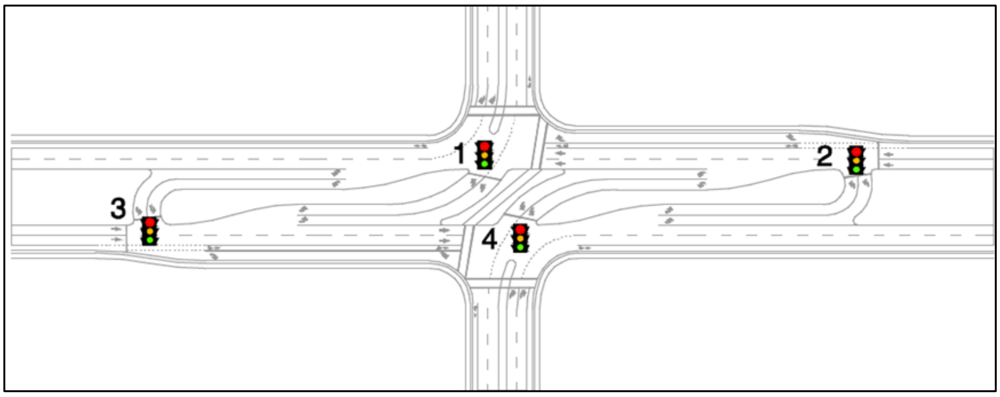

Figure 5.7 A typical RCUT intersection under the signal-control system (FHWA, 2014).

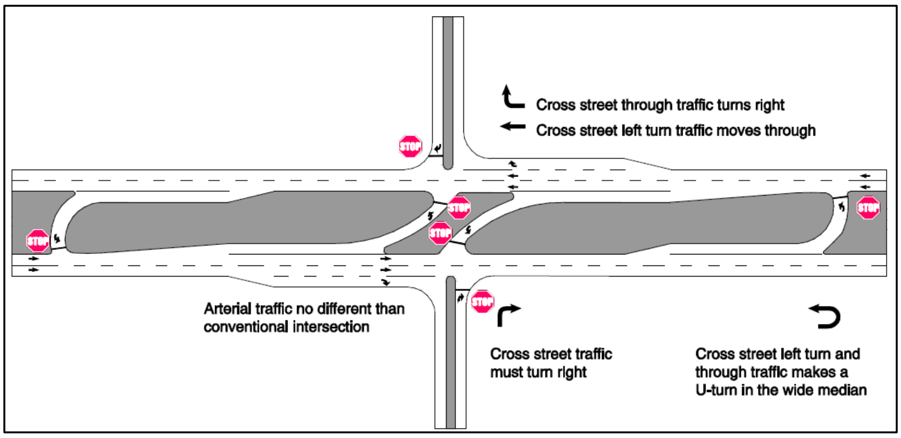

Figure 5.8 A typical RCUT intersection with stop-control signs (FHWA, 2014).

Due to the additional break that is added to allow independent operations for opposite intersections on the arterial streets as Figure 5.2 depicts, four signal controller sets with a two-signal phase scheme are needed to control the primary intersection and the U-turn crossovers as shown in Figure 5.7. The first and the second sets are assigned to control the primary intersection, while the third and the fourth set control the upstream U-turns. At the primary intersection, the first set is designed to control the westbound through-movement, the northbound minor street flow and the eastbound left-turns flow, while the second set is designed to control the eastbound through-movement, the southbound minor street flow and the westbound left-turns flow. Similarly, at the U-turn crossovers, the third controller is allocated to control the westbound through-movement upstream the U-turns and the eastbound U-turning movements, while the fourth controllers control the eastbound through-movement upstream the U-turns and the westbound U-turning movements.

Similarly, as the DLTs proposed intersections' signal timing plans in the previous chapter, the RCUT signal control system is designed based on Webster model as a theoretical calculation method. The critical flow of each controlled stream at each signal ontrol set inside the intersection is employed to determine the split time and the cycle length of each signal control set. Relying on the RCUT unique design concept, the signal control system is designed to achieve the RCUT novelty that allows the arterials' movements independently operation. Hence, each signal control set is designed separately by considering its own signal group critical flow individually. As a result, four signal controller sets with four different cycle lengths are defined at each intersection as Figure 5.9 depicts. The phasing timing plans and sequencing of proposed RCUT designs at AT, AA and ME intersection are exhibited in Figure 5.10 where RCUT (SC1) and RCUT (SC2) are assigned to control the eastbound and the westbound respectively at the primary intersection, while the RCUT (SC3) and RCUT (SC4) are assigned to control the east and the west U-turn crossovers respectively.

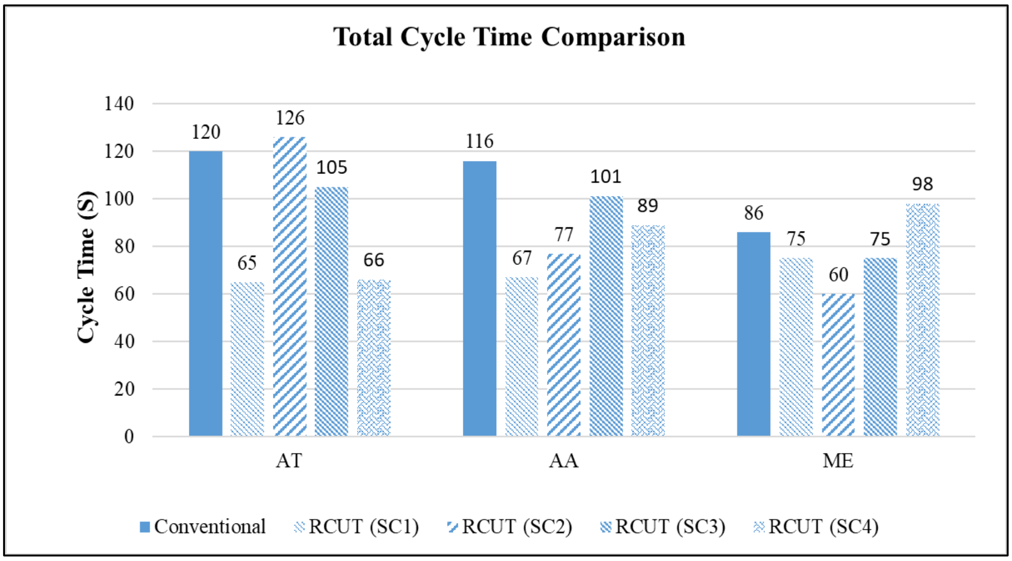

Figure 5.9 The total cycle length comparison between RCUT design and the conventional intersections

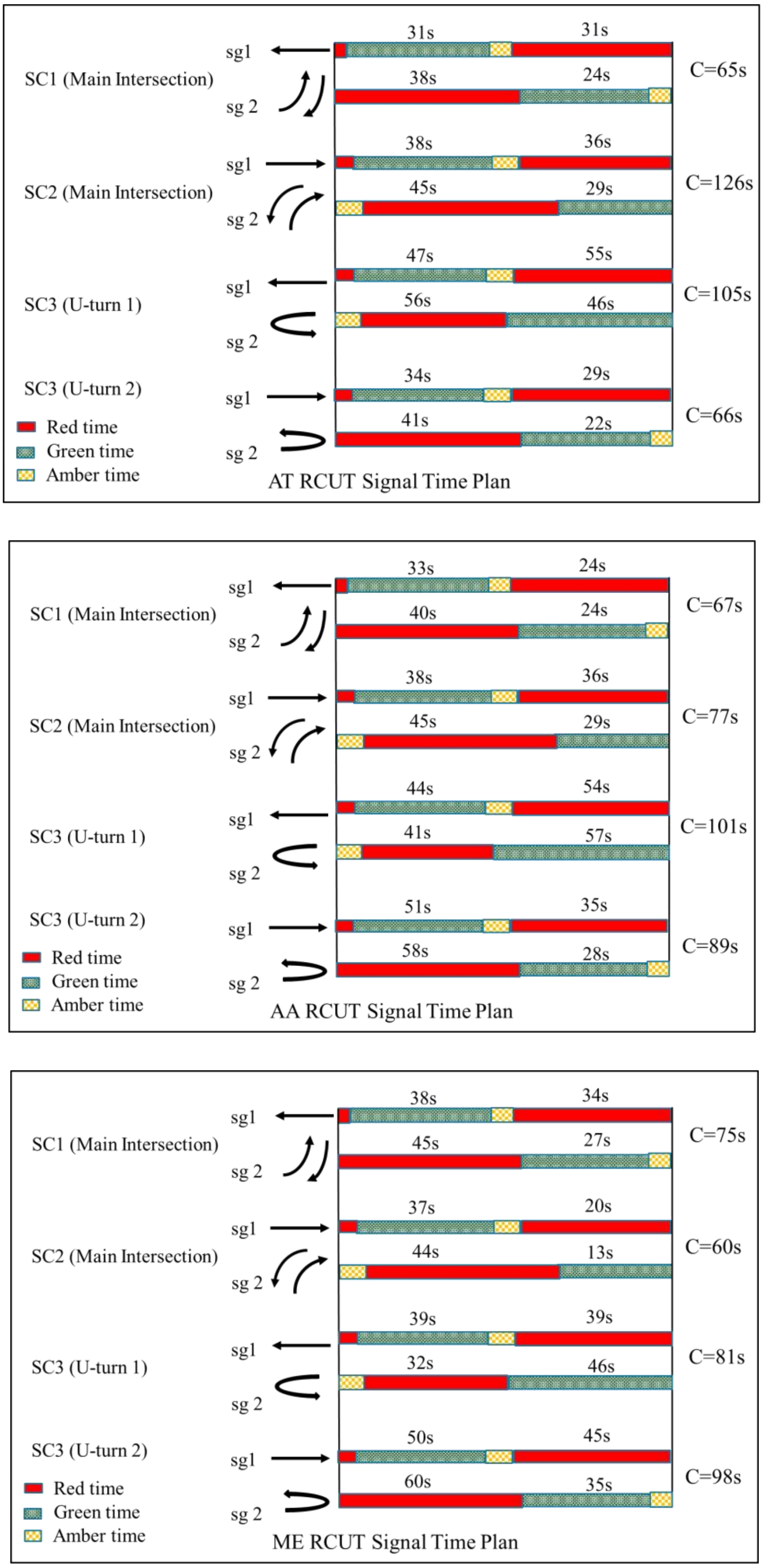

Figure 5.10 The signal-timing plan of the proposed RCUT intersections

For an efficient operation of the presented RCUT design at each intersection, a special attention is paid to coordinate the different signal groups that control the primary intersections and the U-turns crossovers. The four provided signals phases offsets are efficiently coordinated, to achieve a smooth progression along the studied corridor and in order to fulfil the RCUT concept design by ensuring a continuous flow. Taking into the consideration the previous research main finding, the offsets' values are calculated to be equal to the travel time between the primary and the U-turns crossovers (M. Esawey and Sayed 2007).

5.5 Simulation Results and Discussion

Seeking a credible and reliable evaluation, the simulation results are obtained and analyzed. Several evaluation indicators such as the total travel time, the average delay, the queue lengths, the average speeds, and the average number of stops were estimated for the three different studied intersections. The simulation outcomes revealed that RCUT proposed designs at AT, AA and ME intersections overwhelmed the operational performance of the existing conventional intersections as shown in Figure 5.11, Figure 5.12 and Figure 5.13. Due to the independent operation of each direction along the corridor, an undoubted improvement for all performance evaluation indices is exhibited, especially, under the heavy traffic volume levels when the conventional intersections failed to accommodate the traffic demand as mentioned in chapter 3 of this dissertation. The results showed an obvious enhancement in intersections throughputs, the average total delay time per vehicle, the average speeds and the average stopped delay per vehicle as shown in Table 5.2. As it is illustrated, the presented RCUT designs witnessed a reduction in the total travel time by -25.2 %, -79.0% and -13.42% for AT, AA and ME respectively comparing to their conventional counterparts. However, the RCUT proposal experienced longer travel time as a result of the indirect left-turning, especially, for the northbound approach. Meanwhile, due to the short storage length of the western U-turn crossover, as well as its high traffic demand of the southbound approach at AT intersection, RCUT design did not

Table 5.2 The simulation indices comparison

Parameter	(AT) intersection		(AA) intersection		(ME) intersection	
	Conv.	RCUT	Conv.	RCUT	Conv.	RCUT
Total travel time (h)	1107	628	4075	856	708	613
Avg. delay/veh. (s)	333.13	368	786.67	494	199	160
Avg. speed (km/h)	13.58	13.77	2.22	20.5	19.94	22.14
Avg. No. of stops/veh.	16.17	10.88	61.82	3.3	6.05	2.08
Avg. stopped delay/veh. (s)	141.09	49.27	1967.03	44.2	43.07	25.44

prove an improvement in the west flow along the major approach. Similarly, the and eastbound flow witnessed longer delay and travel time as a result of its high traffic demand. In the contrary, only the southbound approach experienced a significant improvement as a result of the shorter cycle length at the main intersections comparing to the conventional intersection as shown in Figure 5.11.

