Development of Model for Pre-Timed Traffic Signal Coordination on Urban Road Corridor

A Thesis submitted to Gujarat Technological University

For the Award of

Doctor of Philosophy

In

Civil Engineering

By

Pranaykumar Mahendrakumar Shah
Enrollment No.: 129990906004

Under supervision of

Dr. H. R. Varia

GUJARAT TECHNOLOGICAL UNIVERSITY
AHMEDABAD
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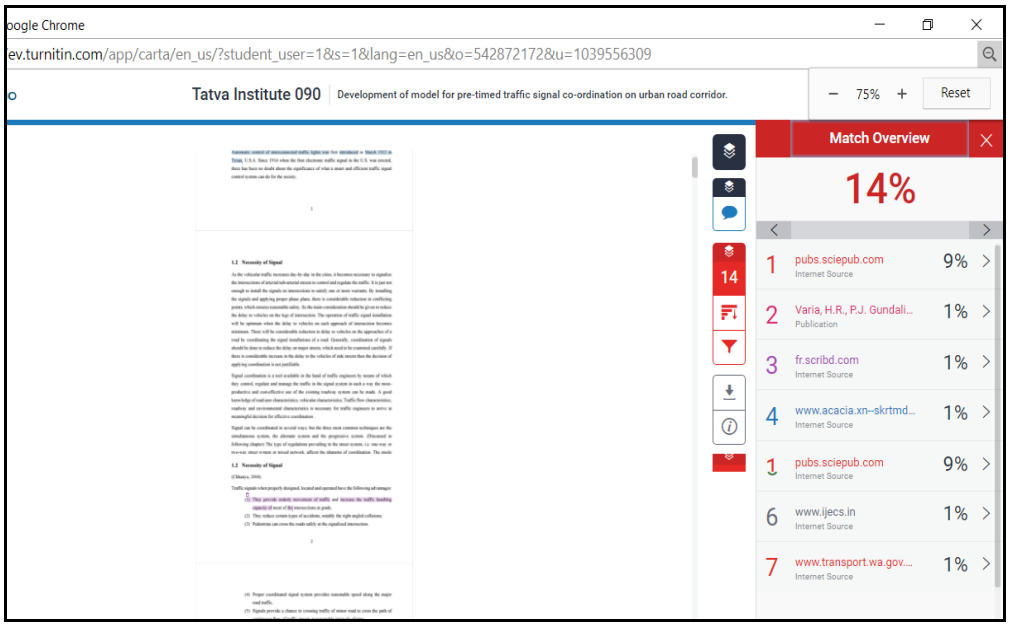

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Abstract

Road traffic congestion is a critical problem accelerated by an exponential growth in the number of vehicles, ceaseless urbanization and has become a phenomenon visible in urban area all over the world. The nature of traffic flow and movement condition in India is quite different than other developed countries. Indian traffic has mixed vehicle flow and it operates through Left Hand Traffic (LHT) with Right Hand Drive (RHD). Traffic congestion on major corridor generally reflects existing lacuna in signal control strategy and limitation of available road space in most of the Indian cities. Signal coordination is a tool which can control, regulate and manage the traffic in the signal system in such a way that most-productive and cost-effective use of the existing roadway system can be made.

If busy urban arterial is having signalized intersections in series with constant spacing, then two-way coordination of pre-timed traffic signals can be achieved by alternative or double alternative or simultaneously progressive system. However, when it has diverse number of approaches and different spacing between each other, then the methodology other than traditional progressive system needs to be worked out. In this research two methodologies: (i) Two way traffic signal coordination strategy1 (TW_TSCS1) and (ii) Two way traffic signal coordination strategy 2 (TW_TSCS2) are developed for the pre-timed signals. The strategies for two way coordination between 3 arm, 4 arm and 5 arm intersections have been discussed considering important parameters, i.e. travel time, link distance, saturation flow rate (SFR), demand flow rate (DFR) and width of approach. Considering the vital importance of SFR and practical difficulty to accurately estimate the SFR, after exploring available past literature on SFR values for Indian traffic condition, the study has proposed “low” SFR values and “high” SFR values that can be conveniently used to compute appropriate cycle length required for two-way coordination on signalized intersections.

This study has developed a computational model to derive optimum cycle length and phase length when travel time in outbound and inbound direction is different. The model culminates with development of obtaining optimum cycle length and phase length required for two-way coordination. The methodology and model leads to development of Delay Minimization Schemes (DMS). A Meta heuristic optimization method of Genetic Algorithm (GA) is applied to obtain the best alternative of DMS. A computer program developed with GA is able to generate optimum phase length for total delay minimization along the corridor. For the selected corridor in Ahmedabad city of India, this study has

derived Dynamic Passenger Car Unit (DPCU) values for the signalized intersection and developed relationship for actual travel time and travel time calculated by space mean speed at mid-block of link. The validation of the derived methodology and model was conducted applying analytical approach and field implementation strategy. Actual implementation of the developed methodology is successful in reduction of about 30 % travel time along corridor and about 21.5% total stopped delay on the corridor. Statistical analysis was performed for the traffic flow parameters observed before and after implementation of two-way coordination. Efforts have also been made to establish relationship of observed stopped delay with delay obtained by time space diagram along with observed flow values and approach width. Satisfactory and encouraging results have been obtained while validation. The developed methodology can be applied effectively for two way coordination of pre-timed signals on the corridor having different link lengths between signalized intersections of 3 arm or 4 arm or 5 arm and also varying speeds of inbound and out bound traffic flow for the Indian mixed traffic conditions.

Acknowledgement

I owe a debt of gratitude to honourable Prof. (Dr.) Harish R. Varia for initial spark and subsequent valuable guidance throughout my thesis work. His teaching background has helped me in formulating the strategy and methodology for my research work, which indeed is the core of the topic. His prolonged contact with me during the formulation of my dissertation report helped to achieve the required result in a pragmatic and presentable manner.

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Above all, I am very much obliged to almighty GOD for giving me this beautiful life and made me able to reach this stage of life.

Pranay Shah

Content

List of Abbreviations

Abbreviations	Full name
AMC	Amdavad Municipal Corporation
BDA	Bangalore Development Authority
OGD	Open Government Data
HCM	Highway Capacity Manual
CRRI	Central Road Research Institute
MORTH	Ministry of Road Transport and Highway
VOC	Vehicle Operation Cost
SMS	Space Mean Speed
DMS	Delay Minimization Scheme
TW_TSCS	Two-way Traffic Signal Coordination Strategy
IRC	Indian Road Congress
FHWA	Federal Highway Administration
LHT	Left Hand Traffic
RHD	Right Hand Drive
SFR	Saturation Flow Rate
DBL	Dedicated Bus lane
AJL	Ahmedabad Janmarg Limited
AMTS	Ahmedabad Municipal Transport Service
MOE	Measure of Effectiveness
DFR	Demand Flow Rate
RTCT	Red Time Countdown Timer
GA	Genetic Algorithm
TRB	Transportation Research Board
PCU	Passenger Car Unit
SPCU	Static Passenger Car Unit
DPCU	Dynamic Passenger Car Unit
PCTR	Projected Per Capita Trip Rate
DBL	Dedicated Bus Lane

List of Symbols

Symbols	Definitions
Pl	Phase length
Po	Phase offset
G	Green time
A	Amber time
AR	All red time
tt	Travel time
n	Number
g_{min}	Minimum green time
C	Cycle length
X	Degree of saturation
g/c	green time/cycle time ratio
v/c	volume/capacity ratio
tt_{ij}	Travel time of traffic stream between intersection *i* to *j*
tt_{ji}	Travel time of traffic stream between intersection *j* to *i*
α_i	Start of green for phase at intersection *i*
α_j	Start of green for phase at intersection *j*
βi	End of green for phase at intersection *i*
β_j	End of green for phase at intersection *j*
P_i	Respective phase time at intersection *i*
P_j	Respective phase time at intersection *j*
θ_1	Angle between band width and link direction (*j* to *i*)
θ_2	Angle between band width and link direction (*i* to *j*)
G_{ip}^{min}	Min. green time for phase *p* of intersection *i*
G_{ip}	Green time for phase *p* of intersection i for the given time period *t*
g/c_{ip}	$\frac{g}{c}$ ratio for the phase *p* of intersection *i*

P_{ik}	Number of phase *k* at intersection *i*
φ_{ij}	Delay at *j* to the right turner coming from *i*
φ_{ji}	Delay at *i* to the right turner coming from *j*
ψ_{ij}	Delay at *j* to the straight movers coming from *i*
ψ_{ji}	Delay at *i* to the straight movers coming from *j*
tts	Travel time by SMS
tta	Actual travel time
x_1	Width of approach in m
x_2	Traffic volume in DPCU
x_3	Calculated time space diagram delay in sec

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CHAPTER: 1
Introduction

1.1 General

The urban traffic congestion has become a global phenomenon. These days, urban arterials are being called upon to carry more users than ever before. The users of these facilities are growing more complex (younger and older drivers, more distractions, larger vehicles, etc.) and the demand for such use continues to outpace transportation supply. Road traffic congestion is a critical problem accelerated by an exponential growth in the number of vehicles, ceaseless urbanization and has become a phenomenon visible in urban area of developing south Asian countries like India. According to the World Population Report by United Nations 2017 revision the population of the world is 7.6 billion in 2017 and it is expected to touch at 11.8 billion in year 2050. India will become world most populous nation by overtaking China in year 2030. In India, a developing country and home of around 1.3 billion people the problem of rapid urbanization and vehicular growth is considerably more severe. The resulted traffic congestion increases delay, energy consumption, environmental pollution and vehicle operation cost (VOC). In view of the increasing traffic problem, heterogeneous nature of moving vehicles and lack of possibilities for infrastructure expansion in urban road networks, the optimal use of available road space can be judiciously achieved by efficient signal control strategies.

