ISTANBUL TECHNICAL UNIVERSITY ★ GRADUATE SCHOOL OF SCIENCE ENGINEERING AND TECHNOLOGY

DELAY-BASED PERFORMANCE ANALYSES OF FOUR-LEGGED SIGNALIZED INTERSECTIONS: A CASE STUDY

M.Sc. THESIS

Farid JAVANSHOUR

Department of Civil Engineering

Transportation Engineering Programme

MAY 2014

ISTANBUL TECHNICAL UNIVERSITY ★ GRADUATE SCHOOL OF SCIENCE ENGINEERING AND TECHNOLOGY

DELAY-BASED PERFORMANCE ANALYSES OF FOUR-LEGGED SIGNALIZED INTERSECTIONS: A CASE STUDY

M.Sc. THESIS

Farid JAVANSHOUR
(501111434)

Department of Civil Engineering

Transportation Engineering Programme

Thesis Advisor: Associate Prof. Dr. Hilmi Berk ÇELİKOĞLU

MAY 2014

İSTANBUL TEKNİK ÜNİVERSİTESİ ★ FEN BİLİMLERİ ENSTİTÜSÜ

DÖRT KOLLU IŞIKLI KAVŞAKLARDA GECİKME ESASLI PERFORMANS ANALİZİ: BİR ÖRNEK ÇALIŞMA

YÜKSEK LİSANS TEZİ

Farid JAVANSHOUR
(501111434)

İnşaat Mühendisliği Anabilim Dalı

Ulaştırma Mühendisliği Programı

Tez Danışmanı: Doç. Dr. Hilmi Berk ÇELİKÖĞLU

MAYİS 2014

Farid-javanshour, a **M.Sc.** student of ITU **Institute of Science and Technology** student ID **501111434**, successfully defended the **thesis/dissertation** entitled “**DELAY-BASED PERFORMANCE ANALYSE OF FOUR-LEGGED SIGNALIZED INTERSECTIONS: A CASE STUDY.**”, which he prepared after fulfilling the requirements specified in the associated legislations, before the jury whose signatures are below.

Thesis Advisor : **Assoc. Prof. Dr. Hilmi Berk ÇELİKOĞLU** ....
İstanbul Technical University

Jury Members : **Prof. Dr. Ergun GEDİZLİOĞLU** ....
İstanbul Technical Universit

Assist. Prof. Dr. Ilgın GÖKAŞAR ....
Boğaziçi University

Date of Submission : 5 May 2014
Date of Defense : 29 May 2014

To my mother and father,

FORWARD

Efficiency of transportation networks is a crucial issue especially for cities with increasing traffic demand. Signalized intersections are significant points in transportation networks on which traffic signals play an important role on their efficiency. When set without aim of optimizing them, traffic signals might cause inefficient use of transportation systems. The main objective of this study is modeling and performance evaluation of four-legged isolated signalized intersections utilizing pre-timed signal controls with reference to a case study located in Istanbul city of Turkey. In the purpose of modeling signal control for the selected intersections three approaches, i.e., conventional methods of Australian (Akçelik) and USA (HCM 2000) as well as dynamic method of VISSIM software, are utilized in analyses. The performances of incorporated signaling approaches are comparatively evaluated following calibration of selected intersection models with site data.

For conducting this research, for most, I would like to thank my supervisor Assoc. Prof. Dr. Hilmi Berk ÇELİKOĞLU for his valuable guidance throughout my period of thesis. I am grateful to Assoc. Prof. Dr. Hilmi Berk ÇELİKOĞLU for his technical and mental support from the beginning of my graduate school life.

Besides my supervisor, I would like to thank Onur DENİZ and Göker AKSOY for their contribution and technical support in conducting the simulation model created in PTV VISSIM software. My sincere thanks also go to Assoc. Prof. Dr. Kemal Selçuk ÖĞÜT and Prof. Dr. Ergun GEDİZLİOĞLU for their valuable courses held on the department of transportation engineering.

May 2014

Farid JAVANSHOUR
(Transportation Engineering)

TABLE OF CONTENTS

Page

FOREWORD.......... ix
TABLE OF CONTENTS xi
ABBREVIATIONS.......... xiii
LIST OF TABLES.......... xv
LIST OF FIGURES.......... xvii
SUMMARY xix
ÖZET xxii
1. INTRODUCTION.......... 1
1.1 Brief Background.......... 1
1.2 Motivation, Problem Statement, and Objectives 1
1.3 Scope of Thesis.......... 2
1.4 Thesis Organization 2
2. FUNDAMENTALS OF INTERSECTION MODELLING AND OPERATIONS 5
2.1 Intersection Fundamentals 5
2.1.1 Basic definitions.......... 5
2.1.2 Intersection geometries 6
2.1.3 Characteristics of conflicting traffic flows 8
2.2 Fundamentals of Signal Operations.......... 9
2.2.1 Basic definitions.......... 9
2.2.2 Performance measures 11
2.2.3 Performance analysis 12
2.3 Evolution of Traffic Control Systems.......... 13
2.3.1 Pre-timed signal control.......... 13
2.3.2 Traffic actuated signal control 13
2.3.2.1 Semi-Actuated signal control.......... 13
2.3.2.2 Fully-Actuated signal control 14
2.3.3 Traffic adaptive signal control.......... 15
2.3.4 Signal priority systems.......... 16
2.4 Signal timing optimization methods.......... 17
2.4.1 Stochastic methods.......... 17
2.4.2 Heuristic methods 17
2.4.3 Hybrid methods.......... 18
3. LITERATURE REVIEW 21
4. METHODS USED TO OBTAIN INTERSECTION PERFORMANCES 23
4.1 Australian Method 23
4.2 US (HCM) Method 43
4.3 Method of VISSIM 63
4.3.1 Review of case studies incorporating simulation.......... 69
5. CASE STUDY 71
5.1 Description of Area and Sample Intersections.......... 71

5.2 Description of Real Data 75
5.3 Development of Sample Intersections at VISSIM Software 79
5.4 Calibration of Sample Intersection Flows Using Real Data 80
5.5 Numerical Implementations Employing Real Data 84
5.5.1 Performances derived by Australian (Akcelik) method 84
5.5.2 Performances derived by HCM 2000 method 88
5.5.3 Performance derived by incorporating VISSIM 91
5.6 Comparative Evaluations 92
6. CONCLUSIONS AND RECOMMENDATIONS 101
REFERENCES 103
CURRICULUM VITAE 107

ABBREVIATIONS

CBD	**:** Central Business District
DVE	**:** Driver Vehicle Element
GEH	**:** Geoffrey E. Havers
HCM	**:** Highway Capacity Manual
ITS	**:** Intelligent Transport Systems
Min	**:** min
MUTCD	**:** Manual on Uniform Traffic Control Devices
NTCIP	**:** National Transportation Communications for ITS Protocol
PF	**:** Progression Factor
PHF	**:** Peak Hour Factor
PTV	**:** Planung Transport Verkehr
RTOR	**:** Right Turn on Red
Sec	**:** Second
STM	**:** Signal Timing Manual
TSP	**:** Transit Signal Priority
Veh	**:** vehicle

LIST OF TABLES

Page

Table 2.1 : Relationship between the amount of control delay per vehicle and LOS 12
Table 4.1: An example of phase-movement matrix 24
Table 4.2: Average saturation flows in through car units per hour for estimation by environment class and lane type 34
Table 4.3: Through car equavalents (t_{cu}/veh) for different types of vehicle and turn 36
Table 4.4: Input data needs for each analysis lane group... 44
Table 4.5: Typical lane groups for analysis 47
Table 5.1 : Specifications of intersection 1 72
Table 5.2 : Specifications of intersection 2 73
Table 5.3 : 15-min volumes for each approach of intersection 1 76
Table 5.4 : 15-min volumes for each approach of intersection 2 77
Table 5.5 : Direction volumes in intersection 1 and intersection 2 78
Table 5.6: Adjusted flow rates of lane groups in intersection 1 and intersection 2 .. 79
Table 5.12: Obtained volume through VISSIM for intersection 1 and intersection 2 using real data 83
Table 5.13: Degree of saturation for approaches in intersection 1 and intersection 2 using real data 84
Table 5.14: Computation of traffic composition (f_c) for each approach in intersection 1 employing Australian method 85
Table 5.15: Computation of saturation flow for each lane group in intersection 1 employing Australian method 85
Table 5.16: Computation of approach delays for intersection 1 employing Australian method 86
Table 5.17: Computation of traffic composition (f_c) for each approach in intersection 2 employing Australian method 87
Table 5.18: Computation of saturation flow for each lane group in intersection 2 employing Australian method 87
Table 5.19: Computation of approach delays for intersection 2 employing Australian method 88
Table 5.20: Computation of saturation flow rates in intersection 1 using HCM 2000 method... 89
Table 5.21: Computation of approach delays based on HCM 2000 method in intersection 1 89
Table 5.22: Computation of saturation flow rates in intersection 2 using HCM 2000 method... 90
Table 5.23: Computation of approach delays based on HCM 2000 method in intersection 2. 91

Table 5.24: Approach delays of intersection 1 derived through VISSIM. 91
Table 5.25: Approach delays of intersection 2 derived through VISSIM 92

LIST OF FIGURES

Page

Figure 2.1 : Relationship among intersection skew and crosswalk length. 7
Figure 2.2 : Diverge, merge and crossing conflicting points in an intersection and a roundabout 8
Figure 2.3 : Vehicle – pedestrian and vehicle – vehicle crossing points 9
Figure 2.4 : Signal phasing diagram. 10
Figure 2.5 : Typical flow rates at a signalized movement. 11
Figure 2.6 : Effect of TSP to adjust signal timing. 16
Figure 4.1 : An example of signal phasing diagram 24
Figure 4.2 : A cycle diagram.. 25
Figure 4.3 : Basic model and definitions 26
Figure 4.4 : Signal timing diagram for the example in Figure 4.1............................ 26
Figure 4.5 : Signal phasing diagram and intersection plan (an example)................. 31
Figure 4.6 : Critical movement search diagram.. 32
Figure 4.7 : Opposed turn saturation flow during the undersaturated part of the opposing movement green period... 37
Figure 4.8 : Signalized intersection methodology 43
Figure 4.9: Illustration of uniform delay... 57
Figure 4.10 : Illustration of random delay 58
Figure 4.11 : Illustration of overflow delay... 59
Figure 4.12 : Communication between traffic simulator and signal state generator 66
Figure 4.13 : Interaction between two vehicles in VISSIM...................................... 69
Figure 5.1 : Location of case studies.. 71
Figure 5.2 : Geometry of intersection 1 72
Figure 5.3 : Signal phasing of intersection 1 72
Figure 5.4 : Geometry of intersection 2 73
Figure 5.5 : Signal phasing of intersection 2 74
Figure 5.6 : Signal timing diagram of intersection 1 74
Figure 5.7 : Signal timing diagram of intersection 2 75
Figure 5.8 : Final model of intersection 1 in VISSIM 80
Figure 5.9 : Final model of intersection 2 in VISSIM 80
Figure 5.10 : Calibration of traffic volume on all approaches of intersection 1 82
Figure 5.11 : Calibration of traffic volume on all approaches of intersection 2 82
Figure 5.12 : Derived delays using Australian method, HCM2000 method and VISSIM in intersection 1 92
Figure 5.13 : Derived delays using Australian method, HCM2000 method and VISSIM in intersection 2 93
Figure 5.14 : Delay functions of Australian and HCM 2000 method on approach 2 of intersection 1... 94

Figure 5.15 : Delay functions of Australian and HCM 2000 method on approach 2 of intersection 1 with logarithmic scale 94
Figure 5.16 : Delay functions of Australian and HCM 2000 method on approach 4 of intersection 1 95
Figure 5.17 : Delay functions of Australian and HCM 2000 method on approach 4 of intersection 1 with logarithmic scale 95
Figure 5.18 : Delay functions of Australian and HCM 2000 method on approach 3 of intersection 2 96
Figure 5.19 : Delay functions of Australian and HCM 2000 method on approach 3 of intersection 2 with logarithmic scale 96
Figure 5.20 : Delay functions of Australian and HCM 2000 method on approach 4 of intersection 2 97
Figure 5.21 : Delay functions of Australian and HCM 2000 method on approach 4 of intersection 2 with logarithmic scale 97
Figure 5.22 : Delays on approach 2 of intersection 1 according to Australian, HCM 2000 and VISSIM methods 98
Figure 5.23 : Delays on approach 4 of intersection 1 according to Australian, HCM 2000 and VISSIM methods 99
Figure 5.24 : Delays on approach 3 of intersection 2 according to Australian, HCM 2000 and VISSIM methods 99
Figure 5.25 : Delays on approach 4 of intersection 2 according to Australian, HCM 2000 and VISSIM methods 100

DELAY-BASED PERFORMANCE ANALYSES OF FOUR-LEGGED SIGNALIZED INTERSECTIONS: A CASE STUDY

SUMMARY

The area in which two or more roads cut one another and different traffic movements in various directions intend to use this space concurrently is called intersection. Intersections are critical points in transportation networks. These points determine both degree of safety in places where face to low traffic volumes and capacity for the congested areas. Thereby, in the cities with growing traffic flow, a profound planning and designing strategies should be conducted for these significant points. An inefficient designing of intersections may affect all network operation inappropriately. The most convenient intersection from the point of geometry and traffic control types should be chosen by traffic engineers. Otherwise, an unsuitable designing may cause traffic congestion, public frustration, high fuel consumption, low capacity conditions and high accident rates.

The geometry of the intersection plays a great role in its efficiency. Unappropriate curve radii may result in low speed for turning vehicles, so traffic congestion can occur. Moreover, sufficient angle between two approaches of an intersection is required to provide sufficient sight triangles for drivers who enter to the intersection to clear this area safely. The exclusive turning lanes will improve intersection capacity with separating the turning movement from through movement, or in rounabouts, radious of central island will significantly affect its efficiency. The slope of the approaches is another parameter that may change capacity of intersections.

For the intersections with high traffic volume, all vehicles are unable to utilize the intersection area concurrently. Thereby, the movements must be directed to stop or proceed alternatively. Traffic signals provide this condition for intersections called signalized intersections. As early as the 1868, the manually operated semaphores first used in London as traffic control signals. In order to decrease the number of accidents in intersections the first traffic signals were used in the United States by alternatively assigning right of way.

Signal timing of signalized intersections determine the efficiency of these areas. The capacity in signalized intersections varies as a function of green time during which vehicles are given right of way and should be optimized to maximize the performance of the intersection. There are several methods for optimizing green times in signalized intersections like English (Webster), Australian (Akcelik), USA (HCM) method or simulation softwares such as PTV VISSIM.

The cycle length of a traffic signal plays a significant role on the efficiency of a signalized intersection. The cycle length should neither be short nor long. The best situation is somewhere between these two extremes. If the cycle length is too short, there will be so many phase changes during an hour which will cause more delays due to this irrational signal timing. On the other hand, if the cycle length is too long,

delays will be lengthened as the vehicles wait for receiving permission to pass through intersection.

Traffic signals fall into four groups, namely, pretimed signal controls, traffic-actuated signal controls, traffic adaptive signal controls and signal priority systems. Pre-timed control is ideally suited to closely spaced intersections where traffic volumes and patterns are consistent on a daily or day-of-week basis and their cycle times and also green times are fixed for all day.

Traffic-actuated signal controls use detection for intersection movements and operate according to the information collected from the detectors. Traffic adaptive signal controls collect the information in the upstream traffic and using an algorithm to predict location and the time during which traffic reach there and make downstream signal adjustments accordingly.

Transit Signal Priority (TSP) is an operational strategy that is applied to reduce the delay transit vehicles experience at traffic signals. TSP involves communication between buses and traffic signals so that a signal can alter its timing to give priority to transit operations. Priority may be accomplished through a number of methods, such as extending greens on identified phases, altering phase sequences, and including special phases without interrupting the coordination of green lights between adjacent intersections.

Nowadays, a vast variety of traffic simulation software has been developed in order to solve too complicated transportation problems utilizing computers. Simulation is important in that it enables traffic engineers to analyse diverse scenerios of transportation models without any need to high cost experiments in the field. Moreover, it helps to visualize the transportation network operations, so the problems of the network may be more obvious. One of these simulation programs is PTV VISSIM, a microscopic simulation tool for modeling multimodal traffic flows and it provides ideal conditions for testing different traffic scenarios in a realistic and highly detailed manner before final implementation. VISSIM is a software to model at the highest level of detail and complexity, Seamless and integrative within Vision Traffic software family and multimodal simulation tool Convincing 2D and 3D films.

There are diverse measures for evaluating the performance of signalized intersections such as average delay per vehicle, number of stops and queue length as basic performance measures. Fuel consumption, pollutant emissions and cost are some other secondary measures of performance. In this study, the average delay per vehicle was utilized for exhibiting the operational circumastances of case studies. Delay is the difference between desired and actual drivers’ travel times.

For conducting this study, two for-legged signalized intersections in Pendik area of Istanbul city were designated as case studies. Traffic flows obtained using camera recording method at sites between 7:00-10:00, 12:00-14:00 and 16:30-19:30 time periods. Then, all the recordings were transformed to the computer and the frequency of different types of vehicles in 15-min periods were counted and tabulated with respect to their travel directions. Subsequently, on each approach and for each travel direction, the hour during which maximum traffic volume occurs was designated as its peak hour. In continue, peak hour factors (PHF) were calculated for each approach by dividing the peak hour volume to the flow rate of that approach. Using this PHF factor the adjusted flow rate was computed for each lane group.

Finally, these flow rates were transformed to the VISSIM, and the established models were calibrated to replicate the field situation subsequently, and the final volumes yielded by the VISSIM after calibration process were used as this study flow rate inputs in all methods. Employing these data the performance of case studies were estimated applying Australian (Akcelik) and USA (HCM 2000) methods as well as PTV VISSIM software. That is, the conventional methods of Australian and USA were compared with dynamic method of VISSIM. Finally, the results derived from these four methods were analyzed and differences between these methods and possible reasons were discussed.

GECİKME ESASLI DÖRT KOLLU IŞIKLI KAVŞAKLARDA PERFORMANS ANALİZİ: BİR ÖRNEK ÇALIŞMA

ÖZET

Kavşak İki ve ya daha fazla kesişen yolların neticesinde oluşan alandır ve çeşitli trafik akımları bu bölgeni ortak kullanmaya çalışırlar. Kavşaklar ulaşım ağlarının kritik noktaları olup trafik hacmin hafif olduğu yerlerde ise güvenliği artıp ve trafik yoğun olduğu yerlerde kapasiteni artarlar. Bu yüzden trafiği artan şehirlerde kavşaklara iyi bir tasarım ve planlama gerekmektedir.

Kavşaklar yetersiz tasarlandığı halde tüm ulaşım ağı etkileyip bu sistemin performansın düşürmekteler. En uygun kavşak geometri ve kontrol tipi bakımından trafik mühendisleri tarafından belirlenmesi gerekmektedir. Aksı takdirde uyumsuz bir tasarım trafik tıkanıklığı, yakıt tüketimin çoğalması, düşük kapasite ve trafik kazaların artmasına neden olacaktır.

Kavşağın geometrisi onun verimli olmasında önemli bir rol oynamaktadır. Dönemeçlerde yetersiz yarıçaplar dönen taşıtların hızını azaltıp ve sonuçta trafik tıkanıklığı gibi sorunlar ortaya çıkmaktadır. Ayrıca, kavşaklarda sürücülerin güvenle kavşağı terk etmeleri için görüş üçgenlerinin sağlanması söz konusu olup iki kol arasındaki açı yeterli olması gerekmektedir. Sağa ve sola dönüş cepleri dönen akımı düz giden akımdan ayırıp kapasiteni artarlar. Yuvarlak ada ise adanın yarıçapı onun verimliliğin etkilemektedir. Kolların eğimi bir başka parametre olup kavşakların performansın değişirler.

Yüksek hacimde çalışan kavşaklarda tüm araçlar kavşak alanın aynı zamanda kullanamazlar. Bu yüzden araçlar sırayla alanı kullanıp belirli sürede kavşağı boşaltırlar. Bu amaç doğrultusunda trafik ışıkları ve sinyalli kavşaklar geliştirilmiştir. 1868 itibariyle semaforlar ilk kez Londra da trafik ışığı olarak çalışmaya başladılar. Amerika da ilk kez trafik ışıkları akımları sırayla kavşaktan geçirip ve bu noktalarda gerçekleşen trafik kazalarının azaltması hedef ile yapılmıştır. Trafik ışıkların zaman ayarlamaları ışıklı kavşakların verimli çalışmaların sağlamaktadır.

Sinyalli kavşaklarda kapasite yeşil süreyle (Araçların harekete geçtikleri süre) bağlı olup kapasitenin artması için optimize edilmeleri gerekmektedir. Bu amaç doğrultusunda çeşitli yöntemler geliştirilmiştir. Örneğin İngiliz (Webster), Avustralya (Akçelik) ve Amerikan (HCM) yöntemleri bu hedefi gerçekleştirmesi doğrultusunda çok genel kullanılan yöntemler olmaktalar. Bunu yanı sıra son yıllarda bilgisayar paket programlarda piyasaya sunulmaktadır. PTV VISSIM bu programların birisi olup trafik problemlerin çözmeye alınmışlar. Işıklı kavşaklarda sinyal devre süresi bu bölgenin verimli çalışmasında önemli bir etkisi olmaktadır. Bu devre süresi ne çok fazla ve nede çok kısa olmalıdır. Ortalama bir değer devre süresi için uygulanması gerekmektedir. Devre süresinin çok kısa olduğu takdirde fazın bir saat içerisinde sık değişiminden dolayı ortalama gecikme taşıt başına artıp kavşağın

performansın düşürecektir. Devre süresi çok uzun ise bu kez taşıtların geçiş hakkı alma sırası uzanmaktan dolayı gecikmeler artacaktır.

Trafik ışıkları dört gruba ayrılmaktalar. Sabit zamanlı ışıklar, trafikle değişen ışıklar, trafik ile adapte olan ışıklar ve öncelikli ışıklar bu dört grupta yer alıyorlar. Sabit zamanlı ışıklar birbirine daha sık yerleşen kavşaklarda ve trafik hacminin gün ve hafta boyunca sabit olduğu durumlarda kullanmaları tavsiye edilmektedir. Bu ışıkların devre ve yeşil süreleri sabit olup gün boyunca değişmez.

Trafikle değişen ışıklar ise trafiğin hacmin detektör kullanarak ölçüp ve bu veriler ışığında çalışmaya devam ederler. Trafik ile adapte olan ışıklar ise yukarıdaki trafiği ölçüp ve bu verileri kullanarak bir algoritma oluştururlar. Bu algoritma neticesinde akım aşağısındaki trafiği öngörüp ne zaman ve hangi konumda olduğunu anlar. Bu veriler ışığında akım aşasında olan sinyallerin yeşil sürelerini ayarlayıp ve kavşağın daha doğru çalışmasına destek olar.

Öncelikli ışıklar toplu taşıma araçların trafikte yaşadıkları gecikmelerin azaltması amaç ile geliştirilmiştir. Bu sistemde otobüs ve trafik ışığın arasında haberleşme kurulup sinyal süreleri otobüse öncelik verme hedef ile otomatik olarak ayarlanır. Öncelik vermek birkaç çeşitli yöntem ile yapılmaktadır. Örneğin bir faz içerisinde yeşil süresini uzatmayla ve ya faz sırasını değişerek bu hedefe ulaşırlar.

Son zamanlarda karmaşık ulaştırma sorunların çözme amaç ile çeşitli bilgisayarda kullanıla bilir trafik benzetim programlar geliştirilmiştir. Benzetim programları trafik mühendislerine çok çeşitli ulaştırma modellerinin sınamasını sağlar ve neticede hiçbir sahada yapılması gereken pahalı denemelere ihtiyaç duyulmaz. Ayrıca benzetim programları ulaştırma ağlarının nasıl çalıştığını görsel bir şekilde sunmakta olup ulaştırma sistemlerinin sorunların daha iyi bir şekilde gösterir. Benzetim programlarının birisi PTV VISSIM olup trafik mühendis lığında çok önem taşımaktadır. Bu program mikroskobik bir program olup çeşitli trafik senaryolarını çok gerçekçe ve detaylı bir şekilde sahada uygulamadan önce sunar. VISSIM çok detaylı ve karmaşık modelleri analiz yapmaktadır. Ayrıca diğer trafik benzetim programlarına göre daha güçlü olup iki ve üç boyutlu filmler yapmaktadır. Işıklı Kavşak performansın ölçmesi için çeşitli ölçütler geliştirilmiştir. Örneğin, ortalama taşıt başına gecikme, durma sayısı ve kuyruk uzunluğu temel performans ölçütleri olarak tanımlanmaktadır. Yakıt tüketimi, kirletici sürümü ve masraf ikinci bir ölçüt olarak kullanılmaktadır.

Önümüzdeki çalışmada ortalama taşıt başına gecikme seç timiz örnek kavşakların çalışma durumunu yansıtmak için kullanılmaktadır. Gecikme sürücünün arzu ettiği ve gerçekte yaşadığı yolculuk zamanların arasındaki fark olarak tanımlanır. Bu çalışmanın yapılmasında İstanbul'daki Pendik mahallesinde olan iki dört kollu ışıklı kavşak örnek olarak alınmıştır. Biran önce 7:00-10:00, 12:00-14:00 ve 16:30-19:30 Saatlar arasında kamera ile çekim yapılmıştır. Sonra kamera görüntüleri bilgisayar ortamına aktarılıp ve bunlar üzerinden türel ayrımı yaparak 15'r dakikalık zaman dilimleri halinde her yöne giden trafik hacimleri belirlenmiştir. Bu bilgiler ışığında her kol da ve her akım için en fazla taşıt bulunan saat zirve saat olarak tanımlanmıştır. Sonra, zirve saat faktörü (ZSF) zirve saat hacmin akım değerine bölerek her kol için elde edilmişte.

Zirve saat faktörün kullanarak her kol için düzeltilmiş hacim hesaplanmıştır. Sonunda bu akım değerleri VISSIM'e aktarılıp ve oluşturdumuz benzetim model bu verilere göre kalibre edilmiştir. Kalibrasiyon süreci modelin gerçeke yakın çalışmasın sağlar ve sonuçta elde edilmiş sonuçlara daha fazla güvenile bilir. Bu çalışmada kalibrasiyon araç sayısına göre tüm kollarda yapılıp ve benzetim modelin gerçeğe ne kadar yaklaştığı GEH değeri ile ölçülmektedir.

Benzetim kalibrasiyonu yapıldıktan sonra elde edilen akım değerleri kayıt edilip ve bu çalışmanın tüm yöntemlerinde akım değeri olarak kullanılmaktadır.

VISSIM'de elde edilen veriler ışığında Amerikan (HCM) ve Avusturalya (Akçelik) yöntemlerin kullanarak seçtimiz kavşakların performansı belirlrnmektedir. Aslında genel Avustralya ve HCM yöntemleri VISSIM'in dinamik yöntem ile karşılaştırılmıştır. Sonunda bu yöntemlerden elde edilen sonuçlar birbiri ile karşılaştırılmış ve olası nedenler tartışılmıştır.

1. INTRODUCTION

The history of incorporating signal controls for increasing the efficiency of transportation networks is discussed in section 1.1. The aim of this study and methods utilized attaining this goal is presented in section 1.2. Finally, Section 1.3 and section 1.4 include the scope of thesis and its organization respectively.

1.1 Brief Background

As early as the 1868, the manually operated semaphores first used in London as traffic control signals. In order to decrease the number of accidents in intersections the first traffic signals were used in the United States by alternatively assigning right of way. Traffic signals play an important role in the transportation network and dissatisfaction of public will occur in the case of inefficient operating of them. It is estimated that many of these signals could be improved by updating equipment or by simply adjusting and updating the timing plans. Inefficient use of signal controls will result in significant amount of delay on urban arterials. Traffic signal optimization is one of the most cost effective ways to improve traffic flow and is one of the most basic strategies to help mitigate congestion. Upon installing the traffic lights, they are often not proactively managed. Maintenance activities are frequently delayed or canceled due to many reasons like the shrinking in budgets and staffs. More than half of the signals in North America are in need of repair, replacement, or upgrading [1].