As an indication of the effectiveness of the studied scheme, the travel time along the major and the minor approaches is used to show the flow progression. The results indicated a reduction in the travel time for the major approach in its both directions west and eastbound under the proposed design is shown in Figure 5.10. Although the proposed scheme overwhelms the conventional intersection by providing shorter travel times for the different approaches, the minor streets flow did not experience the same improvement as the major approach did. Due to the high traffic demand of the minor streets, the southbound travel time was reduced, however, the enhancement was in a few rates. On the contrary, the northbound travel time experienced an adverse impact under the RCUT design due to the indirect left-turning movement restriction at the provided U-turns as shown in Figure 5.3, comparing to the direct movement provided in the existing conventional intersections. The minor approach northbound through and left-turns flow have to use the U-turns instead of a direct movement in case of the existing conventional and the DLT intersections. Therefore, the northbound through and left-turn movements experienced longer travel time and delay at all studied intersections comparing to the existing conventional intersections as shown in Figure 5.11, Figure 5.12 and Figure 5.13. Also, the average stopped delay per vehicle and the average total delay time per vehicle exhibited considerable savings as shown in Table 5.2. The average stopped delay per vehicle improved by 65.03%, 97.76% and 40.93% for AT and ME respectively.

In the contrary, the RCUT design did not prove a significant improvement in the average speeds inside the analyzed intersections comparing to the conventional counterparts as a result of the relatively long total cycle length as Figure 5.9 depicts. Comparing to their conventional counterparts, the enhancement rates of the average speeds recorded 1.4%, 11.0% for AT and ME respectively, while it experienced an obvious enhancement at AA intersection.

On the other hand, the heterogeneous traffic complexities such as the diverse dynamic as well as the aggressive driving behaviour influenced the operational performance of the proposed RCUT scheme. The non-lane based driving system and the diverse dynamic properties of the different vehicles impacted the queues discharging rates, particularly, at U-turn crossovers.

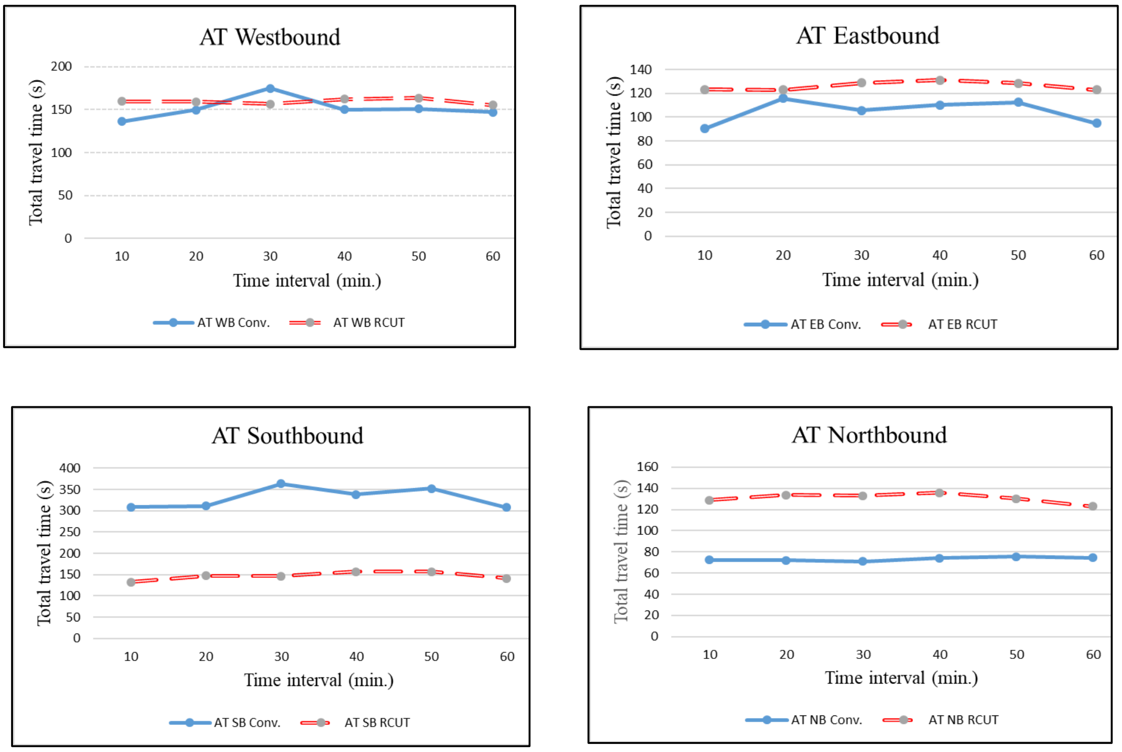

Figure 5.11 Total travel time of AT conventional intersection vs. RCUT design

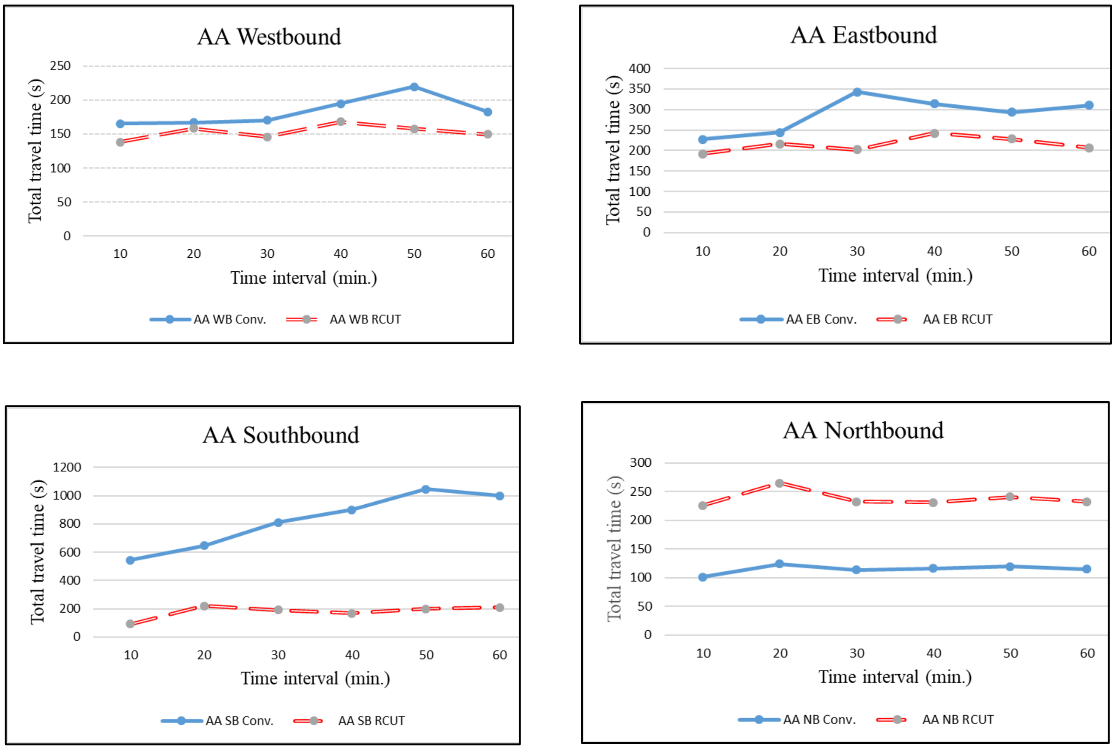

Figure 5.12 Total travel time of AA conventional intersection vs. RCUT design

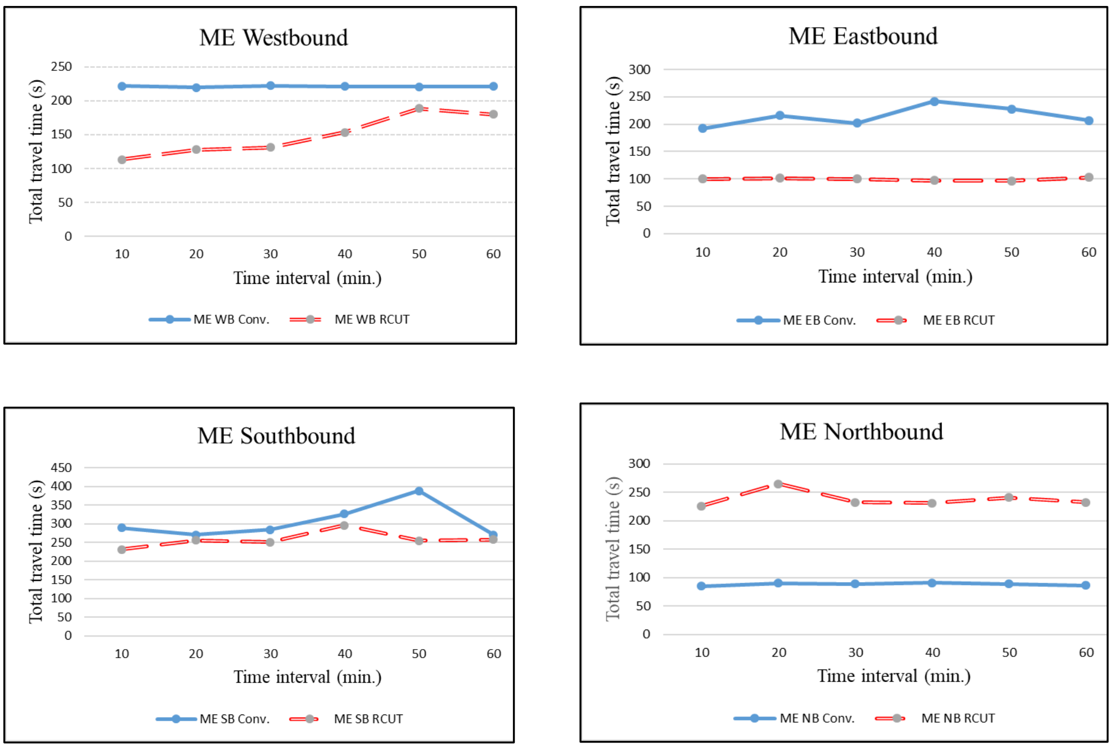

Figure 5.13 Total travel time of AT conventional intersection vs. RCUT design

The heterogeneous conditions considered in this study (i.e the diverse dynamic and static properties of mixed traffic compositions, the non-lane based system, the aggressive driving behaviour…etc) obviously influenced the operational performance improvement rates of the proposed RCUT designs. Hence, the improvement rates in this research are lower than their counterparts where the low complex traffic conditions were considered comparing to main findings in the previous studies.

Based on the reviewed results, it can be concluded that the RCUT is not appropriate for high traffic volumes, practically, under the heterogeneous traffic conditions. However, when the matter of signal coordination is considered, the RCUT proposal still can be considered.