According to the information available for traffic light on Wikipedia the origin of traffic control signals can be traced back to the manually operated semaphores first used in London by engineer J.P. knight as early as on 9th December 1868. The use of this signal was discontinued due to gaseous explosion. Then after the first electric traffic signal was developed in the United States by James Hodge and installed in Cleveland, Ohio, in 1914 (FHWA, 1996). This was followed by the introduction of interconnected signals in 1917 in Salt Lake City, Utah, U.S.A. Automatic control of interconnected traffic lights was first introduced in March 1922 in Texas, U.S.A. Since 1914 when the first electronic traffic signal in the U.S. was erected, there has been no doubt about the significance of what a smart and efficient traffic signal control system can do for the society.

1.2 Necessity of signal

As the vehicular traffic increases day-by-day in the cities, it becomes necessary to signalize the intersections of arterial/sub-arterial streets to control and regulate the traffic. It is just not enough to install the signals on intersections to satisfy one or more warrants. By installing the signals and applying proper phase plans, there is considerable reduction in conflicting points, which ensures reasonable safety. So the main consideration should be given to reduce the delay to vehicles on the legs of intersection. The operation of traffic signal installation will be efficient when the delay to vehicles on each approach of intersection be a minimum. There will be considerable reduction in delay to vehicles on the approaches of a road by coordinating the signal installations of a road. Generally, coordination of signals should be done to reduce the delay on major streets, which need to be examined carefully. If there is considerable increase in the delay to the vehicles of side streets then the decision of applying coordination is not justifiable.

Signal coordination is a tool available in the hand of traffic engineers by means of which they control, regulate and manage the traffic in the signal system in such a way the most-productive and cost-effective use of the existing roadway system can be made. A good knowledge of road user characteristics, vehicular characteristics, Traffic flow characteristics, roadway and environmental characteristics is necessary for traffic engineers to arrive at meaningful decision for effective coordination.

Signal can be coordinated in several ways, but the three most common techniques are the simultaneous system, the alternate system and the progressive system (Discussed in following chapter). The type of regulations prevailing in the street system, i.e. one-way or two-way street system or mixed network, affects the planning of coordination. The single route ‘green wave’ concept can be extended to a whole network, using several signal plans, known as “Area Traffic Control”, through computer programming and centralized control (Chhanya, 2004).

Traffic signals when properly designed, located and operated have the following advantages:

(1) They provide orderly movement of traffic and increase the traffic handling capacity of most of the intersections at grade.
(2) As speed is self-regulated in the coordinated system they reduce certain types of accidents, notably the right angled collisions.
(3) Pedestrian can cross the roads safely at the signalized intersection.

(4) Proper coordinated signal system provides reasonable speed along the major road traffic.
(5) Signals provide a chance to crossing traffic of minor road to cross the path of continuous flow of traffic stream at reasonable intervals of time.
(6) Automatic traffic signal may work out to be economical when compared to manual control.
(7) The quality of traffic flow is improved by forming compact platoons of vehicles, all the vehicles move at approximately at the same speed.

If they are not installed and designed not properly,

(1) The rear end collisions may increase.
(2) Excessive delay may be caused during of-peak hours.
(3) Unwarranted signal installations tend to encourage the disobedience of the signal indications.
(4) Drivers may be induced to use less adequate and less safe routes to avoid delays at signals.
(5) Failures of the signal due to electric power failure or any other defect may cause confusion to the road users.

1.3 Signal coordination- An overview

When traffic signals are located in close proximity, the presence of the upstream traffic signals alters the arrival pattern of traffic at the downstream traffic signals from random arrivals to arrivals in platoons. This means that improved traffic flow can be achieved if the green signal at the downstream traffic signal is arranged to coincide with the arrival of the platoon. To achieve this, traffic signals are coordinated, sometimes called “linked”. Traffic signal coordination is normally implemented to improve the level of service of a road or a network of roads, where the spacing of signals is such that isolated signal operation would cause excessive delays, stops and loss of capacity. The popular concept is that coordinating traffic signals is simply to provide green-wave progression whereby a motorist travelling along a road receives successive green signals. While this is one of the aims, the principal purpose of coordination is to minimize overall delay and/or number of stops along a corridor or network.

Coordination is achieved through three features:

- Traffic signals run on a common cycle time (or in special cases, one half of the cycle time).
- The beginning or end of the green period on the coordinated approach of each intersection is set to occur at the offset time relative to that at the reference intersection. This offset is determined by the distance between the signals, the progression speed along the road, and the queues of vehicles waiting at red signals.
- The optimization of offsets and phase times.

When two or more traffic signals are located in close vicinity, traffic flow on links joining the two signals becomes dependent on timings at these signals. This dependency may be strong or weak depending on a number of factors. These factors include:

• Type of facility,
• Distance between signals,
• Traffic speed on link,
• Traffic volumes, and
• Traffic distribution and origin/destination patterns.

Typically, the first two of these factors remain unchanged for many years. However, the other three may change from one day to the next day and several times within a day.

Green signals at adjacent intersections are set to occur at a given time, relative to that at a reference intersection. It depends on the distance between signals, the progression speed along the road between the signals and the queues of vehicles waiting at red signals. If this time offset is set properly, than coordination of the signal can be achieved as depicted in the following figures 1.1a and 1.1b.

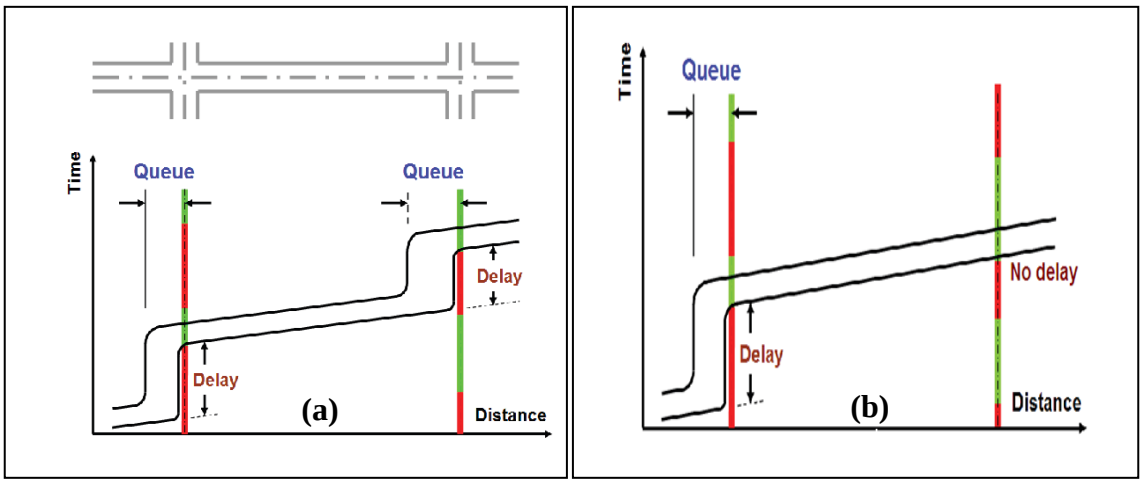


Figure 1.1a & 1.1b: Traffic delay without coordination and with coordination

Figure 1.1a shows that the vehicle is delayed at the second intersection due to an uncoordinated signal time offset. Whereas, Figure 1.1b shows that the vehicle is not delayed at the second intersection due to a coordinated signal time offset. The figure represents one-way coordination which can be applied when the major street flow in one direction only. However, when the traffic flow in both forward and backward direction is considerable than two-way coordination is to be employed for effective traffic management.