1.2 Motivation, Problem Statement, and Objectives

Traffic signals play a significant role in the transportation networks. When set without the aim of optimizing them, traffic signals might cause the inefficient use of the transportation systems. The side effects of such circumstances could be the increase in travel times, fuel consumption, air pollution, frustration for the public and so forth.

The aim of this study is the modeling and performance evaluation of signalized intersections with specific reference to a case study. For this purpose, two four-legged signalized intersections with similar geometric but different traffic flow characteristics have been analyzed. Traffic flow measures at sites are derived from camera recordings that are obtained within 7:00 - 19:30 period on weekdays. Peak hour volumes for conflicting flows are obtained accordingly. In order to figure out the considerable effect of heavy vehicle composition, the relevant percentages have been computed for all approaches.

In the purpose of modeling signal control for the selected intersections three approaches, i.e., the conventional methods of Australian (Akcelik) and USA (HCM 2000) and the dynamic approach of VISSIM software are utilized in the analyses. Differences between the obtained results of these two separate methods have been considered. The performances of incorporated signaling approaches are comparatively evaluated following the calibration of selected intersections' models with site data. The results of the study point out the differences, which are, exist between these three methods and their behavior and sensitivity on different traffic flow conditions.

1.3 Scope of Thesis

The aim of this study is the evaluation of four-legged signalized intersections with reference to two case studies in Pendik area of Istanbul city of Turkey. For this purpose, the average delay per vehicle due to signal control was designated as the measure of performance evaluation. To evaluate the performance of the case studies Australian (Akcelik), USA (HCM 2000), and PTV VISSIM software methods were incorporated. The performance of each approach was evaluated using these three methods. Finally, the differences, which exist among the results obtained from above-mentioned methods, were discussed and the conclusions were discussed.

1.4 Thesis Organization

The thesis is organized into six chapters that the study background, objectives and scope of the thesis are described in the first chapter. This chapter presents a general view on the process with which the thesis has been developed. The second chapter deals with intersection fundamentals and fundamentals of signal operations.

Moreover, basic definitions are discussed in this chapter. The third chapter involves literature review on evolution of traffic control systems and signal timing optimization methods. In addition, the incorporation of simulation software on traffic engineering has been discussed in this chapter. The fourth chapter discusses various methods that have been incorporated for conducting this study. In continue the case study is discussed in the fifth chapter during which the description of area and data, and numerical implementation is presented. Moreover, the comparative evaluation is discussed at the end of this chapter. Chapter 6 is the last chapter that embodies the research results and conclusions.

2. FUNDAMENTALS OF INTERSECTION MODELLING AND OPERATIONS

Sections 2.1 and 2.2 present a general discussion on intersection and signal timing operations. Important definitions about intersections and signal timing, effects of intersection geometry on its efficiency and different kinds of conflicting traffic flows are presented in these sections. Section 2.3 includes discussions about evolution of traffic control systems, and signal timing optimization methods are presented in section 2.4.

2.1 Intersection Fundamentals

In the following sections, basic definitions, intersection geometry and characteristics of conflicting flows have been discussed.

2.1.1 Basic definitions

The area in which two or more roads cut one another and different traffic movements in various directions want to use this space at the same time is called intersection. Intersections are very significant points in a transportation network because diverse traffic movements occupy this area concurrently. In addition, pedestrians try to cross these areas to reach their desired destination. Thereby, the traffic management in intersections plays a serious task in order to operate them in an efficient way and reduce the possible accident risks. An inappropriate design will cause high accident rates, delays and travel times and as a result the overall network performance will decrease. Some important definitions should be described before onset of the discussions as follows:

a- Capacity:

The maximum sustainable flow rate at which vehicles or persons reasonably can be expected to traverse a point or uniform segment of a lane or roadway during a specified time period under given roadway, geometric, traffic, environmental, and control conditions; usually expressed as vehicles per hour, passenger cars per hour, or persons per hour.

b- Flow Rate:

The equivalent hourly rate at which vehicles, bicycles, or persons pass a point on a lane, roadway, or other traffic way; computed as the number of vehicles, bicycles, or persons passing the point, divided by the time interval (usually less than 1 hour) in which they pass; expressed as vehicles, bicycles, or persons per hour.

c- Level of Service (LOS):

A qualitative measure describing operational conditions within a traffic stream, based on service measures such as speed and travel time, freedom to maneuver, traffic interruptions, comfort, and convenience.

d- Volume:

The number of persons or vehicles passing a point on a lane, roadway, or other traffic-way during some time interval, often 1 h, expressed in vehicles, bicycles, or persons per hour [2].

2.1.2 Intersection geometries

The efficiency and safety of an intersection is depended on its overall geometry. Pedestrians are often crossing lanes of a route, whereas vehicular traffic is using the travel lanes provided at the intersection. The number of lanes included in an approach has a significant impact on the capacity of the intersection and as a result the ability for signal timing of that intersection efficiently. For instance, it is obvious that an approach with two lanes has a higher capacity than that of with one lane only. With increasing the number of lanes in an approach, however, the time which is required for pedestrians to cross the leg will also increase and due to growth in clearance time some of the increase in capacity will also be offset.

Geometry details of an intersection have a significant influence on signal timing. The size and geometry of the intersection, coupled with the speed of approaching vehicles and walking pedestrians, affect vehicular and pedestrian clearance intervals, which in turn have an effect on the efficiency of the intersection's operation. For instance, the angle at which two routes intersect influence the length of crosswalks and thus pedestrian clearance time. As shown in Figure 2.1, with decreasing the angle between two roadways from 90 degrees to 60 degrees, the length of crosswalk increase almost 3.5 meters which will increase the pedestrian clearance time. Regardless of the angle of skew, appropriate places should be chosen for signal heads for providing good visibility for approaching vehicles [1].

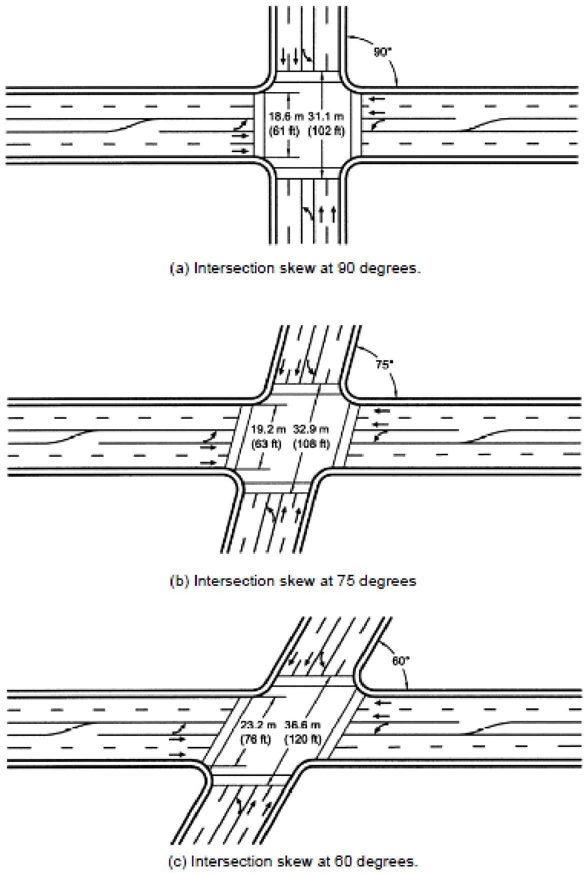

Figure 2.1 : Relationship among intersection skew and crosswalk length [1].

For improving the safety of intersections the following conditions in intersection designs should be considered:

1. The skew of an intersection (the angle at which two roadways intersect) should be nearly 90 degrees.
2. The number of legs in an intersection should be constrained to four legs.
3. Intersection should not include horizontal and vertical curves.
4. Adequate horizontal and vertical traffic signs should be used.
5. Sufficient stopping sight distance and sight triangles should be provided [3].

2.1.3 Characteristics of conflicting traffic flows

The flows enter to the intersection from different approaches conflict with one another due to travel direction changing. These points are called conflicting points, and predominantly accidents are seen in these points. Therefore, a suitable intersection designing will decrease conflicting points that will lead to low degree of accidents. In intersections, there are two different types of conflicting points,

1. Vehicle-Vehicle conflicting points

2. Vehicle-Pedestrian conflicting points [4].

Vehicle-Vehicle conflicting points are characterized with three different types as follows,

1. Diverge 
2. Merge 
3. Crossing 

Figure 2.2 illustrates these different conflicting points in a four-legged intersection as well as a roundabout.

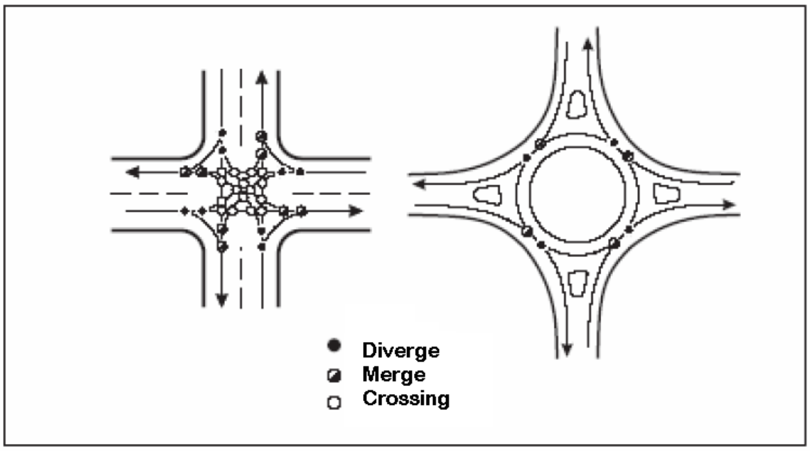

Figure 2.2 : Diverge, merge and crossing conflicting points in an intersection and a roundabout [3].

Vehicle – Pedestrian crossing points as well as Vehicle – Vehicle crossing points are obvious in Figure 2.3.

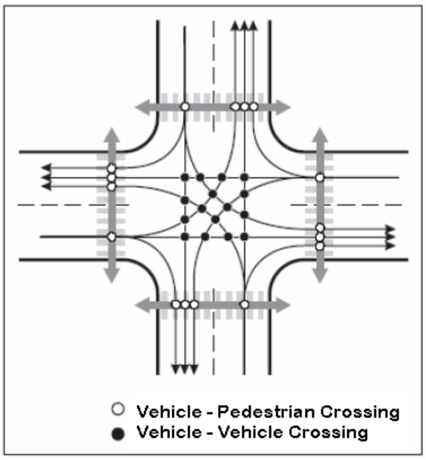

Figure 2.3 : Vehicle – pedestrian and vehicle – vehicle crossing points [3].

2.2 Fundamentals of Signal Operations

In the following sections, basic definitions, performance measures and performance analyses have been discussed.

2.2.1 Basic definitions

Initially, this section presents some important definitions, which are essential for understanding the signal operations.

a- Saturation flow rate:

The equivalent hourly rate at which previously queued vehicles can traverse an intersection approach under prevailing conditions, assuming that the green signal is available at all times.

b- Delay:

The additional travel-time experienced by a driver, passenger, or pedestrian.

c- Cycle:

A complete sequence of signal indications.

d- Cycle length:

The total time for a signal to complete one cycle.

e- Phase:

As shown in Figure 2.4, the part of the signal cycle allocated to any combination of traffic movements receiving the right-of-way simultaneously during one or more intervals.

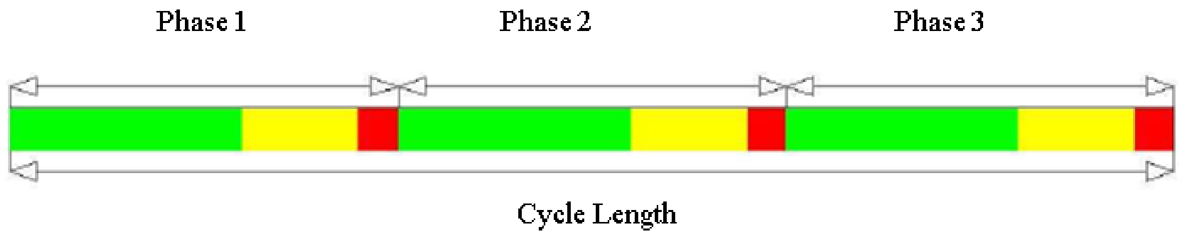

Figure 2.4 : Signal phasing diagram [5].

f- Clearance lost time:

The time, in seconds, between signal phases during which an intersection is not used by any traffic.

g- Start-up lost time:

The additional time, in seconds, consumed by the first few vehicles in a queue at a signalized intersection above and beyond the saturation headway, because of the need to react to the initiation of the green phase and to accelerate.

h- Lost time:

The time, in seconds, during which an intersection is not used effectively by any movement; it is the sum of clearance lost time plus start-up lost time.

i- Effective green time:

The time during which a given traffic movement or set of movements may proceed; it is equal to the cycle length minus the effective red time.

j- Lane group:

A set of lanes established at an intersection approach for separate capacity and level-of-service analysis.

k- Flow ratio:

The ratio of the actual flow rate to the saturation flow rate for a lane group at an intersection.

l- Critical lane group:

The lane groups that have the highest flow ratio for a given signal phase.

m- Isolated intersection**:**

An intersection at least 1.6 km from the nearest upstream signalized intersection.

Figure 2.5 describes the typical flow rate at a signalized movement and how vehicular traffic passes through intersection. The horizontal and vertical axes depict the signal display and flow rate respectively. During the red indication vehicles arrive and form a queue and there is no flow. Upon turning red light to green indication it takes a few seconds for the first driver to understand this change and start to drive. The next vehicles also take some time to accelerate. This is defined as the start-up lost time or start-up delay and is commonly assumed to be approximately 2 seconds. After approximately the fourth vehicle, flow rate stabilize at the maximum amount called saturation flow rate. This condition continues until the last vehicle departs the intersection and queue disappears. Upon termination of the green indication, some vehicles continue to pass through the intersection during the yellow change interval; this is known as yellow extension. The usable amount of green time, that is, the duration of time between the end of the start-up delay and the end of the yellow extension, is referred to as the effective green time for the movement. The unused portion of the yellow change interval and red clearance interval is called clearance lost time [1].

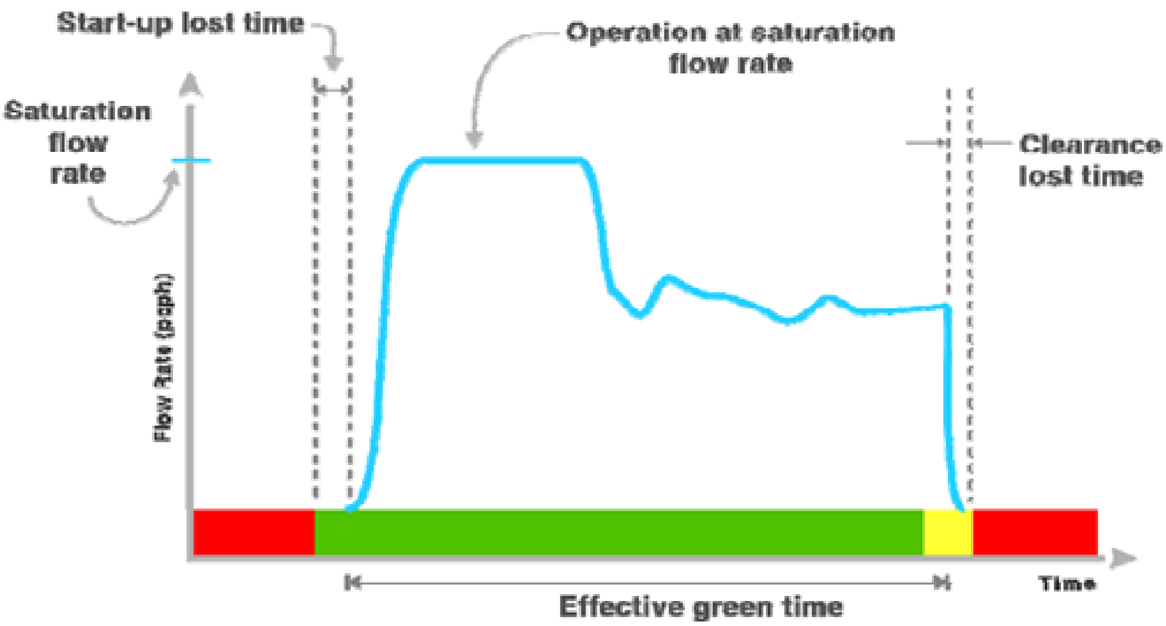

Figure 2.5 : Typical flow rates at a signalized movement [1].

2.2.2 Performance measures

There are two primary parameters for evaluating the performance of the individual intersections, namely, delay and queue length.

- Control delay

Delay is defined as “the additional travel time experienced by a driver, passenger, or pedestrian.” [2]. Control delay is due to any control devise like signal heads and stop signs plus the time decelerating to a queue, waiting in queue, and accelerating from a queue [1].

Table 2.1 : Relationship between the amount of control delay per vehicle and LOS [2].

LOS	Control delay per vehicle (s/veh)
A	≤10
B	>10-20
C	>20-35
D	> 35-55
E	>55-80
F	>80

- Queue length

Queue length is a measurement that considers the physical space vehicles will occupy while waiting to proceed through an intersection. It is usually used to calculate the length of turning lengths and whether the vehicles from one intersection will physically spill over into an adjacent intersection. Several queue length estimations are commonly used with signalized intersections. Average queue and 95th-percentile queue are commonly estimated for the time-period for which the signal is red. However, it is sometimes useful to include the queue formation that occurs during green while the front of the queue is discharging and vehicles are arriving at the back of queue. Queues measured in this way are often noted as average back of queue or some percentile of back of queue [1].

2.2.3 Performance analysis

The most predominant methods for determining the performance of an isolated intersection encompasses Australian (Akcelik), and US (HCM) Method. Moreover, the method used in PTV VISSIM simulation software is presented in this thesis. All above-mentioned approaches will be discussed in detail in sections 4.1, 4.2 and 4.3.

2.3 Evolution of Traffic Control Systems

A traffic control signal is defined as any highway traffic signal by which traffic is alternatively directed to stop and permitted to proceed. Traffic signals operate in either pre-timed or actuated mode or some combination of the two [1].

2.3.1 Pre-timed signal control

Pre-timed signal control is ideally suited to closely spaced intersections where traffic volumes and patterns are consistent on a daily or day-of-week basis. Such conditions are often found in downtown areas. They are also better suited to intersections where three or fewer phases are needed. Pre-timed control has several advantages. For example, it can be used to provide efficient coordination with adjacent pre-timed signals, since both the start and end of green are predictable. Also, it does not require detectors, thus making its operation immune to problems associated with detector failure. Finally, it requires a minimum amount of training to set up and maintain. On the other hand, pre-timed control cannot compensate for unplanned fluctuations in traffic flows, and it tends to be inefficient at isolated intersections where traffic arrivals are random. Modern traffic signal controllers do not explicitly support signal timing for pre-timed operation, because they are designed for actuated operation. Nevertheless, pre-timed operations can be achieved by specifying a maximum green setting [1].

2.3.2 Traffic actuated signal control

Traffic actuated signal controls fall into two categories, namely, semi-actuated and fully-actuated signal controls, and are discussed in following sections.

2.3.2.1 Semi-actuated signal control

Semi-actuated control uses detection only for the minor movements at an intersection. The phases associated with the major-road through movements are operated as "non-actuated." That is, these phases are not provided detection information. In this type of operation, the controller is programmed to dwell in the non-actuated phase and, thereby, sustain a green indication for the highest flow movements (normally the major street through movement).

The signal normally will remain in the major street phase except there is a conflicting call.
Semi-actuated control is most suitable for application at intersections that are part of a coordinated arterial street system. Semi-actuated control may also be suitable for isolated intersections with a low-speed major road and lighter crossroad volume.
Semi-actuated control has several advantages. Its primary advantage is that it can be used effectively in a coordinated signal system. Also, relative to pre-timed control, it reduces the delay incurred by the major-road through movements (i.e., the movements associated with the non-actuated phases) during periods of light traffic. Finally, it does not require detectors for the major-road through movement phases and hence, its operation is not compromised by the failure of these detectors.
The major disadvantage of semi-actuated operation is that continuous demand on the phases associated with one or more minor movements can cause excessive delay to the major road through movements if the maximum green and passage time parameters are not appropriately set. Another drawback is that detectors must be used on the minor approaches, thus requiring installation and ongoing maintenance. Semi-actuated operation also requires more training than that needed for pre-timed control [1].

2.3.2.2 Fully-actuated signal control

Fully-actuated control refers to intersections for which all phases are actuated and hence, it requires detection for all traffic movements. Fully-actuated control is ideally suited to isolated intersections where the traffic demands and patterns vary widely during the course of the day. Most modern controllers in coordinated signal systems can be programmed to operate in a fully-actuated mode during low-volume periods where the system is operating in a "free" (or non-coordinated) mode. Fully-actuated control can also improve performance at intersections with lower volumes that are located at the boundary of a coordinated system and do not impact progression of the system. Fully-actuated control has also been used at the intersection of two arterials to optimize green time allocation in a critical intersection control method.
There are several advantages of fully-actuated control. First, it reduces delay relative to pre-timed control by being highly responsive to traffic demand and to changes in traffic pattern. In addition, detection information allows the cycle time to be

efficiently allocated on a cycle-by-cycle basis. Finally, it allows phases to be skipped if there is no call for service, thereby allowing the controller to reallocate the unused time to a subsequent phase.

The major disadvantage of fully-actuated control is that its cost (initial and maintenance) is higher than that of other control types due to the amount of detection required. It may also result in higher percentage of vehicles stopping because green time is not held for upstream platoons [1].

2.3.3 Traffic adaptive signal control

Adaptive traffic signal control is a concept where vehicular traffic in a network is detected at an upstream and/or downstream point and an algorithm is used to predict when and where the traffic will be and to make signal adjustments at the downstream intersections based on those predictions. The signal controller utilizes these algorithms to compute optimal signal timings based on detected traffic volume and simultaneously implement the timings in real-time. This real-time optimization allows a signal network to react to volume variations, which results in reduced vehicle delay, shorter queues, and decreased travel times. Adaptive systems can still have a profound impact even starting from well-designed baseline signal timings where:

- Traffic conditions fluctuate randomly on a day-to-day basis
- Traffic conditions change rapidly due to new or changing developments in land use
- Incidents, crashes, or other events result in unexpected changes to traffic demand
- Other disruptive events, such as preemption, require a response

The underlying reason for adaptive system performance is quite simple; when system conditions are stable, there is little need to modify the control parameters if they are set appropriately. In the situation where fixed parameters have not been chosen by thorough analysis, adaptive systems can find better traffic control parameters.

All adaptive control systems are driven by a similar conceptual process:

1. Collect data in real-time from sensor systems to identify traffic conditions
2. Evaluate alternative signal timing strategies on a model of traffic behavior
3. Implement the “best” strategy according to some performance metric
4. Repeat steps 1,2,3 again and again

Each adaptive system is distinguished by how it uses different components or approaches to these four key steps in the control of the traffic system [1].

2.3.4 Signal priority systems

Transit Signal Priority (TSP) is an operational strategy that is applied to reduce the delay transit vehicles experience at traffic signals. TSP involves communication between buses and traffic signals so that a signal can alter its timing to give priority to transit operations. Priority may be accomplished through a number of methods, such as extending greens on identified phases, altering phase sequences, and including special phases without interrupting the coordination of green lights between adjacent intersections. Ultimately, TSP has the potential to improve transit reliability, efficiency, and mobility. With TSP, there are two basic methods to adjust the signal timing at an intersection for an approaching bus: reducing the red time (red truncation) or extending the green time (green extension). Figure 2.6 illustrates these methods. Transit signal priority has a limited effect on signal timing because it adjusts to normal timing and logic to serve a specific vehicle type. The priority algorithm modifies the green allocation and may work within the constraints of coordination settings or maximum green. The NTCIP 1211 standard requires that priority allow the coordination logic to be maintained without a recovery or transition period after the priority request.

The most commonly reported benefits of using signal priority include reduced signal delay for transit vehicles and improved transit travel time [1].

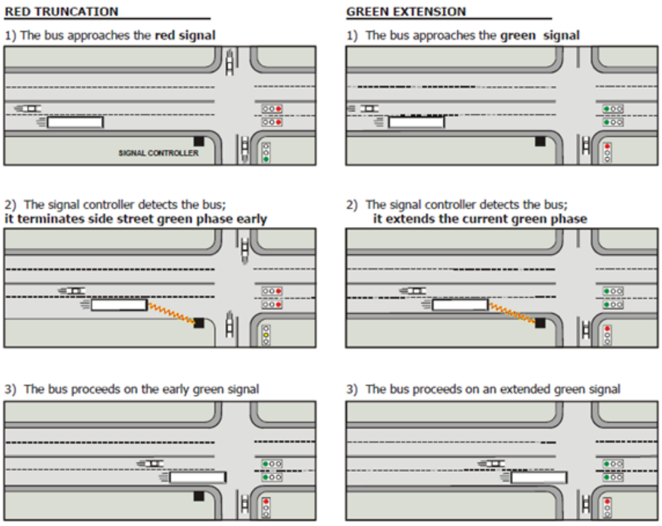

Figure 2.6 : Effect of TSP to adjust signal timing [1].

2.4 Signal Timing Optimization Methods

In the following sections, three different methods for optimization, namely, stochastic, heuristic and hybrid methods have been discussed.

2.4.1 Stochastic methods

In the past 50 years, since the development of digital computers, many investigators have studied the problem of numerically optimizing an objective function [7]. Stochastic optimization is the problem of finding a local optimum for an objective function whose values are not known analytically but can be estimated or measured. Classical stochastic optimization algorithms are iterative schemes based on gradient estimation. Proposed in the early 195Os, Robbins-Monro and Kiefer-Wolfowitz are the two most commonly used algorithms for unconstrained stochastic optimization [8]. These algorithms converge extremely slowly when the objective function is flat and often diverge when the objective function is steep. Additional difficulties include absence of good stopping rules and handling constraints. More recently, a stochastic optimization algorithm that converges under more general assumptions than these classical algorithms was proposed [9]. Better results with the Robbins-Monro algorithm were reported when applied in a finite-time singlerun optimization algorithm than when applied in a conventional way [10]. In the stochastic counterpart method (also known as sample path optimization) a relatively large sample is generated and the expected value function is approximated by the corresponding average function [11]-[12]. The average function is then optimized by using a deterministic nonlinear programming method. This allows statistical inference to be incorporated into the optimization algorithm which addresses most of the difficulties in stochastic optimization and increases the efficiency of the method [13].

2.4.2 Heuristic methods

Heuristic stems from the Greek word heuriskein which means to find or discover. It is used in the field of optimization to characterize a certain kind of problem-solving methods. There are a great number and variety of difficult problems, which come up in practice and need to be solved efficiently, and this has promoted the development of efficient procedures in an attempt to find good solutions, even if they are not optimal. These methods, in which the process speed is as important as the quality of the solution obtained, are called heuristics or approximate algorithms [14].

In addition to the need to find good solutions of difficult problems in reasonable time, there are other reasons for using heuristic methods, among which we want to highlight:

– No method for solving the problem to optimality is known.

– Although there is an exact method to solve the problem, it cannot be used on the available hardware.

– The heuristic method is more flexible than the exact method, allowing, for example, the incorporation of conditions that are difficult to model.

– The heuristic method is used as part of a global procedure that guarantees to find the optimum solution of a problem.

A good heuristic algorithm should fulfill the following properties:

– A solution can be obtained with reasonable computational effort.

– The solution should be near optimal (with high probability).

– The likelihood for obtaining a bad solution (far from optimal) should be low.

There are many heuristic methods that are very different in nature. Therefore, it is difficult to supply a full classification. Furthermore, many of them have been designed to solve a specific problem without the possibility of generalization or application to other similar problems [14].

2.4.3 Hybrid methods

Problems of combinatorial optimization have been the subject of intensive study by two communities of researchers: those in mathematical programming (often classified under "operations research") and those in constraint satisfaction programming (often classified under "artificial intelligence"). Recent years have seen increasing interaction between these two initially separate communities. The workshop will endeavor to help fostering this confluence. Traditional methods of combinatorial optimization come in two distinct flavors. Heuristic search algorithms (with variants such as simulated annealing, tabu search, genetic algorithms, neural networks) aim at quickly finding a satisfactory even if not necessarily optimal solution; where these methods leave off, the exact algorithms (typically some variation on the theme of branch-and-bound) proceed, often through much time-consuming work, to find a provably optimal solution. The more recent hybrid methods borrow ideas from both sources. Development of novel hybrid algorithms will be one of the themes of the workshop [15].