5.6 Conclusion

This chapter presented the RCUT as one of the innovative alternative intersection design under the heterogeneous traffic complexities. Owing to explore the functional operations of such design as a real-world practical application, the proposed RCUT proposals in this context was examined and applied for three realistic entire signalized intersections as a credible case of study. Based on the simulation outputs, the different indices emphasized significant improvement for, particularly, the major corridor movements under the heavy traffic volume levels as an indication of the RCUTs outperformance over their conventional counterparts. The intersections throughputs, the average total delay time per vehicle, and the average stopped delay per vehicle experienced an obvious enhancement. However, due to the high demand at the minor approach as well as the indirect left-turn of their movements, the minor approach northbound experienced longer travel time comparing to the conventional intersections. Therefore, the average speeds did not exhibit a significant enhancement of the analyzed intersections under the proposed RCUT design. On the other hand, comparing to main findings in the previous studies where the low complex traffic conditions were considered, the improvement rates in this research are lower than their counterparts. Due to the heterogeneous conditions considered in this study (i.e. the diverse dynamic and static properties of mixed traffic compositions, the non-lane based system, the aggressive driving behaviour…etc.) were obviously influenced the operational performance improvement rates of the proposed RCUT designs.

In conclusion, the RCUT scheme revealed an improvement for all performance evaluation indices of the entire existing conventional intersections. However, the heterogeneous conditions could also influence the operational performance improvement rates of the proposed RCUT designs. Also, it can be concluded that the RCUT is not appropriate for high traffic volumes, practically, under the heterogeneous traffic conditions. However, when the matter of signal coordination is considered, the RCUT proposal still can be considered.

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CHAPTER 6

OPERATIONAL PERFORMANCE COMPARISON AMONG DLT, RCUT DESIGNS AND THEIR CONVENTIONAL COUNTERPARTS

6.1 Introduction

Considering the heterogeneous traffic complexities as the pillar of this study, a comparison of unconventional designs and typical existing conventional counterparts has been conducted to determine the types of intersections which are more preferable. The primary aim of this chapter is to compare the operational efficiency of existing conventional signalized intersections with the two proposed UAIDs schemes namely; the Displaced Left-Turn (DLT) intersection and Restricted U-turn Crossover (RCUT) intersection. This comparison focuses on comparing the operational performance of each proposed alternative scheme with their corresponding conventional counterpart. Seeking an equitable comparison, the same traffic demand, the same geometric layout and the other circumstances are fixed for all designs. Also, this chapter discusses the impacts of the considerable complexities of such heterogeneous traffic conditions on both alternative designs. The influences of such specific traffic conditions on the operation performance as well as the geometric designs or each alternative scheme are highlighted. Likewise, the signal control timing plans of DLT and RCUT intersection including the cycle length and the split time are compared with the existing time plans of the conventional intersections. The obtained simulation results of the three analyzed intersections as a realistic case study of such traffic conditions, which was discussed in the previous chapters are used to accomplish this comparison.

6.2 Signal Phasing and Timing Plans

Utilizing Webster's method (1966) as a mathematical approach, the signal-timing plans for both proposed UAIDs were designed in this present study. According to Webster's, the green time is optimized based on the flow ratio of each phase to achieve the optimal delay inside an intersection.

Based on the two-signal phase scheme design principle applied for the proposed UAIDs to facilitate through traffic movements by reducing the total cycle length time, the signal time plans were estimated for both designs. With a lost time of 5.0 seconds per each phase, the total cycle length and split times were estimated. However, due to the heterogeneous traffic complexities, the saturation flow rate of 1800 veh/h/ln was used to design the different control plans instead of 1900 veh/h/ln for the less complex traffic conditions. As mentioned previously, the proposed DLT design in this context is a full DLT design which includes crossovers to control both major and minor approach. Therefore, each intersection under such design consists of six signal sequences as shown in Figure. 6.1. On the other hand, the RCUT design requires four signal controller sets with a two-signal phase for each signal set to control the primary intersection and the U-turn crossovers as shown in Figure 6.2. For the DLT design, only one cycle is assigned for the all signal groups at an intersection and is estimated based on the critical flow ratios of the primary intersection. Consequently, the split time of each phase inside the primary intersections and the left-turn crossovers of major and minor approaches are determined based on the critical flow for each direction and the single pre-calculated cycle length as shown in Figure 6.1. However, in order to achieve the RCUT unique design concept that allows the arterials' movements independently operation, each signal control set is designed separately by considering its own signal group critical flow individually. Accordingly, four signal controller sets with four different cycle lengths are defined at each intersection as shown in Figure 6.2.

Regarding the cycle length, as a result of applying the two-phase system as the main design concept for UAIDs, both proposed schemes DLT and RCUT significantly provided shorter cycle time, particularly, inside the primary intersections than the existing conventional intersections. Meanwhile, the total cycle length at DLT design experienced a shorter time than the RCUT cycle times as Figure 6.3 depicts due to the directional ratios distributions. Hence, the critical traffic volumes that are used to estimate the cycle length at the U-turn crossovers for the RCUT design are bigger than the critical traffic volumes that assigned for the DLT intersections. Although the RCUT design provided a shorter

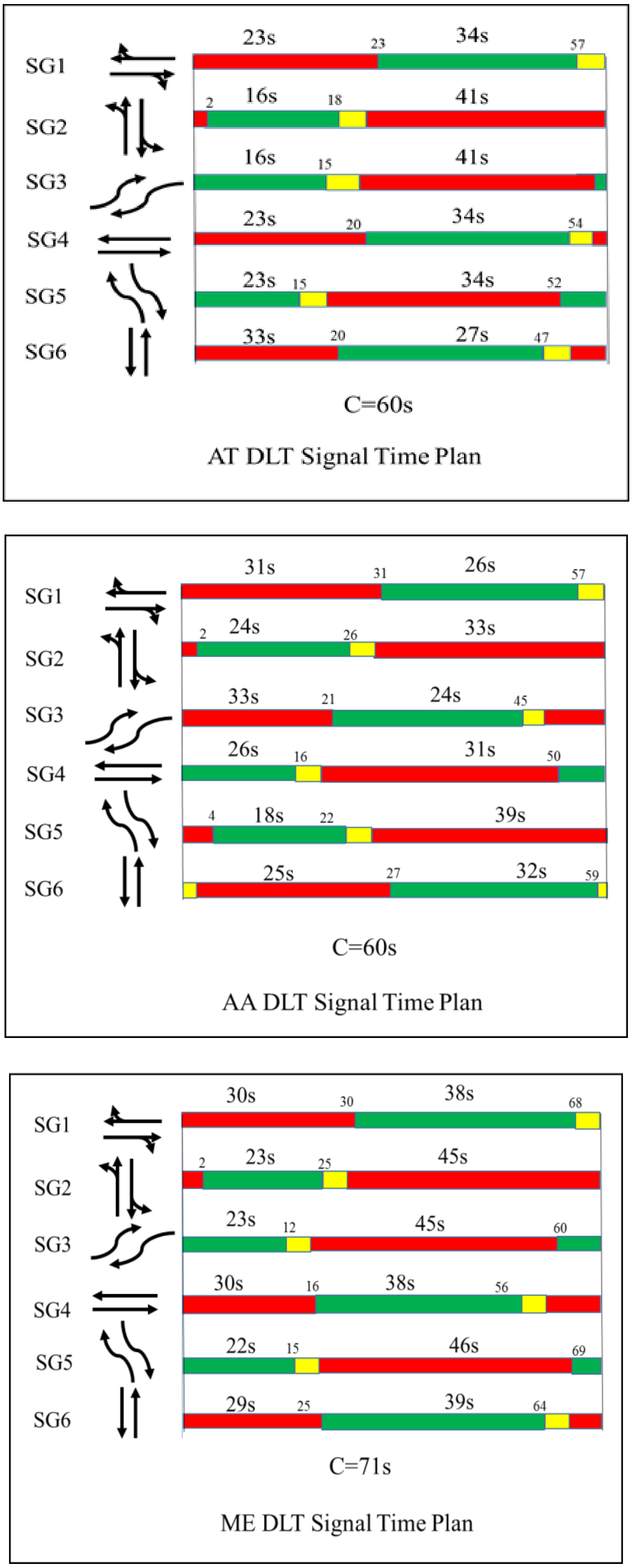

Figure 6.1 The signal-timing plan of the proposed DLT intersections

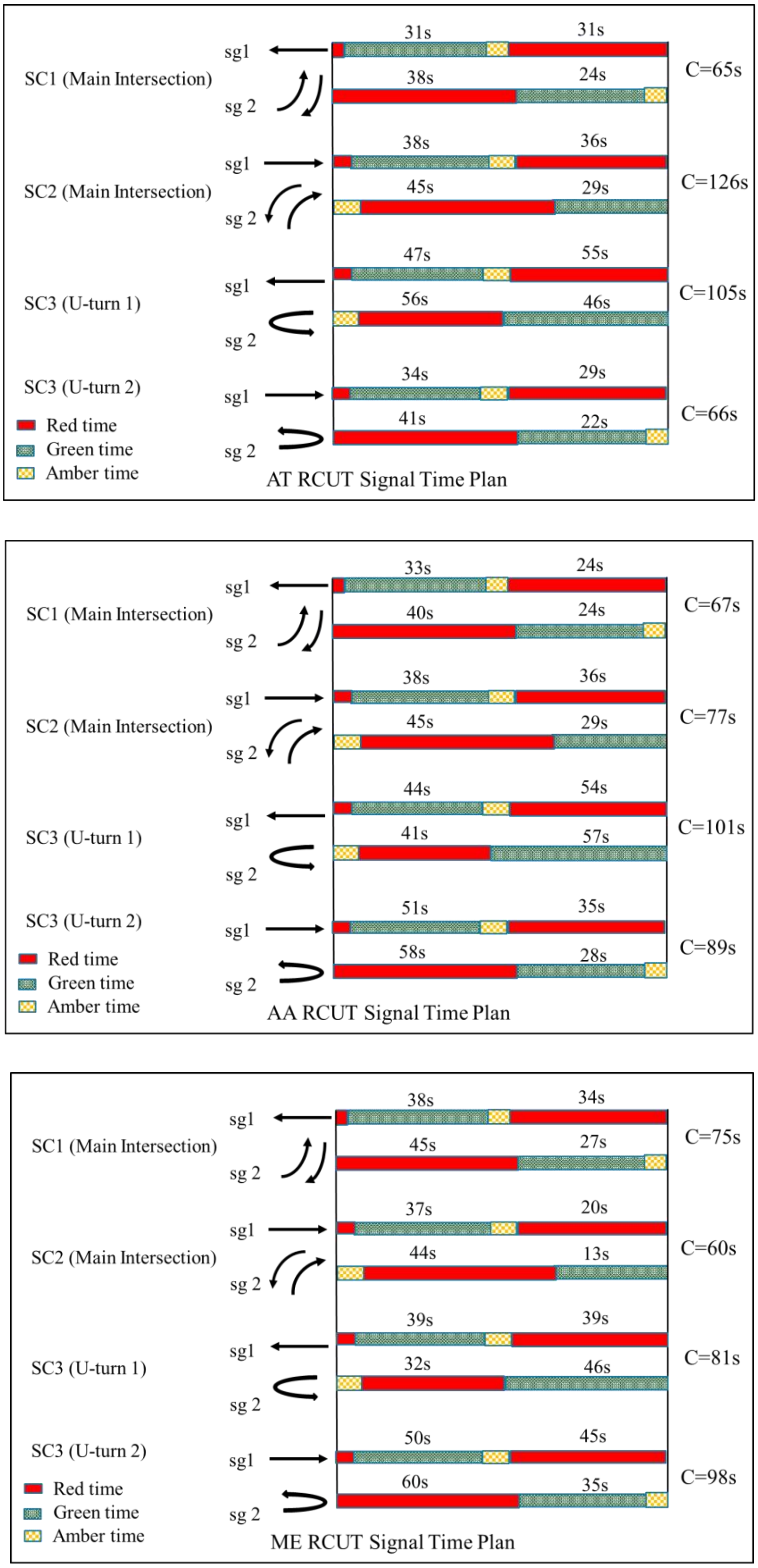

Figure 6.2 The signal-timing plan of the proposed RCUT intersections

cycle length at the primary intersections than the existing conventional intersection, the U-turn crossovers experienced longer time comparing to the primary intersections at the conventional intersections. The signal groups at the U-turns crossovers have to control the full approach high traffic demand before starting distributing for the different directions.