Coordination of two or more signals on a signalized arterial requires the determination of following four signal-control parameters:

• Cycle length,
• Green splits,
• Phase sequence, and
• Offsets.

1.4 Motivation of the research

As per Census 2011 data 32 percent population of India lives in urban area while for Gujarat state urban population is quite higher than the national average and it is around 45 percentage. Detail study of the census data presents challenging trends of Indian urbanization pattern because 468 Class-I cities having population more than 1 lakh constitute about 70% urban population of the country. Despite all odds Indian economy is the fastest growing major economy in the world. The pace of urbanization resulting from the economic development of the country will become quicker in coming years as compared to other developing countries (China 45%, Indonesia 54%, Brazil 87% and Mexico 78%) Indian urban population is quite low. Just three countries—India, China and Nigeria— together are expected to account for 37 per cent of the projected growth of the world's urban population between 2014 and 2050. India is projected to add 404 million urban dwellers, China 292 million and Nigeria 212 million (World Urbanization Trends, United Nations -2014). New Delhi is second biggest city of the world with 25 million populations next only to Tokyo, Japan. In India the inclination of the people for own vehicle ownership and private mode of transport is increasing day by day despite the best efforts by Union and State Governments to promote mass transportation system in the country. Vehicle ownership in the country had witnessed threefold increase in the past fifteen years. According to available information on the Open Government Data (OGD) platform of India (https://data.gov.in/) the number of

registered motor vehicles per 1000 population in India was 53 in 2001 which was increased to 167 in 2015.

Car is considered as a social status symbol of the effluent middle class people in our country. As per the data only 6% Indian household is having ownership of car in 2014. The present economic growth has resulted in the increase in the income level of the people. Purchasing power of people is accelerating the vehicle ownership growth of the consumption driven economy. Shortfall of required road space mandatory as per norms and temporary/permanent encroachments on the roads of Indian cities create chaos due to increased traffic congestion. This chaos is especially obvious at signalized intersection where competition exists among vehicles to use available space. As per the latest report of 2015 on road accident by Ministry of Road Transport and Highway (MORTH) traffic junctions are regarded as the accident prone areas, about 49% of the total reported accidents (1,46,133) took place at the junctions itself.

The similar trends were observed in the city of Ahmedabad which is selected in this research. Despite the best of its class and award winning mass rapid transits system, Bus Rapid Transits (BRT) in city which covers more than 130 km length of the city, growth in the private vehicle ownership in the city refuses to subside. As on March 31, 2010 (When BRT started its operation) there were 14.62 lakh two-wheelers and 3.09 lakh four- wheelers in the city. These numbers have increased to 21.93 lakh and 5.29 lakh respectively up to July 2015. The ever increasing vehicles on the limited road space are responsible for reduction of the operating speed of the traffic stream. The report submitted in 2015 by Ahmedabad Municipal Corporation (AMC) to Ministry of Urban development, Government of India states average operating speed of traffic on city road is 9 to 17 kmph. The similar or even further challenging trend is observed in other Indian cities as well. For example, the recently published (April 2017) revised master plan 2031 of Bangalore Development Authority (BDA) said 1.18 crore citizens waste 60 crores (600 million) man-hours annually, and this translates to a loss of rupees 3,700 crore, including rupees 1,350 crore on fuel alone, and the rest on productivity (i.e. man hours). BDA admits that the shortage of public transport options has only added to the dominance of private vehicles on the city's landscape, directly contributing to congestion and reducing vehicle speed. In India peak hours speed of the vehicles on the some highly congested stretches of the megacities are increasingly approaching to the walking speed of the healthy person i.e. 4 to 5 kmph.

Traffic signal retiming is one of the most cost effective ways to improve traffic flow and is one of the most basic strategies to mitigate congestion. The benefits of up-to-date signal timing include shorter commute times, improved air quality, reduction in certain types and severity of crashes, and reduced driver frustration. The ability to synchronize multiple intersections to enhance the operation of one or more directional movements in a system is called traffic signal coordination. The decision to use coordination can be considered in a variety of ways. There are numerous factors used to determine whether coordination would be beneficial. Establishing coordination is easiest to justify when the intersections are in close proximity and there is a large amount of traffic on the coordinated street. The traffic signals located within 300 meters (1000 feet) to 800 meters (0.5 miles) of each other along a corridor should be coordinated unless operating on different cycle lengths. (Manual of Uniform Traffic Control Devices for Streets and Highways (MUTCD) Published by FHA, USDOT 2015.)

The vehicle-to-vehicle and vehicle-to-pedestrian crashes at intersections are decreased in different crash severity levels and types, especially for angle and rear-end ones after signal timing optimization. Similar results are found for multi-vehicle rear-ends crashes on street segments. These indicate that intersection signal timing optimization in dense urban street networks has a potential for improving traffic mobility, vehicle and pedestrian safety at intersections, and vehicle safety on street segment (Roshandeh et al., 2016). Speed is self-regulated in coordinated signal system; as over speeding is one of the main culprits of the road crash; signal coordination can improve road safety as well.

1.5 Issues and challenges

Traffic signal coordination for heterogeneous traffic condition prevailing in most of the South Asian countries poses greatest ever challenges because of several reasons such as lack of traffic discipline, poverty, poor literacy rate, lack of proper enforcement, lack of awareness of traffic rules and regulation, ease of obtaining driving license and variety of mixed vehicle plying on the road. The traffic in mixed flow is comprised of fast moving, slow moving, motorized and non-motorized vehicles. The vehicles also vary in size, manoeuvrability, control, and static and dynamic characteristics. Traffic is not lane based and smaller size vehicles which constitute about 50% to 70% of traffic mix in urban area often squeeze through any available gap between large size vehicles, move in a haphazard

manner, reach the head of the queue and pose dangerous safety problem. Highway Capacity Manual (HCM) is indispensable resource for proper planning, design and operation of road traffic facilities in any country. In view of limited research and few standardized guidelines and codes were available previously for signal control and analysis in India, the first ever Indo-HCM is released recently on 12th February 2018. It was jointly developed by seven academic institutions including IIT-Roorkee, Bombay and Guwahati, School of Planning and Architecture, New Delhi, Indian Institute of Engineering and Science and Technology, Shibpur, Sardar Vallabhai Patel National Institute of Technology, Surat and Anna University, Chennai under the umbrella organization CSIR-Central Road Research Institute (CRRI), New Delhi, India. The Indo- HCM 2018 covers details related to signalized intersections in chapter 6.

There are various online and offline methods, mathematical models and simulation software are available for optimization of signal setting parameters in the literature but all these methods, models and software mostly developed in foreign country based on the extensive research carried out depicting their local prevailing conditions. These are not relevant in Indian mixed traffic condition where prevailing roadway and traffic condition are quite different having Left Hand Traffic (LHT) and Right Hand Drive (RHD) (Arasan and Khosy, 2005). The applicability of these models and software to replicate Indian mixed traffic condition is a debatable issue. Deployment of an adaptive area traffic control system is expensive; physical sensors require installation, calibration, and regular maintenance. Because of low budgets and technical capacity in resource-constrained economies, area traffic control systems found minimally functioning; throughout in World Bank partner countries (Lu et al., 2017). All these mathematical models and simulation software requires extensive computational efforts, licensing issues, technical manpower, sensors, huge data collection, and proves costly. Now a days the possibility of signal data hacking poses security concerns as well.

The Highway Capacity and Quality of Service Committee of the Transportation Research Board (the Official Creator of the HCM) “does not” review software nor make any statement concerning the degree to which it faithfully replicates the HCM (Roess et al., 2010). The HCM model for signalized intersection analysis is extremely complex and includes many iterative elements. As a result, there are traffic agencies that still use the methodology of the Interim Materials (Published by Transportation Research Board in 1980) to analyse the

signalized intersections including the California Department of Transportation (Roess et al., 2010).

As per authors' knowledge a few literature is available pertaining to pre-timed two-way traffic signal coordination for a typical Indian mixed traffic situation. The topic of signal coordination had not attained due importance in Indian context. Coordination of signal itself requires suitable understanding and coordination among different agencies like Traffic Department of Municipal Corporation, City Traffic Police and private agency responsible for maintaining and adjusting signal timing. As per available information to author presently no such optimization software is available which can optimize all four signal control parameters - cycle length, phase sequence, offset and green split simultaneously in Indian condition. The signalized intersections having other than four numbers of approaches are also prevailing on the urban corridors with different spacing between them. It makes difficult to obtain two-way signal coordination by adopting traditional alternative or double alternative or simultaneously progressive system, which generally gives through band in both directions for equal spacing between four arm junctions. To overcome this situation an effort is made in this research to present simple, lucid and easy to understand two –way traffic signal coordination methodology for pre-timed signal having 3 arm, 4 arm and 5 arm intersections for peculiar Indian traffic situation.