3. LITERATURE REVIEW

Control of traffic signals is by far the most common type of control at heavily trafficked intersections in urban areas. Inefficient use of the transportation system results when traffic signals are set without the aim of optimizing them. The byproducts of such situations include greater fuel consumption, increased vehicle emissions, increased travel time, higher accident rate, and less reliable services [16]. Traffic congestion leads to delays, decreasing flow rate, higher fuel consumption and thus has negative environmental effect. It is caused by irregular occurrences such as traffic accidents, vehicle disablements, and so on. Traffic congestion and flows can be modeled in various ways under different assumptions [17].

Traffic signal control is one of the most important traffic management techniques for coordinating traffic in orderly movements at intersections. Statistical data indicate that two thirds of urban vehicle miles traveled in the U.S. are on signal-controlled roadways [18]. A poorly designed traffic signal degrades traffic operations at intersections by lengthening vehicle delays, increasing rates of vehicle crashes, and introducing disruptions to traffic progression. Researchers at Oak Ridge National Laboratory estimated that poor signal timing caused 296 million vehicle hours of delay annually [18]. A recent evaluation on the more than 260 000 traffic signals in the U.S. rated the overall performance of these traffic signals at a very disappointing level of D. Hence, it is widely recognized that optimal traffic signal timing is one of themost cost effective solutions to mitigate congestion and enhance traffic mobility at signalized intersections [18].

Microscopic traffic-simulation tools are increasingly being applied to evaluate the impacts of a wide variety of intelligent transport systems (ITS) applications and other dynamic problems that are difficult to solve using traditional analytical models. The accuracy of a traffic-simulation system depends highly on the quality of the traffic-flow model at its core, with the two main critical components being the car-following and lane-changing models [19]. Car-following behavior, which describes how a pair of vehicles interact with each other, is an important consideration in traffic-simulation models. Understanding driving behavior is a key issue in evaluating model performance. A number of factors have been found to influence car-following behavior.These factors can be classified into two categories. The first category is individual differences consisting of age, gender, risk-taking behavior, driving skill, vehicle size, and vehicle performance characteristics. The second category is situational factors involving both the environment and the individual. These include factors such as time of day, day of week, and weather and road conditions. Individual factors include situations of distraction, impairment due to alcohol, drugs, stress and fatigue, trip purpose, and length of driving. Headways have been found to increase with driver age andmales are reported to choose shorter headways than females [19].

Advancements in traffic flow theory concepts and computing technology have led to widespread creation and use of traffic simulation models by traffic engineers and transportation planners during recent years. A simulation model is a computer program that uses mathematical models to conduct experiments with traffic events on a transportation facility or system over extended periods of time [2]. In the case of traffic simulation, the typical process of selecting a simulation model for use in a specific project varies based on the type of problem, available features, ability to replicate real-world conditions, ease of use, complexity, and cost [20]. Traffic simulation models are used in many cases to visually display analysis results and provide a system-wide analysis instead of isolated components (i.e. intersections, ramp merge / diverge, weaving sections, etc). Other common reasons for their use are to analyze complex or unique roadway geometries that other analysis programs cannot evaluate or to evaluate a combined system of arterial and freeway facilities [21].

4. METHODS USED TO OBTAIN INTERSECTION PERFORMANCES

In the following sections, different conventional methods for estimating the performance of the signalized intersections are discussed. All the computations in this study were conducted according to the approaches and processes described in the following sections.

4.1 Australian Method

Australian method is a movement-related approach, which carries out all analyses based on movement lost times instead of phase lost times and leads to the definition of the intersection lost time as the sum of critical movement lost times rather than the sum of phase lost times. Moreover, the process during which saturation flow rate is computed differs from the HCM method. The following sections describe the Australian method process in intersection performance analyses.

- **Movements and phases**

Each separate queue leading to intersection and characterized by its direction, lane usage and right of way provision is called a movement. The allocation of rights of way to individual movements is determined by phasing system. Signal phase is a state of signals during which one or more movements receive right of way. An example of signal phasing diagram and related phase-movement matrix is shown in Figure 4.1 and Table 4.1 respectively [22].

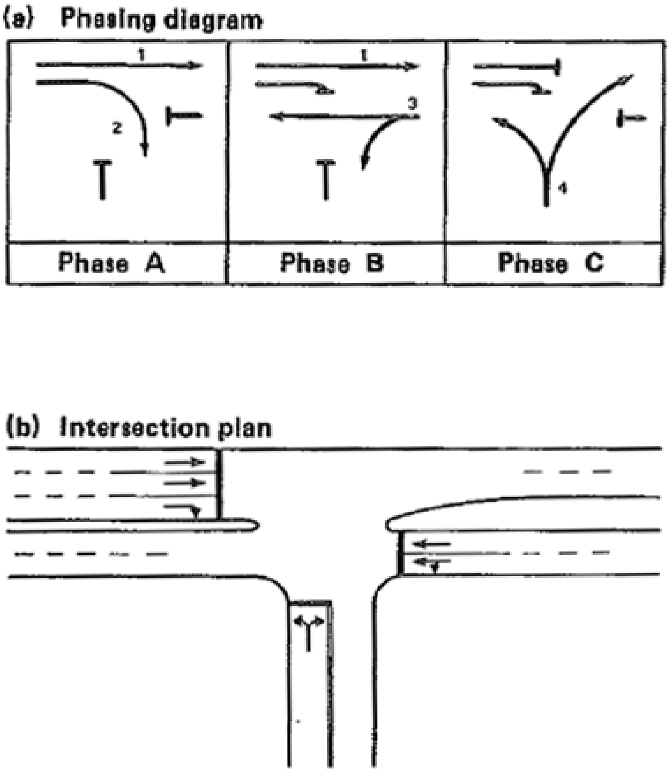

Figure 4.1 : An example of signal phasing diagram [22].

Table 4.1 : An example of phase-movement matrix [22].

Movement	Starting phase	Terminating phase
1	A	C
2	A	B
3	B	C
4	C	A

- **Signal cycle**

One complete sequence of signal phases is called signal cycle. An example of a cycle diagram is shown in Figure 4.2. The time from the end of the green period on one phase to the beginning of the green period on next phase is called intergreen time which consists of yellow and all red periods. From Figure 4.2, it is seen that the phase change times (F) are defined as phase termination times which occur at the end of green period and the intergreen is the initial part of the phase. Thereby, the green period starts at time (F+I). If the displayed green time for a phase is G, then the green time ends at time (F+I+G). This is the phase change time for the next phase. It is obvious that the sum of all intergreen and green times is the cycle time as shown in equation **(4.1)** [22].

$$C = \sum(I + G) \quad \textbf{(4.1)}$$

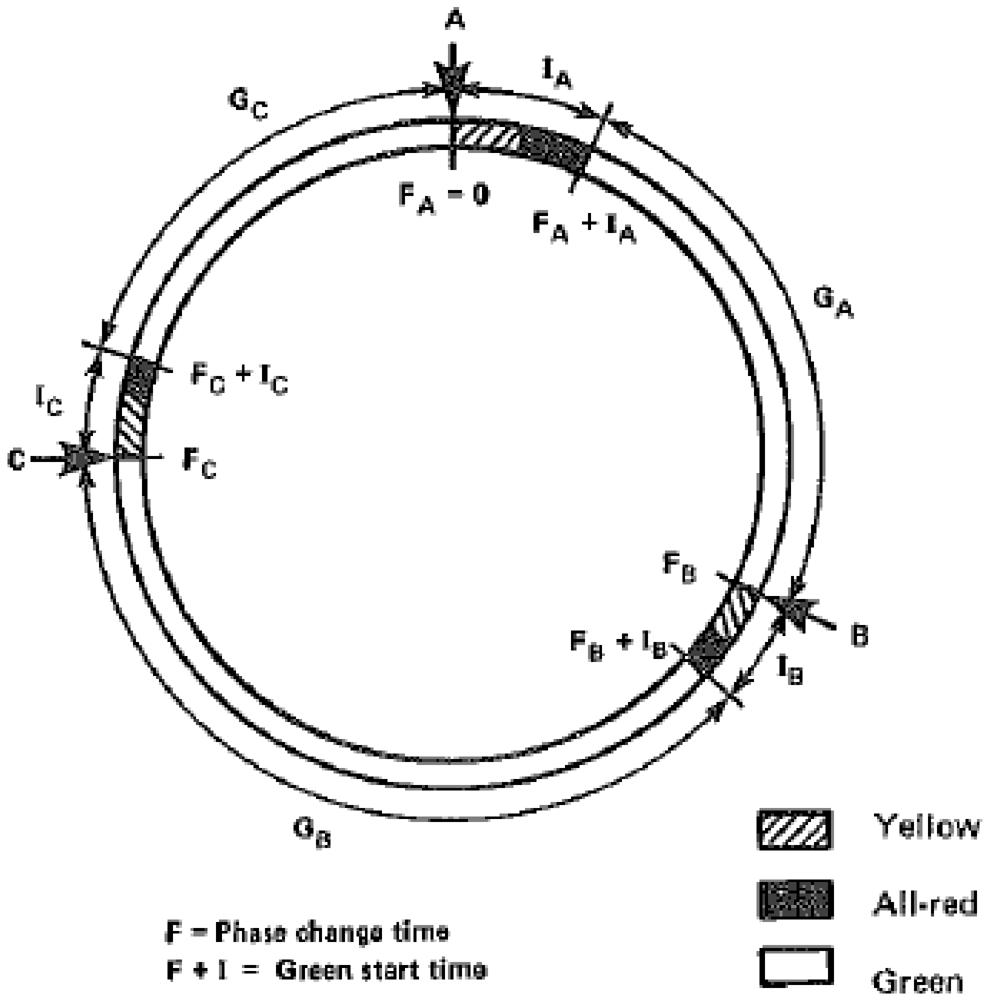

Figure 4.2 : A cycle diagram [22].

- **Movement characteristics**

The basic movement characteristics are illustrated in Figure 4.3 in relation to a corresponding phasing arrangement. The complete signal timing diagram for the example in Figure 4.1 is illustrated in Figure 4.4 [22].

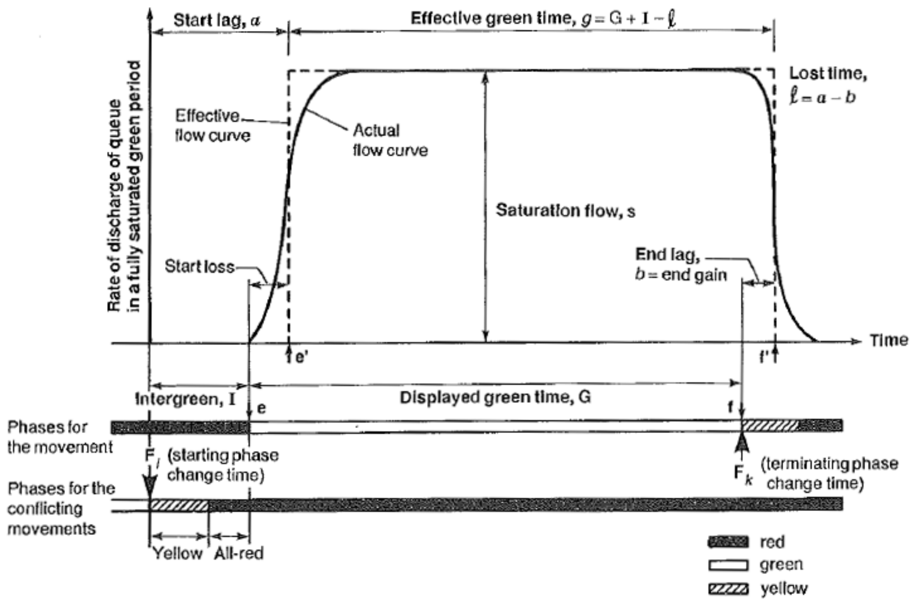

Figure 4.3 : Basic model and definitions [22].

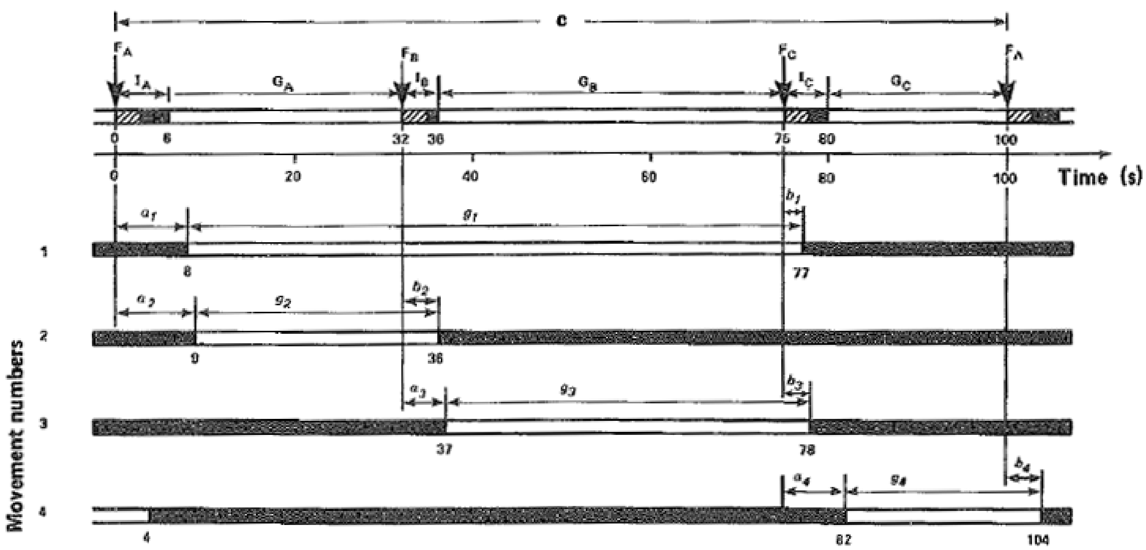

Figure 4.4 : Signal timing diagram for the example in Figure 4.1 [22].

- **Critical movements**

The movements which determine the capacity and timing requirements of intersection are called critical movements. All movements will have sufficient capacity provided that sufficient time is allocated to each critical movement. If all movements were non-overlap movements, there would be one critical movement per phase. This would be the movement which requires the longest time (g+l) in the phase. But, if there is overlap movements, identification of the critical movement will be complex. A method in order to determine the critical movements will be discussed in section [22].

- **Intersection lost time**

It is clear that $C = \sum(g + l) = \sum g + \sum l$ and an intersection lost time can be defined as $L = \sum l$ where the summation is for critical movements. In contrast with traditional methods, the intersection lost time is defined as the sum of critical movements lost times [22].

- **Capacity and degree of saturation**

The capacity of a movement at signalized intersections depends on the saturation flow (s) and the proportion of the cycle time which is effectively green for that movement (g/c) and is given by equation **(4.2)**.

$$Q = s\frac{g}{c} \tag{4.2}$$

Where:

Q: capacity of the movement (veh/h)

S: saturation flow rate (veh/h)

g: effective green time for that movement (s)

C: the cycle length (s)

The proportion of the effective green time to cycle time is called the green time ratio for the movement and is defined as equation **(4.3)**.

$$u = \frac{g}{c} \tag{4.4}$$

Where:

u: green time ratio

g: effective green time for that movement (s)

C: the cycle length (s)

Another useful movement parameter is the ratio of arrival flow (q) to saturation flow. This is called the flow ratio and is defined with equation **(4.5)**.

$$y = \frac{q}{s} \tag{4.5}$$

Where:

y: flow ratio

q: arrival flow (veh/h)

s: saturation flow rate (veh/h)

The degree of saturation of a movement is defined as the ratio of the arrival flow to the movement capacity. It is given by equation **(4.6)**.

$$X = \frac{q}{Q} = \frac{qc}{sg} = \frac{y}{u} \qquad \textbf{(4.6)}$$

The flow ratio (y) and green time ratio (u) can be defined as demand and supply respectively. Degree of saturation (x) is a parameter that relates these two parameters. To provide sufficient movement capacity the inequalities **(4.7)** and **(4.8)** should be met.

$$Q > q \ \ or \ \ X < 1 \qquad \textbf{(4.7)}$$

$$sg > qc \ \ or \ \ u > y \ \ or \ \ supply > demand \qquad \textbf{(4.8)}$$

By summing the above-mentioned inequalities for the critical movements the inequality **(4.9)** must hold for the intersection.

$$U = \sum u > Y = \sum y \qquad \textbf{(4.9)}$$

Where:

U : intersection green time ratio

Y: intersection flow ratio

Moreover, the intersection green time ratio (U) is equal to the ratio of the total green time to cycle time as equation **(4.10)**.

$$U = \frac{(C-L)}{C} \qquad \textbf{(4.10)}$$

Note that, $(C - L) = \sum g$ where $\sum g$ is the sum of critical movements green times.

The intersection degree of saturation (X) is defined as the largest (movement) degree of saturation. The minimum intersection degree of saturation for a given cycle time is obtained when the critical movements green times are proportional to the corresponding flow ratios [22]. This is an equal degree of saturation method [22]-[24]. This is obvious in equation **(4.11).**

$$x_1 = x_2 = \cdots = x_c = X \qquad \textbf{(4.11)}$$

Where x_1, x_2,…, x_c are the critical movement degrees of saturation. The intersection degree of saturation can also be computed without need to calculate green times. It is given by equation **(4.12)**.

$$X = \frac{Y \times C}{C - L} \qquad \textbf{(4.12)}$$

Where:

X: intersection degree of saturation

Y: intersection flow ratio

L: intersection lost time

The operating conditions that satisfy the inequality $X<1$ are usually referred to as undersaturated conditions [22].

- **Practical degrees of saturation**

In practice, the amount of arrival flow that is less than intersection capacity is desirable. In other words, degree of saturation should be less than 1 and it is called as practical degree of saturation. It is denoted with x_p and X_p for movement and intersection practical degree of saturation respectively. The use of x_p (or X_p)=0.9 is recommended as general-purpose value [22].

- **Critical movement identification**

Critical movements have the highest degrees of saturation, are used for determination of the capacity, and signal timing requirements of the intersection. The time allocated to a movement is the sum of effective green time, g, and lost time, l, and is given by equation **(4.13)**. The summation is for the phases during which the movement has right of way.

$$t = g + l = G + L = \sum_i^{k-1}(G + L) \qquad \textbf{(4.13)}$$

Where

i: starting phase

k: terminating phase

G: displayed green time for the movement (s)

L: intergreen time of starting phase (s)

t: movement time (s)

The required movement time can be calculated from equation **(4.12)**.

$$t = UC + l \qquad \textbf{(4.12)}$$

Where

t: movement time (s)

U: required green time ratio

C: cycle time (s)

Moreover, another parameter should be defined to control the sufficiency of the t and this is t_m which is given by equation **(4.13)**.

$$t_m = G_m + L = g_m + l \qquad \textbf{(4.13)}$$

Where

G_m: minimum displayed green time (s)

L: intergreen time (s)

g_m: minimum effective green time (s)

l: lost time for the movement under consideration (s).

Note that the constraint $t \geq t_m$ should be met. Otherwise, the movement required time (t) must be set equal to t_m ($t = t_m = G_m + L$). G_m is determined according to the minimum time that vehicles or pedestrians need to clear the intersection.

The required green time ratio is calculated to achieve maximum acceptable (practical) degrees of saturation, X_p, and is given by equation **(4.14)**.

$$U = y/X_p \qquad \textbf{(4.14)}$$

Where

U: required green time ratio

y: movement flow ratio

X_p: practical degrees of saturation

The critical movement identification method is based on the comparison of the required movement time (t) values. When movements are non-overlap, i.e. receive right of way during one phase only the identification of the critical movement is simple. But in the case of over-lap movements, condition gets a little complex [22].

- **Non-overlap movements**

The critical movement for a phase is the one with the largest required movement time (t). If the lost times for all movements in a phase are same, also assuming an equal degree of saturation solution, the critical movement can be identified as the one with the largest flow ratio (y) [22].

- **Overlap movements**

For an over-lap movement, the t value must be compared with the sum of the t values of non-overlap movements which represent the corresponding phases. The overlap movement is critical if its t value is the largest. Otherwise, the non-overlap movements representing the relevant phases are the critical movements [22].

- **The procedure**

The procedure is described by using an example phasing diagram shown in Figure 4.5.

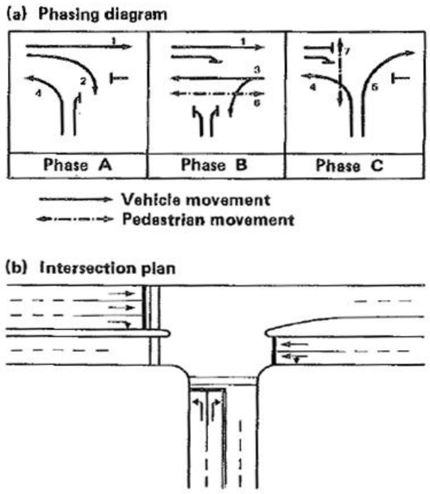

Figure 4.5 : Signal phasing diagram and intersection plan (an example) [22].

Step 1: Critical Movement Search Diagram

1. Draw the phase names as the nodes in order, e.g. A, B, C, A, in Figure 4.6.
2. Connect the nodes by links (movements) according to the start and end phase data.

3. If all movements are not exhausted using nodes A to A, extend the diagram adding the nodes B, C and etc. as required, e.g. movement 4 needs addition of node B in Figure 4.6.a.

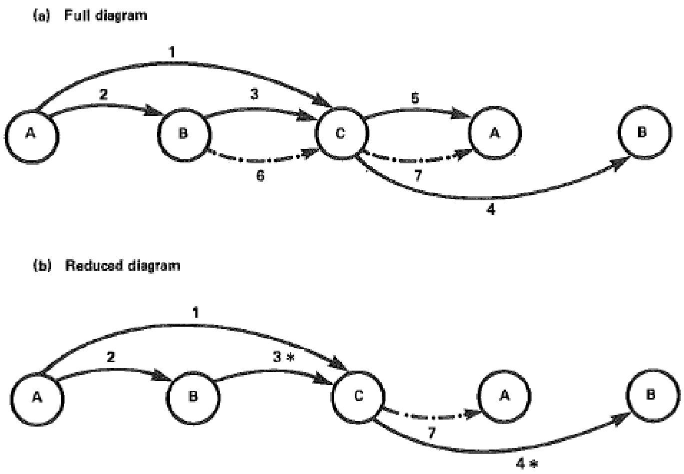

Figure 4.6 Critical movement search diagram [22].

Step 2: Non-Overlap Movements

Compare t values of the non-overlap movements in each phase, choose the movement with largest movement value and eliminate the others. For example, $t_3 > t_6$ in phase B and $t_7 > t_5$ in phase C, hence, eliminate movements 6 and 5 and prepare a reduced search diagram as shown in Figure 4.6.b. For a phasing system with non-overlap movements the critical movements are simply those left in the reduced diagram.

Step 3: Over-Lap Movements

Similarly, compare the t values of the over-lap movements which receive right of way during the same phases, i.e. have the same starting and terminating phase numbers, choose the movement with largest t value and eliminate others. Identify possible movement combinations which complete one cycle in the reduced search diagram. For example, (1 and 7) or (2,3 and 7) from node A to node A, and (3 and 4) from node B to node B. Consider a condition in which t_1=27, t_7=22, t_2=22, t_3=37 and t_4=60. So,

$T_{1,7} = t_1 + t_7 = 27+22= 49$

$T_{2,3,7} = t_2 + t_3 + t_7 = 22+37+22= 81$

$T_{3,4} = t_3 + t_4 = 37+60= 97$

Because $T_{3,4}$ is the largest required total time the critical movements are 3 and 4 [22].

- **Saturation flows**

Saturation flow concept is very significant in capacity and signal timing analyses of signalized intersections. The saturation flow is the maximum constant departure rate from the queue during the green period. Some of the important factors that influence the saturation flow are as follows.

a. Environment class defined according to the degree of interference to the free movement of vehicles
b. Lane type defined according to the characteristics of turning maneuvers
c. Lane width
d. Gradient
e. Traffic composition [22].

- **Estimation of saturation flows**

The following sections describe a method for calculating saturation flows.

- **The Method**

The following method is recommended for estimating saturation flows.

a. For each lane allocated to the movement choose a base saturation flow value from Table 4.2 which gives average saturation flows in through car unit per hour (tcu/h).
b. Adjust the base saturation flow value with various factors in order to obtain saturation flow in vehicle per hour for the particular movement.
c. Add lane saturation flows to determine the movement saturation flow.

The environmental classes are as follows.

Class A: Ideal or nearly ideal conditions for free movement of vehicles (both approach and exit sides), good visibility, very few pedestrians, almost no interference due to loading and unloading of goods, vehicles or parking turnover.

Class B: Average conditions: adequate intersection geometry, small to moderate numbers of pedestrians, some interference by loading and unloading of goods, vehicles and parking turnover.

Class C: Poor conditions, large numbers of pedestrians, poor visibility, interference from standing vehicles, loading and unloading goods, vehicles, taxis and buses and high parking turnover.

The lane types are as follows.

Type 1: Through lane: a lane which contains through vehicle only.

Type 2: Turning lane: a lane which contains any type of turning traffic (exclusive left turn or right turn lane, or a shared lane from which vehicles may turn left or right or continue straight through), Adequate turning radius and negligible pedestrian interference to turning vehicles.

Type 3: Restricted turning lane as for type 2 but turning vehicles are subject to small radius and some pedestrian interference.

Table 4.2 : Average saturation flows in through car units per hour for estimation by environment class and lane type [22].

Environmental class	Lane type		
	1	2	3
A	1850	1810	1700
B	1700	1670	1670
C	1580	1550	1270

- **Adjustment factors**

The base saturation flow values taken from Table 4.8 must be adjusted using equation **(4.15)**.

$$S = \frac{f_w f_g}{f_c} S_b \qquad \textbf{(4.15)}$$

Where

S: saturation flow rate in vehicle per hour (veh/h)

S_b: base saturation flow in through car units per hour (tcu/h)

f_w: lane width factor

f_g: gradient factor

f_c: traffic composition factor.

- **Lane width and gradient**

If the lane width is between 3 to 3.7 m, no adjustment is required. Otherwise, the equation **(4.16)** should be used.

$$f_w = \begin{cases} 1 & \text{for } w = 3 \text{ to } 3.7 \\ 0.55 + 0.14w & \text{for } 2.4 \leq w < 3 \\ 0.83 + 0.05w & \text{for } 4.6 \geq w > 3.6 \end{cases} \qquad \textbf{(4.16)}$$

Where

f_w : lane width factor

w: lane width (m)

In order to account for the effects of the gradient on saturation flow rate the equation **(4.17)** can be used.

$$f_g = 1 \pm 0.5(\frac{G_r}{100}) \qquad \textbf{(4.17)}$$

Where

f_g: Gradient factor

+ G_r: Percent gradient for downhill gradient

$-G_r$: Percent gradient for uphill gradient.

- **Traffic compositions**

Traffic composition factor is used to convert the saturation flow value in standard through car units to vehicles and is calculated using equation **(4.18)**.

$$f_c = \frac{\sum e_i \, q_i}{q} \qquad \textbf{(4.18)}$$

Where

q_i : flow in vehicles for vehicle-turn type i

q : total movement flow ($\sum q_i$)

e_i: through car equivalent of vehicle-turn type I ($\frac{t_{cu}}{veh}$).

The through car equivalent can be taken from Table 4.3.

Table 4.3 : Through car equavalents (t_{cu}/veh) for different types of vehicle and turn [22].

Vehicle Type	Through	Unopposed turn		Opposed turn
		Normal	Restricted	
Car	1	1	1.25	e_o
HV	2	2	2.5	e_o+1

The definitions of the vehicle and turn types used in Table 4.3 are given below.

- **Types of vehicles**

In the Australian method only two vehicle types are considered. Any vehicle with more than two axles or with dual tires on the rear axle is defined as a heavy vehicle (HV). All other light vehicles are defined as CARs.

- **Types of turn**

The effect of turning traffic (left or right) depends on whether it is an opposed movement (i.e. if it has to give way to a higher priority opposing movement), or it is an unopposed movement. The opposing movement may be a vehicle or a pedestrian movement.

a. Unopposed turn:

(i) Normal: This applies to both left-turning and right-turning vehicles and represents the conditions where the radius of curvature of turn is reasonably large and there is no (or very little) pedestrian interference.