Owing an effective operational performance to ensure the continuous flow with no stopping for both DLT and RCUT scheme, a special attention is paid to achieve the needed coordination between the different signal groups at the primary intersection and either the left-turn crossovers in case of DLT design or the U-turns signal groups in case RCUT design. The offsets between the coordinated signal groups were carefully estimated as equal to the travel time between the primary and the secondary intersections (M. Esawey and Sayed 2007). Similarly, the offsets of DLT signal groups, the RCUT signal groups can also be coordinated such the offset is equal to the travel time between the primary and the U-turn crossovers.

6.3 Geometric Layouts

As a result of the evaluation of such alternative scheme in the urban networks where the land use issue and the right of way requirements (i.e. same footprint/land area) are a considerable concern, the same total width of the studied corridor was kept fixed for both proposed alternative designs regardless the number of lanes. Therefore, the geometric layouts of the DLT the RCUT designs were allocated to fulfil each intersection requirements. For the DLT scheme, different geometric configurations were configured and assigned for each studied intersection based on the directional traffic volumes obtained from the real-world field observation. Although the same geometric layout was assigned for the minor approach for all studied intersections, the geometric layout was different for the major approach in the studied intersections. For instance, at AT intersection, three lanes were employed for the through-traffic westbound flow, while only one lane was employed for the left-turn crossover movements west and eastbound movements. Meanwhile, three lanes were allocated for both AA west and east through traffic. Accordingly, the right channeled lane of the westbound as well as

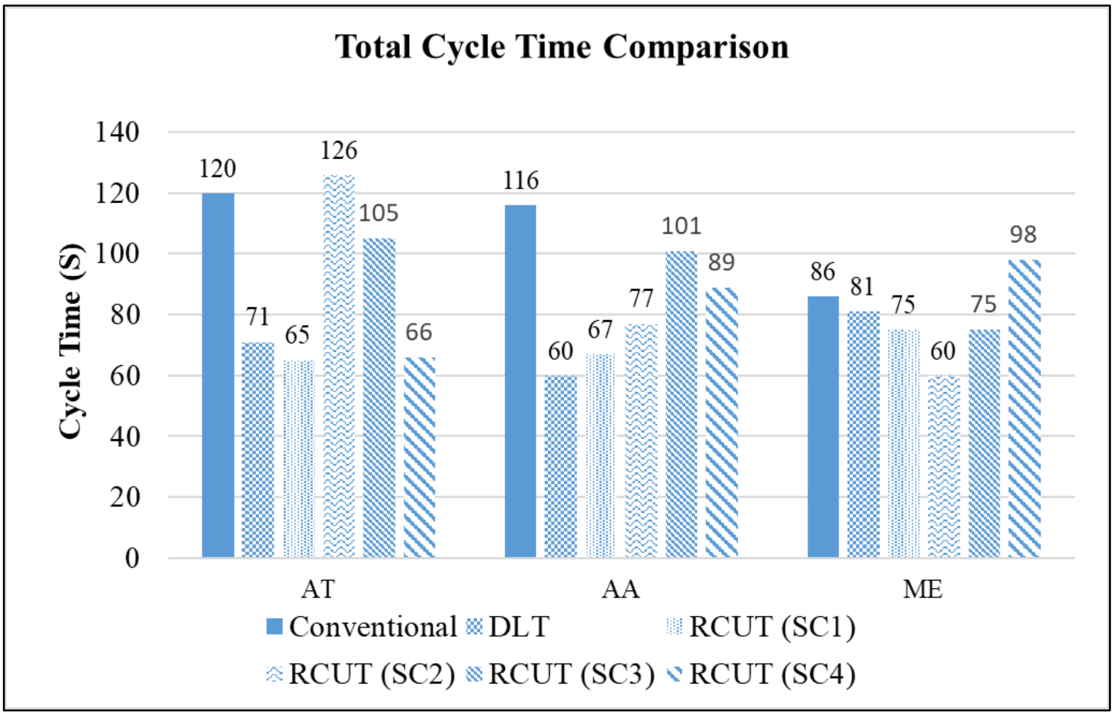

Figure 6.3 The total cycle length comparison between DLT, RCUT design and the conventional intersections

the southbound was eliminated and one lane was allocated for left-turn west and eastbound. For ME intersection, two exclusive lanes were allocated for each direction for through traffic flows, while the other two lanes were installed as crossover lanes.

Similarly, the RCUT geometric design along the major approach was assigned as three-lanes for the through-traffic east and westbound movements at the primary intersection, while a pair of lanes of 3.5m width is employed for the left-turn movements at AT, AA and ME intersections. Because the heterogeneous traffic complexities, particularly, the non-lane based system, the RCUT signal control was designed with a corresponding saturation flow rate of 1800 veh/h/ln and a midblock of three lanes per approach, the U-turn signal group failed to accommodate the assigned traffic volumes of 3040 veh/h and 3578 veh/h of AT and AA intersections respectively. As a result, and to keep the same road cross-section with the same right of way, the RCUT proposed midblock geometric was changed as four lanes for the eastbound approach and two lanes for the westbound.

According to the field observation and the simulated models, it can be concluded that the heterogeneous traffic complexities such as the diverse dynamic and the aggressive driving behaviour influenced the operational performance of the proposed UAIDs. The non-lane based driving system, as well as the queues discharging rates in both main intersections and left-turn crossovers in case of DLT designs and U-turns in case of RCUT designs, affected the intersections' LOS. Therefore, in order to fulfill such conditions, the geometric design elements (i.e. the spacing distance between the main intersections and major crossovers for DLTs and the U-turns for RCUTs) were adjusted and reallocated based on the traffic demand. In this context, for the proposed DLTs, the ME intersection was designed with two crossover left-turn lanes, whereas the AT and AA intersections with one crossover left-turn lane. Likewise, the spacing distance designed as 200.0 m, 105.0 m and for 85.0 m ME, AT and AA respectively, while the recommended distance is 91.4 m to 152.4 m (FHWA: Displaced Left Turn Intersection Informational Guide, 2014).

6.4 Results and Discussion

To evaluate the effectiveness of the proposed intersection treatment, and seeking a credible evaluation, simulation-based assessment results were obtained and analyzed for a number of simulation runs. For a comprehensive evaluation, a number of evaluation indices such as the total travel time, the average delay, the queue lengths, the average speeds, and the average number of stops were estimated for the three entire analyzed intersections. The simulation results highlighted the superiority of the DLT and RCUT designs proposed in this study. The outputs revealed that DLTs consistently reported better results and overcame over the existing conventional intersections as well as the RCUT design for the three intersections studied. As a measure of effectiveness for the studied intersection, the travel time along the major and the minor approaches was estimated. As it is illustrated in Table 6.1, the total travel time was dropped by -46.2 %, -79.4% and -24% for AT, AA and ME intersections respectively for DLT designs. Meanwhile, the RCUT witnessed a reduction in the total travel time -25.2%, -78.8% and -13.42% for AT, AA and ME intersections respectively as shown in

Table 6.1 The simulation indices comparison

Parameter	(AT) intersection			(AA) intersection			(ME) intersection		
	Conv.	DLT	RCUT	Conv.	DLT	RCUT	Conv.	DLT	RCUT
Total travel time (h)	1107	596	828	4075	839.67	856	708	537	613
Avg. delay/veh. (s)	333.13	124.63	368	786.67	168.52	494	199	132.13	160
Avg. speed (km/h)	13.58	24.71	13.77	2.22	21.13	20.5	19.94	23.56	22.14
Avg. No. of stops/veh.	16.17	1.45	10.88	61.82	2.64	3.3	6.05	1.73	2.08
Avg. stopped delay/veh. (s)	141.09	16.87	49.27	1967.03	28.62	44.2	43.07	24.34	25.44

Figure 6.4. Among the all analyzed intersections, the DLT design overwhelmed the entire conventional counterparts as well as the RCUT design. The travel time of the DLT design for the different approaches; west, east, south and northbound showed a clear improvement as shown in Figure 6.5, Figure 6.6 and in Figure 6.7. On the other hand, the results indicated a reduction for the travel time for the major approach in west and eastbound under the RCUT design. Also, the southbound travel time was saved, however, the enhancement was in a few rates.

Although the superiority of both UAIDs over the conventional intersections by providing shorter travel times for the different approaches, conversely, the RCUT proposal experienced longer travel time for the northbound approach as an adverse impact of the indirect left-turning as shown in Figure 6.5, Figure 6.6 and in Figure 6.7. The minor approach northbound through and left-turns flow have to use the U-turns instead of a direct movement in case of the existing conventional and the DLT intersections. Likewise, due to the short storage length of the western U-turn crossover, as well as its high traffic demand of the southbound approach at AT intersection, the RCUT design did not prove an improvement in the west flow along the major approach. Similarly, the and eastbound flow witnessed longer delay and travel time as a result of its high traffic demand. In the contrary, only the southbound approach experienced a significant improvement as a result of the shorter cycle length at the main intersections comparing to the conventional intersection as shown in Figure 6.5.

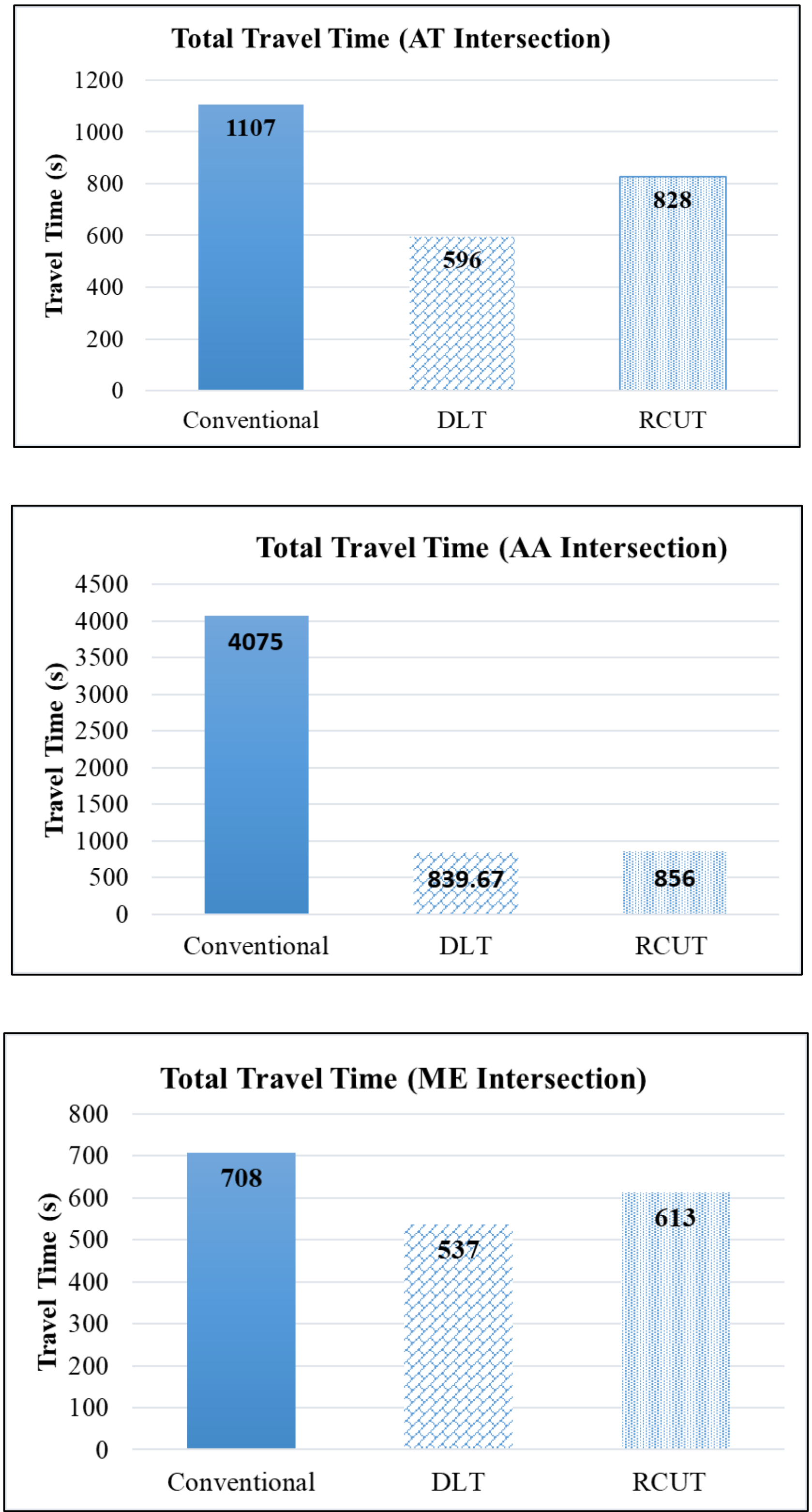

Figure 6.4 The total travel time comparison among DLT, RCUT design and the conventional intersections