1.6 Objective and scope of work

Owing to serious limitations of the online signal coordination methods there is a need to develop simple and lucid method of signal coordination which can be easily implemented. The scope of the research work is to develop a methodology for the two-way coordination of pre-timed signal for 3 arm, 4 arm and 5 arm intersections for combinations considering travel time and link distance between two intersections. The main emphasis of the research then after devoted to evolve robust signal coordination methodology and model for commonly available four arm signalized intersection. Considering the focus of the research the main objectives of the research are:

- To derive robust methodology for two-way coordination of pre-timed traffic signal which can be used to optimize all four signal control parameters simultaneously (cycle length, green split, phase sequence and phase offset).
- To develop a model for two- way coordination of pre-timed traffic signal for the four arm signalized intersections along urban corridor for mixed traffic composition.

1.7 Stages involved in the study

The various stages involved in the study are presented in the figure 1.2.

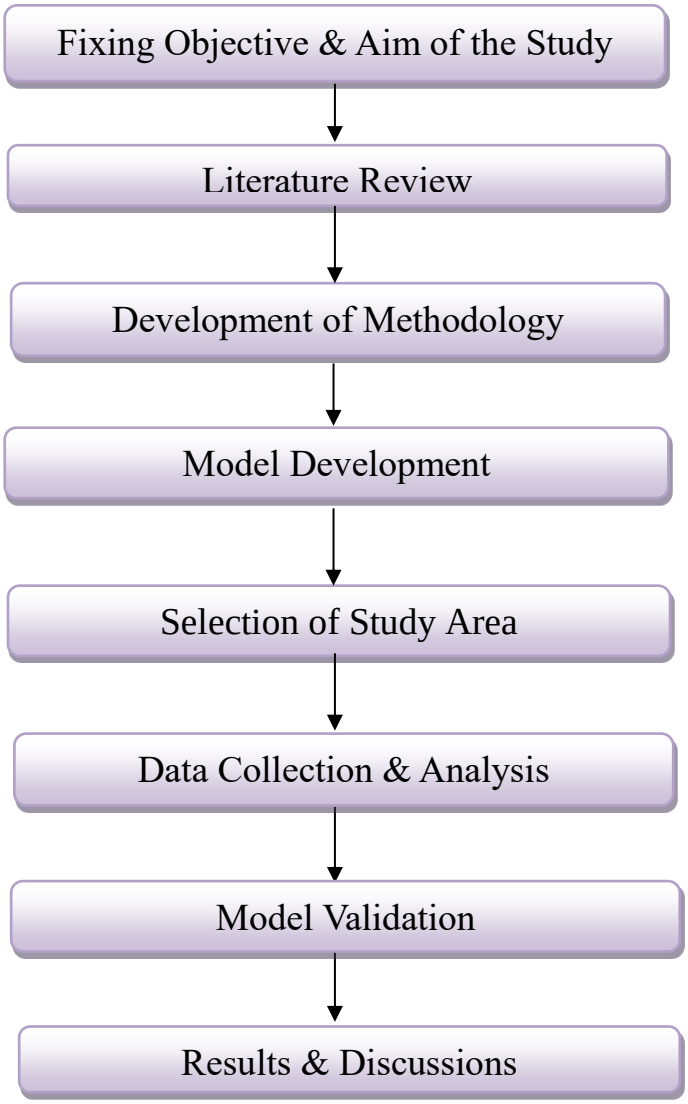


Figure 1.2: Stages involved in the study

1.8 Organization of report

The research work is presented in total eight chapters including Introduction

Chapter 1 deals with the introduction, necessity of signals and motivation of research. It covers critical issues and challenges faced for two way signal coordination with typical condition of Indian traffic, objective, aim of the study and different stages involved in the study.

Chapter 2 states about different types of signals and traditional signal coordination techniques. It includes understanding of basic fundamental concept required to achieve the goal of signal coordination.

Chapter 3 presents extensive literature review regarding different signal coordination techniques, saturation flow rate, and passenger car unit etc. carried out to accomplish research objectives. Finally chapter culminates with the inference derived from the discussed literature review.

Chapter 4 covers development of methodology for two way traffic signal coordination scheme coded as TW_TSCS1 and TW_TSCS2. The chapter elaborates signal coordination scheme based on key factors like space mean speed, link distance, saturation flow rate (SFR), demand flow rate (DFR) and width of approach. It includes methods of deriving optimum cycle length for coordination.

Chapter 5 explains central focus of this research via robust model for two way signal coordination. It exhibits mechanism and rule for coordination even when the travel time in forward and backward direction is different. Generalization of the developed model and optimization with Genetic Algorithm (GA) is also explained in this chapter.

Chapter 6 describes procedure followed for study area selection and data collection. It covers different techniques adopted for data reduction and analysis. Lastly the chapter ends with the derived values of various parameters required to ascertain operational quality of signal control.

Chapter 7 includes details of process followed for analytical validation of the developed methodology TW_TSCS1 and TW_TSCS2 for actual noon peak and evening peak data. The chapter also summarizes validation of optimum signal setting plan generated by GA. At the end the chapter presents comparison of delay reduction by different methods and flow chart of developed model.

Chapter 8 comprises details of the initiative taken for validation of the developed methodology i.e. field implementation and testing. It recapitulates various performance

measures of signal coordination observed before implementation and after implementation. The chapter covers statistical analysis via t test and relationship developed to predict observed stopped delay.

Chapter 9 offers conclusion derived from the research. It offers easy to implement signal coordination technique which is extremely useful for resource crunch country like India. The chapter covers future scope and limitations of the model as well.

1.9 Summary

This chapter gives brief overview of the necessity of signal, with motivation behind the research. It includes issues and challenges faced in the Indian scenario along with objective and scope of the study. At last it explains various stages involved in the study. The next chapter deals with the review of literature carried out to obtain research objective of this study.

CHAPTER: 2

Basics of Signal Coordination

2.1 General

The traffic signal is a device that allows traffic engineers to leave their intelligence at an intersection to operate it in their absence. Traffic signal across the world today operates with the three light systems i.e. Green, Red and Amber (Yellow). Except few British colonial countries (Including India) which operate with Right Hand Drive (RHD) and Left Hand Traffic (LHT), the traffic in many countries follow Left Hand Drive (LHD) with Right Hand Traffic (RHT). Preliminary from the invention to the development of traffic signal coordination techniques was contributed by these developed countries having LHD and RHT. India is the big country with unique nature of traffic movement and traffic composition. This chapter covers fundamental concept and basic theory of signal coordination to achieve the research objectives.

2.1.1 History of traffic signal

The first ever traffic signal used in the field can be traced back to the manually operated semaphores used in London as early as 1868 (Webster and Cobbe, 1966). This signal was inspired by the semaphores signal used in the railway line. The signal operating with gas was exploded and the use of signal was discontinued. Then after the first electric traffic signal in the United States was developed by James Hoge and installed in Cleveland, Ohio, in 1914 (Benesh, 1915). This was followed by the introduction of interconnected signals in 1917 in Salt Lake City, Utah (Marsh, 1927). The modern electric traffic light is an American invention. As early as 1912 in Salt Lake City, Utah, policeman Lester Wire invented the first red-green electric traffic lights. On 5 August 1914, the American Traffic Signal Company installed a traffic signal system on the corner of East 105th Street and Euclid Avenue in Cleveland, Ohio. It had two colors, red and green, and a buzzer, based on the design of James Hoge, to provide a warning for color changes. The design by James Hoge allowed police and fire stations to control the signals in case of emergency. The first four-way, three-color traffic light was created by police officer William Potts in Detroit, Michigan in 1920. In

1922, T.E. Hayes patented his "Combination traffic guide and traffic regulating signal". Ashville, Ohio claims to be the location of the oldest working traffic light in the United States, used at an intersection of public roads until 1982 when it was moved to a local museum.

The first interconnected traffic signal system was installed in Salt Lake City in 1917, with six connected intersections controlled simultaneously from a manual switch. Automatic control of interconnected traffic lights was introduced March 1922 in Texas. The first automatic experimental traffic lights in England were deployed in Wolver Hampton in 1927. In 1923; Garrett Morgan patented his own version. The Morgan traffic signal was a T-shaped pole unit that featured three hand-cranked positions: Stop, in all -directional stop position. This third position halted traffic in all directions to allow pedestrians to cross streets more safely. It's one "advantage" over others of its type was the ability to operate it from a distance using a mechanical linkage. The color of the traffic lights representing stop and go might be derived from those used to identify port (red) and starboard (green) in maritime rules governing right of way, where the vessel on the left must stop for the one crossing on the right. Timers on traffic lights originated in Taipei, Taiwan, and brought to the US after an engineer discovered its use. Though uncommon in most American urban areas, timers are still used in some other Western Hemisphere countries. Timers are useful for drivers/pedestrians to plan if there is enough time to attempt to cross the intersection before the light turns red and conversely, the amount of time before the light turns green. These may also be useful to stop the engines, if there is the long red time.