(ii) Restricted: This applies to both left-turning and right-turning vehicles which are subject to a smaller turning radius and some interference by pedestrians.

b. Opposed turns: Its value depends on the opposed turn equivalent (e_0) and is calculated according to a method given below.

- **Opposed turns**

The opposed turn equivalent (e_0) should be calculated using equation **(4.19)**.

$$e_0 = \frac{0.5\, g}{s_u g_u + n_f} \tag{4.19}$$

Where

g : green time for the movement with opposed turns (s)

s_u : opposed turn saturation flow which is obtained from the Figure 4.7 (veh/s)

g_u : unsaturated part of the opposing movement green (s)

$s_u g_u$: number of turning vehicles per cycle which can depart during the period g_u

n_f : Number of turning vehicles per cycle which can depart after the green period from the shared lane.

The unsaturated part of the opposing movement green period can be calculated from equation **(4.20)**.

$$g_u = \frac{g - yc}{1 - y} \qquad \textbf{(4.20)}$$

Where

g : effective green time for the opposing movement

y : flow ratio for the opposing movement

C: cycle time

Note that the equation **(4.20)** should only be used in the case that the opposing movement is undersaturated, i.e. sg > qc [22].

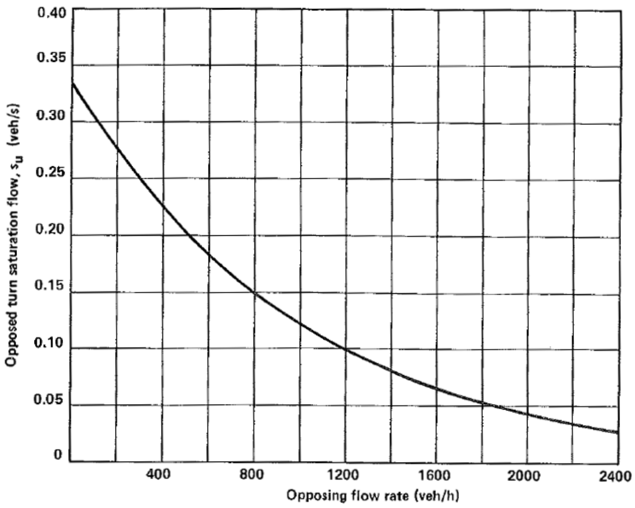

Figure 4.7 : Opposed turn saturation flow during the undersaturated part of the opposing movement green period [22].

- **Measures of performance**

There are many factors in order to evaluate the performance of an intersection. Delay, number of stops and queue length are significant parameters for determining the performance of an intersection. The basic concept of the overflow queue which is applicable to both undersaturated and oversaturated cases is discussed before presenting the formulas of delay, number of stops and queue length [22].

- **Overflow queue**

Overflow queues are due to oversaturation which may last only for a few signal cycles. The equation **(4.21)** has been derived for predicting average overflow queues in both undersaturated and oversaturated conditions at isolated fixed-time signals [22].

$$N_0 = \begin{cases} \frac{QT_f}{4}\left(z + \sqrt{z^2 + \frac{12(x-x_0)}{QT_f}}\right) & for\ \ x > x_0 \\ \text{Zero} & \text{Otherwise} \end{cases} \qquad \textbf{(4.21)}$$

Where

N_0 : Average overflow queue in vehicles (where there are several lanes of vehicles, this is the total number of vehicles queued in all lanes)

Q : Capacity in vehicles per hour

T_f : Flow period, i.e. the time interval in hours, during which an average arrival flow rate (q) persists

QT_f : The maximum number of vehicles which can be discharged during interval T_f

x : q/Q, degree of saturation

$z = x - 1$

x_0 : The degree of saturation below which the average overflow queue is approximately zero and is given by

$x_0 = 0.67 + {}^{sg}/_{600}$

- **Delay**

The total delay for a traffic movement at isolated fixed-time signals can be computed from equation **(4.22)** [22].

$$D = \frac{qc(1-u)^2}{2(1-y)} + N_0 x \qquad \textbf{(4.22)}$$

Where

D : total delay in vehicle-hours per hour or simply vehicles

qc : average number of arrivals in vehicles per cycle (q= flow in vehicles per second, c= cycle time in seconds) u: green time ratio

y: flow ratio

N_0 : average overflow queue in vehicles

If the average delay per vehicle is desired the equation **(4.23)** can be applied.

$$d = \frac{D}{q} \tag{4.23}$$

Where

D : total delay in vehicles

q : flow in vehicles per second

d: Average delay per vehicle.

- **Number of stops**

The average number of complete stops per vehicle is called the stop rate and denoted by (h). The stop rate for a movement at isolated fixed-time signals can be calculated from equation **(4.24**).

$$h = 0.9 \left(\frac{1-u}{1-y} + \frac{N_0}{qc}\right) \tag{4.24}$$

Where u, y, qc, N_0 are as equation (**4.22)**.

The number of complete stops per unit time (H) experienced by a movement with a flow rate q is given by the general equation **(4.25)**.

$$H = qh \tag{4.25}$$

Where

h : average number of complete stops per vehicle

q : flow rate in vehicles per unit time

H: number of complete stops per unit time [22].

- **Queue length**

The average number of vehicles in the queue at the start of the green period (N) and the maximum back of the queue (N_m) which is reached sometime after the start of the green period can be obtained using Equations **(4.26)** and **(4.27)** respectively.

$$N = qr + N_0 \tag{4.26}$$

$$N_m = \frac{qr}{1-y} + N_0 \qquad \textbf{(4.27)}$$

Where

N: average number of vehicles in the queue at the start of the green period

N_m : maximum back of the queue

q : arrival flow rate (veh/s)

r = c – g = effective red time in seconds

N_0 : average overflow queue in vehicles

y : Flow ratio [22].

- **Signal timing**

The method for choosing appropriate cycle length and green times are described in the following sections.

- **Cycle length**

In order to obtain efficient cycle length initially a cycle length called optimum cycle length should be calculated using equation **(4.28)**.

$$C_o = \frac{(1.4+k)L+6}{1-Y} \qquad \textbf{(4.28)}$$

Where

C_o : approximate optimum cycle time (s)

L : intersection lost time (s)

Y : intersection flow ratio

K : K/100 is the stop penalty parameter

The following values can be used as stop penalty parameter in equation,

K=0.4 for minimum fuel consumption

K=0.2 for minimum cost

K= 0 for minimum delay.

For determining the lower constraint of cycle length another parameter called practical cycle time has been formulated.The minimum cycle time which ensures that degree of saturation of all movements are below specified maximum acceptable degrees of saturation, $x < x_p$ is called the practical cycle time and is given by equation (**4.29)**.

$$C_p = \frac{L}{1-U} \tag{4.29}$$

Where

L : intersection lost time (s)

U : intersection green time ratio

A cycle time calculated using equation and equation can be used in practice. It is recommended that both C_o and C_p are calculated and a suitable cycle time between these two values is chosen for use. It should be noted that the selected cycle time should be below the maximum cycle time (C_{max}) which lies usually between 120 and 150 s [22].

- **Green times**

The movement green times for a specific cycle time are calculated in two steps:

1. Calculate the critical movement green times
2. Calculate the non-critical movement green times

For calculating the critical movement green times the equation (**4.30)** can be used.

$$g = \frac{c-L}{U} \times u \tag{4.30}$$

Where

g : critical movement green time (s)

c : cycle time (s)

u : movement green time ratio

U : intersection green time ratio

In order to calculate green times for non-critical movements there are three cases,

1. There are no overlap movement

In this case calculation of critical movement green times is sufficient to determine all non-critical movements green times and is calculated using equation (**4.31)**.

$$g = g_c + l_c - l \tag{4.31}$$

Where

g : non-critical movement green time (s)

g_c : critical movement green time (s)

l_c : critical movement lost time (s)

l : non-critical movement lost time

2. There are a single overlap movement where the overlap movement is critical

In this case the green times for non-overlap movements cab be calculated by treating the critical movement times as a sub-cycle time, $C^* = g_c + l_c$ where g_c and l_c are the critical movement green time and lost time. The non-critical movement green time can be calculated using equation **(4.32)**.

$$g = \left(\frac{c^* - L^*}{U^*}\right) \times u \tag{4.32}$$

Where

g : non-critical movement green time (s)

c^*: sub-cycle time (s)

U^*: sum of non-overlap movement green time ratios

u : non-critical movement green time ratio.

3. There are multiple overlap movements

In this case when non-overlap movements are critical, the non-critical overlap movement green time can be calculated from equation (**4.33)**.

$$g = \sum g_c + \sum l_c - l \tag{4.33}$$

Where

g : the non-overlap movement green time (s)

g_c : the critical non-overlap movement green time

l_c : the critical non-overlap movement lost time

l : the non-critical movement lost time [22].

4.2 US (HCM) Method

US (HCM) method gives a methodology for analyzing the capacity and level of service (LOS) of signalized intersections. The main objective of this methodology is determination of LOS for known or projected conditions. Capacity is evaluated in terms of the ratio of demand flow rate to capacity (v/c ratio), whereas LOS is evaluated on the basis of control delay per vehicle (in seconds per vehicle).

This method includes two limitations:

1. The potential impact of downstream congestion on intersection is overlooked.
2. Impacts of turn-pocket overflows on through traffic and intersection operation are not taken into account.

Figure 4.8 shows required input and the basic computation procedure. The primary output of the methodology is level of service (LOS) [2].

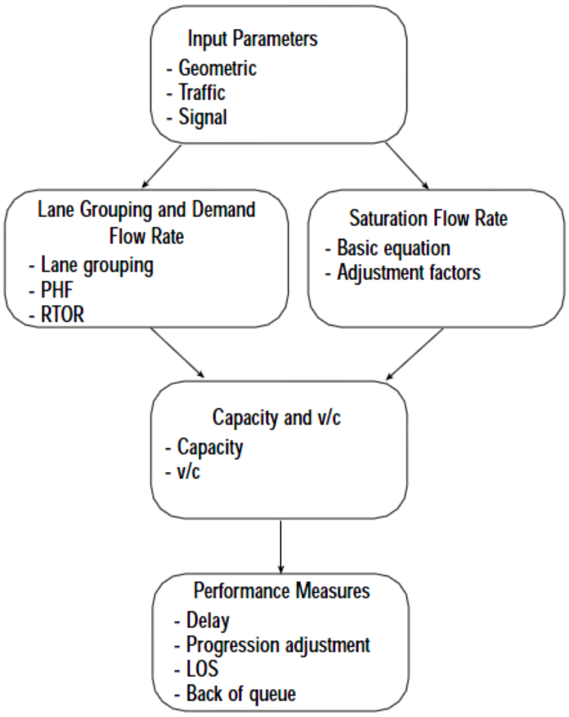

Figure 4.8 : Signalized intersection methodology [2].

- **Input parameters**

Input parameters are categorized into three main groups, namely, geometric conditions, traffic conditions and signalization conditions. The most important constituents of these three groups are shown in Table 4.4.

Table 4.4 : Input data needs for each analysis lane group [2].

Type of Condition	Parameter
Geometric conditions	Area type Number of lanes, N Average lane width, W (m) Grade, G (%) Existence of exclusive LT or RT lanes Length of storage bay, LT or RT lane, Ls (m) Parking
Traffic conditions	Demand volume by movement, V (veh/h) Base saturation flow rate, so (pc/h/ln) Peak-hour factor, PHF Percent heavy vehicles, HV (%) Approach pedestrian flow rate, vped (p/h) Local buses stopping at intersection, NB (buses/h) Parking activity, Nm (maneuvers/h) Arrival type, AT Proportion of vehicles arriving on green, P Approach speed, SA (km/h)
Signalization conditions	Cycle length, C (s) Green time, G (s) Yellow-plus-all-red change-and-clearance interval (intergreen), Y (s) Actuated or pretimed operation Pedestrian push-button Minimum pedestrian green, Gp (s) Phase plan Analysis period, T (h)

- **Geometric conditions**

Geometric specifications of an intersection are significant factors on performing the performance evaluation of that intersection. Area type, number of lanes, approach grades and so forth are the parameters that should be considered in capacity and level of service analyses [2].

- **Traffic conditions**

Traffic volumes for each approach of an intersection should be designated in vehicle per hour for the 15-min analysis period. In the case that the 15-min flow rates are unknown, they may be estimated using hourly volumes and peak-hour factors (PHF). The percentage of heavy vehicles (HV%) is a factor that accounts for the frictional effects of these vehicles on traffic movements according to their size and difficulties in driving them. The number of buses that stop at the intersection on each approach in order to pick up or discharge passengers should also be identified. If the local buses pass through intersection without any stop, they should be considered as heavy vehicles. Another decisive factor in performance evaluation of a signalized intersection is the quality of the progression. Arrival type (AT) is the factor that describes this characteristic. The phase during which vehicles arrive at the intersection is an important factor in the amount of delays experienced by the drivers. For instance, if most of the vehicles arrive to the intersection at the start of the green phase, they would obviously discharge the intersection with no delay. However, when reaching to the intersection at the start of the red phase, vehicles would experience maximum delay at the same intersection. equation **(4.34)** formulates these types of arrivals as a function of proportion of all vehicles in movement arriving during green phase (P), cycle length (C) and effective green time for movement or lane group (g_i). P may be estimated or observed in the field, whereas gi and C are computed from the signal timing. The value of P may not exceed 1.0.

$$R_p = \frac{P}{\frac{g_i}{C}} \qquad \textbf{(4.34)}$$

Where:

Rp : platoon ratio,

P : proportion of all vehicles in movement arriving during green phase,

C : cycle length (s), and

g_i : effective green time for movement or lane group (s) [2].

- **Signalization conditions**

All the information about signalization includes phase plan, cycle length, green times, red times, yellow times and full red times should be prepared for onset an analysis. Initially, the pedestrian minimum green time should be calculated in the case that the pedestrian volumes are high. The minimum green times for pedestrians are derived using equations **(4.35)** and **(4.36)**.

$$G_p = 3.2 + \frac{L}{S_p} + 0.81\frac{N_{ped}}{W_E} \quad for\ W_E > 3.0\ m \tag{4.35}$$

$$G_p = 3.2 + \frac{L}{S_p} + 0.27N_{ped} \quad for\ W_E \leq 3.0\ m \tag{4.36}$$

where

G_p: minimum green time (s),

L: crosswalk length (m),

S_p: average speed of pedestrians (m/s),

W_E: effective crosswalk width (m),

3.2 : pedestrian start-up time (s), and

N_{ped} : number of pedestrians crossing during an interval (p).

Computation of equations **(4.35)** and **(4.36)** lie in the fact that the 15th-percentile walking speed of pedestrians crossing a street is 1.2 m/s. Upon calculating the minimum pedestrian green times, this amount should be compared with the green times needed for vehicles to discharge the intersection [2].

- **Lane grouping**

For analyzing a signalized intersection, a method based on segmenting the intersection into lane groups that include either geometry of the intersection and distribution of traffic movements is recommended [2]. In other words, these lane groups describe the operation of the intersection. Lane groups are designated as follows.

a- An exclusive left-turn lane should be designated as a separate lane group.

b- An exclusive right-turn lane should be designated as a separate lane group.

c- On approaches with no exclusive turn lanes, all the lanes should be classified as a one lane group unless there are so many left-turns that act as an exclusive left-turn lane called de facto left-turn lane. If the proportion of left-turns in a shared lane equals 1, the shared lane should be considered a de facto left-turn lane.

Upon determining the lane group, whole group should be treated as a single entity even if there is more than one lane. Different types of lane groups have been illustrated in Table 4.5 [2].

Table 4.5 : Typical lane groups for analysis [2].

Number of Lanes	Movement by Lanes	Number of Possible Lane Groups
1	LT + TH + RT	① (Single-lane approach)
2	EXC LT TH + RT	②
2	LT + TH TH + RT	① OR ②
3	EXC LT TH TH + RT	② OR ③

- **Determining the flow rate**

For calculating the flow rate during peak 15-min, hourly volume (veh/h) and peak hour factor should be known. The flow rate is derived using equation **(4.37)**.

$$V_p = \frac{V}{PHF} \quad \textbf{(4.37)}$$

Where

vp : flow rate during peak 15-min period (veh/h),

V : hourly volume (veh/h), and

PHF : peak-hour factor.

The existing peak hour factor (PHF) is applied not only for all approaches of an intersection but also for all volumes during every interested hour. In converting the hourly volume to peak flow rates it is assumed that all approaches reach to their peak volume during the same 15-min period. Thereby, high estimates of control delay will result. Note that it is conservative if different PHF values are assumed for each movement [2].

- **Adjustment for right turn on red**

When right turn on red (RTOR) is permitted, the volumes turning right during the red phase should be subtracted from the right-turn volume. This reduction should be done before deriving flow rate and commence to analyze capacity and LOS.
The number of vehicles able to turn right depends on several parameters as follows:
• Approach lane allocation (shared or exclusive right-turn lane),
• Demand for right-turn movements,
• Sight distance at the intersection approach,
• Degree of saturation of the conflicting through movement,
• Arrival patterns over the signal cycle,
• Left-turn signal phasing on the conflicting street, and
• Conflicts with pedestrians.
If field data are not available, it is recommended to ignore RTOR, except in special cases. Free-flowing right turns that are not under signal control should be removed entirely from the analysis [2].

- **Determining saturation flow rate**

Saturation flow rate is the maximum flow rate in vehicles per hour that can be conveyed by the lane group provided that the green phase is available for all the time. HCM2000 proposes the equation **(4.38)** in order to compute the saturation flow rate for each lane group.

$$S = S_0 \times N \times f_w \times f_{hv} \times f_g \times f_p \times f_{bb} \times f_a \times f_{LU} \times f_{LT} \times f_{RT} \times f_{lpb} \times f_{Rpb} \quad \textbf{(4.38)}$$

Where

S : saturation flow rate for subject lane group, expressed as a total for all lanes in lane group (veh/h);

So : base saturation flow rate per lane (pc/h/ln);

N : number of lanes in lane group;

f_w: adjustment factor for lane width;

f_{hv}: adjustment factor for heavy vehicles in traffic stream;

f_g: adjustment factor for approach grade;

f_p: adjustment factor for existence of a parking lane and parking activity adjacent to lane group;

f_{bb}: adjustment factor for blocking effect of local buses that stop within intersection area;

f_a: adjustment factor for area type;

f_{LU}: adjustment factor for lane utilization;

f_{LT}: adjustment factor for left turns in lane group;

f_{RT}: adjustment factor for right turns in lane group;
f_{lpb}: pedestrian adjustment factor for left-turn movements; and
f_{Rpb}: pedestrian-bicycle adjustment factor for right-turn movements [2].

- **Base saturation flow rate** (S_0)

The saturation flow computation has been commenced after choosing an amount for base saturation flow rate usually 1900 passenger cars per hour per lane (pc/h/ln) [2].

- **Adjustment factor for lane width** (f_w)

The lane width adjustment factor accounts for the effects of lane width on the amount of saturation flow rate. According to HCM2000, the standard lane width is 3.6 m. When the lane width is less than 3.6 m, this adjustment factor decreases the amount of saturation flow rate in order to reflect the reduction in the capacity of the lane. Equation **(4.39)** is proposed for calculating the adjustment factor for lane width in which the amount of this factor only varies as a function of lane width [2].

$$f_w = 1 + \frac{w-3.6}{9} \qquad \textbf{(4.39)}$$

Where:
f_w: adjustment factor for lane width
w: lane width

It should be considered that the lane width (w) must always be greater and equal to 2.4m. If the lane width (w) is greater than 4.8 m, a two-lane analysis may be considered [2].

- **Adjustment factor for heavy vehicles** (f_{hv})

This factor embodies the effects of heavy vehicles on the traffic movements. Heavy vehicles are those with more than four tires touching the pavement. It is obvious that not only the space that is occupied by heavy vehicles but also the operating capabilities of them is different from that of passenger cars. According to HCM2000, this factor can be computed by the equation **(4.40)**.

$$f_{hv} = \frac{100}{100+\%HV(E_t-1)} \qquad \textbf{(4.40)}$$

Where
f_{hv}: adjustment factor for heavy vehicles
HV: percentage of heavy vehicles for lane group volume
E_t: passenger car equivalent(proposal amount is 2) [2].

- **Adjustment factor for approach grade** (f_g)

The effects of approach grade on a traffic flow and the operation of vehicles are treated by the factor f_g. Equation **(4.41)** is proposed for computing this factor [2].

$$f_g = 1 - \frac{\%G}{200} \qquad \textbf{(4.41)}$$

Where
f_g: adjustment factor for approach grade
G: grade on a lane group approach [2].

- **Adjustment factor for parking** (f_p)

The parking lane near an intersection affects the traffic flow on an adjacent lane group. The vehicles that park throughout these lanes influence the movement of vehicles on the adjacent lane group when moving into and out of parking spaces. The adjustment factor for parking (f_p) accounts for these frictional effects of parking lanes on traffic flow. The required time for each maneuver (either in or out) is 18 seconds. The parking lane will affect the traffic flow on adjacent lane group as long as locates within 75 m upstream from the stop line. The equation (**4.42)** is proposed for computing the parking adjustment factor [2].

$$f_p = \frac{N-0.1-\frac{18Nm}{3600}}{N} \tag{4.42}$$

Where,

N: number of lanes in lane group

N_m: number of parking maneuvers per hour

$0 \leq N_m \leq 180$

$fp \geq 0.050$

fp : 1.00 for no parking [2].

If there are more than 180 maneuvers per hour, the amount of 180 should be used. If parking lane is on an exclusive turn lane, parking adjustment factor only affects the traffic on this exclusive lane. On a one-way street with no exclusive turn lane the computations are a little different. In this condition, all parking maneuvers within 75 m upstream are taking into account for both sides of the lane group. It should be noted that the case with zero maneuvers is different from the no parking situation. N_m takes the zero amounts provided that there are no maneuvers. But, in a no parking situation the amount of one should be selected for f_p and there is no need for using the equation (**4.42)** [2].

- **Adjustment factor for bus blockage** (f_{bb})

The bus blockage adjustment factor (f_{bb}) accounts for the frictional effects of public transport buses that stop for pick up or discharge passengers at a bus stop that locates within 75 m upstream and downstream of the stop line. This factor should only be used in the case of generating traffic congestion in the lane group on behalf of the stopping buses. This means that if there is an exclusive bus stop lane, f_{bb} takes the amount 1 and the stopping buses have no effect on the traffic flow. The equation **(4.43)** is proposed for computing the f_{bb} [2].

$$f_{bb} = \frac{N - \frac{14.4 N_B}{3600}}{N} \tag{4.43}$$

Where

N: number of lanes in the lane group

N_B: number of buses stopping per hour

$0 \leq N_B \leq 250$

$f_{bb} \geq 0.050$ [2].

- **Adjustment factor for area Type** (f_a)

The area type adjustment factor accounts for (f_a) the negative effects of business districts on the performance of intersections. The business districts embodies narrow streets with high traffic congestion, frequent parking maneuvers, taxi and bus activity, high pedestrian activity, dense population and many other situations that worsen the traffic condition in a designated intersection. In other words, business districts are places with specific conditions in comparison to other locations that cause significant increase in vehicle headways to the point in which the capacity of intersection is affected adversely. Proposed amount for this factor is 0.90 for central business districts (CBD) and 1 for all other areas [2].

- **Adjustment factor for lane utilization** (f_{LU})

The lane utilization adjustment factor (f_{LU}) accounts for the unequal distribution of traffic among the lanes in a lane group with more than one lane. This circumstance occurs due to roadway downstream or upstream characteristics such as changes in the number of lanes or traffic flow characteristics such as existing high volume turning movements. Using actual lane volume distribution is recommended if observed in the field. Moreover, it is proposed a lane utilization factor of 1 for the case that a uniform traffic distribution can be assumed across all lanes in the lane group or the lane group includes just a single lane. The equation (**4.44)** can be used for computing the amount of lane utilization factor [2].

$$f_{lu} = \frac{V_g}{V_{g1} N} \tag{4.44}$$

Where

f_{lu} : lane utilization adjustment factor

v_g : unadjusted demand flow rate for lane group (veh/h)

v_{g1} : unadjusted demand flow rate on single lane with highest volume in lane group (veh/h)

N : number of lanes in lane group [2].

- **Adjustment factor for right turns** (f_{RT})

The right-turn adjustment factor accounts for the effects of geometry on the traffic flow. The amount of this factor varies with respect to this matter whether there is an exclusive turning lanes or not as well as the proportion of right-turning vehicles in the shared lanes. If there is any right-turn exclusive lane the amount of 0.85 should be used for the right-turn adjustment factor. If there is no exclusive lane for right-turning vehicles equations, **(4.45)** and **(4.46)** should be used,

a) If there is only one lane in the lane group

$$f_{rt}= 1-0.135P_{rt} \quad \textbf{(4.45)}$$

b) If there are more than one lane in the lane group

$$f_{rt}= 1-0.15P_{rt} \quad \textbf{(4.46)}$$

P_{rt}: proportion of vehicles turning right in the lane group

Note: $f_{rt} \geq 0.05$ [2].

- **Adjustment factor for left turns** (f_{LT})

The function of Left turn adjustment factor is analogous to that of right turn adjustment factor. This function varies based on:

1) Whether there is any exclusive left turn lanes or not,
2) Type of phasing (protected, permitted, or protected-plus-permitted),
3) Proportion of left-turning vehicles using a shared lane group, and
4) Opposing flow rate when permitted left turns are made.

Left-turn adjustment factor is computed based on the left turns are made from exclusive or shared lanes as well as the type of phasing as follows:

• Case 1: Exclusive lane with protected phasing,
• Case 2: Exclusive lane with permitted phasing,
• Case 3: Exclusive lane with protected-plus-permitted phasing,
• Case 4: Shared lane with protected phasing,
• Case 5: Shared lane with permitted phasing, and
• Case 6: Shared lane with protected-plus-permitted phasing.

The amount of left-turn adjustment factor for the protected phasing type computes as follows:

a) If there is an exclusive left-turn lane the amount of 0.95 should be used for the left-turn adjustment factor.
b) For other cases equation **(4.47)** may be used,

$$f_{LT} = \frac{1}{1+0.05P_{LT}} \qquad \textbf{(4.47)}$$

P_{Lt}: proportion of vehicles turning left in the lane group [2].

• Adjustment factor for pedestrian-bicycle blockage (f_{LPb}),(f_{RPb})

In order to consider the effects of pedestrians and bicycles on the capacity of the signalized intersections, equations **(4.48)** and (**4.49)** are recommended for computing these factors [2].

For left turn adjustment:

$$f_{LPb} = 1-P_{LT}(1-A_{PbT})(1-P_{LTA}) \qquad \textbf{(4.48)}$$

where

P_{LT}: proportion of vehicles turning left in the lane group

A_{pbT}: permitted phase adjustment

P_{LTA}: proportion of LT protected green over total LT green

For right turn adjustment:

$$f_{RPb} = 1-P_{RT}(1-A_{PbT})(1-P_{RTA}) \qquad \textbf{(4.49)}$$

Where

P_{RT}: proportion of vehicles turning right in the lane group

A_{pbT}: permitted phase adjustment

P_{RTA}: proportion of RT protected green over total RT green [2].

- **Determining capacity and V/C ratio**

The following sections describe capacity and volume/capacity ratio concepts.

- **Capacity**

In signalized intersections capacity for a lane group is determined as the maximum number of vehicles that can pass through the intersection during a cycle length. This amount is a function of green time, cycle length and saturation flow rate of that lane group. The flow ratio for a given lane group is defined as the ratio of the actual or projected demand flow rate for the lane group (V_i) and the saturation flow rate (S_i). The flow ratio for lane group i is shown with symbol $(V/S)_i$ and the capacity of given lane group may be computed using Equation 3.

$$C_i = S_i \frac{g_i}{C} \qquad \textbf{(4.50)}$$

where
C_i: capacity of lane group i (veh/h),
S_i: saturation flow rate for lane group i (veh/h), and
g_i /C: effective green ratio for lane group I [2].