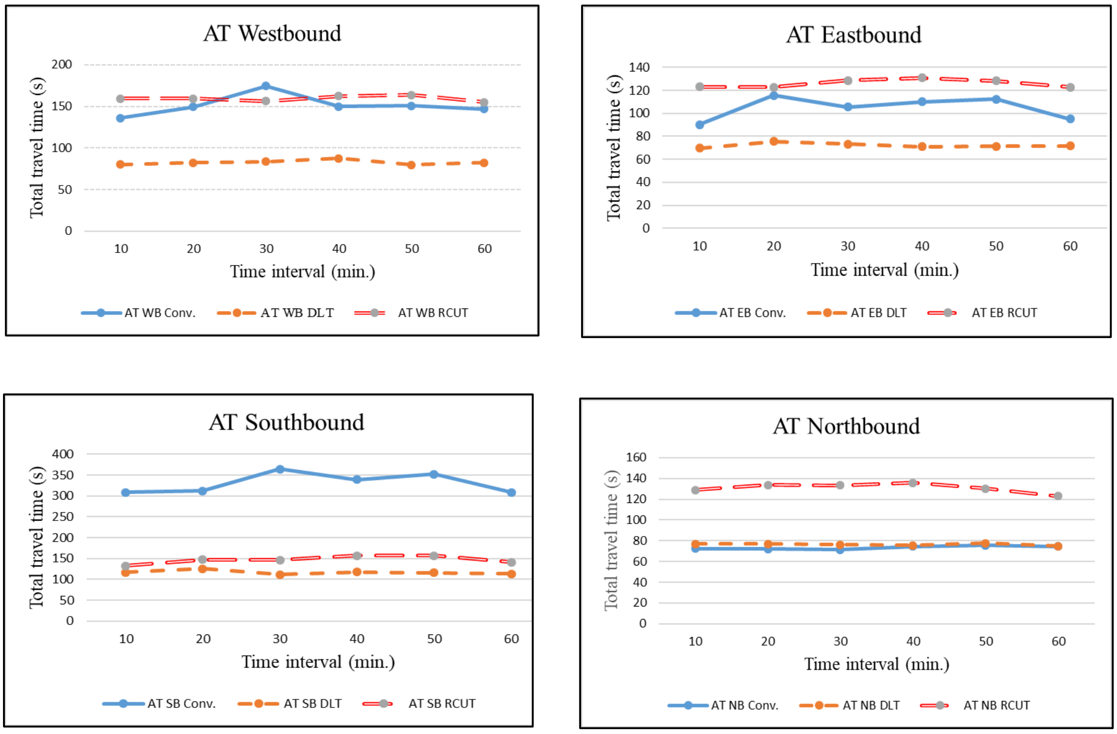

Figure 6.5 Total travel time of AA conventional intersection vs. RCUT design

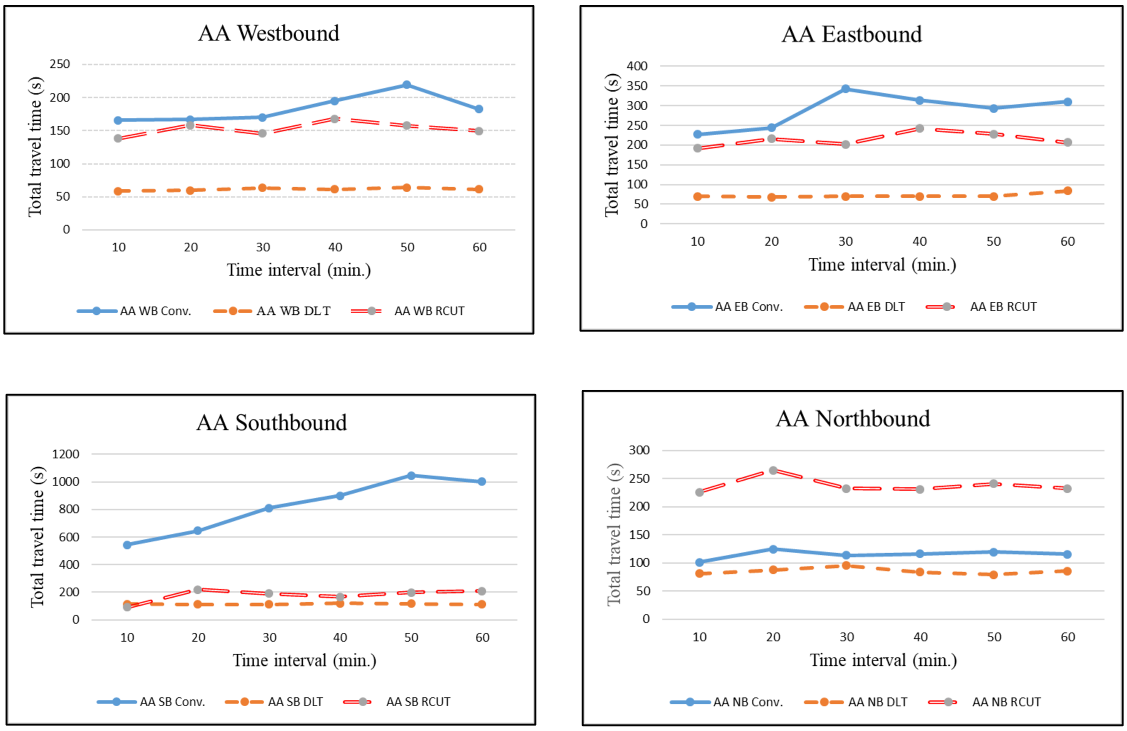

Figure 6.6 Total travel time of AA conventional intersection vs. RCUT design

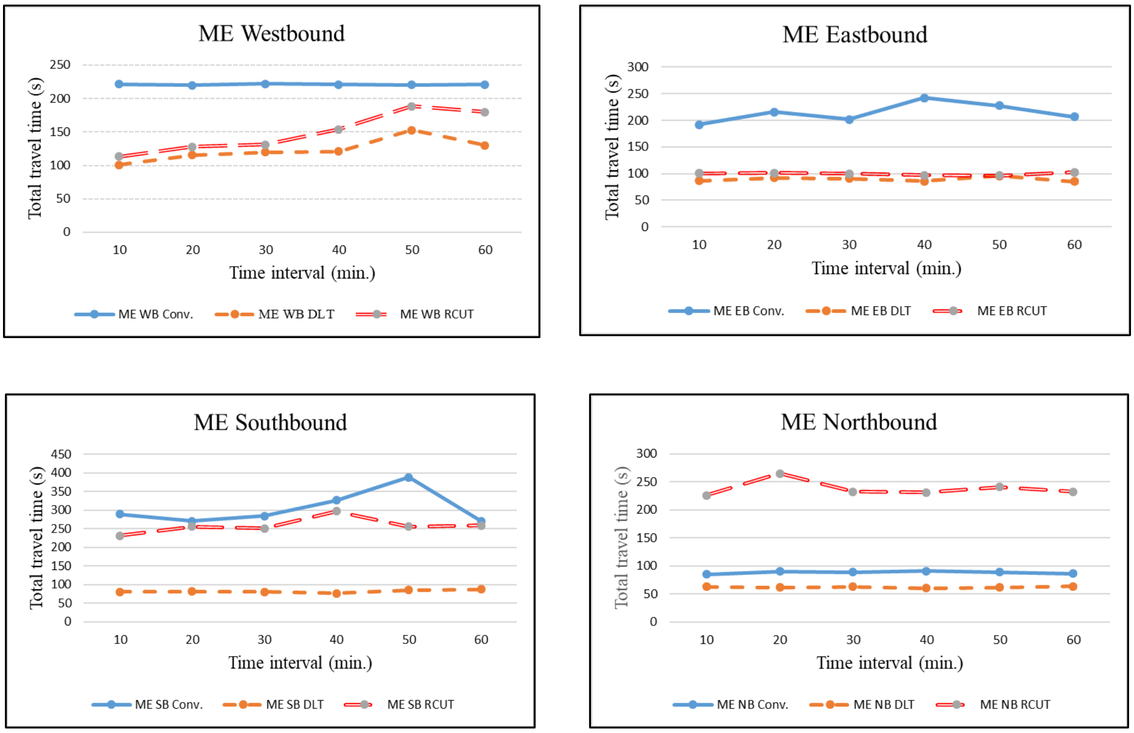

Figure 6.7 Total travel time of ME conventional intersection vs. RCUT design

Accordingly, the DLT the average delay for all approaches experienced shorter than the existing intersections and the RCUT design as Figure 6.5 depicts. The results referred that the average delay savings are 62.26%, 78.58% and 33.6% for AT, AA and ME intersections respectively, while the savings are 25.33%, 37.2% and 19.6% for AT, AA and ME intersections respectively.

On the other hand, the average speeds experienced an obvious improvement under DLT operation, however, the RCUT design did not prove a significant improvement in the average speeds inside the analyzed intersections comparing to the conventional counterparts as a result of the relatively long total cycle length as shown in Figure 6.3. Comparing to their conventional

counterparts, the enhancement rates of the average speeds recorded 1.4%, 11.0% for AT and ME respectively, while it experienced an obvious enhancement at AA intersection. Also, the average stopped delay per vehicle and the average total delay time per vehicle exhibited considerable savings as shown in Table 6.1. Although it was found that when using the RCUT design the overall indices were improved; the enhancement rate was not as significant as the DLT design because of the indirect left-turning restriction of the minor approach.

Based on the previous results, it can be concluded that the RCUT is not appropriate for high traffic volumes, practically, under the heterogeneous traffic conditions. However, when the matter of signal coordination is considered, the RCUT proposal still can be considered. The RCUT provides better coordination system than the DLT design.

6.5 Conclusion

This chapter investigated the evaluation of the two proposed UAIDs schemes DLT and RCUT intersection with an objective assessment of the possibility of implementing these alternatives under the heterogeneous traffic conditions. The simulation-based assessment output results are analyzed and evaluated to investigate the efficiency of each proposed scheme. The level of service (LOS) at each intersection is evaluated through different indices such as the travel time, the average delay, the average speeds and the average number of stops. The simulation results highlighted the superiority of the DLT and RCUT designs proposed in this study. The outputs revealed that DLTs consistently reported better results and overcame over the existing conventional intersections as well as the RCUT design for the three intersections studied. The results indicated to the obvious improvement of the LOS of the studied the intersections. It was found that the proposed UAIDs schemes reduced the overall delay and the total travel time while the average speed was increased significantly. Also, it was concluded that the heterogeneous traffic influenced the proposed UAIDs geometric design as well as the operational efficiency.

Based on the previous findings, the RCUT design is not appropriate for high traffic levels it can be concluded and emphasized as the previous research revealed that although the UAIDs improved the operational performance of the entire existing conventional intersections, they are not universal solutions for solving traffic problems.