Since then, the technologies of traffic signal control have been growing rapidly and the traffic signal has become a critical element of today's modern traffic control and management systems. Today's traffic signals are, by definition, "operated traffic control devices which alternately direct traffic to stop and to proceed" (FHWA 2003, 1A-14). From the application's point of view, traffic signals are used to control the assignment of vehicular or pedestrian right-of-way at locations where potentially hazardous conflicts exist or where passive devices (signs, markings) do not provide adequate control of traffic.

2.1.2 Classification of signals

The signals can be classified as,

(I) Traffic control signals:

(a) Pre-timed (fixed time) signal,

(b) Semi actuated signals,

(c) Fully actuated (automatic) signal,

(d) Volume-density signals.

(II) Pedestrian signal,

(III) Special traffic signal.

Traffic signals at isolated intersections classified with respect to their flexibility in terms of cycle length and phase length. Pre-timed traffic signals are those signals whose cycle length is fixed for Time of Day (TOD) intervals or for entire day. Basically there are three kinds of TOD intervals exercised for setting of cycle length for Pre-timed signal; i.e. Morning Peak, Evening Peak and Off Peak time. Depending on the factors like traffic composition, area type, directional flow, corridor type; the morning and evening peaks are bifurcated suitably in required intervals of one to two hour. Off peak hours may include afternoon, late night, midnight and early morning off peak.

Semi actuated signals have controllers on major approach which sets the cycle and phase lengths to be varied according to the arrival rate and volume conditions on the minor or side street approaches. It is more useful where one major street of high volume intersects with minor street with low volume. Thus, principle advantage of the semi actuated signal is to extend the green interval to the main street traffic in off-peak periods.

The volume-density controller is a more complex fully actuated type of signal that permits almost continual adjustment for both the phase and cycle lengths in accordance with split second data on vehicular arrivals, waiting time and arrival headways on all intersection approaches or legs; these data are detected by detectors installed on all approaches, stored within the signal memory, and continually analysed for setting (or resetting) minimum green times, green time increments, and maximum green times. It is also known as Adaptive Traffic Control Signal

Pedestrian signals are provided on busy urban streets for crossing of the pedestrians. They temporarily show a RED light for the traffic to stop, permitting the pedestrians to cross the road. Sometimes additional traffic lights and cycles are provided at intersections for pedestrians to cross, when vehicles are stopped, such as WALK, DON’T WALK should be used.

Sometimes special types of traffic signals are provided as per condition or situation of the intersections, such as flashing signals, bus priority signals are provided. Flashing signals are provided to warn the approaching drivers. Occasionally 'yellow flashing signals' on the main road and 'red flashing signals' on the crossroad are provided. At fixed time traffic intersections, sometimes flashing lights are used during the late night non-peak hours of traffic. Bus priority signals are used at special intersections, with the concept that whenever there is a bus in the traffic stream, it should find a green indication of signal immediately, so that the total delay to people sitting inside the bus is minimized.

2.2 Types of signal system coordination schemes

The main purpose of signal coordination is to discharge the maximum amount of main street traffic without enforced halts while allowing adequate capacity for cross-street traffic. Traditionally there are two types of traffic signal coordination schemes available:

(i) A band width system which permits continuous movement in progression bandwidth.

(ii) A disutility system, which is aimed at minimizing travel cost (delays, stops, fuel emissions, etc.) along an arterial or through an area (Roess et al., 2010).

2.2.1 Bandwidth system

The concept of the bandwidth has been introduced by reference "windows" of green through which platoons of vehicles can move. The bandwidth concept is very popular in traffic engineering practice, because (a) the windows of green are easy visual images for both working professionals and public presentations, and (b) good solutions can often be obtained manually, by trial and error. The most significant shortcoming of designing offset plans to maximize bandwidths is that internal queues are overlooked in the bandwidth approach. When internal queues exist, historic bandwidth based solutions can be misleading and erroneous.

When the signal displays green (at time = t), the first vehicle starts and moves, after some lag time (t_1). Down the street it reaches the downstream signal at time (t_2). The first vehicle trajectory is the front edge of the bandwidth. If the indication of the downstream signal is green the vehicle continuous, otherwise it stops. The vehicles which are accumulated behind the first vehicle forming a platoon of vehicles. The last vehicle in the platoon which crosses the first signal and passes the downstream signal form the end edge of the bandwidth. Hence, bandwidth is a measure of the platoon size that can cross an intersection without halting.

Normally, a platoon will maintain its shape over 350m and then disperses over time based on the platoon size, progressive speed, and link length.

Efficiency of a band width or progressive system is defined as the ratio of the bandwidth (measured in the seconds) to the cycle length, expressed as a percentage. The efficiency is a measure of how much of the cycle length has been used by the bandwidth. Thus, the closer efficiency to 100 % the larger bandwidth is, thus producing more efficient progression. The bandwidth is limited by the minimum green in the direction of interest.

$$\text{Efficiency} = (\text{Bandwidth} / \text{Cycle length}) \times 100\ \% \qquad \ldots\ldots (2.1)$$

Attainability is the ratio of bandwidth to minimum green in each direction. An attainability of 100% implies that no larger bandwidth can be found for the given traffic conditions. Thus, the attainability is a measure of the progression's ability to utilize the available greens of the intersections within the artery.

$$\text{Attainability} = (\text{Bandwidth} / \text{Minimum artery through green}) \times 100\% \qquad \ldots. (2.2)$$

2.2.2. Disutility system

Another approach to obtain coordination is by minimizing disutility. Disutility is a measure of effectiveness, which can be expressed by delays, stops, and other terms, e.g. fuel consumption, queue length, environmental pollution (air, noise) etc. It is common to consider the benefits of a coordination plan in terms of a "cost" or "disutility" function. A general disutility function can have the form of equation.

$$\text{Disutility} = A\ (\text{total stops}) + B\ (\text{total delays}) + C\ (\text{other terms}) \qquad \ldots\ldots. (2.3)$$

The weights A, B and C are coefficients to be specified by the engineer or analyst. The coefficients may be selected on the basic of a judgment of how important every element is to the public or can be estimated based on the economic cost. For example, perhaps one stops are as bothersome as 5 seconds of delay, so that A=5B. The amounts, by which various timing plans to reduce the cost shown in equation, can then be used in a cost-benefit analysis to evaluate alternative plans.

In practice, numeric values of the improvement in stops and delay are usually obtained with timing plans done with signal optimization computer packages, such as the TRANSYT (Traffic Analysis Study Tool) program. For those done manually, the engineer usually tries

to make the number of vehicles stopped as small as possible or tries to minimize delay. This is usually acceptable.

2.2.3 Time – Space Diagram

In planning a system of coordinated signal control, it is often expedient to indicate the system diagrammatically by what is known as a "time-space" diagram. The time space diagram is simply the plot of signal indications as a function of time for two or more signals. The diagram is scaled with respect to distance, so that one may easily plot vehicle positions as a function of time. Time-space diagrams can be plotted manually or by using various computer software.

For the best bandwidth coordination result, a time-space diagram should be constructed. On this diagram, the time and signal settings (expressed in seconds) are indicated along the horizontal axis to a suitable scale, whereas the signal spacing (expressed in meter or kilometre) is plotted vertically to a suitable scale. The first step in constructing a time-space diagram is to calculate the required green splits and cycle length for each intersection and also to determine the offset for consecutive intersections. Then allocate on the time axis the green and red splits for each intersection by considering offset. The offset for one signal is relative to the immediately preceding one. Based on the split allocation, three types of bandwidth system can be formed – simultaneous, alternate, and simple progressive system. Flexible progressive system may also be formed which is an improvement over simple progressive system, in which the simple progression is revised two or more times in a day. In all system a key requirement is to have a common cycle length throughout the artery. These coordinated signal systems are explained below.

2.3 Various types of coordinated signal systems

(i) Simultaneous system (Synchronized system)

Under this system all the signals along a given street always display the same indication to the same traffic stream at the same time, i.e. all of the signals turn green at the same time (see Figure 2.1). The division of the cycle time is the same at all intersections. A master controller is employed to keep the series of signals in step. For very closely spaced signals and high vehicle speed or for vastly spaced signals and low vehicle speed, it may be best to have a simultaneous system.