- **V/C Ratio**

The ratio of flow rate to capacity which is often called volume to capacity ratio is given the symbol X in intersection analysis. It is usually defined as degree of saturation. For a specific lane group i, X_i is computed using equation **(4.51).**

$$X_i = \left(\frac{v}{c}\right)_i = \frac{v_i}{c_i\left(\frac{g_i}{c}\right)} = \frac{v_i C}{s_i g_i} \qquad \textbf{(4.51)}$$

Where
X_i: $(v/c)_i$: ratio for lane group i,
v_i : actual or projected demand flow rate for lane group i (veh/h),
s_i : saturation flow rate for lane group i (veh/h),
g_i : effective green time for lane group i (s), and
C : cycle length (s).

Note: $0 < X_i < 1$

The X_i takes zero value provided that there is no demand flow rate. The values above 1 occur when there is an excess of demand over capacity. Given that rarely all movements of an intersection become saturated at the same time of day, the capacity of entire intersection is not a significant concept [2].

- **Critical lane groups**

Critical lane group is the concept which should be considered in analyzing intersections. This v/c ratio is for the entire intersection. The lane group, in which the amount of v/c is highest, should be designated as critical lane group for a specific phase. Sufficient amount of green time for each signal phase is determined according to the v/c ratio of critical lane group. The critical v/c ratio for the intersection is determined by using equation **(4.52)**:

$$X_c = \sum \left(\frac{v}{s}\right)_{ci} \left(\frac{c}{c-l}\right) \quad \textbf{(4.52)}$$

where

Xc: critical v/c ratio for intersection;

$\sum \left(\frac{v}{s}\right)_{ci}$: summation of flow ratios for all critical lane groups i;

C : cycle length (s); and

L : total lost time per cycle, computed as lost time for critical path of movements (s) [2].

- **Delay**

Delay due to a signal control or stop sign is called control delay. The control delay of an intersection is derived as the sum of three different types of delays and includes movements at slower speeds and stops on intersection approaches as vehicles move up in queue position or slow down upstream of an intersection. In order to compute the average control delay of an intersection the equation **(4.53)** is used.

$$d = d_1(PF) + d_2 + d_3 \quad \textbf{(4.53)}$$

where

d : control delay per vehicle (s/veh);

d1 : uniform control delay assuming uniform arrivals (s/veh);

PF : Progression Factor

d2 : incremental delay (s/veh);

d3 : initial queue delay, (s/veh) [2].

- **Uniform delay**

Uniform delay is based on assuming uniform arrivals, stable flow, and no initial flow. Figure 4.9 shows this concept. All vehicles discharge the intersection during one green phase. Total uniform delay during this period is the sum of all triangular areas between arrival and departure function [25].

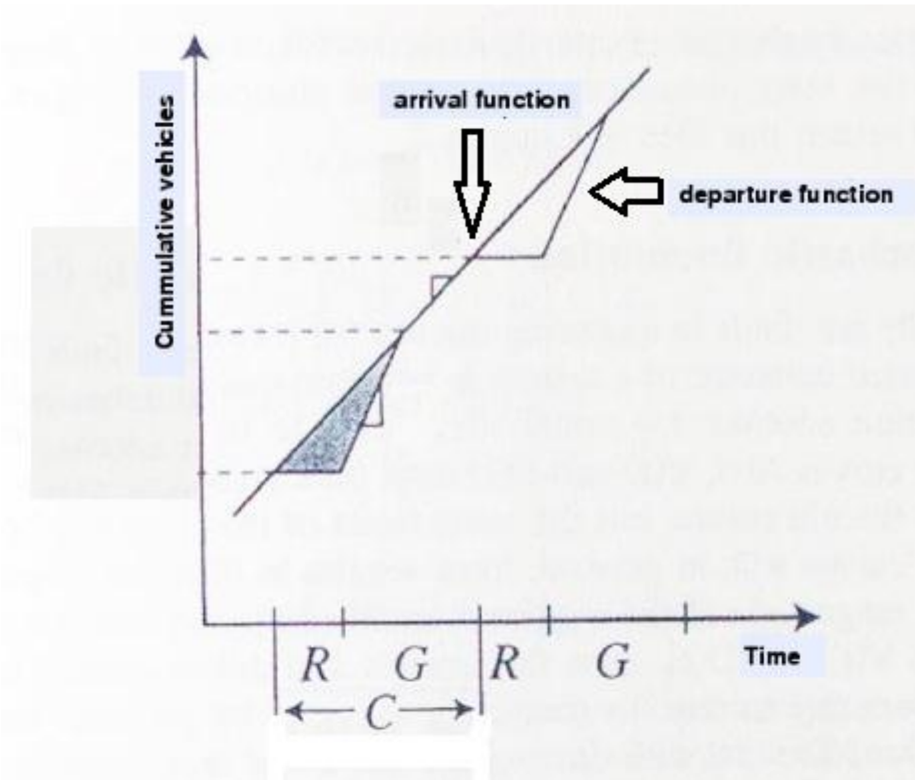

Figure 4.9 : Illustration of uniform delay [26].

Equation (**4.54)** gives an estimate of uniform delays in a signalized intersection.

$$d_1 = \frac{0.5C\left(1-\frac{g}{C}\right)^2}{1-\left[\min(1,X)\frac{g}{C}\right]} \quad \textbf{(4.54)}$$

where

d1: uniform control delay assuming uniform arrivals (s/veh);

C: cycle length (s); cycle length used in pretimed signal control, or average cycle length for actuated control;

g : effective green time for lane group (s); green time used in pretimed signal control, or average lane group effective green time for actuated control

X: v/c ratio or degree of saturation for lane group.

• **Incremental delay**

This delay includes random delay and oversaturation delay. Random delay is the additional delay, above and beyond uniform delay because flow is randomly distributed rather than uniform in isolated intersections. As shown in Figure 4.10, a part of vehicles should wait for more than one green phase in order to discharge the intersection. But, until the end of period condition gets better and again all vehicles discharge the intersection during one green phase. Thereby, some individual green phases fail during the period and the sum of areas between arrival function and capacity function during these failures is known as random delay.

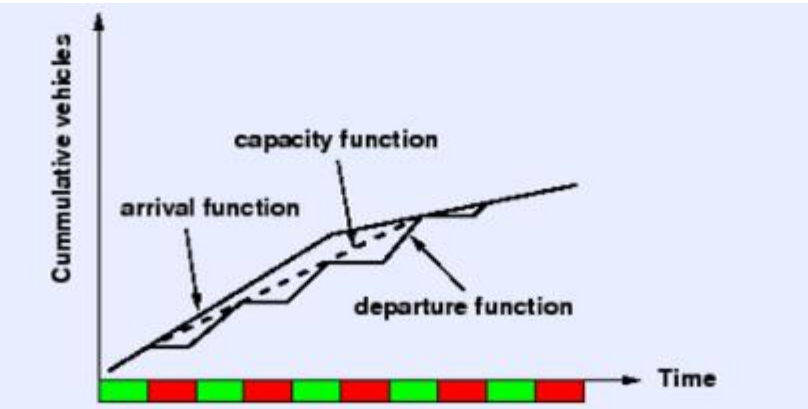

Figure 4.10 : Illustration of random delay [25].

Overflow delay is the additional delay that occurs when the capacity of an individual phase or series of phases is less than the demand or arrival flow rate. Figure 4.11 describes this condition during which every green phase fails and the residual queue of vehicles increase throughout the analysis period. As observed in Figure 4.11, overflow delay unlike random delay which occurs during a short period, continues to grow for a long period of time [25].

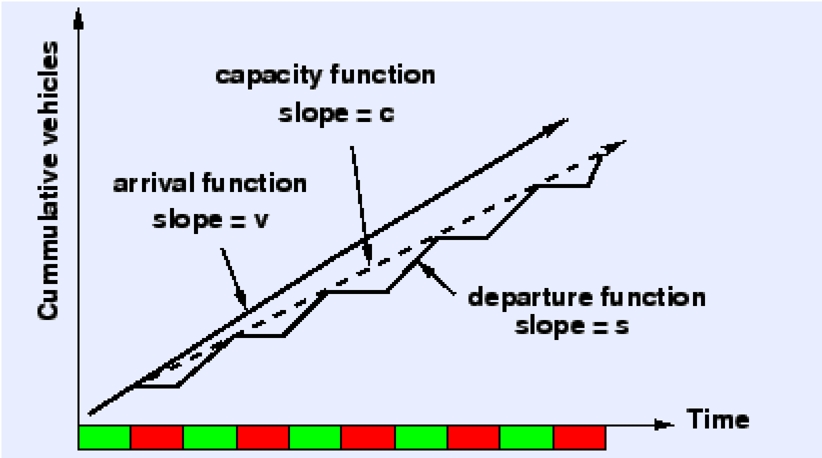

Figure 4.11: Illustration of overflow delay [25].

Equation **(4.55)** is used to estimate the incremental delay which assumes that there is no initial queue.

$$d_2 = 900T\left[(X-1) + \sqrt{(X-1)^2 + \frac{8KIX}{cT}}\right] \quad \textbf{(4.55)}$$

where

d2 : incremental delay to account for effect of random and oversaturation

queues, adjusted for duration of analysis period and type of signal control (s/veh);

T : duration of analysis period (h);

k : incremental delay factor that is dependent on controller settings;

I : upstream filtering/metering adjustment factor;

c : lane group capacity (veh/h); and

X : lane group v/c ratio or degree of saturation.

Incremental delay factor (k) incorporates the effect of controller type on delay. For pretimed signals a value of k=0.5 is used. Upstream filtering/metering adjustment factor (I) incorporates the effects of metering arrivals from upstream signals. For an isolated signalized intersection a value of 1 for I is used [2].

- **Initial delay**

When all vehicles cannot discharge the intersection during the green phase, due to the time needed for this residual queue to clear the intersection at the next green phase, an initial delay for the vehicles that arrive to the intersection at the start of the next green time will occur. This delay is referred as initial delay [2].

- **Progression adjustment factor (PF)**

The more proportion of vehicles arrives on green, the better signal progression will result. Progression adjustment factors are applied to account for the effects of the upstream traffic signal on the delay at the downstream signal [27]. Progression primarily affects uniform delay, and for this reason, the adjustment is applied only to d_1. PF can be estimated applying equation **(4.56)**.

$$PF = \frac{(1-p)f_{PA}}{1-\left(\frac{g}{C}\right)} \tag{4.56}$$

where

PF : progression adjustment factor,

P : proportion of vehicles arriving on green,

g/c : proportion of green time available, and

f_{PA} : supplemental adjustment factor for platoon arriving during green [2].

- **Aggregated delay estimates**

The procedure for calculating delay states the control delay in terms of second per vehicle per lane group. It is often desirable to aggregate these values to provide delay for an intersection approach and for the intersection as a whole. Thereby, the equation (**4.57)** is used for computing the delay for a specific approach.

$$d_A = \frac{\sum d_i v_i}{\sum v_i} \tag{4.57}$$

where

d_A: delay for Approach A (s/veh),

d_i: delay for lane group i (on Approach A) (s/veh), and

v_i: adjusted flow for lane group i (veh/h).

Moreover, in order to compute the entire intersection average control delay, the equation (**4.58)** can be used which is aggregation of all average control delay of approaches.

$$d_l = \frac{\sum d_A v_A}{\sum v_A} \tag{4.58}$$

where

d_i: delay per vehicle for intersection (s/veh),

d_A: delay for Approach A (s/veh), and

v_A: adjusted flow for Approach A (veh/h) [2].

- **Determining level of service**

For determining the level of service of an intersection the average control delay per vehicle of the entire intersection should be calculated. Then, using the Table 2.1 the LOS of the intersection will be obtained.

- **Allocation of green time**

The average cycle length and effective green time for each lane group must be defined. The most desirable way to obtain these values is by field measurement; however, there are many cases in which field measurement is not possible. A procedure for estimating the signal timing characteristics is therefore an important traffic analysis tool [2].

- **Timing plan design for pre-timed control**

Equalizing the v/c ratios for critical lane groups is the simplest strategy and the only one that may be calculated without excessive iteration. Minimizing the total delay to all vehicles is generally proposed as the optimal solution to the signal timing problem [2].

• Procedure for Equalizing Degree of Saturation

Once a phase plan and signal type have been established, signal timing may be estimated using equations (**4.59), (4.60), (4.61)** and **(4.62)** [2].

$$X_i = \frac{V_i \times C}{S_i \times g_i} \quad \textbf{(4.59)}$$

$$X_c = \sum_i \left(\frac{V}{S}\right)_{ci} \left(\frac{C}{C-L}\right) \quad \textbf{(4.60)}$$

$$C = \frac{LX_c}{\left[X_c - \sum_i \left(\frac{V}{S}\right)_{ci}\right]} \quad \textbf{(4.61)}$$

$$g_i = \frac{V_i C}{S_i X_i} = \left(\frac{V}{S}\right)_i \left(\frac{C}{X_i}\right) \quad \textbf{(4.62)}$$

where

C : cycle length (s)

L : lost time per cycle (s)

Xc : critical v/c ratio for the intersection

Xi : v/c ratio for lane group i (note that target v/c ratio is a user-specified input with respect to this procedure; default value suggested is 0.90)

$(v/s)_i$: flow ratio for lane group i

S_i : saturation flow rate for lane group i (veh/h).

The shortest cycle time can be computed from equation **(4.63)**,

$$C_{minimum} = \frac{L}{\left[1 - \sum_i \left(\frac{V}{S}\right)_{ci}\right]} \quad \textbf{(4.63)}$$

4.3 Method of VISSIM

Simulation is a method that provides a condition in which behavior studying of specific network traffic in various circumstances is by far simple and economical. Most of the subsequent effects due to any modification in a transportation network can be assessed in a computer without any need to establish high-cost experiments in the field that is also impossible in some cases. In other words, simulation is a technique which permits the study of a complex traffic system in the laboratory rather than in the field. Some other reasons that we utilize simulation methods are as follows:

a) Simulation provides a condition in which gathering the data in a systematic way becomes possible and as a result the study of traffic characteristics and operation get much more possible.
b) Simulation of complex traffic operations clarifies the importance degree of different variables and how they relate. This may lead to significant analytic formulations.
c) Simulation is a method in order to test the authenticity of analytic solution [28].

There are a vast variety of traffic simulation software like PTV VISSIM, PARAMICS, SYNCHRO, SIDRA, SUMO and so forth.

PTV VISSIM is a commercial software solution which assists you in intelligently simulating and controlling daily road traffic. PTV VISSIM is part of the Vision Traffic Suite software package. It is the microscopic simulation tool for modeling multimodal traffic flows and it provides ideal conditions for testing different traffic scenarios in a realistic and highly detailed manner before final implementation. Today, PTV VISSIM is used by customers around the world, including public authorities, consulting firms and universities [29]. VISSIM can be very effectively used to compare various types of intersections for a particular location, capacity analysis of freeways and arterial corridors, analysis of traffic management systems, analysis of delay times for various alternative routes, simulating evacuation plans, airport studies, and other highly complex engineering problems like simulating a multimodal transit center, underground train station, and feasibility analysis of large networks [20].

Use cases of PTV VISSIM are as follows:

1. Comparison of junction geometry
- model any junction geometry
- simulate the traffic passing different junction variants
- Account for the interaction between different modes of transport (motorized, rail, cyclists, pedestrians)
- Analyze numerous planning variants according to criteria such as traffic level of service, delays or queue length
- Graphical depiction of traffic flows.

2. Development planning
- Model and evaluate the effects of urban development concepts.
- Implementation and coordination of construction sites.
- Pedestrian simulation inside and outside buildings.
- Simulate the search for parking space, the size of parking lots and their effects on parking operations.

3. Capacity analyses
- realistically model traffic flows at complex intersection systems
- account for and graphically depict the impact of throngs of arriving traffic, interlacing
- Traffic flows between intersections and irregular intergreen times

4. Traffic control systems
- investigate and visualize traffic on a microscopic level
- analyze simulations regarding numerous traffic parameters (e.g. speed, queue length, travel time, loss of time)
- examine the effects of traffic-actuated control and variable message signs
- develop measures to increase the traffic flow

5. Signal systems operations and re-timing studies
- Simulate travel demand scenarios for signal-controlled intersections

- Analyze traffic-actuated control with efficient data input, even for complex algorithms.
- create and simulate construction and signal plans for traffic calming before starting implementation
- PTV VISSIM provides numerous test functions concerning the impact of signal control

6. Public transit simulation

- Model all details for bus, bus rapid transit, streetcar, light rail transit and commuter rail operations
- Analyze transit specific operational improvements by using built-in industry standard signal priority
- Simulate and compare several approaches, showing different courses for special public transport lanes and different stop locations (during preliminary draft phase)
- Test and optimize switchable, traffic-actuated signal controls with public transport priority [30].

Traffic simulation programs are fall into two categories based on the method they apply for analyzing the network performances. First category packages use deterministic (empirical or analytical) models such as equations and correlations (e.g. delay, queues and capacity). The latter category packages apply stochastic micro-simulation models using an interval-based simulation to describe traffic operations [31]. PTV VISSIM lies in the second category which utilizes stochastic simulation models for network evaluations. In other words, VISSIM is a time step and behavior-based simulation model [32]. VISSIM provides a flexible space that enables user to model a transportation network easily. The user can generate a network with drawing different links which are connected to one another using connectors. Then different parameters like the vehicle volumes, desired speeds, vehicle compositions, priority rules, signal heads and other macroscopic and microscopic parameters are assigned to these links. The simulation package VISSIM consists internally of two different parts, exchanging detector calls and signal status through an interface. The traffic simulator is a microscopic traffic flow simulation model including the car following and lane change logic. The signal state generator is a signal control software polling detector information from the traffic simulator on a discrete time step basis (down to 1/10 of a second). It then determines the signal status for the following time step and returns this information to the traffic simulator. Figure 4.12 illustrates this communication process.

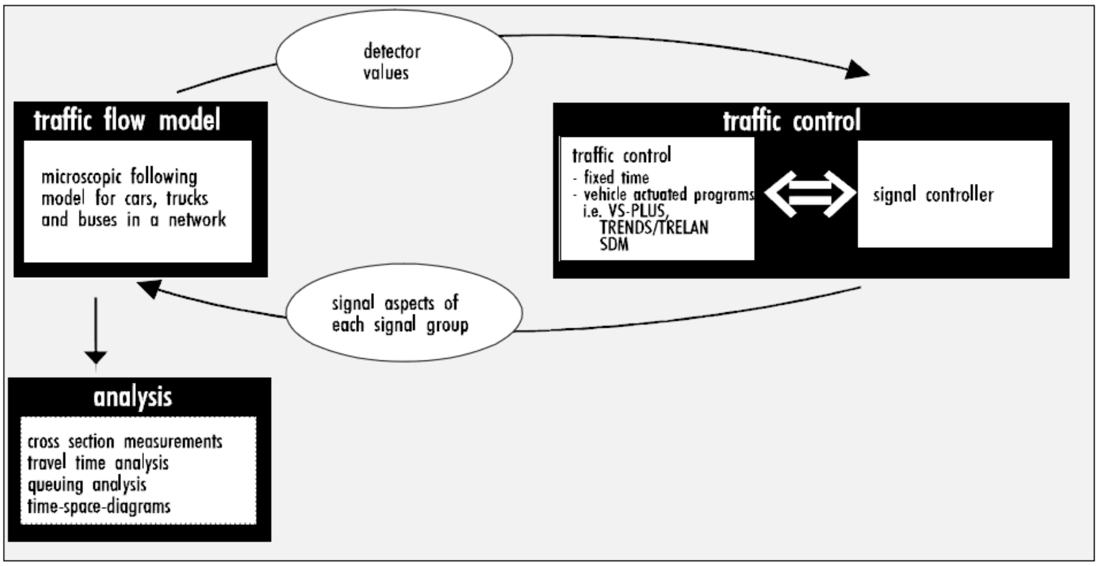

Figure 4.12 : Communication between traffic simulator and signal state generator [32].

The accuracy of a traffic simulation model is mainly dependent on the quality of the vehicle modeling, e.g. the methodology of moving vehicles through the network. In contrast to less complex models using constant speeds and deterministic car following logic, VISSIM uses the psycho-physical driver behavior model developed by Wiedeman in 1974. The basic concept of this model is that the driver of a faster moving vehicle starts to decelerate as he reaches his individual perception threshold to a slower moving vehicle. Since he cannot exactly determine the speed of that vehicle, his speed will fall below that vehicle’s speed until he starts to slightly accelerate again after reaching another perception threshold. This results in an iterative process of acceleration and deceleration.

Stochastic distributions of speed and spacing thresholds replicate individual driver behavior characteristics. The model has been calibrated through multiple field measurements at the Technical University of Karlsruhe, Germany. Periodical field measurements and their resulting updates of model parameters ensure that changes in driver behavior and vehicle improvements are accounted for VISSIM’s traffic simulator not only allows drivers on multiple lane roadways to react to preceding vehicles (4 by default), but also neighboring vehicles on the adjacent travel lanes are taken into account. Furthermore, approaching a traffic signal, results in a higher alertness for drivers at a distance of 100 meters in front of the stop line. VISSIM simulates the traffic flow by moving “driver-vehicle-units” through a network. Every driver with his specific behavior characteristics is assigned to a specific vehicle. As a consequence, the driving behavior corresponds to the technical capabilities of his vehicle. Attributes characterizing each driver-vehicle unit can be discriminated into three categories:

- Technical specification of the vehicle, for example:
  - Length
  - Maximum speed
  - Potential acceleration
  - Actual position in the network
  - Actual speed and acceleration
- Behavior of driver-vehicle units, for example:
  - Psycho-physical sensitivity thresholds of the driver (ability toestimate, aggressiveness)
  - Memory of driver
  - Acceleration based on current speed and driver’s desired speed
- Interdependence of driver-vehicle units, for example:
  - Reference to leading and following vehicles on own and adjacent travel lanes
  - Reference to current link and next intersection
  - Reference to next traffic signal [32].

- **The “Wiedemann” car following model**

The traffic flow model in VISSIM is a discrete, stochastic, time step based, microscopic model with driver-vehicle-units as single entities. The model contains a psycho-physical car following model for longitudinal vehicle movement and a rule-based algorithm for lateral movements. The model is based on the continued work of Wiedemann. The basic idea of the Wiedemann model is the assumption that a driver can be in one of four driving modes [32].

1) Free-driving mode, where no influence is exerted from leading vehicles. In this mode, the driver attempts to reach and maintain a desired speed.

2) Approaching mode, when the driver of the follower vehicle consciously observes that she is approaching a slower vehicle in front.

3) Following mode, where the headway for a pair of vehicles is between the maximum following headway and the safe headway. In this mode, the follower vehicle is able to accelerate or decelerate in accordance with the vehicle in front.

4) Braking mode, when the headway between vehicles drops below a desired safety distance [19].

The car-following behavior switches from one mode to another according to predetermined perceptual threshold levels that form the basis of the psychophysical models. These thresholds are defined as a combination of speed and headway differences [33]-[34]. In VISSIM, each driver–vehicle unit is described as a driver–vehicle element (DVE). Figure 4.13 shows the interaction between two vehicles where DVEj is moving faster than and approaching a slower vehicle DVEi. Driver j begins to decelerate until an individual threshold, which is a function of acceptable speed difference and spacing, is reached. Driver j then maintains a speed at or below the current speed of DVEi, until other thresholds are reached and the driver then accelerates again [35]. Driver-specific perception abilities and individual risk behavior is modeled by adding random values to each of the parameters. AX Desired distance between the front sides of two successive vehicles in a standing queue. ABX Desired minimum following distance, which is a function of AX, a safety distance, and speed. SDV Action point when a driver consciously observes approaching a slower vehicle. SDV increases with increasing speed differences. In the original work of Wiedemann, an additional threshold is applied to model additional deceleration by usage of brakes. OPDV Action point when drivers of follower vehicles notice that they are traveling slower than the leading vehicles and start to accelerate again. SDX Perception threshold to model the maximum following distance, which is about 1.5–2.5 times ABX. A driver reacts to the leading vehicle when the distance between the two vehicles approaches 150 m. The minimum acceleration and deceleration rate is set to be 0.2 m/s2. Maximum rates of acceleration depend on vehicles’ technical features. The model also includes a rule for exceeding the maximum deceleration rate in case of emergency [19].

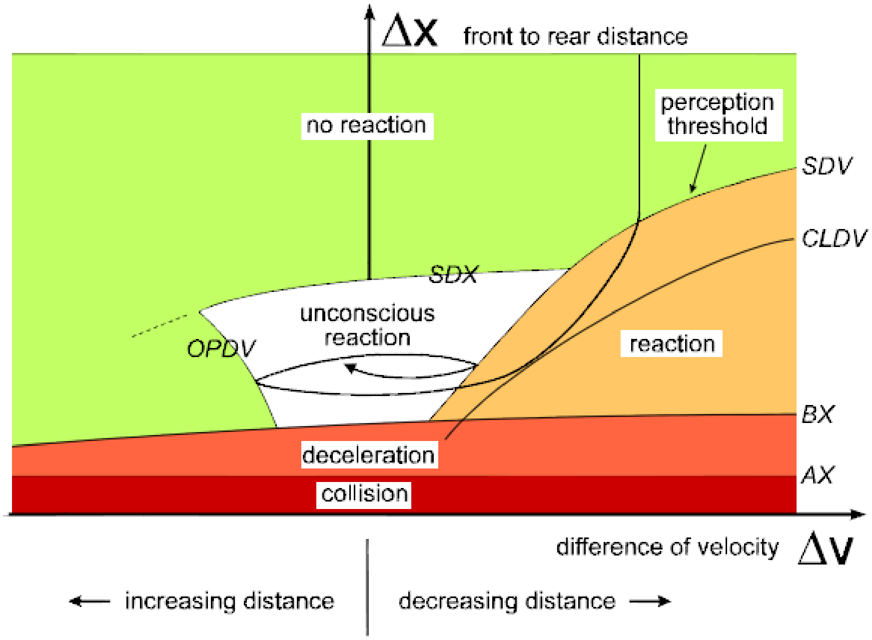

Figure 4.13 : Interaction between two vehicles in VISSIM [32]

4.3.1 Review of case studies incorporating simulation

A microscopic simulation model calibration and validation for freeway work zone network were carried out. The case study network is a freeway segment with a work zone located in the City of Covington, VA. . The performance of the procedure was tested by comparing distribution of simulation outputs and field travel time data. The results based on the case study of a freeway work zone network indicate that default and best-guessed parameters of VISSIM were not able to replicate field travel time, while the calibrated parameter set can. Thus, the validity of the procedure is proven for a freeway network [37].

In another case study, SYNCHRO and CORSIM simulation software were applied for evaluation of route diversion strategies. In this study, a methodology was presented to estimate the effects of incident management and signal-timing modification using two computer simulation packages, SYNCHRO and CORSIM, and data transferring between these two programs. Moreover, this study documents the application of the results of the CORSIM-based simulation method to evaluate and assist in the implementation of a demonstration route diversion plan for the I-75 corridor in Sarasota County [38].

5. CASE STUDY

Location and other specifications of case studies like signal timing, phasing diagrams and geometric characteristics are discussed in section 5.1.

5.1 Description of Area and Sample Intersections

Two four-legged signalized intersections in Istanbul city of Turkey in the area of Pendik located in the south west of Asian part of Istanbul city was chosen as case studies for conducting this study. Both intersections are connected to each other with Adnan Menderes street ends to Mostar avenue, a 3*2 divided main road. As shown in Figure 5.1, the Adnan Menderes street is connected to the Fatih street with intersection 1, while intersection 2 locates in the intersecting point of Adnan Menderes with Süreyya Paşa streets. Moreover, intersection 1 is close to the Pendik clover intersection.This area exhibits central business district (CBD) characteristics. Figure 5.2 and Figure 5.4 illustrate geometry of intersection 1 and intersection 2 respectively.

Figure 5.1 : Location of case studies [39].

In Figure 5.2, traffic flows are shown with arrows on which the value at the start and end points represent the lane group number and the traffic volume in term of vehicle per hour on that direction respectively. Figure 5.2 illustrates that all approaches except approach 4 which consists of one lane, include two lanes in each direction separated from opposite direction flow through a boulevard.