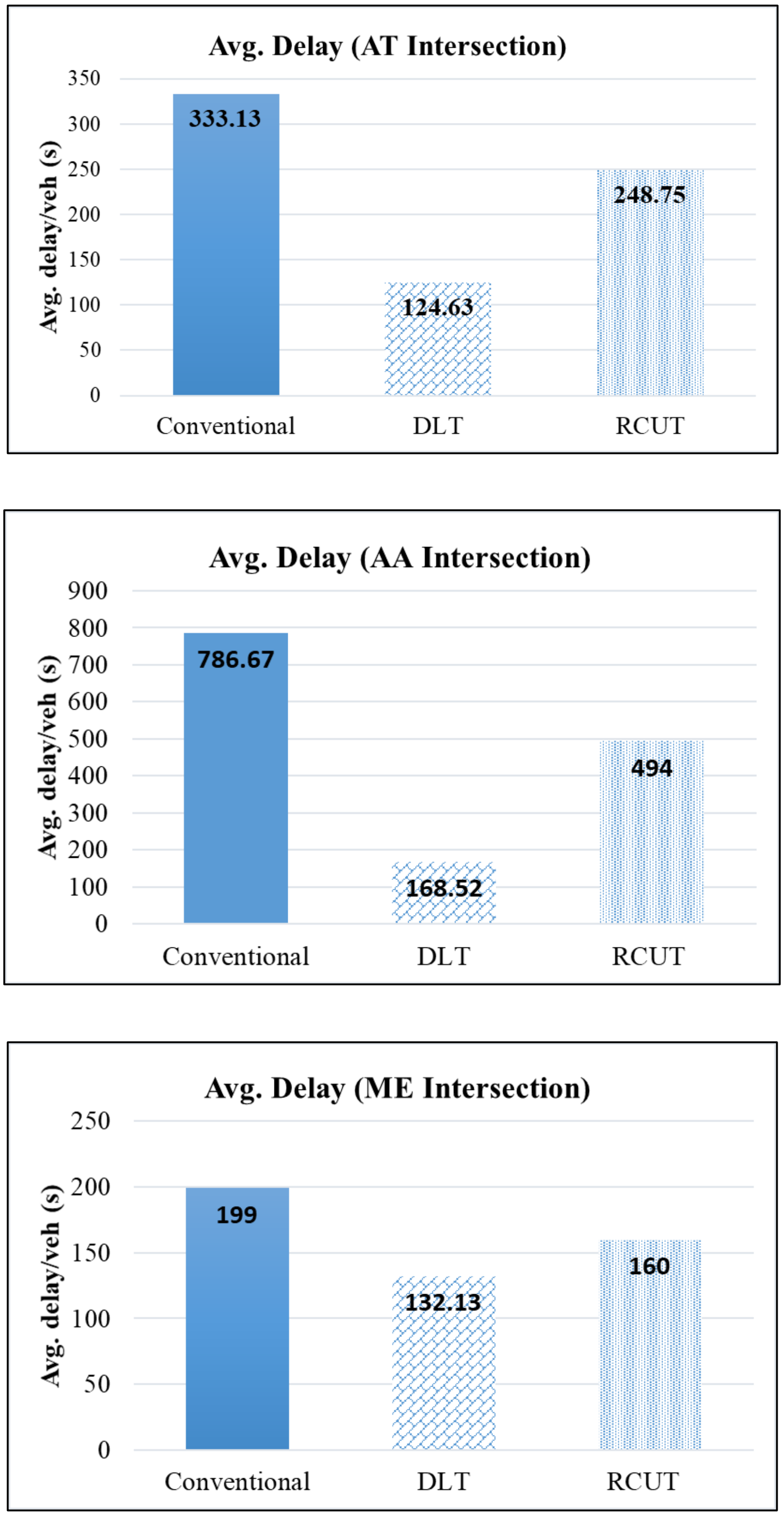

Figure 6.8 The average delay comparison among DLT, RCUT design and the conventional intersections.

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CHAPTER 7

THE UAIDs COST-AND-BENEFIT ANALYSIS

7.1 Introduction

The Cost-and-Benefit Analysis (CBA) is a consistent procedure in which the merits of a particular project(s) are evaluated in a systematic way with a conscious effort to ascertain whether or not to pursue a public investment when the public entities compare among several alternatives. From purely a financial point of view, the CBA helps the policy-makers to assess the value of a transaction or a decision aiming to estimate the strengths and weaknesses of different projects in order to decide the best proposal and which alternative should be considered to accrue the most beneficial sense. The outcomes of this approach will determine whether the project is financially feasible or the analysts should pursue another project. Accordingly, a final selection that offers the greatest overall benefits for incurred costs is considered. By using a common unit of monetary measurement, this approach depends on estimating the potential costs and revenues that may associate with different potential benefits of implementing or even propose several options for financial impacts. Hence, this approach mainly depends on the way in which the present values of various alternative projects are appraised for some future benefits by taking into account the possible comprehensive list of all the costs and benefits could be associated with that project or decision. Therefore, it is essential to classify the different cost and benefit components carefully before conducting the CBA. Form a financial point of view, the cost is defined by the value of money that has been used to produce or present a certain project including direct and indirect costs, principle costs, personnel costs, running costs and the depreciation rates. However, from the economic or social perspectives, the cost is the way in which the available resources are used properly in a specific project or a particular activity. From the transportation economic point of view, the cost of a transportation investment is the value of the resources consumed to achieve expected benefits (Steven et al., 2006). On the other hand, the benefits are defined from the financial point of view as the monetary values of desirable consequence of a certain project, an economic policy or even a decision. Whereas, from economic or social perceptions, the benefits are the value of the goods and services produced. The CBA discussed in this research as an economic assessment approach of this dissertation aims to define and formulate the different costs and benefits' components associated with the implantation of the proposed UAIDs intersections in order to produce enough information of the proposed alternatives schemes and to ascertain whether these it should be undertaken as a cost-effective treatment. Hence, the costs and benefits components are estimated for each proposed treatment including: the conventional at-grade intersection as base-case and the grade-separated intersection (selected as overpass flyover for this study), the DLT and the RCUT intersections as proposed treatments.

After monetizing the various costs and benefits beneficiary items, the Net Present Value (NPV) of both cost and benefit is estimated by applying the future rates of interest to discount the future value of costs and benefits to the today's values. Consequently, the net present values of both costs and benefits of each alternative to decide the best alternative.

7.2 Cost Analysis and Estimation

Numerous components should be considered while estimating the costs as a key factor and the first step in the CBA process. The cost considered in this study is classified into three major components: the construction cost, the running cost and the maintenance cost. First, for the at-grade intersections (i.e. the existing conventional intersections, the DLT and RCUT intersections), the construction cost consists of the pavement construction, the signal heads and detectors installation. The pavement construction that includes the cost of the asphaltic concrete including the surface street remarks painting and installed sidewalks' instruction signs. The signal heads cost consists of the purchasing and installing of the signal heads' units as well as the detectors' units. On the other hand, the grade-separated intersection costs include the construction cost (i.e. the base and subbase concrete elements, the asphaltic concrete, the sidewalks and the Lighting poles). The right of way cost, however, is not taking into account for the base-case as well as for the alternative projects due to the given attention of using the same existing right of way. The estimated costs for the above-mentioned items were obtained from the 2018 annual report of the Egyptian Holding Company for Roads Bridges and Land Transport (a governmental company under the Egyptian Ministry of Transportation) as a guidance that provides an item average unit cost (The Egyptian Holding Company for Roads Bridges and Land Transport, 2018 annual report). A cost breaking down is conducted to calculate the total cost of the existing conventional intersection as well as each of the alternative projects for the three intersections along the studied corridor as illustrated in Table 7.1. As the cost components of the different proposed treatments along the studied corridor obviously shown that the construction cost is much higher in case of the grade-separated intersections comparing to the at-grade ones. Similarly, the construction costs for the both proposed UAIDs designs, DLT and RCUT are higher than the conventional intersections. Due to the extra number of installed signal heads required to control the left-turn crossovers movements in both major and minor approach as well as the main intersections for the DLT intersection comparing to the less signal heads needed to control the RCUTs' main intersections and their U-turns as shown in Table 7.1, the DLT construction cost exceeded the RCUT construction cost. Furthermore, the DLTs construction costs includethe left crossover left-lanes barrier installation that separate the crossover exclusive lanes in case of the DLT intersection as well as the installed detectors which lead to raise the initial construction cost.

Second, based on the engineering judgment and the discussion with Aswan local municipality, the running cost of each alternative is calculated as illustrated in Table 7.1. The running cost includes the electricity needed for the signal heads operation as well as the fixing cost of unexpected sudden malfunctions that may occur. Similarly, and because of the more signal head as well as the detectors units required for the DLTs, the running cost of the DLTs exceeded its counterparts for the conventional, RCUT and grade-separated intersection. On the contrary, the running cost of the grade-separated intersection expressed a reduction because of using only two signal heads to control the

Table 7.1 The values of the cost components of the different alternatives

Cost Component	Value in Egyptian pound (LE)			
	Conventional Intersection	UAIDs Intersection		Grade-Sep. Intersection
		DLT	RCUT	
Construction Cost	27,630,000	30,570,000	29,250,000	300,220,000
Purchasing and installing new signal heads and detectors units	1,320,000.00	3,960,000.00	2,640,000.00	660,000.00
Running Cost/year	200,000	250,000	220,000	100,000
Maintenance Cost/year	160,000	180,000	165,000	250,000
Total Cost (30 years of lifespan)	41,070,000.00	51,390,000.00	46,080,000.00	312,040,000.00

minor approach movements instead of four signal head units needed to control the conventional intersections.

Third, the maintenance cost includes the annual periodical maintenance cost for the pavement surface including the marks painting, the sidewalks and separation barriers maintaining, the signal heads and detectors maintenance for both at-grade conventional and UAIDs intersections. For the separate-grade intersections maintenance cost, however, is more expensive than the at-grade intersections due to the high repairing cost of the pavement surface including ramps, the concrete elements (i.e. retaining walls, girders...etc.) and the sidewalks and other constructed elements. As a result, the grade-separated intersection maintenance is costly comparing to the at-grade intersections. In this research, the lifespan of the proposed projects is assumed to be 30 years as recommended in similar studies related to road traffic costing in Egypt (Mohamed A. Ismail and Samar M. M. Abdelmageed, 2010). The signal heads and detector units' lifespan are assumed to be 10 years.

7.3 Users' Benefit Estimation

Generally, the improvement in the transportation system leads to either users or non-users' benefits. The term "user benefits" referees to the benefits that may accrue to users of the highway system, while the non-user benefits refer to the indirect benefits, or societal benefits such as the environmental impacts (e.g. the noise of an adjacent roadway that may impose a cost on homes or shops along the roadway) (User Benefit Analysis for Highways, 2003). The users' benefits are estimated that redound to a reduction in a user's cost trip as a result of the traffic condition improvement along the studied corridor. Hence, the users' estimated benefits result from savings in operating costs due to the implantation of the proposed treatments comparing to the base-case as adverse components of traffic congestion. However, the indirect benefits or the non-user benefits consider the CO_2 emissions as an environmental impact. Therefore, the estimated benefits are classified into three components as follow:

1. Cost savings of travel time delay;
2. The fuel consumption cost savings;
3. The associated cost savings of CO_2 emissions.

Following the Cairo traffic congestion report published by the World Bank as a secondary data source, the users' benefits components are formulated and monetized for this study.

7.3.1 The Cost savings of Travel Time Delay

First, the annual delay cost is primarily estimated based on the methodology developed by Texas Transportation Institute (Texas A & M Transportation Institute : Urban Mobility Information, Technical report,1992), which was followed in the previous studies in Cairo such as JICA studies and the World Bank reports (The World Bank report, 2011; JICA master plan, 2009). The methodology is outlined and developed as a part of estimating travel delay (the extra amount of time spent travelling due to the congestion) as illustrated in Equation 7.1.

$$DC = N \times O \times (1+\alpha) \times L \times (\frac{1}{V_p} - \frac{1}{V_f}) \times VOT \tag{7.1}$$

Where:

DC The annual delay cost (LE/year);

N The annual total number of vehicles on the corridor during peak hours (veh/h);

O The vehicle occupancy factor;

α The road incident delay factor;

L The congested corridor length (km);

V_p The average speed during peak hours (km/h);

V_f The free flow speed (km/h);

VOT The value of time of passenger car users (LE/h).