The disadvantages of a simultaneous system are:

- It is not conductive to give continuous movement of all vehicles.
- It encourages speeding of drivers between stops.
- The overall speed is often reduced.
- Because the division of the cycle time is the same at all the intersections, inefficiency is inevitable at some intersections.
- The simultaneous stoppage of a continuous line of traffic at all intersections often results in difficulty for the side street vehicles in turning into or crossing the main side street.

The efficiency of simultaneous systems depends upon the no. of signals involved.

For N Signals, it is given by:

$$[(1/2) - \{(N-1) \times L / (V \times C)\}] \times 100\ \% \qquad \ldots\ldots (2.4)$$

Where,

L = Block length (m)

V = Platoon Speed (m/s)

C = Cycle Length (sec)

N = Number of signals involved

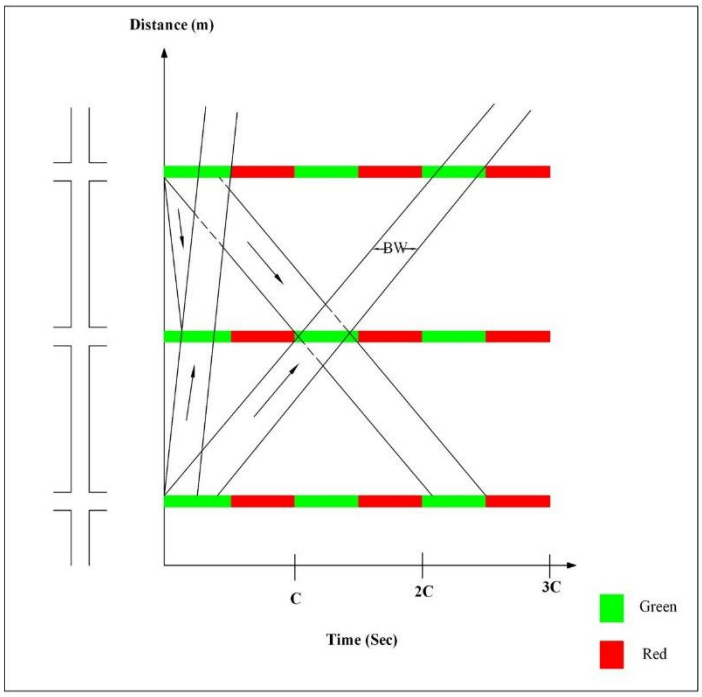


Figure 2.1: Time-space diagram for simultaneous system

(ii) Alternate system (Limited Progressive system)

Under this system, consecutive signal installations along a given road show contrary indications at the same time (see Figure 2.2). This permits the vehicles to travel one block in half the cycle time. For uniform block lengths with 50:50 splits and the travel time of each

platoon is very near to one-half the cycle length, it may be best to have an alternate system. It also brings about a certain measure of speed control since speeding drivers are stooped at each signal. When the travel time of each platoon is exactly one-half the cycle length, then the efficiency of the signal system is 50% in each direction because all of the green is used in each direction. There is no limit to the number of signals, which can be involved in this system.

Some of the disadvantages of this system are:

- The green times for both the main and side street have to be substantially equal, resulting inefficiency at most of the intersections. If the splits are not 50:50 at some signals, then (a) if they favour the main street, they simply represent excess green, suited for accommodating miscellaneous vehicles, and (b) if they favour the side street, they reduce the bandwidths.
- In situations where the block lengths are unequal, the system is not well suited.
- Adjustments are difficult for changing traffic conditions.

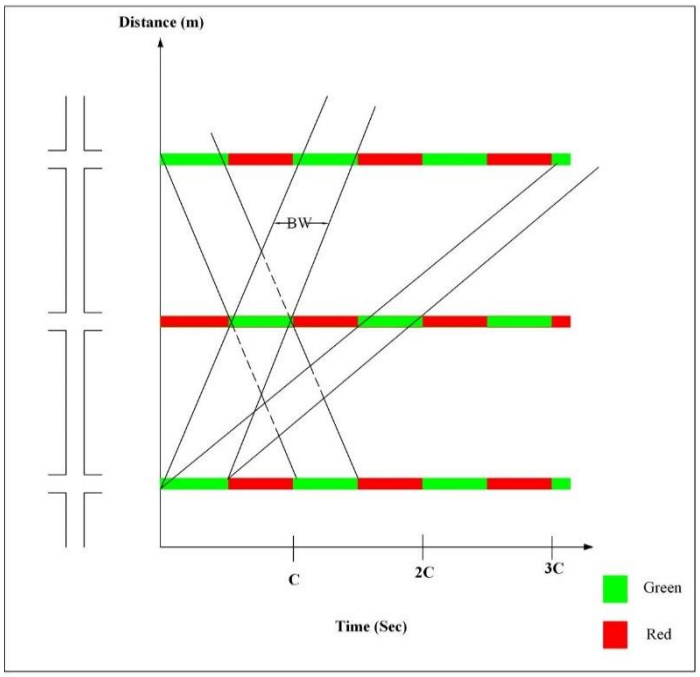


Figure 2.2: Time-space diagram for alternative system

(iii) Double alternate system

Under this system, consecutive signal installation in pairs along a given road show contrary indications at the same time (see Figure 2.3). For uniform block lengths with 50:50 splits and the travel time of each platoon along two consecutive blocks is very near to one-half the

cycle length, it may be best to have a double alternate system. When the travel time of each platoon along two consecutive blocks is exactly one-half of a cycle length then the efficiency of the double alternate system is 25% in each direction because only half of the green is used in each direction. There is no limit to the number of signals, which can be involved in this system. It has the same disadvantages as alternate system.

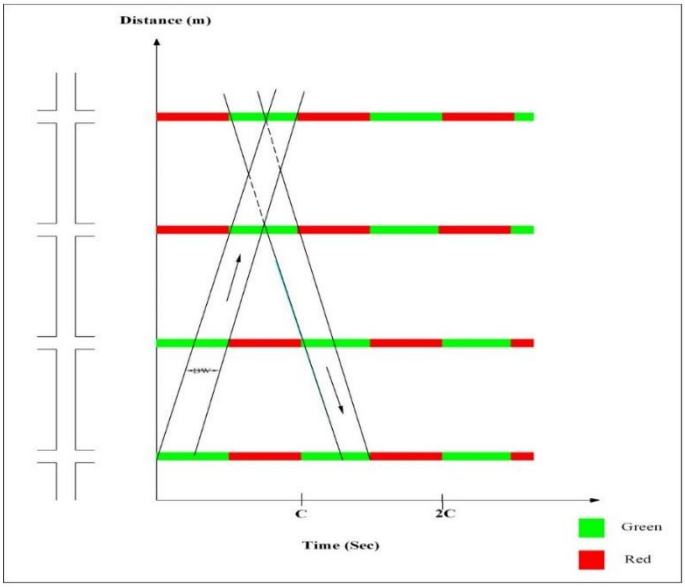


Figure 2.3: Time-space diagram for double alternative system

(iv) Simple progressive system

With this system, the various signals along a street display green aspects in accordance with time schedule to permit, as nearly as possible. Continuous operations of platoons of vehicles along the street at a planned rate of motion, which may vary in different parts of the system. In a progressive system, the green displays are staggered in relation to each other according to the desired road speed (see Figure 2.4).

For irregular block lengths and the travel times of platoon in each block are different, it may be best to have a simple progressive system. This system can be used to favour one direction over the other, e.g. inbound flow in the morning peak at the expense of fewer vehicles travelling in the opposite direction, and vice versa in the evening peak.

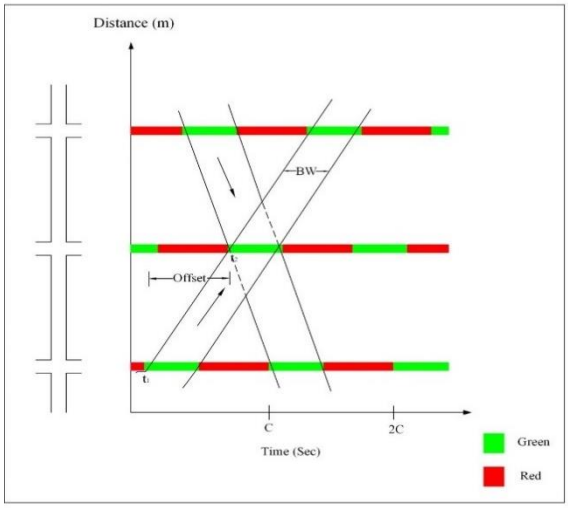


Figure 2.4: Time-space diagram for simple progressive system

The offset, as well as the phase plan, at each installation is determined so as to secure the best continuous movement of platoons in both directions. These offsets are fixed and cannot be altered at different periods of the day. The signal installation may have a cycle division different from the others, but that division different from the others remains fixed throughout the day.