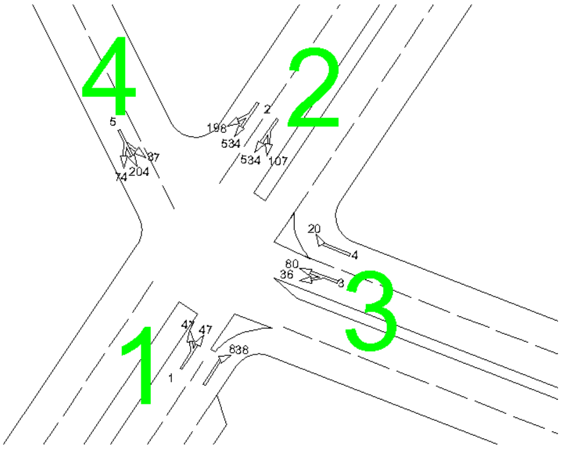

Figure 5.2 : Geometry of intersection 1.

Table 5.1 presents various characteristics such as green times, cycle lengths and lane widths of intersection 1 employed in performance evaluations.

Table 5.1 : Specifications of intersection 1.

Approach No	Lane Group	Green(sec)	Yellow(sec)	All red(sec)	Cycle Length(sec)	Number of Lanes	Lane width(m)	Slope%
1	1-2,4	13	3	3	110	1	2.7	-1
	1-3	All Green	3	3	110	1	2.7	-1
2	2-1,3,4	39	3	3	110	2	3	1
3	3-2	18	3	3	110	1	2.8	-2
	3-4,1	18	3	3	110	1	2.8	-2
4	4-1,2,3	18	3	3	110	1	3.2	0

Figure 5.3 represents signal phasing of intersection 1 on which departure and destination approaches for each traffic flow are designated with the numbers at the start and end of the arrows respectively.

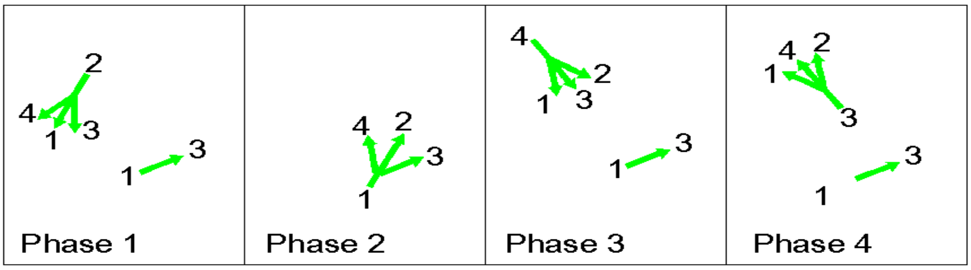

Figure 5.3 : Signal phasing of intersection 1.

As explained for Figure 5.2, similarly, in Figure 5.4 traffic flows are shown with arrows on which the value at the start and end points represent the lane group number and the traffic volume in term of vehicle per hour on that direction respectively. Figure 5.4 depicts that all approaches include two lanes in each direction separated from opposite direction flow through a boulevard.

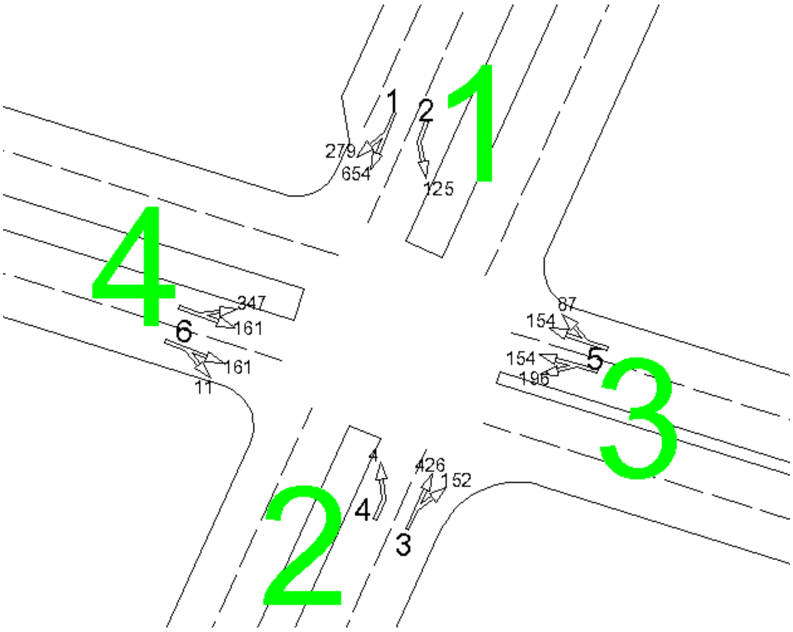

Figure 5.4 : Geometry of intersection 2.

Table 5.2 exhibits diverse characteristics such as green times, cycle lengths and lane widths of intersection 2 employed in performance evaluations.

Table 5.2 : Specifications of intersection 2.

Approach No	Lane Group	Green(sec)	Yellow(sec)	All red (sec)	Cycle Length(sec)	Number of Lanes	Lane width(m)	Slope%
1	1-2,4	40	3	3	119	2	2.5	0
	1-3	20	3	3	119	1	2.5	0
2	2-1,3	29	3	3	119	2	2.75	0
	2-4	10	3	3	119	1	2.75	0
3	3-1,2,4	27	3	3	119	2	2.5	0
4	4-1,2,3	23	3	3	119	2	2.5	0

Like Figure 5.3, Figure 5.5 represents signal phasing of intersection 2 on which departure and destination approaches for each traffic flow are designated with the numbers at the start and end of the arrows respectively.

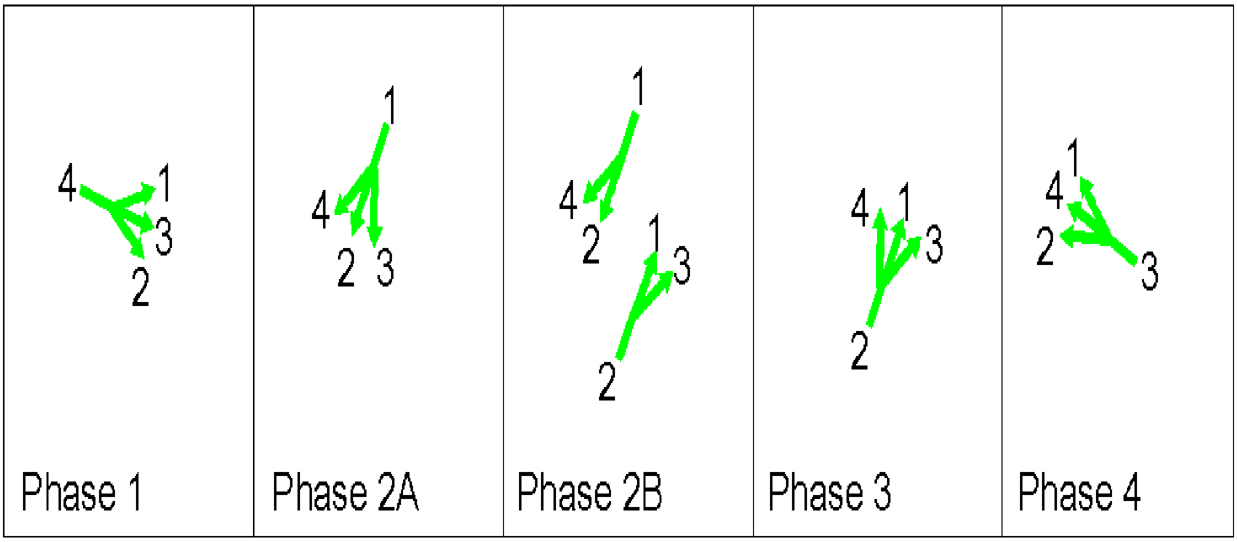

Figure 5.5 : Signal phasing of intersection 2.

Figure 5.6 illustrates signal timing of intersection 1. Cycle length and all green times for each movement are shown on the diagram.

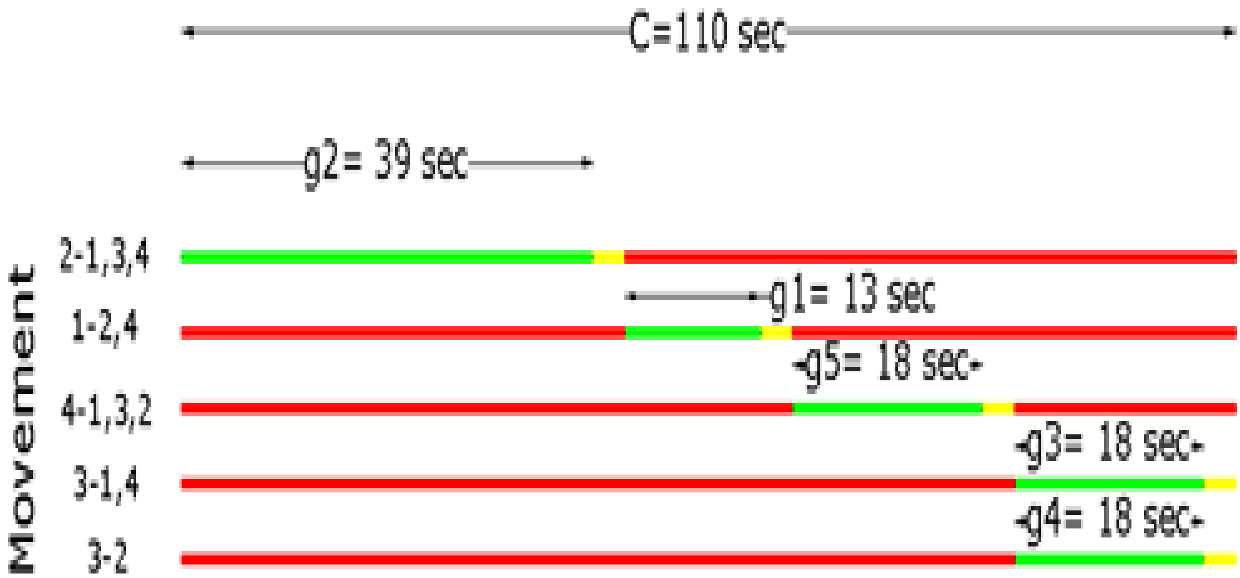

Figure 5.6 : Signal timing diagram of intersection 1.

Similarly, Figure 5.7 demonstrates signal timing of intersection 2 including cycle length and green times for all movements.

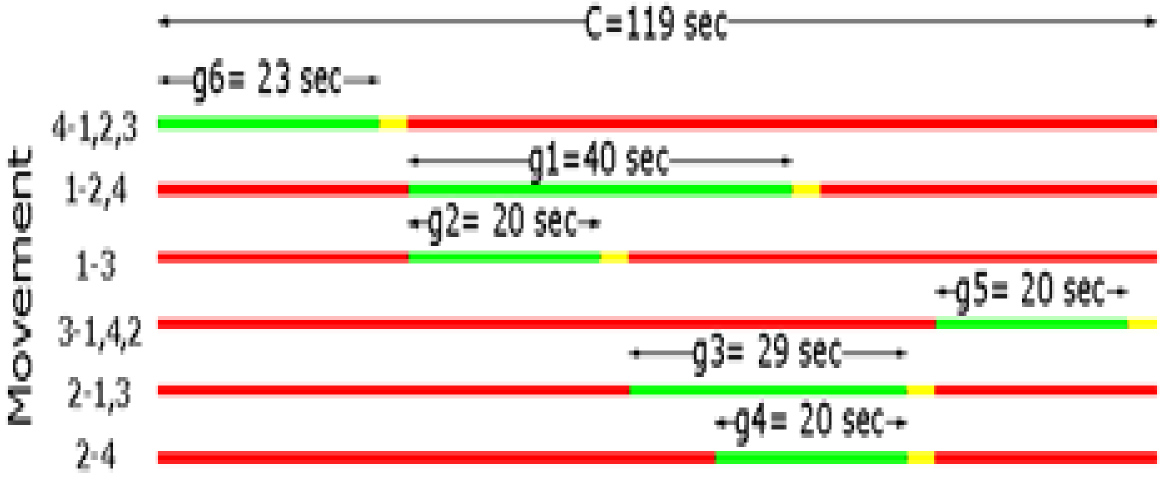

Figure 5.7 : Signal timing of diagram of intersection 2.

5.2 Description of Real Data

Traffic flow measures obtained using camera recording method at sites between 7:00-10:00, 12:00-14:00 and 16:30-19:30 time periods. Then, all the recordings were transformed to the computer and the frequency of different types of vehicles in 15-min periods were counted and tabulated with respect to their travel directions. Subsequently, on each approach and for each travel direction, the hour during which maximum traffic volume occurs was designated as its peak hour. Then, peak hour factors (PHF) were calculated for each approach by dividing the peak hour volume to the flow rate of that approach. Using PHF, the adjusted flow rate was computed for each lane group, and these flows were used as demand flows in all performance evaluations of case studies. The 15-min traffic volumes for each lane group and their PHF factors are obvious in Table 5.3 and Table 5.4 for intersection 1 and intersection 2 respectively. Table 5.5 shows the peak hour volumes and adjusted flow rates for each travel direction in intersection 1 and intersection 2.

Table 5.3 : 15-min volumes for each approach of intersection 1.

Intersection 1																
	Approach 1				Approach 2				Approach 3				Approach 4			
15-min time interval	1-2	1-3	1-4	Total volume (veh/h)	2-1	2-3	2-4	Total volume (veh/h)	3-1	3-2	3-4	Total volume (veh/h)	4-1	4-2	4-3	Total volume (veh/h)
0-15	8	194	17	219	250	20	43	313	7	4	17	28	23	7	49	79
15-30	11	189	9	209	258	26	61	345	7	3	15	25	12	4	39	55
30-45	6	217	10	233	265	30	47	342	10	7	17	34	16	10	43	69
45-60	19	188	8	215	263	28	41	332	6	3	18	27	10	9	36	55
	Hourly flow rate (veh/h)	233*4=932	PHF	0.94	Hourly flow rate (veh/h)	345*4=1380	PHF	0.97	Hourly flow rate (veh/h)	34*4=136	PHF	0.84	Hourly flow rate (veh/h)	316	PHF	0.82

Table 5.4 : 15-min volumes for each approach of intersection 2.

Intersection 2																
	Approach 1				Approach 2				Approach 3				Approach 4			
15-min time interval	1-2	1-3	1-4	Total volume (veh/h)	2-1	2-3	2-4	Total volume (veh/h)	3-1	3-2	3-4	Total volume (veh/h)	4-1	4-2	4-3	Total volume (veh/h)
0-15	145	20	62	227	101	44	0	145	18	48	70	136	74	5	58	137
15-30	159	42	63	264	100	31	1	132	26	42	77	145	83	2	85	170
30-45	150	27	58	235	89	33	0	122	19	48	80	147	58	1	63	122
45-60	141	25	71	237	98	30	3	131	21	52	72	145	80	1	68	149
	Hourly flow rate (veh/h)	264*4=1056	PHF	0.91	Hourly flow rate (veh/h)	145*4=580	PHF	0.91	Hourly flow rate (veh/h)	147*4=588	PHF	0.97	Hourly flow rate (veh/h)	170*4=680	PHF	0.85

Table 5.5 : Direction volumes in intersection 1 and intersection 2.

Intersection name	Approach NO	Direction	Peak hour volume (veh/h)	Adjusted flow rate (veh/h)
Intersection 1	1	1-2	44	47
		1-3	788	838
		1-4	44	47
	2	2-1	1036	1068
		2-3	104	107
		2-4	192	198
	3	3-1	30	36
		3-2	17	20
		3-4	67	80
	4	4-1	61	74
		4-2	30	37
		4-3	167	204
Intersection 2	1	1-2	595	654
		1-3	114	125
		1-4	254	279
	2	2-1	388	426
		2-3	138	152
		2-4	4	4
	3	3-1	84	87
		3-2	190	196
		3-4	299	308
	4	4-1	295	347
		4-2	9	11
		4-3	274	322

Table 5.6 exhibits the peak hour volume, PHF factor and percentage of heavy vehicles for each lane group for both intersections. Note that the peak hour factor (PHF) is the proportion of observed hourly traffic volume to the flow rate. Moreover, flow rate is the equivalent hourly rate at which vehicles pass a point on a lane, roadway, or other traffic way; computed as the number of vehicles, divided by the time interval (usually less than 1 hour) in which they pass [2].

Table 5.6 : Adjusted flow rates of lane groups in intersection 1 and intersection 2.

	Approach No	Lane group	HV%	Peak hour volume (veh/h)	PHF	Adjusted flow rate (veh/h)
Intersection 1	1	1-2,4	0	88	0.94	94
	2	2-1,3,4	2.3	1332	0.97	1373
	3	3-1,4	3.55	97	0.84	115
		3-2		17		20
	4	4-1,2,3	2	258	0.82	319
Intersection 2	1	1-2,4	2.75	849	0.91	933
		1-3 (left turn lane)		114		125
	2	2-1,3	10	526	0.91	578
		2-4 (left turn lane)		4		4
	3	3-4,2,1	4.4	573	0.97	591
	4	4-1,3,2	2.4	578	0.85	680

5.3 Development of Sample Intersections at VISSIM Software

Initially, the precise plan of intersections including lane widths, angle between roads and other geometric characteristics of them with a specific scale were drawn in AUTOCAD software. Then, this AUTOCAD map was inserted into the VISSIM platform as background. It is also possible to insert Google map of intersection in interest to work on it. However, in order to provide more accuracy in drawing process the AUTOCAD plans were utilized. Subsequently, the links and connectors were drawn on the background. The number and width of lanes in each direction, parking lots, and bus stops, peak hour traffic volumes for each approach, traffic compositions, signal-timing specifications and other characteristics of intersections were assigned to the links. The final developed model in VISSIM for intersection 1 and intersection 2 are shown in Figure 5.8 and Figure 5.9.

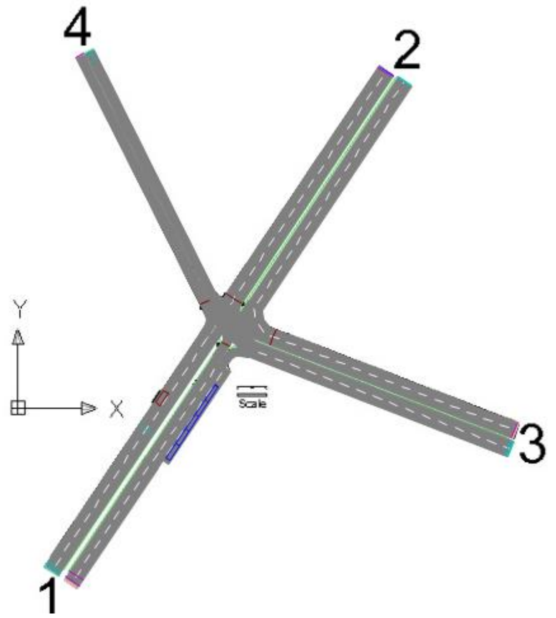

Figure 5.8 : Final model of intersection 1 in VISSIM.

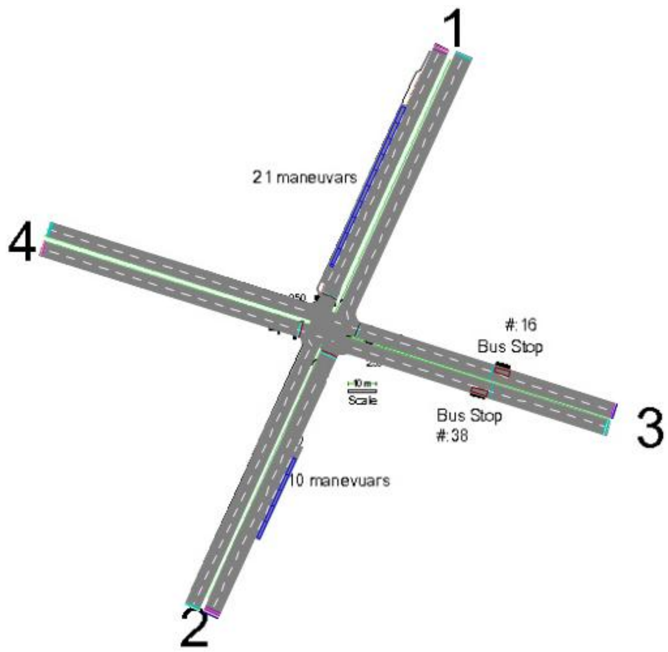

Figure 5.9 : Final model of intersection 2 in VISSIM.

5.4 Calibration of Sample Intersection Flows Using Real Data

The final goal of calibration is minimizing the difference between reality, as measured by a set of observed data, and the model results, described by another set of data that has been produced or constructed from the simulation model [40] The components or parameters of a simulation model that require calibration most frequently include: traffic control operation, traffic flow characteristics, vehicle performance characteristics, and driver behavior [41]. It is common practice to ensure that the calibration parameters from simulation runs are statistically within 5% or one standard deviation of observed values for a given time period [42].

In this study, camera recordings were conducted at sites during 7:00-10:00, 12:00-14:00 and 16:30-19:30 time periods. Then, using these data the adjusted flow rate derived for each direction on that approach. In continue, these volumes were assigned to the approaches modeled in VISSIM as 15-min periods, the travel directions for each approach were designated, and the relevant proportion of volume as well as traffic composition was specified for them. Note that the obtained volumes through camera recording were assigned to the routes in VISSIM as stochastic volumes.

In order to calibration, vehicle count was designated as the measure of performance for attaining this purpose. The data collected from the simulation model runs for the existing condition was compared against the empirical data using the GEH statistic. The GEH statistic is a formula used in traffic engineering, traffic forecasting, and traffic modeling to compare two sets of traffic data. Its mathematical form is similar to a x-square test. The equation **(5.1)** is used for the calculation of GEH statistic.

$$GEH = \sqrt{\frac{2(M-C)^{\wedge}2}{M+C}} \qquad \textbf{(5.1)}$$

Where

M: hourly traffic volume from the traffic model

C: real-world hourly traffic count

The GEH statistic is recognized as a useful measure as it considers both the absolute values of those being compared, and their relative differences. A general rule of thumb is that the GEH value resulting from a flow rate comparison calculation should be around 1.00 and less than 5.00 in a predetermined number of cases, otherwise there is a need to recalibrate [41].

The calibration was carried out for driving behaviours including following, lane change, and lateral.

For each approach, observed vehicle counts were compared to the vehicle counts resulted from simulation for 15-min time periods. Then, GEH statistic was calculated for all the 15-min time periods in each approach. Except for the last 15-min of approach 3 of intersection 1whose GEH value was calculated 1.22, all approaches have the GEH values below 1. Figure 5.10 and Figure 5.11 illustrates development of calibration on different approaches of intersection 1, and intersection 2 respectively.

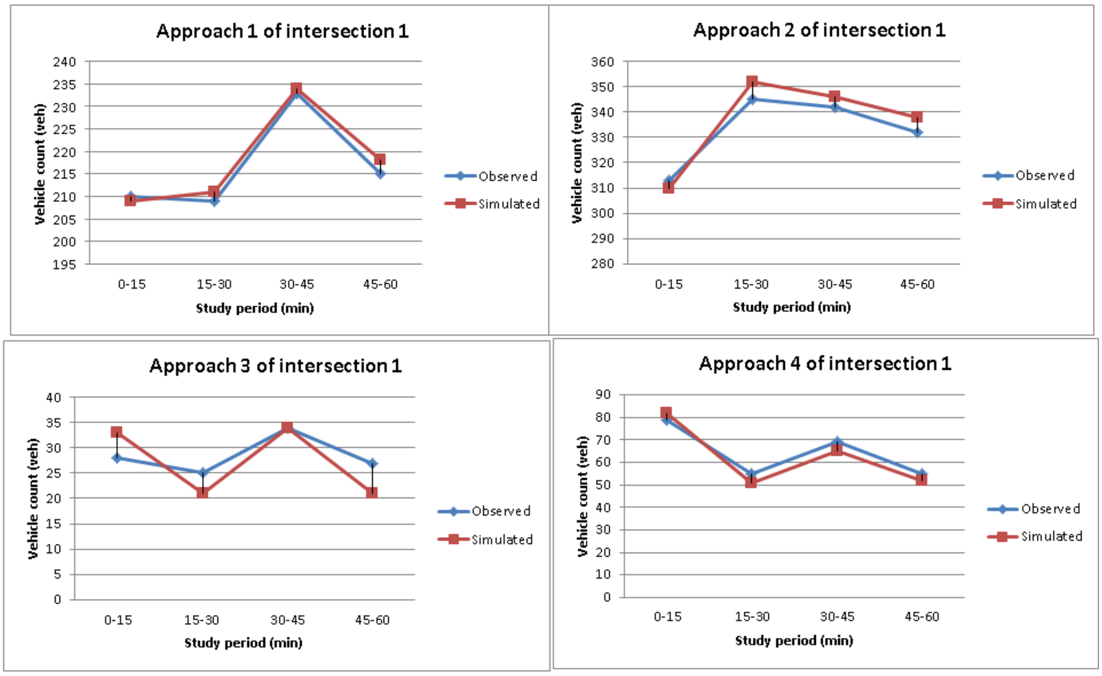

Figure 5.10 : Calibration of traffic volume on all approaches of intersection 1.

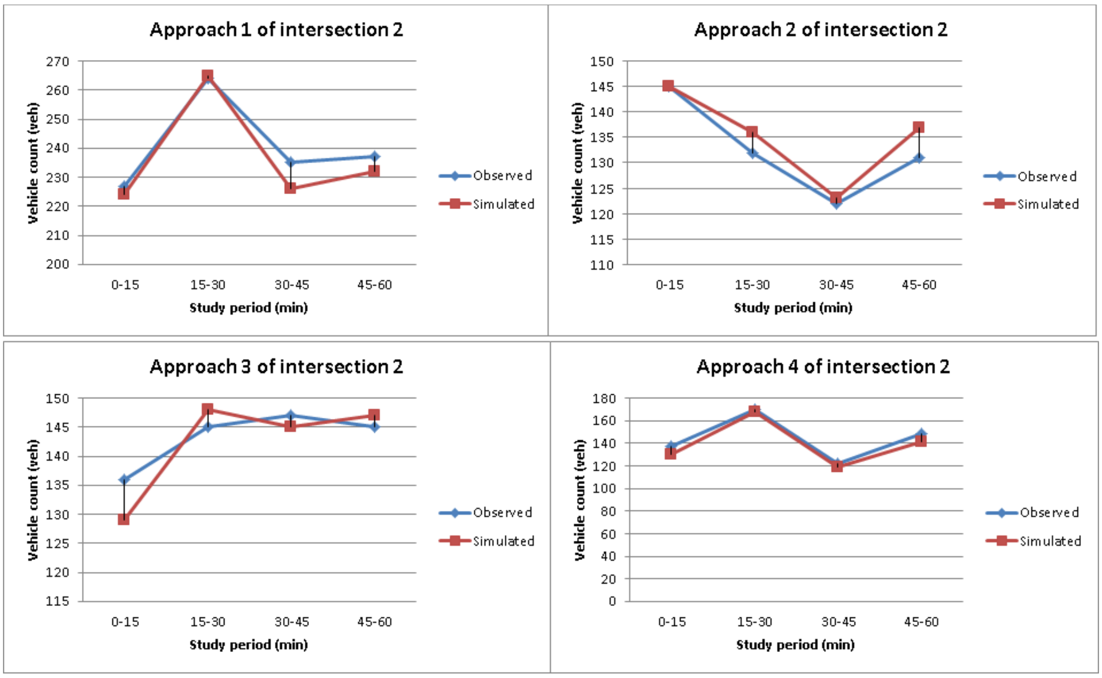

Figure 5.11 : Calibration of traffic volume on all approaches of intersection 2.

The calibration was carried out on the Following and Lane changing parameters. In Following section, the value of 4 and 8 was assigned as observed vehicles for intersection 1 and intersection 2 respectively. For the car following model parameters , the amount of CC0 (Standstill Distance) was set to 1.50m in intersection 1 and 1.30m in intersection 2. Moreover, the value of 0.90s was selected for CC1 (Headway Time) in intersection 1, while the value of 0.80s was assigned to this parameter in intersection 2. In lane changing section, maximum deceleration parameter was set to -4m/s^2 and -3m/s^2 in intersection 1 and intersection 2 respectively. In addition, value of 60s was selected as waiting time before diffusion parameter for intersection 1, and 20s for intersection 2. Moreover, maximum deceleration for cooperative braking parameter was set to -3m/s^2 in intersection 1, and -1m/s^2 in intersection 2. No adjustments were found to be necessary for other driving behavior parameters.