Based on the traffic counting data survey made available by the Department of Civil Engineering, Ain Shams University, Egypt, which was discussed at chapter 3 of this dissertation, the annual total number of vehicles on the corridor during peak hours is estimated as shown in Table 7.2. According to the traffic data collected from the field observation, the maximum observed traffic volumes were 8384.0 veh/h, 8795.0 veh/h and 7448.0 veh/h for AT, AA and ME intersection respectively with 4.0 peak hours per day. The followed method is built on estimating the travel delay which is the difference between the amount of time it takes to travel in the same segment of a road during the peak-period at the average speed and at free- flow speed which is calculated by the term [$L\times (1/V_p-1/V_f)$] as shown in Equation 7.1. Therefore, the average delay per vehicle is used to represent the travel delay along the corridor during the peak hours for the base-case and the alternative projects.

On the other hand, it is necessary to use the average vehicle occupancy factor to estimate the number of road users for computing the travel time reliability measures. Based on a national household travel survey in 2017, the Federal Highway Administration (FHWA) provides an average vehicle occupancy as illustrated in Equation 7.2.

Table 7.2 The values of the cost components of the different alternatives

	Conventional intersections	Grade Separated intersections	DLT intersection	RCUT intersection
ADT (veh/h)	24,624			
Annual number of vehicles during peak hours (veh/year)	35,955,420			
The vehicle occupancy factor	1.7			
The road incident delay factor	1.1			
The value of time of passenger car users (LE/h)	18.19			
Avg. delay/veh. (s)	409.11	245.45	192.17	254.52
Total annual delay cost (LE/year)	277,975,335	166,774,330	130,572,512	172,937,064
Annual delay cost savings (LE/year)	0	111,201,004.7	147,402,822.7	105,038,270.7
Total delay cost savings (30 years)	0	3,336,030,142.0	4,422,084,682	3,151,148,122

$$O = \sum_{r=1}^{R} \left(\frac{(TRPMILES)_r \times (NUMONTRP)_r \times (WTTRDFIN)_r}{(TRPMILES)_r \times (WTTRDFIN)_r} \right) \tag{7.2}$$

Where:

O The vehicle occupancy factor;

r A record in the queried “trippub” data;

R The total number of records in the queried “trippub” data;

$(TRPMILES)_r$ The trip distance in miles for the data record “r”;

$(NUMONTRP)_r$ Number of people on trip including respondent for the data record “r”;

$(WTTRDFIN)_r$ The final trip weight for the data record “r”;

In this study context, the average vehicle occupancy value is taken equal to 1.7 as recommended in the previous reports by the World Bank and JICA studies for the Greater Cairo Region. Likewise, to include the encountered delay by road users that results from random or unpredictable occasions such as incidents, security checks, vehicles' breakdowns or even random stops of public transport, the road incident delay factor is taken into account. The incident delay factor reflects the abovementioned unpredictable events by multiplying the recurring delay (the existing delay when the traffic volume on the roadway exceeds its capacity at a particular location during a predictable and a repeated time of day), by a ratio. According to the World Bank published report, the value of the road incident delay factor is allocated by 1.1 (Cairo Traffic Congestion Study Phase 1 Final Report, 2011). Finally, in order to monetize the delays to costs, the value of time is required. Hence, the value of time considered in this study is based on the conducted survey by Japan International Cooperation Agency (JICA) master plan study of the greater Cairo region in 2009 (JICA master plan, 2009). According to the conducted survey in 2007, the average household income was estimated at 1,134 LE/ HH /month. By figuring out the expected hourly worker's income considering the household income level in 2017 provided in the same report, the value of time is adjusted for diverse transport user classes in 2017. In this study context, the value of time is calculated as 18.19 LE/hr by considering the value of time for the passenger car users as dominant users by 80.0% along the studied corridor.

7.3.2 The Fuel Consumption Cost savings

The excess fuel consumption cost due to traffic congestion can be estimated by calculating the wasted consumption caused by the congestion as the difference between the fuel consumed amount at peak speeds and the free-flow speeds. Based on the derived formulas as shown from Equation 7.3 to Equation 7.9 by The Texas Transportation Institute that also has been used in the Cairo traffic congestion related studies such as the Cairo traffic study final report by the World Bank, the average fuel economy calculation is estimated in this study.

$$EFC = EGC + EDC \tag{7.3}$$

$$EGC = EGW \times 1.8 \tag{7.4}$$

$$EDC = EDW \times 1.0 \tag{7.5}$$

$$EFS = EGS + EDS \tag{7.6}$$

$$EGS = EGW \times 2.2 \tag{7.7}$$

$$EDS = EDW \times 1.1 \tag{7.8}$$

$$EGW / EDW = \frac{AVKT}{Free.Speed} \times \left(\frac{Avg.Speed}{Avg.FuelEco} - \frac{Cong.Speed}{Avg.FuelEco} \right) \tag{7.9}$$

Where:

EFC The total annual excess fuel cost (LE/year);

EGC The annual excess gasoline cost (LE/year);

EDC The annual excess diesel cost (LE/year);

EGW The annual excess gasoline wasted (litre);

EDW The annual excess diesel wasted (litre);

EFS The total annual fuel subsidy (LE/year);

EGS The annual excess gasoline subsidy (LE/year);

EDS The annual excess diesel subsidy (LE/year);

$AVKT$ The annual vehicle kilometre travel (veh/km/year).

Certain steps are applied to estimate the excess fuel consumption cost. First, the Average Daily Traffic (AVT) volume along the studied corridor is calculated. Accordingly, the Daily Vehicle

Kilometer of Travel (DVKT) and the Annual Vehicle Kilometer of Travel (AVKT) are estimated for the running vehicle along the studied corridor. Second, using the obtained average speeds from simulating each proposed alternative (i.e. the grade-separated intersections, DLT, RCUT), the average fuel economy and the DVKT, the annual excess gasoline wasted (EGW) values and the annual excess diesel wasted (EDW) values are estimated for each proposal as shown in Table 7.2. The average fuel economy which is used to estimate the fuel consumption of the running vehicles during the congested time, this value is allocated by 20 as recommended in the World Bank study (Cairo Traffic Congestion Study Phase 1 Final Report, 2011). This value was calculated by adjusting the fuel formula for the Cairo case by the World Bank report consultants. Based on the given data, the average fleet age of running vehicles is considered as 10 to 12 years with average fuel consumption of 10 litre/100 km of engine size of most passenger cars of 1600 CC on a speed of 60 km/h. In this research, the estimated excess fuel consumption cost considers the costs for both the road users as 45% of the total costs, and the costs for the government as provided subsidy as 55% of the total costs. Hence, it should be noted the fuel subsidy is 2.2 LE/litre for gasoline and 1.1 LE/litre for diesel as illustrated in Equation 7.7 and Equation 7.8. Accordingly, the fuel consumption cost savings of the different proposed treatments along the studied corridor is calculated and compared to the base-case to estimate the potential benefits of implementing such alternatives as shown in Table 7.3.

Table 7.3 The cost of excess fuel consumption of the different alternatives

	Conventional intersections	Grade Separated intersections	DLT intersection	RCUT Intersection
ADT (veh/h)	24,624			
DVKT(veh/h/km)	93,571,200			
AVKT(veh/h/km)	34,153,488,000			
Avg. fuel economy in congestion	20			
The free flow speed (km/h)	50.0			
The average speed (km/h)	19.0	25.0	42.0	23.0
EGW (litre)	952,882,315.2	768,453,480	245,905,113.6	829,929,758.4
EDW (litre)	105,875,812.8	85,383,720	23,907,441.6	92,214,417.6
EFC (LE/year)	1,821,063,980	1,468,599,984	466,536,646.1	1,586,087,983
EFS (LE/year)	2,212,804,488	1,784,519,748	567,289,435.7	1,927,281,328
Total annual excess fuel consumption (LE/year)	4,033,868,468	3,253,119,732	1033826082	3,513,369,311
Annual cost savings (LE/year)	0	780,748,735.7	3,000,042,386	520,499,157.1
Total cost savings(30 years)	0	23,422,462,070	90,001,271,578	15,614,974,714

7.3.3 The Cost savings of CO2 emissions

Based on the standard CO_2 emission rates due to the excess fuel consumption during the congestion in Cairo given in the related studies as shown in Table 7.4, the associated cost savings of CO2 emissions due to the excess fuel consumption is accordingly calculated as illustrated in Equation 7.10 and 7.11. The given rates depend on the fuel type and the vehicle type but the engine type is not considered. This value is allocated by 2.4 for gasoline engines and 2.41 for the diesel engines as shown in Table 7.3 and recommended in the related report (Cairo Traffic Congestion Study Phase 1 Final Report, 2011). Before using Equation 7.10, attention should be paid to convert the annual excess wasted gasoline and diesel volume units in litres to weight units in kilograms as 1.0 litre of gasoline is equal 0.77 kg and 1.0 litre of diesel is equal 0.832 kg to the fit the Equation 7.11. Consequently, the annual CO2 emission cost of the different proposed treatments along the studied corridor is calculated and compared to the base-case to estimate the associated cost savings as potential benefits of implementing such alternative as shown in Table 7.5.

$$\mathrm{CCO_2} = \mathrm{WCO_2} \times U\mathrm{CCO_2} \tag{7.10}$$

$$\mathrm{WCO_2} = GW \times 2.40 + DW \times 2.41 \tag{7.11}$$

Where:

$\mathrm{CCO_2}$ The annual CO_2 emission cost (LE/year);

$\mathrm{WCO_2}$ The annual CO_2 emission weight (Kg);

$U\mathrm{CCO_2}$ The unit cost of CO_2 (LE/ton);

GW The annual weight of wasted gasoline (kg);

DW The annual weight of wasted diesel (kg).

Table 7.4 The emission rate for the different vehicle types

Vehicular mode	CO_2 emission rate (Kg/L)
Cars (diesel and gasoline)	2.40
Motorcycle	2.42
Taxi	2.40
Bus	2.41

Table 7.5 The CO_2 emission cost savings of the different alternatives

	Conventional intersections	Grade Separated intersections	DLT intersection	RCUT Intersection
EGW (litre)	952,882,315.2	768,453,480	245,905,113.6	829,929,758.4
GW (kg)	733,719,382.7	591,709,179.6	189,346,937.5	639,045,914
EDW (litre)	105,875,812.8	85,383,720	23,907,441.6	92,214,417.6
DW (kg)	88088676.25	71039255.04	19890991.41	76722395.44
WCO_2 (kg)	1,973,220,228.25	1,591,306,635	502,369,939.23	1,718,611,166
CCO_2 (LE/year)	112,473,553,010	90,704,478,234.	28,635,086,536	97,960,836,492
Annual cost savings (LE/year)	0	21,769,074,776	83,838,466,474	14,512,716,517
Total CO_2 emission cost savings (30 years)	0	653,072,243,286	2,515,153,994,220	435,381,495,524

7.4 The Net Present Value

Owing to evaluate and compare the cash flows for both the cost and benefits of each proposed alternative over the project lifespan, it is necessary to apply the Net Present Value (NPV) as intrinsic evaluation approach. Within this hypothetical method, the difference between the present value and the future cash flows from an investment is estimated by discounting the future values of the current estimated values. The NPV is determined by applying a social discount rate which is yield to the costs as negative cash flows as well as to the benefits which are positive cash flows for each period of an investment as shown in Equation 7.12. The total cost savings over the 30 years of the project lifespan for the different alternatives without considering the discount rate are shown in Table 7.6. According to the Central Bank of Egypt, the social discount rate is 17.25% (Central Bank of Egypt, 2018). Applying the given equation by assuming the lifespan of the proposed projects as 30 years for the pavements and other construction elements, however, the signal heads and detector units' lifespan are assumed to be 10 years, the NPV of both the cost and benefits of each proposed alternative over the project lifespan is calculated. Accordingly, the Benefit Cost Ratio (BCR) is calculated to show the economic efficiency of each alternative are shown in Table 7.7. The estimated BCR indicated to the efficiency of the proposed UAIDs two proposed intersections; DLT and RCUT, over the grade-separated intersection. The BCR for the DLT design exceeded 4.0, and it recorded 2.99 for the RCUT alternative, while the grade-separated BCR is less than 1.0.

$$NPV = \sum_{t=0}^{T} \left(\frac{C}{(1+i)^t} \right) \tag{7.12}$$

Where:

NPV The Net Present Value;

T The project lifespan;

C The annual project cash flow;

i The social discount rate.