(v) Flexible progressive system

This system is an improvement over the simple progressive system with the following provisions.

- It is possible to vary the cycle time and division at each signal depending upon traffic.
- It is possible to vary the offset, thus enabling two or more completely different plans.
- It is possible to introduce flashing or shut down during off-peak hours.

Flexible progressive systems require a master controller, which keeps the local controllers at each intersection in step.

2.4 Area traffic control

Area traffic control is a further extension of the same principles of co-ordination to include signals in a substantial area. Area traffic control can be defined as a technique which provides for a centralized control of numerous signal installations distributed throughout an urban area, such that there is a planned co-ordination between signals at different junctions. The technique invariably employs digital computers for achieving the desired objective. The earliest known scheme of Area traffic control was in Toronto begun in modest scale in 1959. The success of the project promoted its application in many important cities of the world. In

U.K. controlled experiments were initiated in certain areas of Glasgow and London to assess the amount of benefit that could be expected from different system of centralized control. By now, nearly 100 cities have some form of Area traffic control. The subject has grown into a very interesting and promising field with extensive literature. In this section only a general introduction is sought to be given.

The objectives in an Area control systems are one or more of the following:

(i) Minimizing journey time for vehicles.

(ii) Minimizing vehicular stops, resulting in less noise, reduced pollution and minimize consumption of fuel.

(iii) Reducing accidents.

(iv) Discouraging use of certain areas.

(v) Minimizing person-time.

Area control system has proved to be a very efficient tool in tackling the serious problem of congestion at signalized intersection having homogeneous traffic situation, proper lane discipline and under saturated traffic demand. New Delhi is an example of typical heterogeneous traffic where area traffic control was introduced in year 2008 via global tender and later on in 2011 its use was discontinued.

2.4.1. Traffic control methods

The following are the main types of methods in general use:

Fixed time plans based on historical data and calculated off line by a computerized optimizing technique. The information about vehicle movement is obtained manually or through detectors and fed to the computer, which then determines the signal settings, and transmits the settings to the signals. Example of this type of the combination method is TRANSYT (Traffic Analysis Study Tool). Examples of Co-ordinated systems with local response at each signal are the FLEXIPROG (Flexible Progressive) and EQUISAT (Equally Saturated). Fully responsive systems are such as S.P.G., (Signal Plan Generation) and PLIDENT (Platoon Identification) (kadiyali 2008).

2.4.2 Combination method

This method has been developed by the Road research laboratory, UK and uses delay/offset relationship to obtain the relative timings or off-sets or traffic signals in a network. It applies a rigorous optimization process to a reasonable model of the traffic and assumes that the

cycle time, green times, flow and saturation flows are known and then chooses the offsets of the signals to minimize delay over the whole network. The technique can be applied on an area basis, subject to some restriction in assuming that the delay between two signals depends solely on the relative settings of the two signals, which is a good approximation only in heavily loaded conditions.

2.4.3 TRANSYT (Traffic Analysis Study Tool)

The TRANSYT, implemented by a digital computer programme, is a method of automatically determining fixed time signal plans to meet known network conditions. It is a widely used system, and as demonstrated by the Glasgow experiments, is an improvement over the combination method. The traffic model makes allowance for flow intersection between successive sections of roads and represents average traffic behaviour more correctly than the combination method. It represents platoon dispersion effectively. The solution time is short and good convergences on the optimum signal setting is achieved by a “hill climbing” type of optimization procedure. The overall impedance to traffic is measured by a “performance index” that can be chosen with any desired balance between journey time and number of stops. The optimization procedure minimizes the performance index by altering the points within the signal cycle at which each stage starts. Thus, both signal offsets and green times are included in the optimization procedure.

2.4.4 FLEXIPROG (Flexible progressive)

This is a vehicle-actuated progressive system and requires the use of detectors on the approach arms. With a continuous stream of traffic all over the detectors, the signal behaves as a fixed time system. Under lighter traffic conditions, the signals can change after detecting suitable gap in the traffic. Stages are missed if there is no demand for them. This system has not proved better than the combination method in the Glasgow experiments.

2.4.5 EQUISAT (Equally Saturated)

Under this system, the cycle time and offset pattern are fixed. The allocation of green time is varied to equalize the degree of saturation of each stage. Detectors on each approach arm and computer logic are used to measure or deduce both flow and saturation flow. This system too did not prove to be measurably better than the combination method in the Glasgow experiments.

2.4.6 PLIDENT (Platoon Identification)

This is a system developed by the Road Research laboratory; U.K. the system identifies platoons of vehicles and operates signals to allow unimpeded passage to them on priority streets just the amount of green they require at times which avoid delay to the approaching platoons. This scheme though successfully implemented in the Glasgow experiment, produced the largest average journey times there.

2.4.7 SPG (Signal Plan Generation)

This is fully responsive system and generates cycle times, splits and offsets on line using measured data on traffic condition. This type of system is installed in Madrid and Barcelona.

2.4.8 SCOOT (Split Cycle Offset Optimization Technique)

The SCOOT is a new entrant into the field of traffic signal controlled network. It has been developed in the U.K. by the transport and Road Research Laboratory and the British industry. A large number of detector loops are laid across the road which transmits the data on the traffic to the central control system which adjusts the signal plans according to the actual needs. Traffic delay can be substantially reduced when compared to fixed time plans.

2.5 Saturation Flow Rate (SFR)- A key parameter

SFR is perhaps the most important parameter in signalized intersection control strategy. Despite its central importance, it is extremely difficult to accurately measure SFR at current stage of research. (Roess et al., 2010) Even the HCM suggests eleven different adjustment factors to estimate SFR and gives default value of 1900 pcu/hr/ln. Indian Roads Congress (IRC) in its Special Publication (SP)-41(IRC 1994) suggest empirical formula 525w to estimate SFR where w is in m for the width of approach 5.5 m to 18m. Selection of PCU values and availability of continuous one-hour green signal as well as saturation condition makes task of calculating exact SFR insurmountable. Many researchers have worked to find the saturation flow rate. Saturation flow rate (S) is ‘The maximum rate of flow of vehicles that can pass through the intersection per unit time of effective green expressed in pcu/hr or pcu/sec’. This is a service flow rate. It depends directly on width of approach. It also depends on allowed turning movements of vehicles on given approach, opposing flow of traffic, composition of traffic, approach gradient, parking activity on approach, blocking effect of bus stop etc. (Varia, 1995). Saturation flow rate (S) can be obtained by: (i) using models, (ii) field measurements (by measuring time headway or by measuring flow rate).

The famous models are given as follows:

(i) Road Research Laboratory (U.K.) Model-Widths from 5.5m to 18m:

$$S = 525W \qquad \text{……... (2.5)}$$

Where, S = Saturation flow rate (pcu/hr)

W = Width of approach road in meter at stop line

(ii) HCM (2000) model:

$$S = So\ N\ fw\ fHV\ fg\ fp\ fbb\ fa\ fLU\ fLT\ fRT\ fLpb\ fRpb, \qquad \text{…….. (2.6)}$$

Where,

S	=	saturation flow rate for the subject lane group, expressed as a total for all lanes in the lane group, veh/h,
So	=	base saturation flow rate per lane, pc/h/ln,
N	=	number of lanes in the lane group,
fw	=	adjustment factor for lane width (3.6 m width is base condition),
fHV	=	adjustment factor for heavy vehicles in the traffic stream,
fg	=	adjustment factor for approach grade,
fp	=	adjustment factor for the existence of a parking lane and parking activity adjacent to the lane group,
fbb	=	adjustment factor for the blocking effect of local buses that stop within the intersection area,
fa	=	adjustment factor for area type,
fLU	=	adjustment factor for lane utilization,
fLT	=	adjustment factor for left turns in the lane group,
fRT	=	adjustment factor for right turns in the lane group,
fLpb	=	pedestrian adjustment factor for left-turn movements, and
fRpb	=	pedestrian/bicycle adjustment factor for right-turn movements.