Final vehicle counts derived through VISSIM after calibration were used as input data for computations with HCM and Australian methods. The final volumes for each lane group in intersection 1 and intersection 2 obtained employing VISSIM are shown in Table 5.12.

Table 5.12 : Obtained volume through VISSIM for intersection 1 and intersection 2 using real data.

	Approach No	Lane group	HV%	Volumes derived by VISSIM (veh/h)
Intersection 1	1	1-2,4	0	68
		All GREEN		762
	2	2-1,3,4	2.3	1425
	3	3-1,4	3.55	127
		3-2		25
	4	4-1,2,3	2	332
Intersection 2	1	1-2,4	2.75	982
		1-3 (left turn lane)		152
	2	2-1,3	10	544
		2-4 (left turn lane)		10
	3	3-4,2,1	4.4	624
	4	4-1,3,2	2.4	731

Table 5.13 includes degrees of saturations of both intersection 1, and intersection 2 based on their lane groups. Note that saturation flow rates displayed in Table 5.13 are calculated according to the HCM 2000 method.

Table 5.13 : Degree of saturation for approaches in intersection 1 and intersection 2 using real data.

	Approach No	Lane group	Volumes derived by VISSIM q (veh/h)	Saturation flow rate s (Veh/h)	Green time g (sec)	Cycle time c (sec)	Degree of saturation x = q/(s*g/c)
Intersection 1	1	1-2,4	68	1551.57	13	110	0.37
	2	2-1,3,4	1425	2848.78	39	110	1.41
	3	3-1,4	127	1325.89	18	110	0.59
		3-2	25	1291.69	18	110	0.12
	4	4-1,2,3	332	1364.00	18	110	1.49
Intersection 2	1	1-2,4	982	1065.64	40	119	2.75
		1-3 (left turn lane)	152	1372.74	20	119	0.66
	2	2-1,3	544	981.69	29	119	2.27
		2-4 (left turn lane)	10	1548.50	10	119	0.02
	3	3-4,2,1	624	2618.07	27	119	1.05
	4	4-1,3,2	731	2605.30	23	119	1.45

5.5 Numerical Implementations Employing Real Data

In the following sections, performances of case studies are evaluated employing Australian method, HCM 2000 method and VISSIM software using real data.

5.5.1 Performances derived by Australian (Akcelik) method

In the following sections the performance of case studies are evaluated utilizing Australian (Akcelik) method.

- **Intersection 1**

In order to estimate performance of intersection 1 based on Australian (Akcelik) method the process described in section 4.1 was followed. Initially, traffic compositions (f_c) for each approach were calculated, and the results were tabulated in Table 5.14. In continue, the saturation flow rate for each lane group estimated. Finally, delay for each approach was calculated. The obtained saturation flow rates for each lane group and their relative parameters are shown in Table 5.15.

Table 5.14 : Computation of traffic composition (f_c) for each approach in intersection 1 employing Australian method.

Intersection 1									
		Left		Through		Right		Total	fc
		Car	HV	Car	HV	Car	HV		
Approach 1	qi (veh)	44	0	44	0	641	147	876	1.168
	ei (tcu/veh)	1	2	1	2	1	2		
	eiqi (tcu)	44	0	44	0	641	294	1023	
		Left		Through		Right		Total	fc
		Car	HV	Car	HV	Car	HV		
Approach 2	qi (veh)	103	1	897	139	188	4	1332	1.108
	ei (tcu/veh)	1	2	1	2	1	2		
	eiqi (tcu)	103	2	897	278	188	8	1476	
		Left		Through		Right		Total	fc
		Car	HV	Car	HV	Car	HV		
Approach 3	qi (veh)	28	2	65	2	17	0	114	1.035
	ei (tcu/veh)	1	2	1	2	1	2		
	eiqi (tcu)	28	4	65	4	17	0	118	
		Left		Through		Right		Total	fc
		Car	HV	Car	HV	Car	HV		
Approach4	qi (veh)	29	1	165	2	61	0	258	1.012
	ei (tcu/veh)	1	2	1	2	1	2		
	eiqi (tcu)	29	2	165	4	61	0	261	

Table 5.15 : Computation of saturation flow for each lane group in intersection 1 employing Australian method.

Approach	Lane group	f_w	f_g	f_c	S_b	$S=(f_w*f_g/f_c)*S_b$
1	1-2,4	0.935	0.995	1.17	1670	1330.174
2	2-1,3,4	1	1.005	1.11	3340	3029.513
3	3-1,4	0.942	0.99	1.04	1670	1504.743
	3-2	0.942	0.99	1.04	1670	1504.743
4	4-1,2,3	1	1	1.01	1670	1650.198

Upon calculating flow ratios (y), green ratios (u), degree of saturation (x), total delays (D), lane group delays and approach delays, results were tabulated in Table 5.16.

Table 5.16 : Computation of approach delays for intersection 1 employing Australian method.

Approach	Lane group	Degree of saturation (x=y/u)	Total delay (D)	Lane group delay (sec/veh)	Approach delay (sec/veh)
1	1-2,4	0.4326	0.8514	45.0723	3.6927
2	2-1,3,4	1.3267	261.7822	661.3445	661.3445
3	3-1,4	0.5158	1.4823	42.0191	41.5428
	3-2	0.1015	0.2717	39.1227	
4	4-1,2,3	1.2295	54.2866	588.6494	588.6494

- **Intersection 2**

As for intersection 1, performance evaluation of intersection 2 was conducted based on the process described in section 4.3.1. Initially, traffic compositions (f_c) for each approach were calculated, and the results were tabulated in Table 5.17. In continue, the saturation flow rate for each lane group estimated. Finally, delay for each approach was calculated. The obtained saturation flow rates for each lane group and their relative parameters which influence the final capacity of the signalized approach, namely, lane width factor, slope factor and traffic composite factor are shown in Table 5.18.

Table 5.17 : Computation of traffic composition (f_c) for each approach in intersection 2 employing Australian method.

		Intersection 2							
		Left		Through		Right		Total	fc
		Car	HV	Car	HV	Car	HV		
Approach 1	qi (veh)	111	3	524	71	185	69	963	1.148
	ei (tcu/veh)	1	2	1	2	1	2		
	eiqi (tcu)	111	6	524	142	185	138	1106	
		Left		Through		Right		Total	fc
		Car	HV	Car	HV	Car	HV		
Approach 2	qi (veh)	4	0	367	21	109	29	530	1.094
	ei (tcu/veh)	1	2	1	2	1	2		
	eiqi (tcu)	4	0	367	42	109	58	580	
		Left		Through		Right		Total	fc
		Car	HV	Car	HV	Car	HV		
Approach 3	qi (veh)	141	49	279	20	84	0	573	1.120
	ei (tcu/veh)	1	2	1	2	1	2		
	eiqi (tcu)	141	98	279	40	84	0	642	
		Left		Through		Right		Total	fc
		Car	HV	Car	HV	Car	HV		
Approach4	qi (veh)	145	150	252	22	9	0	578	1.298
	ei (tcu/veh)	1	2	1	2	1	2		
	eiqi (tcu)	145	300	252	44	9	0	750	

Table 5.18 : Computation of saturation flow for each lane group in intersection 2 employing Australian method.

Approach	Lane group	f_w	f_g	f_c	S_b	$S=(f_w * f_g/f_c)*S_b$
1	1-2,4	0.9	1	1.148	1670	1309.233
	1-3	0.9	1	1.148	1670	1309.233
2	2-1,3	0.935	1	1.094	1670	1427.285
	2-4	0.935	1	1.094	1670	1427.285
3	3-4,2,1	0.9	1	1.120	3340	2683.929
4	4-1,3,2	0.9000	1	1.298	3340	2315.871

Upon calculating flow ratios (y), green ratios (u), degree of saturation (x), total delays (D), lane group delays and approach delays obtained results were tabulated in Table 5.19.

Table 5.19 : Computation of approach delays for intersection 2 employing Australian method.

Approach	Lane group	Degree of saturation (x=y/u)	Total delays (D)	Lane group delay (sec/veh)	Approach delay (sec/veh)
1	1-2,4	2.2382	646.653 3	2363.0000	2066.2000
	1-3	0.6908	5.9072	139.9064	
2	2-1,3	1.5640	170.579 0	1128.8000	1124.9000
	2-4	0.0167	0.0278	49.9902	
3	3-4,2,1	1.0247	40.9304	236.1368	236.1368
4	4-1,3,2	1.6331	251.581 5	1239.0000	1239.0000

5.5.2 Performances derived by HCM 2000 method

In following sections, the performance of case studies are evaluated utilizing HCM 2000 method.

- **Intersection 1**

For computing the performance of intersection 1 according to the HCM2000 method the process which was described in section 4.2 was utilized. Initially, the saturation flow rate for each lane group was estimated. Computed saturation flow rates and their relative parameters for each lane group are shown in Table 5.20. The capacity, volume/capacity ratio, uniform delay (d_1), incremental delay (d_2), control delay (d) and approach delays were calculated according to the discussions presented in section 4.3.2 and final results are shown in Table 5.21.

Table 5.20 : Computation of saturation flow rates in intersection 1 using HCM 2000 method.

Lane group NO	1	2	3	4	5
Lane group names	1-2,4	2-1,3,4	3-1,4	3-2	4-1,2,3
So	1900	1900	1900	1900	1900
Number of lanes (N)	1.00	2.00	1.00	1.00	1.00
Lane width (m)	2.75	3.00	2.80	2.80	3.20
HV%	0.00	2.30	3.55	3.55	2.00
Slope%	-0.01	0.01	-0.02	-0.02	0.00
Number of bus stops per hour	NO	29	29	NO	29
Number of parking maneuver	Forbidden	Forbidden	Forbidden	Forbidden	Forbidden
Fw	0.906	0.933	0.911	0.911	0.956
Fhv	1.000	0.978	0.966	0.966	0.980
Fg	1.005	0.995	1.010	1.010	1.000
Fp	1.000	1.000	1.000	1.000	1.000
Fbb	1.000	0.942	0.884	1.000	0.884
Fa	0.900	0.900	0.900	0.900	0.900
Flu	1.000	1.000	1.000	1.000	1.000
Flt	0.997	0.996	0.987	1.000	0.994
Frt	1.000	0.978	1.000	0.850	0.969
Saturation flow rate	1551.574	2848.783	1325.892	1291.688	1364.000

Table 5.21 : Computation of approach delays based on HCM 2000 method in intersection 1.

Approach	Lane group	Flow rates derived through VISSIM (veh/h)	Capacity C=S*g/c	Volume/Capacity (v/c)	Approach delay (sec/veh)
1	1-2,4	68	183.4792	0.3706	4.4287
2	2-1,3,4	1425	1010.2	1.4106	265.4254
3	3-1,4	127	216.9152	0.5855	59.7371
	3-2	25	211.3671	0.1183	
4	4-1,2,3	332	223.2298	1.4873	307.1945

- **Intersection 2**

As for intersection 1, the methods described in section 4.2 were utilized for computing the performance of the intersection 2 as well. The computed saturation flow rates and their relative parameters for each lane group are shown in Table 5.22. In continue, the capacity, volume/capacity ratio, uniform delay (d_1), incremental delay (d_2), control delay (d) and approach delays were calculated according to the discussions presented in section 4.3.2, and are shown in Table 5.23.

Table 5.22 : Computation of saturation flow rates in intersection 2 using HCM 2000 method.

NO	1	2	3	4	5	6
	1-2,4	1-3 (left turn lane)	2-1,3	2-4 (left turn lane)	3-4,2,1	4-1,3,2
So	1900	1900	1900	1900	1900	1900
Number of lanes (N)	1	1	1	1	2	2
Lane width (m)	2.5	2.5	2.75	2.75	2.5	2.5
HV%	2.75	2.75	10	10	4.4	2.4
Slope%	0	0	0	0	0	0
Number of bus stops per hour	no	no	38	no	16	38
Number of parking maneuver	21	forbidden	10	forbidden	forbidden	forbidden
Fw	0.878	0.878	0.906	0.906	0.878	0.878
Fhv	0.97	0.97	0.91	0.91	0.96	0.98
Fg	1.00	1.00	1.00	1.00	1.00	1.00
Fp	0.80	1.00	0.85	1.00	1.00	1.00
Fbb	1.00	1.00	0.85	1.00	0.97	0.92
Fa	0.90	0.90	0.90	0.90	0.90	0.90
Flu	1.00	1.00	1.00	1.00	1.00	1.00
Flt	1.00	0.99	1.00	1.00	0.98	0.98
Frt	0.960	1.000	0.960	1.000	0.980	0.998
Saturation flow rate	1065.64	1372.74	981.69	1548.50	2618.07	2605.30

Table 5.23 : Computation of approach delays based on HCM 2000 method in intersection 2.

Approach	Lane group	Flow rates derived through VISSIM (veh/h)	Capacity C=S*g/c	Volume/Capacity (v/c)	Approach delay (sec/veh)
1	1-2,4	985	374.757	2.6284	698.6474
	1-3	152	243.062	0.6254	
2	2-1,3	544	237.386	2.2916	649.7273
	2-4	10	118.296	0.0169	
3	3-4,2,1	624	606.533	1.0288	136.4124
4	4-1,3,2	731	509.446	1.4349	284.9243

5.5.3 Performance derived by incorporating VISSIM

In following sections, the performances of intersections are derived upon modeling them in VISSIM software.

- **Intersection 1**

Upon modeling the intersection 1 in VISSIM platform and assigning required parameters, the simulation defined for exhibiting peak hour characteristics was started and after simulation finished the results were recorded. Table 5.24 exhibits the performances derived by VISSIM for each approach of intersection 1.

Table 5.24 : Approach delays of intersection 1 derived through VISSIM.

Approach	Delay (sec/veh)
1	5.24
2	34.66
3	40.84
4	46.10

- **Intersection 2**

As for intersection 1, the intersection 2 was modeled in VISSIM, and subsequently all necessary factors were defined for all model elements. Finally, the simulation defined for exhibiting peak hour characteristics was run and at the end of simulation all obtained results were noted. Table 5.25 exhibits the performances derived by VISSIM for each approach of intersection2.

Table 5.25 : Approach delays of intersection 2 derived through VISSIM.

Approach	Delay (sec/veh)
1	40.49
2	40.74
3	44.25
4	47.03

5.6 Comparative Evaluations

The performances of intersections according to the Australian, HCM 2000 and VISSIM were derived in sections 4.1, 4.2 and 4.3, and the results are shown in Figure 5.12 and Figure 5.13 for intersection 1 and intersection 2 respectively.

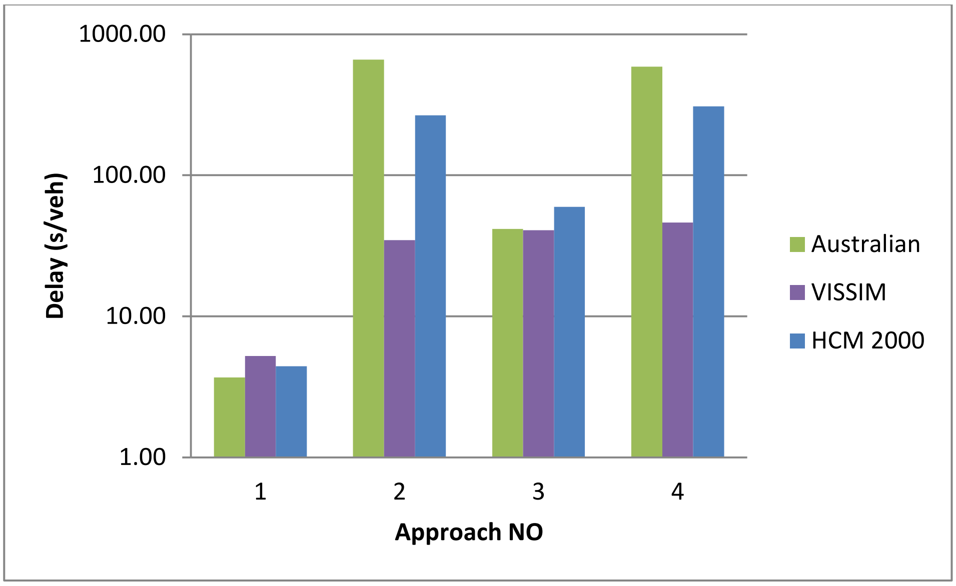

Figure 5.12 : Derived delays using Australian method, HCM2000 method and VISSIM in intersection 1.

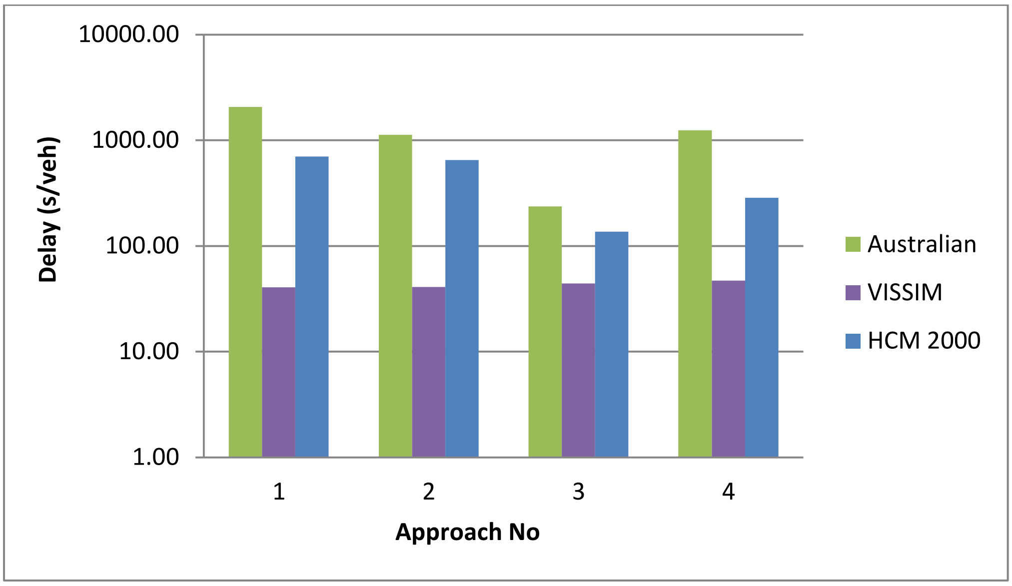

Figure 5.13 : Derived delays using Australian method, HCM2000 method and VISSIM in intersection 2.

As it is obvious from Figure 5.12 and Figure 5.13, on each approach of intersection 1 and intersection 2, amounts of delays differ from one another. For intersection 1, on approach 1, delay obtained from VISSIM has the maximum amount compared to the other methods. For approach 2, Australian and HCM 2000 results are almost identical, and greater than VISSIM delay. On approach 3, all delays are nearly equal, and finally, on approach 4 delay derived utilizing Australian method is greater than other delays, while delay of VISSIM is the lowest delay on this approach. For intersection 2, similarly, on each approach either of methods result in different delays.

For understanding the reason that causes differences among the obtained delays of Australian and HCM 2000 methods, and regarding that the approaches which consist of one lane group possess only one degree of saturation, and for the sake of simplicity, in each intersection, two approaches that include only one lane group was selected. Then, delays for different degrees of saturations which ranges from little amounts near zero to the amounts above 1 on designated approaches were calculated incorporating MATLAB software. Figure 5.14 until Figure 5.21 illustrate variations of delays as a function of saturation degree in intersection 1 and intersection 2. As it is obvious from these graphs, the greater degree of saturation is, the greater amount of delay from Australian method compared to the HCM 2000 method will be derived. According to Figure 5.14 and Figure 5.15, on approach 2 of intersection 1, the amount of delays obtained from Australian method is less than HCM 2000 delays until the degree of saturation is 1. This condition is more visible in Figure 5.15 whose vertical axis is in logarithmic scale. However, as degree of saturation takes the amounts above 1, delays derived using Australian method are always greater than that of HCM 2000.

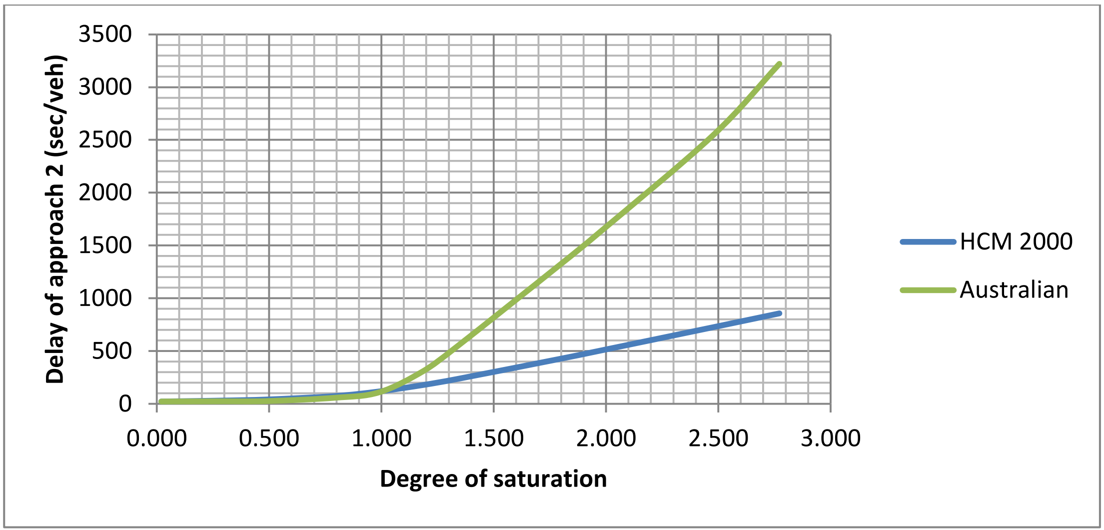

Figure 5.14 : Delay functions of Australian and HCM 2000 method on approach 2 of intersection 1.

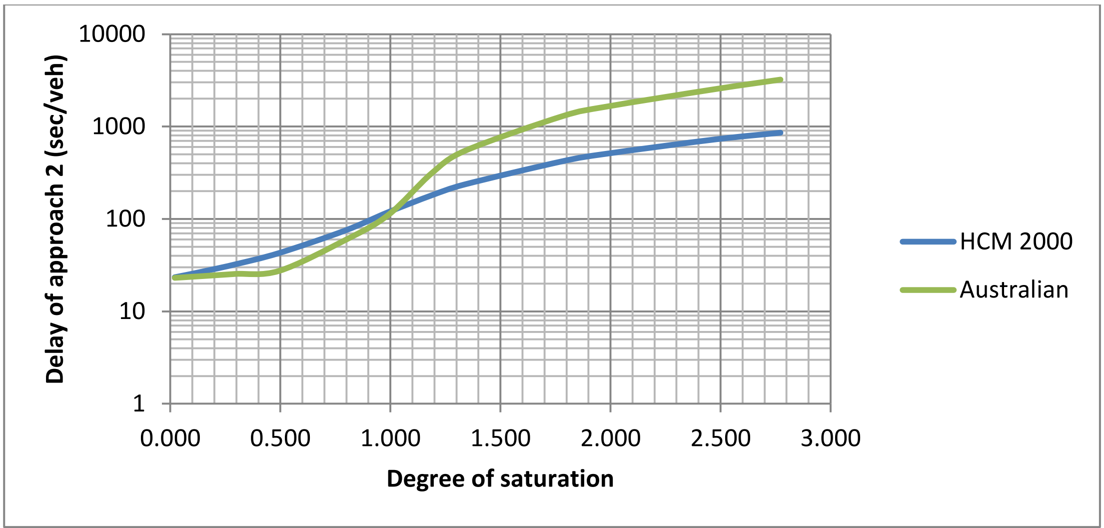

Figure 5.15 : Delay functions of Australian and HCM 2000 method on approach 2 of intersection 1 with logarithmic scale.

Similarly, as shown in Figure 16 and Figure 17, for approach 4 of intersection 1, delays of Australian method outgo that of HCM 2000 method at the point on which degree of saturation is 0.95. For observing the point at which Australian method function outgoes that of HCM 2000 method clearly, the vertical axis of Figure 5.17 is drawn in a logarithmic scale.

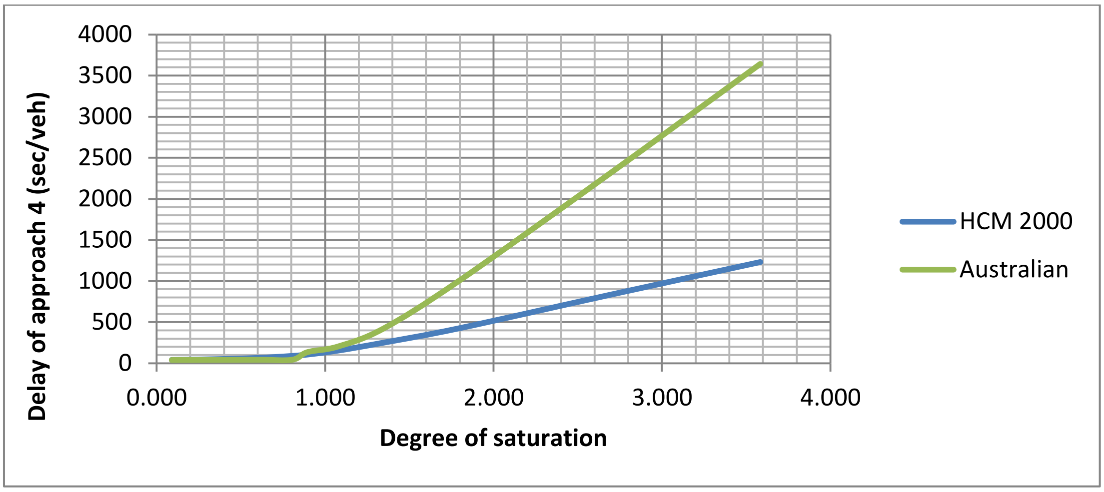

Figure 5.16 : Delay functions of Australian and HCM 2000 method on approach 4 of intersection 1.

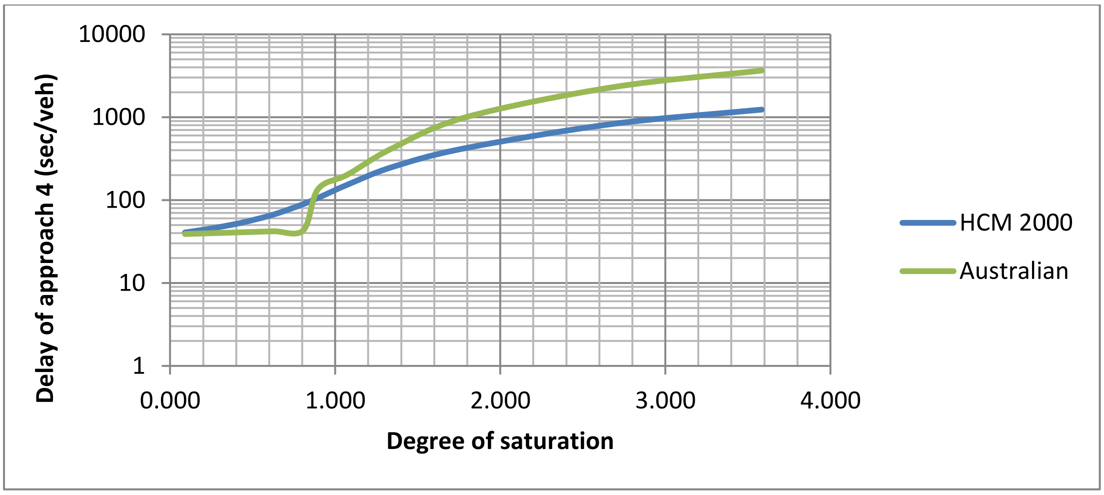

Figure 5.17 : Delay functions of Australian and HCM 2000 method on approach 4 of intersection 1 with logarithmic scale.

Figure 5.18 and Figure 5.19 illustrate the variation of delays as a function of degree of saturation on approach 3 of intersection 2, and it is obvious that Australian method delays outgo delays of HCM 2000 method at the point whose degree of saturation is 0.80. Figure 5.19 presents delays in a logarithmic scale.