Table 7.6 The total cost savings over the project lifespan of the different alternatives

	Grade Separated intersections	DLT intersection	RCUT Intersection
Delay cost savings	3,336,030,142.0	4,422,084,682.0	3,151,148,122.0
Fuel consumption cost savings	23,422,462,070	90,001,271,578	15,614,974,714
Total CO_2 emission cost savings	653,072,243,286	2,515,153,994,220	435,381,495,524
The total cost savings	679,830,735,498	2,609,577,350,480	454,147,618,360

Table 7.7 The BCA ratio of the different alternatives

	NPV (Costs)	NPV (Benefits)	BCA ratio
Grade Separated intersections	300,384,731.61	951,235.92	0.32
DLT intersection	31,544,286.05	1,291,286.38	4.09
RCUT Intersection	29,900,354.48	895,152.73	2.99

7.5 Results Analysis and Discussion

Based on the detailed calculation illustrated in the previous sections, the results are obtained and discussed. The achieved enhancement in the operational performance indices leads to obvious potential benefits by considering the alternative projects instead of the current conventional intersections. First, the delay cost savings results indicate to possible benefits in case of implementing the different alternatives. The grade-separated intersection may save 43.0% for the delay costs, while the DLT and RCUT intersections may also save 53.0 and 38.0% respectively. The results also showed that the DLT save 30% higher than the cost could be saved at the grade-separated intersection and 40% than the RCUT intersection of delay cost savings, while the cost savings of the grade-separated intersection is higher than the RCUT alternative by 5.0% as shown in Table 7.2.

Second, as a result of the smooth travel along the corridor that may occur because the enhancement in the average speeds for the proposed alternatives, the fuel consumption cost experienced significant savings for the proposed projects comparing to the conventional intersection as a base-case. The fuel consumption cost analysis outputs referred to significant savings for the grade-separated intersection by 19.0%, while the DLT and RCUT intersections may also save 74.0 and 13.0% respectively. Likewise, the results showed that the cost savings of the DLT intersection are higher than the grade-separated intersection by 74.0% as well as the RCUT alternative by 83.0%. On the other hand, the cost savings of the grade-separated intersection excessed the cost savings by 33.0% of the RCUT design as shown in Table 7.3.

In terms of the associated cost savings of CO2 emissions due to the excess fuel consumption, the results reported significant expected cost savings as a result of the alternative implementation. According to the analysis results, the expected savings by 19.0, 75.0 and 13.0% for the grade-separated intersection, the DLT and RCUT intersections respectively. The results also indicated to the superiority of the DLT intersection in terms of the associated cost savings of CO2 emissions. The DLT may save

74.0% more than the grade-separated intersection and 83.0% more than the RCUT design as shown in Table 7.5.

In conclusion, the CBA results emphasized the expected cost savings as potential benefits due to the implementation of the different proposed alternatives. The BCR revealed the efficiency of the two proposed UAIDs intersections; DLT and RCUT, over the grade-separated intersection. Despite the expected benefits of the grade-separated intersection, the BCR is less than 1.0 which is interpreted as an economically ineffective alternative, however, the BCR for the DLT design exceeded 4.0, and it recorded 2.99 for RCUT intersection. Hence, based on the CBA, it can be concluded that the UAIDs are economically effective compared to the grade-separated intersections as recommended alternatives to alleviate the congestions at the conventional at-grade intersections.

CHAPTER 8

FINAL REMARKS

8.1 Summary and Conclusions

Indeed, the primary aim of this research is to gain a better understanding of the performance, various functions, benefits, vulnerabilities and limitations of the two proposed UAIDs schemes namely; the Displaced Left-Turn (DLT) intersection and the Restricted Crossing U-turn (RCUT) intersection under the heterogeneous traffic conditions. This study provided a clearer idea to traffic analysts and the policymakers with an objective assessment of the possibility of implementing these schemes as proposed alternative designs to substantially alleviate the traffic congestions in the developing cities around the world where the heterogeneous traffic conditions are dominant. Although academic in nature, this research was built upon actual realistic data that were made available for typical existing conventional intersections located in Cairo, the capital city of Egypt. Due to the limitation of time and resources, only Cairo, Egypt is selected as a case study in this research as it satisfies the traffic conditions mentioned above, however, the outcome of this study can be applied in any other developing city around the world where the heterogeneous traffic is dominant. By bringing together the existing knowledge and defining future research directions, this study opens the doors to investigate the possibility of implementing different designs of UAIDs using other case studies. This study, mainly, focuses on the operational performance evaluation and cost-and-benefit assessments. However, other research areas include, but are not limited to, safety performance, environmental impacts (i.e. emissions) and pedestrian movements still need more examination and evaluation.

In this study, the methodology enhancements could be achieved based on employing VISSIM, as a powerful simulation-based assessment approach as well as a widely psychophysical car-following model, highly recommended to analyze the operational performance of UAIDs. This research introduced the needed procedures to represent such UAIDs under the different complexities of the heterogeneous traffic conditions as close as the real world. In other words, a special attention was paid to fulfil the heterogeneous traffic needs include the geometric layout design, driving behaviour, vehicle molding, non-lane based traffic and the signal controller calculations. Also, seeking a reliability in the real world and in order to obtain a credible representation close to the actual field, the model calibration and validation process were carefully conducted through a conditional match of the simulated parameter values with observed traffic field data to ensure the models' effectiveness and practicability.

For the academic contribution, this dissertation developed new methodologies in which both the pre-timed and real-time coordination systems of the proposed DLT design are considered by utilizing the most common coordination techniques; bandwidth maximization and delay minimization. The branch-and-bound algorithm was utilized to improve a bandwidth maximization approach as a pre-timed (fixed-time) coordination approach, while the delay minimization approach is employed to develop a real-time demand-responsive signal control algorithm by creating a programming code on the solid foundation of the dynamic optimization principles. This entire algorithm was built based on developing a mathematical model and then was examined by initiating an inter-process communication between VISSIM as a versatile simulator-based and MATLAB as a multi-paradigm numerical computing environment.

Likewise, to determine the types of intersections which are more preferable, an equitable comparison among the presented unconventional schemes, DLT and RCUT designs and their existing conventional counterparts has been conducted. This comparison focused on comparing the operational performance of each proposed alternative scheme with their corresponding conventional counterparts in terms of travel time, average delay, average speed and the other performance indices.

Finally, the CBA discussed in this research as an economic assessment approach that aims to define and formulate the different costs and benefits' components associated with the implantation of the proposed UAIDs intersections in order to produce enough information of the proposed alternatives schemes and to ascertain whether these alternatives should be undertaken as cost-effective treatments.

Hence, the costs and benefits components are estimated for each proposed treatment including: the conventional at-grade intersection as base-case and the grade-separated intersection (selected as overpass flyover for this study), the DLT and the RCUT intersections as proposed treatments.

8.2 The Key Findings

The overall results are presented to draw up the research main findings as follows:

1. This research emphasized the possibility of implementing the DLTs and RCUTs in the developing cities around the world where the heterogeneous traffic conditions are dominant. However, special attention is needed to be taken into consideration for a required modification in both geometric and signal controller designs to fulfil the heterogeneous traffic needs.
2. This study reported the possibility of implementing the proposed UAIDs in an urban area where the same right-of-way is usually limited, as long as the geometric designs are modified to fulfil the existing traffic environment.
3. Based on the simulation results, the superiority of the DLT and RCUT designs proposed in this study was recognized having overwhelmed their conventional counterparts. The performance used indices to evaluate the level of service (LOS) at each intersection were experienced a significant improvement.
4. The outputs revealed that DLTs consistently reported better results and overcame over the existing conventional intersections as well as the RCUT design for the three intersections studied. The results indicated to the obvious improvement of the LOS of the studied the intersections. It was found that the proposed UAIDs schemes reduced the overall delay and the total travel time while the average speed was increased significantly.
5. It can be concluded that the heterogeneous traffic complexities (i.e. the diverse dynamic and static properties of mixed traffic compositions, the non-lane based system, the aggressive driving

behaviour…etc.) obviously influenced the proposed UAIDs geometric design as well as the operational efficiency.

6. The simulation findings emphasized that the RCUT design is not appropriate for high traffic levels, similar to the previous studies findings.

7. The CBA results revealed that the UAIDs are economically effective treatments compared to the grade-separated intersections as recommended designs to alleviate the congestions at the conventional at-grade intersections. The CBA results emphasized significant expected cost savings as potential benefits due to the implementation of the different proposed alternatives. The Benefit Cost Ratio (BCR) revealed the efficiency of the two proposed UAIDs intersections; DLT and RCUT, over the grade-separated intersection. However, the DLT intersection BCR shows more economical feasibility than the RCUT design and the grade-separated intersection.

8. In conclusion, although the UAIDs improved the operational performance of the entire existing conventional intersections, they are not universal solutions for solving traffic problems.

8.3 Research Recommendations

Aiming to draw up a clearer perception for improving the operational efficiency of the recent traffic situation, it is necessary to come up with some recommendations to traffic analysts, other agencies and policy makers. Indeed, an overall inclusive plan is required to raise the level of service of the entire existing intersections. This enhancement should be presented based on comprehensive traffic studies by involving new techniques, advanced technologies and a collaboration between both the industrial and academic sides. However, public hearing includes public users, drivers, traffic experts, urban planners, traffic police, academic staff and policy makers is needed to ensure the appropriate decision-making among the strategies presented in this study.

This study strongly recommends the urgent need for enhancing the signalized intersections performance, particularly, under such heterogeneous traffic complexities. Due to the noteworthy influence of such complexities, especially, the non-lane based operation or the lack of lane discipline, thus the main recommendation of this study is the separation of the traffic movements by considering and activating the lane lines. Hence, the surface marks such as lane lines and direction arrows should be clearly painted. Meanwhile, the on-street parking, particularly near the intersections, should be prohibited to increase the capacity of the intersections. Likewise, traffic regulations have to be enforced and warning signs have to be installed. In addition, pedestrian signals and cross lines should be set up for enhancing the pedestrian movements.

As a result of the significant impact of the aggressive driving behaviour on the performance of intersections, principally, this study highly recommends the necessity of enforcing the traffic laws. On the other hand, before initiating these alternative schemes as unknown designs in most of the developing countries, this study also emphasizes the necessity of paying much efforts to avoid drivers' confusion by explaining the operational mechanism for the different road users. The different media platforms can play a great role and also by teaching the traffic altitudes in the education systems would have a great effect to improve the corresponding congestions in the future.

8.4 Directions for Future Studies

Towards more comprehensive assessment of the studied alternative intersections and in order to provide further improvements on the performance, this study suggests some valuable ideas for further research as follow:

1. Regarding the DLT coordination considered in this work, since the optimization of this study is formulated individually at each intersection, it is recommended to examine the optimization where it is performed in all intersections simultaneously. On an analytical aspect, it would be interesting to conduct a sensitivity analysis to estimate the optimal values the relative importance coefficient of major and minor approach delays, as a future extension of this work.

2. More generalized discussion on the use of other evolutionary algorithms with different optimizers that may be examined to enhance the optimization problem outcomes is still lacking.
3. The assessment of land use impacts is required, practically, the accessibility to roadside facilities and retailers through investigating the interactions and the impacts of such proposed UAIDs on the abutting land use.
4. It would be useful to consider other alternative designs such as USC, QR, Bowtei, Jughandle…etc to be examined under such heterogeneous traffic conditions.
5. Other indirect or non-users' benefits such as the safety conditions are needed to be included in the Cost-and-Benefit analysis for the proposed UAIDs.
6. The recommended research areas that may need more evaluations include, but not limited to, safety performance and environmental impacts (i.e. emissions), pedestrian movement analysis and other aspects of such alternative should also be examined.
7. It is recommenced to consider other case studies with different geometric and traffic characteristics for UAIDs applicability investigation.

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ANNEX 1