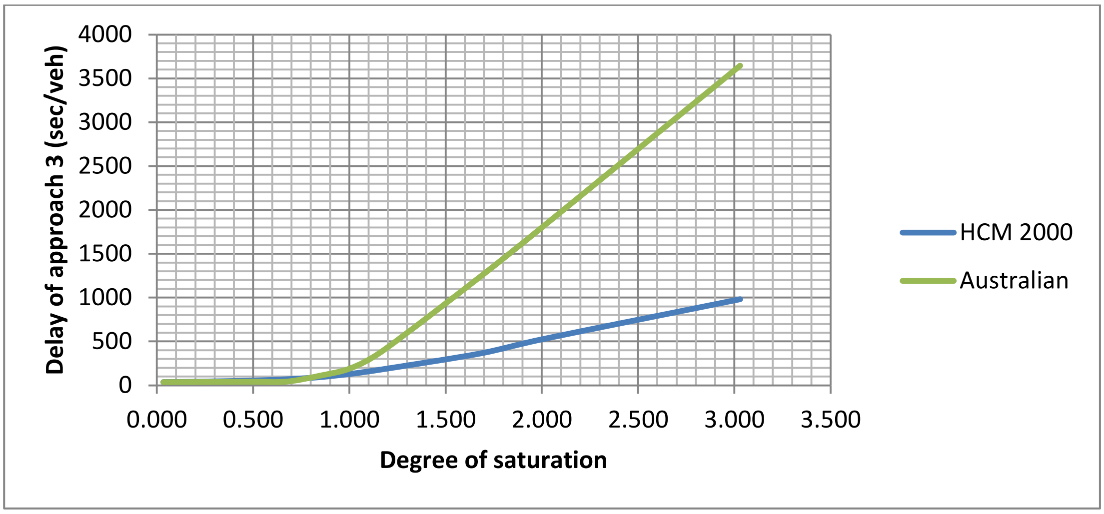

Figure 5.18 : Delay functions of Australian and HCM 2000 method on approach 3 of intersection 2.

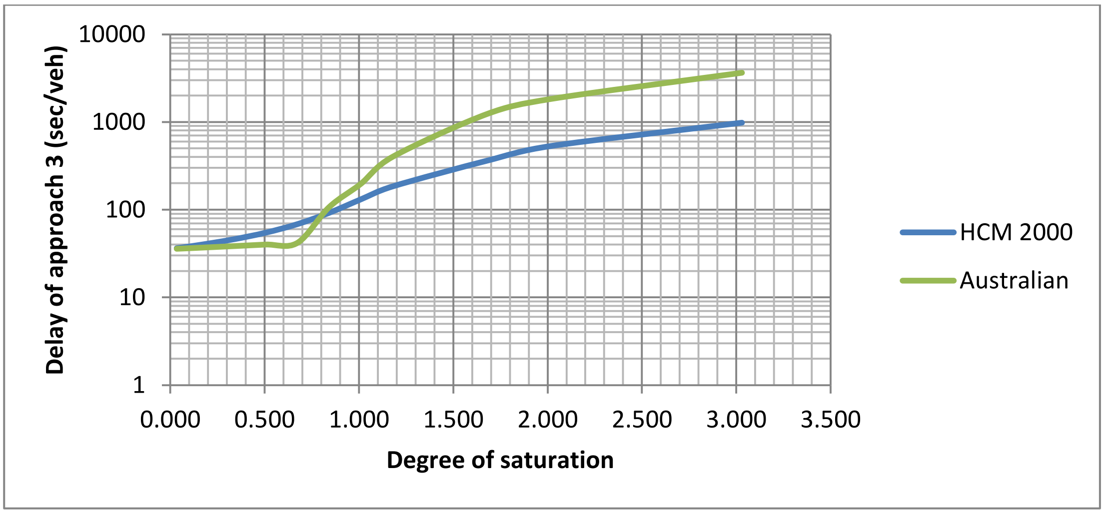

Figure 5.19 : Delay functions of Australian and HCM 2000 method on approach 3 of intersection 2 with logarithmic scale.

Finally, Figure 5.20 and Figure 5.21 demonstrate delay alteration on approach 4 of intersection 2 as a function of degree of saturation. Delays of Australian method are below the delay function of HCM 2000 until the degree of saturation of 0.70, but after this point Australian delay function lies on top of that of HCM 2000 method. Figure 5.21 shows delays in a logarithmic scale.

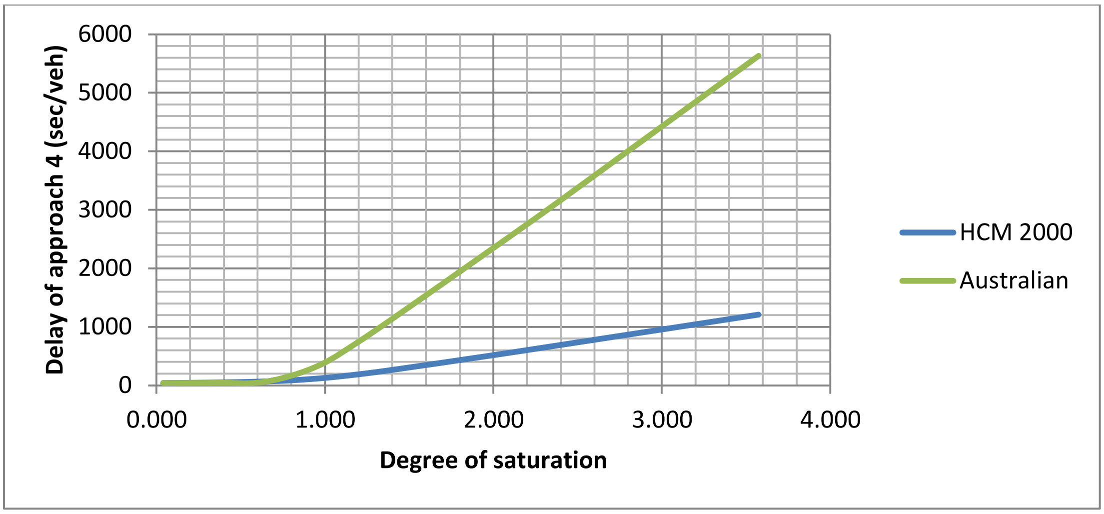

Figure 5.20 : Delay functions of Australian and HCM 2000 method on approach 4 of intersection 2.

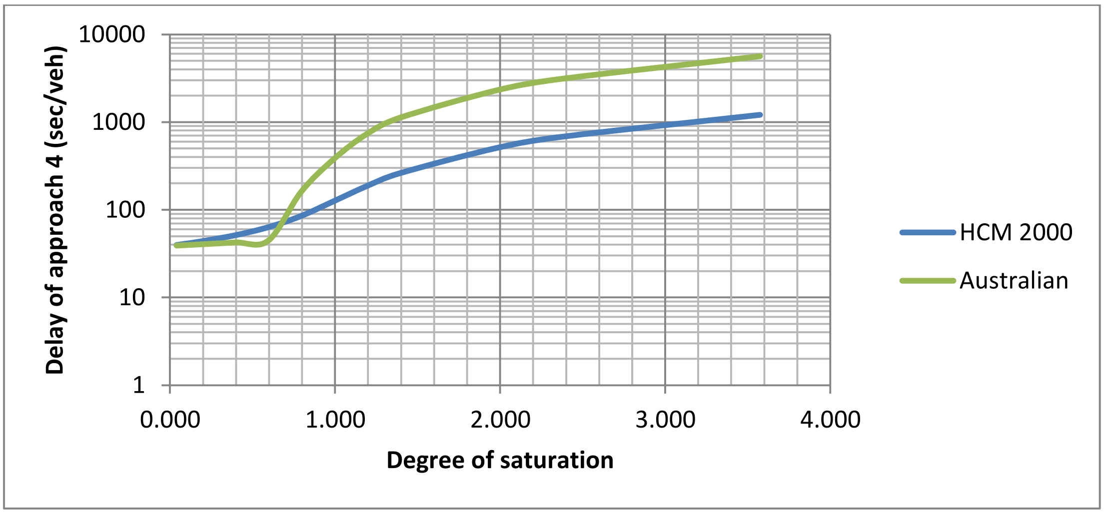

Figure 5.21 : Delay functions of Australian and HCM 2000 method on approach 4 of intersection 2 with logarithmic scale.

Considering all above-mentioned discussions, that Australian method is more sensitive to the variation of saturation degree than HCM 2000 method can be inferred. In summary, delays derived using Australian method is less than that of HCM 2000 method until the degree of saturations of about 1, but after this point slope of delay function increase, and outgoes the HCM 2000 method such that the more degree of saturation is, the greater gap between these two methods will be. Thereby, for the degree of saturations less than 1 both Australian and HCM 2000 methods calculate nearly identical delays, but for the saturation flow rates more than one which is the case during the peak hours, delays obtained using Australian method is always more than that of HCM 2000 method. That is, for the oversaturated four-legged signalized intersections the Australian method is more appropriate than HCM 2000 method for calculating cycle lengths whose formulating is based on minimizing delays for a particular degree of saturation.

As shown in Figure 5.22, Figure 5.23, Figure 5.24 and Figure 5.25, in order to consider the sensitivity of VISSIM to the degree of saturation compared to the sensitivity of Australian and HCM 2000 methods to the variations in degrees of saturations, on approaches 2 and 4 of intersection 1, and approaches 3 and 4 of intersection 2, delays derived for various degrees of saturations utilizing Australian and HCM 2000 methods as well as VISSIM program. As it is clear from Figure 5.22, once degree of saturation on approach 2 of intersection 1 is about 0.20 delays derived from VISSIM are almost equal to other methods, but for the degree of saturations above 0.80 delays of VISSIM are always less than that of other methods.

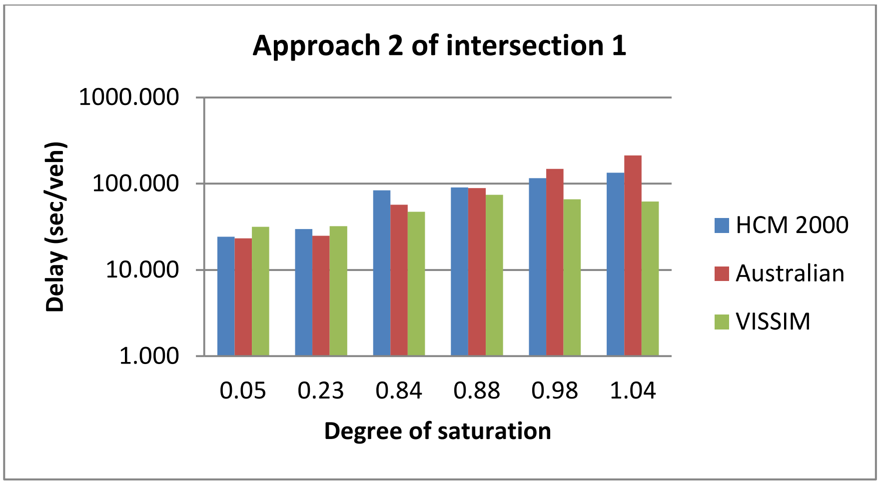

Figure 5.22 : Delays on approach 2 of intersection 1 according to Australian, HCM 2000 and VISSIM methods.

Similarly, from Figure 5.23, it can be discerned that on approache 4 of intersection 1, as degree of saturation is 0.09 delays of all methods are almost equal to one another. For the degree of saturations above 0.80, however, delays derived incorporating VISSIM is less than that of other methods.

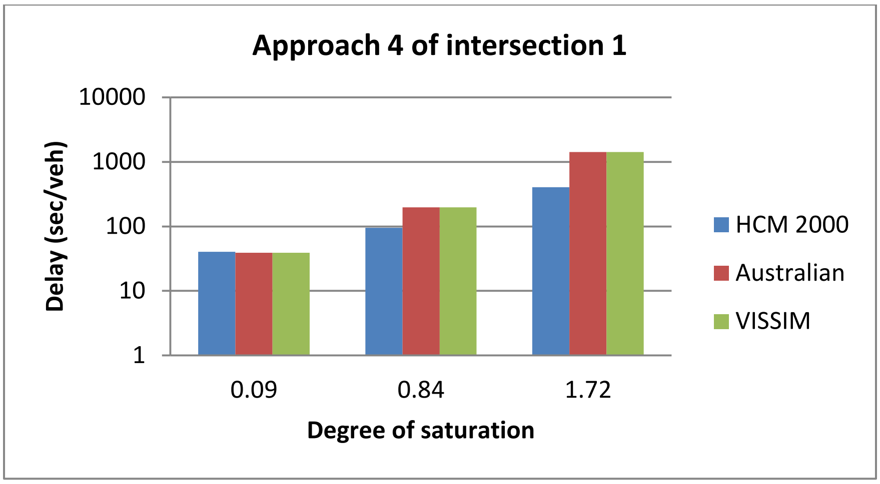

Figure 5.23 : Delays on approach 4 of intersection 1 according to Australian, HCM 2000 and VISSIM methods.

Figure 5.24 illustrates that when degree of saturation is 0.07, delay of VISSIM is nearly equal to other methods, whereas for the degrees of saturations above 0.34 delays of VISSIM is the lowest amount.

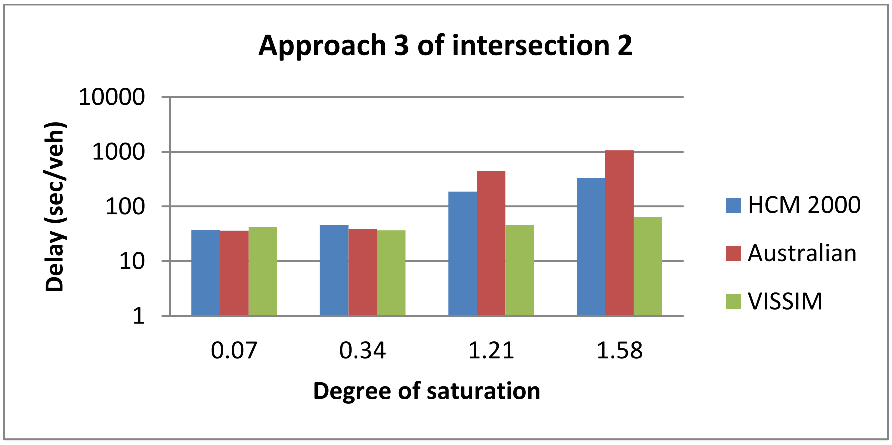

Figure 5.24 : Delays on approach 3 of intersection 2 according to Australian, HCM 2000 and VISSIM methods.

Finally, Figure 5.25 exhibits that on approach 4 of intersection 2 for the degree of saturation 0.06 delays derived from all methods is approximately identical, but as degree of saturation increases the gap between delays of VISSIM enhance with that of other methods so that delays derived using VISSIM is less than other methods.

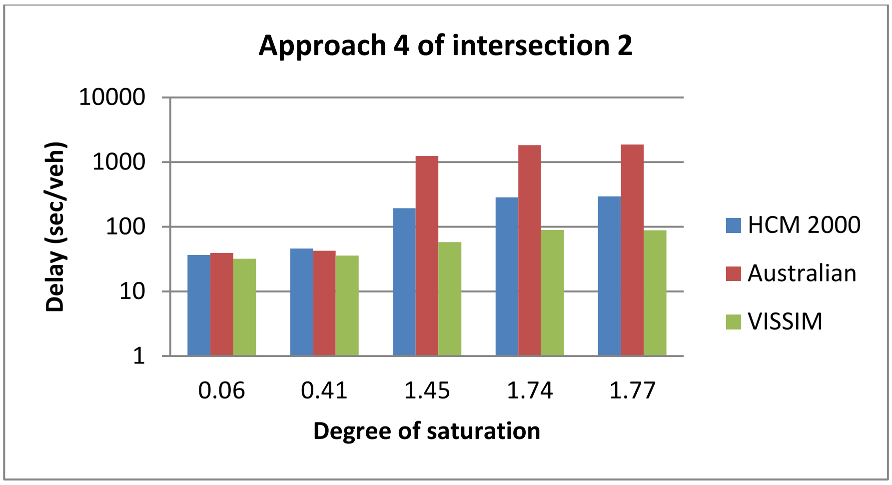

Figure 5.25 : Delays on approach 4 of intersection 2 according to Australian, HCM 2000 and VISSIM methods.

6. CONCLUSIONS AND RECOMMENDATIONS

In this study, two four-legged signalized intersections in Pendik area of Istanbul city of Turkey were designated as case studies. Through camera recording methods traffic volumes obtained during the 7:00-10:00, 12:00-14:00 and 16:30-19:30 time periods. Then, the peak hour volumes were calculated through these recordings, and were used as input data for computing the delays, the performance measure for this study, in all approaches of the intersections incorporating Australian (Akçelik) and USA (HCM 2000) methods as well as VISSIM program. Considering all previous-mentioned discussions and analyses, the following conclusions can be drawn:

Delays derived incorporating VISSIM on all approaches of both intersections except approach 1 of intersection 1 are less or equal to that of Australian and HCM2000 methods. Delays derived using Australian method are greater or equal to that of HCM2000 method on all approaches of both intersections. Moreover, Australian method is much more sensitive to the degree of saturation than HCM2000 method as well as VISSIM so that delays derived utilizing Australian method are less than that of HCM2000 method until the degree of saturation about 1, and after this point Australian method outgoes HCM2000 method. Therefore, the more degree of saturation is, the greater gap between delays of these two methods will be. As a result, it can be concluded that for the oversaturated four-legged signalized intersections the Australian method is more appropriate than HCM 2000 method for calculating cycle lengths whose formulating is based on minimizing delays for a particular degree of saturation.

Delays derived incorporating VISSIM except the cases that degree of saturation is little (about 0.20), are less than that of Australian and HCM2000 method, and this gap is increased as degree of saturation enhances.

As volumes increase, obtained delays are enhanced in Australian and HCM2000 methods (deterministic methods), so that the slope variation of delay-volume function of Australian method after the degree of saturations of about 1 increases by far rapidly compared to that of HCM2000 method.

The above-mentioned discrepancies among the outcomes of various performance evaluation methods are result due to the basic differences between the hypotheses and initial assumptions that the model developers of each of these methods have utilized for developing an equation to replicate the realistic circumstances of a transportation network. For instance, as discussed before, Australian method is by far sensitive to the variation of degree of saturation, and this is because of the different assumptions above which the model has been developed compared to the other methods. For VISSIM, its nature, and the process it employs for evaluating a transportation network is completely different with Australian and HCM 2000 methodologies. VISSIM is a microscopic simulation software which uses dynamic and stochastic methods for conducting an evaluation unlike the Australian and HCM 2000 methods which utilize deterministic approaches. Moreover, in VISSIM saturation flow rate is variable and varies during the study period which is the case in realistic conditions. However, Australian, and HCM 2000 methods assume that the saturation flow rate is constant during the study period. Thereby, this is one of the significant factors that generate a discrepancy between the results of the VISSIM software with that of Australian, and HCM 2000 methods.

In addition, it should be noted that the type of signal controls employed in this study were pre-timed, and the signal timing were fixed all over the day. However, deploying actuated signal controls whose signal timing varies as a function of traffic volume during a day can improve the performance of the network with allocating the most appropriate signal timing for each period of time considering the amount of traffic volume is available for that time interval in the transportation network.

This study was conducted for the signalized case studies with four leg, while similar studies can be done for T intersections. Moreover, in this study only one type of phasing diagram as well as cycle length was considered for the case studies. Therefore, more researches with different phasing diagrams and cycle length can consider various aspects of performance evaluation issue in signalized intersections.

REFERENCES

[1] **STM**. (2008). Signal Timing Manual. Department of Transportation, *Federal highway administration FHWA*. Washington, D.C, USA.

[2] **HCM**. (2000). Transportation Research Board Publications, *2000 Executive Committee,* Washington, D.C, USA.

[3] **DKTR**. (2005). Dönel Kavşaklar Tasarım Rehberı. *Karayolları Genel Müdürlüğü Planlama Şubesi Müdürlüğü.* Ankara, Türkiye.

[4] **Oğut, K**. (2013). Trafik Yönetimi Ders Notları. İstanbul Teknik Üniversitesi. İstanbul, Türkiye.

[5] **Eraslan, O**. (2008). Işıklı Kavşaklarda Amerikan ve Avustralya yöntemler ile Gecikme Analizi ve Örnek Bir Kavşak Çözümü. *Yüksek lisans tezi*. İstanbul Teknik Üniversitesi. İstanbul, Türkiye.

[6] **MUTCD**. (2009). Manual On Uniform Traffic Control Devices. Department of Transportation, *Federal highway administration FHWA*. Washington, D.C, USA.

[7] **Draper, D and Fouskakis, D.** (2002). Stochastic Optimization: a Review. *Internatinal statistical institute*, Netherland.

[8] *http://chercheurs.lille.inria.fr/~ghavamza/RL-EC-Lille/Lecture5-a.pdf.* date retrieved 2014.04.02.

[9] **Andradottir, S.** (1990). A New Algorithm for Stochastic Optimization, *Proceedings of the Winter Simulation Conference, 364-366.* Wisconsin university, USA, 9-12 September.

[10] **Leung, Y, T. and Suri, R.** (1990). Finite-Time Behavior of Two Simulation Optimization Algorithms, *Proceedings of the Winter Simulation Conference*, *372-376.* State university of New York at Binghamton, USA, 9-12 December.

[11] **Shapiro, A.** (1996). Simulation Based Optimization, *Proceedings of the Winter Simulation Conference, 332-336.* Georgia Institute of Technology, USA, 8-11 December.

[12] **Gurkan, G., Ozge, A, Y, and Robinson, S. (**1994). Sample-Path Optimization in Simulation, *Proceedings of the Winter Simulation Conference, 247-254.* University of Wisconsin, USA, 11-14 December

[13] **Carson, Y and Maria, A,** (1997). Simulation optimization : Methods and applications. *Proceedings of the Winter Simulation Conference,118-126.* State university of New York at Binghamton, USA, 7-10 December.

[14] **Marti, R & Reinelt, G**, (n.d.). The linear ordering problem, Exact and Heuristic methods in combinatorial optimization. Date retrieved: 2013.11.05, address: http://www.springer.com/978-3-642-16728-7

[15] **Combinational Optimization Theme Semester,** (n.d.). Date retrieved: 10.10.2013,address: /http://www.crm.umontreal.ca/Hybrid06/index_e.html

[16] **Hajjiri, S, A. and Stephanedes, Y, J**. (1988). Urban congestion reduction for energy conservation control strategies for urban street systems a state of the art.*The center for transportation studies*, Department of civil and mineral engineering,University of Minnesota, USA.

[17] **Udoh, N and Ekpenyong,E**. (2012). Analysis of Traffic Flow at Signalized Junctions in Uyo Metropolis. Studies in Mathematical Sciences ISSN 1923-8452,Online. Vol. 5, no. 2, pp.72-89.

[18] **Zhang, G and Wang, Y. (**2011). Optimizing Minimum and Maximum Green Time Settings for Traffic Actuated Control at Isolated Intersections. IEEE Transaction on intelligent transportation systems, Vol. 12, no. 1. pp. 164-173.

[19] **Panwai, S and Dia, H. (**2005). Comparative evaluation of microscopic car-following behavior. IEEE Transactions on intelligent transportation systems, Vol 6, no 3, pp. 314-315.

[20] **Pulugurtha, S and Desai, A. (**2007). Comparative Evaluation of Synchro and VISSIM Traffic Simulation Software to Model Railroad Crossings. *Proceding of the Workshop on Computing in Civil Engineering,* American society of civil engineers (ASCE), Pittsburgh, Pennsylvania, USA, July 24-27.

[21] **Choa, F, Milam, T and Stanek, D**. (2003). CORSIM, PARAMICS, and VISSIM: What the Manuals Never Told You. *Proceedings of the*

Ninth TRB Conference on the Application of Transportation Planning Methods, Radisson Hotel and Conference Center, Baton Rouge, LA, USA, April 6-10.

[22] **Akçelik, R. (**1981). Traffic Signals: Capacity and Timing Analysis. *Australian Road Reasearch Board*. Research Report ARR NO.123-7th reprint.

[23] **Ohno, K. and Mine, H.** (1973). Optimal traffic signal settings: Criterion for undersaturation of a signalized intersection and optimal signal setting. *Transportation research,* Vol.7, Issue 3, pp. 243-267.

[24] **Webster, F.V. and Cobbe, B.M.** (1966). Traffic Signals, *Road Research Technical Paper* no.56, HMSO London.

[25] <http://www.civil.iitb.ac.in>, date retrieved 20.10.2013.

[26] **Daganzo, C** (1997). Fundamentals of Transportation and Traffic Operations, Pergamon-Elsevier, Oxford, U.K.

[27] **El-Zohairy, Y and Benekohal, R**. (2001). Multi-Regime arrival rate uniform delay models for signalized intersections. *Transportation research part A: policy and practice*. Vol 35, Issue 7, pp. 625-667.

[28] **Drew, D, R.** (1968). Traffic flow theory and control, McGraw-Hill. Chapter eleven. New York, USA.

[29] *http://www.mtechthailand.com/mtech/news/partnership-ptv-group/* date retrieved 2014.03.04.

[30] *http://vision traffic.ptvgroup.com/fileadmin/files_ptvvision/Downloads_N/0_General/2_Products/2_PTV_Vissim/EN-US_PTV_Vissim_Brochure.pdf.* date retrieved 2014.04.09

[31] **Gallelli, V & Vaiana, R.** (2008). Roundabout intersections: evaluation of geometric and behavioural features with vissim. *TRB national roundabout conferenc*, Kansas city, Missouri, USA. 18-21 May.

[32] **PTV AG. (**2011). Planung Transport Verkehr. *Vissim 5.40-01 user manual.* Karlsruhe, Germany.

[33] **Wiedemann, R.** (1974). Simulation des Straßenverkehrsflusses. Schriftenreihe des Instituts für Verkehrswesen der Universität Karlsruhe, Heft 8.

[34] **Wiedemann, R.** (1991). Modelling of RTI-elements on multi-lane roads, *Drive conference*, Vol 2, pp. 1001-10, Brussels, Belgium, March 16-18.

[35] **Fellendorf, M**. (1994). VISSIM: A microscopic Simulation Tool to Evaluate Actuated Signal Control including Bus Priority. *Technical paper, Session 32. 64th ite meeting,* October, Dallas.

[37] **Park, B and Qi, H.** (2006). Microscopic Simulation Model Calibration and Validation for Freeway Work Zone Network – A Case Study of VISSIM. *IEEE Intelligent Transportation Systems Conference*, , Toronto, Canada, September 17-20.

[38] **Huaguo, Z. (**2008). Evaluation of route diversion strategies using computer simulation. *Journal of transportation engineering and information technology*. Vol 8, no 1, pp.61-67.

[39] *https://www.google.com/maps/place/Adnan+Menderes+Bulvar, date retrieved: 2014.02.04.*

[40] **Antoniou, C, Barcelo, J, Brackstone, M, Celikoglu, H, Ciuffo, B, Punzo, V,** et all. (2014). Traffic Simulation : Case for Guidelines. *JRC Science and Policy Reports,* European Commision.

[41] **Young, W., Taylor, M. A. P., and Gipps, P. G.** (1989). Microcomputers in traffic engineering, *Research Studies Press,* Taunton, England.

[42] **Archer, J and Young, W.** (2010). Signal Treatments to Reduce the Likelihood of Heavy Vehicle Crashes at Intersections: Microsimulation Modeling Approach. *Journal of transportation engineering*. American society of civil engineers (ASCE), Vol. 136, July, Issue 7, pp. 632-639.

CURRICULUM VITAE

Name Surname: Farid Javanshour

Place and Date of Birth: Iran- Tabriz- 1985.11.28

E-Mail: fjcivil@yahoo.com

B.Sc.: Civil Engineering- University of Tabriz

Professional Experience and Rewards:

Working in IRDAK Company, a prefabricated concrete factory, as a civil engineer.

List of Publications and Patents:

Javanshour.F and Oğut.K, 2013. "Comparison of Minibus, a Paratransit Mode and Conventional Bus Transportation In Istanbul". *Transist 6th International Transportation Symposium, December 25-26, Istanbul, TURKEY.*

Javanshour.F and Çelikoğlu.H, 2014. "Performance Analyses of Four-Legged Signalized Intersections: A Case Study". *Euro Working Group On Transportation (EWGT), July 2-4, Seville/Spain.*

PUBLICATIONS/PRESENTATIONS ON THE THESIS

- **Javanshour.F and Çelikoğlu.H, 2014.** "Performance Analyses of Four-Legged Signalized Intersections: A Case Study". *Euro Working Group On Transportation (EWGT), July 2–4. Seville/ Spain.*