For the Indian conditions, Sarana and Malhotra (1967) have given

$$S = 431.7W + 103.5 \qquad \text{…….. (2.7)}$$

Bhattacharya and Bhattacharya (1982) have given

$$S = 490W - 360 \qquad \text{…….. (2.8)}$$

Chandra (1994) has given

$$S = 293W + 1241 \qquad \text{…….. (2.9)}$$

Gundaliya and Raval (2011) have given

$$S = 626W + 268 \qquad \text{…… (2.10)}$$

Saturation flow rate (S) can also be obtained by field measurements:

- By Measuring Headway

$$S = 3600/h \qquad \text{……... (2.11)}$$

Where, S = saturation flow rate (pcu/hg/ln)

W = width of approach (m)

H = saturation headway (sec/veh)

- By Measuring Flow rate

No. of departures shall be recorded against short duration of green time like 3 to 5 sec at stop line (by videography). Figure 2.5 illustrates this method. Here,

$$G + A = g + (l1 + l2) \qquad \text{……. (2.12)}$$

Where, G = Green time (sec)

A = Amber time (sec)

g = Effective green time (sec)

$l1$ = Starting lost time (sec)

$l2$ = Clearance lost time (sec)

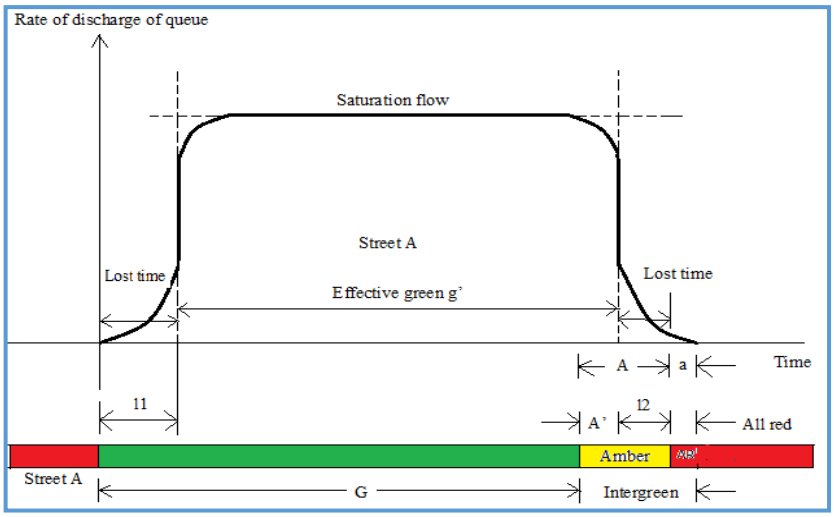


Figure 2.5: Measurement of saturation flow rate at stop line

Table 2.1 summarizes the previous studies on saturation flow rates. (Pc/h/ln = passenger car per hour per lane). Table 2.2 reviews recommendation of global standards for influential factors responsible for saturation flow rate.

Table 2.1: Previous study on saturation flow rate (Partha et al., 2014)

Study	Country	Mean (pc/h/ln)	Sample Size
Webster & Cobbe	UK	1800	100
Kimber et al.	UK	2080	64
Miller	Australia	1710	-
Branston	UK	1778	5
H.E.L.Athens	Greece	1972	35
Shoukry & Huizayyin	Egypt	1617	18
Hussain	Malaysia	1945	50
Coeyman & Meely	Chile	1603	4
Bhattacharya & Bhattacharya	India	1232	20
De Andrade	Brazil	1660	125

All eight standards totally agreed that width of approach and gradient are the key influential factor on the saturation flow rate. It forms basis for development of our two-way traffic signal coordination scheme (TW_TSCS2) explained in chapter 4.

Table 2.2: Influential factors on SFR at signalized intersections (Tuan and Man, 2015)

Influence Factors	Descriptions	Guidelines							
		HBS 2001	HCM 2000	TRRL 67	ARRB 123	TAC 2008	JTCR	IHCM 1997	MHCM
Road conditions	Approach width	Y	Y	Y	Y	Y	Y	Y	Y
	Gradient	Y	Y	Y	Y	Y	Y	Y	Y
	Intersection geometry	Y							
Traffic conditions	Vehicle type	Y	Y		Y		Y		Y
	Turning vehicle		Y	Y	Y	Y	Y	Y	
	Pedestrian crossing	Y	Y		Y	Y			
	Green time					Y			
Weather and other condition	Weather			Y	Y			Y	
	Parking		Y			Y			
	Bus stop		Y		Y	Y			
	Lane utilization		Y	Y	Y				

2.6 Common cycle length- A mandatory requirement

Cycle length is probably the most important variable for finding operational efficiency of signal coordination. Coordination of signalized intersection across network or on corridor requires common cycle length at all intersections. The present research has developed two-way traffic signal coordination scheme TW_TSCS1 (explained in chapter 4) for 3 arm, 4 arm and 5 arm intersection which require derivation of common cycle length for coordination. The cycle length should be selected in such a way that;

1. It allows safe crossing of maximum possible vehicular traffic flow from all approaches,
2. It provides minimum overall delay to vehicles.

If the cycle time is small, the proportion of time lost to the cycle time will be high, resulting in an insufficient signal operation and lengthy delays. On the other hand, if the cycle is long, the proportion of the time lost to the cycle time will be small. But sizable portion of the green time will be used by unsaturated flow of traffic, which again leads to inefficiency. So, for the each traffic flow volume there is an optimum cycle time, which shall be neither too long nor very short. Such signal design again affected by upstream and downstream traffic signals. Hence, coordination is necessary to obtain efficient traffic flow.

IRC 93-1985 reprinted in 2010 gives guideline about signal cycle time determination based on Webster's methodology which fails to give optimum signal cycle time for saturated condition. The Australian Road Research Board (ARRB) and Highway Capacity Manual (HCM) method for calculating signal cycle time also have serious limitations to replicate highly heterogeneous and saturated Indian traffic condition. It must be noted, however, that it is virtually impossible to develop a complete and final signal timing that will not be subject to subsequent fine-tuning when the proposed design is analyzed using the HCM 2010 analysis model or some other analysis model or simulation. This is because no straightforward signal design and timing process can hope to include and fully address all of the potential complexities that may exist in any given situation. (Roess et al., 2010)

2.6.1 Factors affecting cycle time

The Cycle length of a signal is the period required for one complete sequence of signal indications. The cycle length is normally divided into a number of phases, where each phase is a part of the time cycle allocated to one or more traffic or pedestrian movement. When it is designed for optimum signal cycle time, it is clear that arrival rate i.e. demand flow rate (q), saturation flow rate i.e. service flow rate (S), pedestrian requirement and total lost time (L) per cycle are deciding parameters are to be studied for optimum cycle time. It is also essential to study in details about other parameters of intersection, which are affecting optimum cycle time. Theses all parameters can be grouped under (a) Geometrical features (b) Signal phasing, (c) Traffic features, and (d) Pedestrian flow.

2.7 Summary

In this chapter brief overview of fundamental concept of signal coordination, classical methods for traffic signal coordination, key parameters of signal coordination have been carried out. The chapter covers exhaustive review of the on line and offline signal coordination techniques, saturation flow rate and passenger car unit values and common cycle length requirement for coordination. Next chapter presents literature review carried out to meet research objectives.

CHAPTER: 3

Literature Review

3.1General

Except few British colonial countries (Including India) which operate with Right Hand Drive (RHD) and Left Hand Traffic (LHT), the traffic in many countries follow Left Hand Drive (LHD) with Right Hand Traffic (RHT). The invention to the development of traffic signal coordination techniques was largely contributed by these developed countries having LHD and RHT. The nature of traffic movement and traffic composition is unique in India. This chapter present critical analysis of past research work related to research topic to achieve the research objectives.

3.2 Previous studies on signal coordination

Providing two-way signal progression for congested arterials has long been viewed by traffic professionals as one of the most effective control strategies. Over the past several decades, researchers in the traffic control community have proposed a variety of signal progression models to maximize the progression bands for the target movements. Depending on the selected measures of effectiveness (MOEs), most of those existing studies have made impressive contributions on mitigating arterial traffic congestion. The effectiveness of all progression-based models, in general, are conditioned on the common traffic pattern where through traffic constitutes the primary volume on the arterial, and consequently turning flows are not the main concern of the signal. Morgan and Little (1964) are the pioneers who first presented a model to maximize the total two-way progression bandwidth on an arterial. Following the same principle, Little (1966) further proposed an advanced model to concurrently optimize the common cycle length, progression speeds, and offsets with integer programming.

Purdy (1967) has given methodology for two- way coordination applying bandwidth concept. He has used space mean speed and distance between intersections to find out ideal offset required to coordinate traffic signal. In his method he had emphasized only arterial flow through movement for coordination and considered same speed for both forward and

backward movements. As per the observation in most of the Indian cities, four arm signalized intersections have considerable side street flow also, which cannot be neglected at all. The significant speed variation can be observed for forward and backward traffic flow directions.

Sen and Head (1997) have shown optimization of a variety of performance indices such as delay, stops and queue lengths. Based on dynamic programming and the simulation study, they produced very promising results that indicate that the COP (Controlled Optimization of Phases) algorithm is capable of significantly reducing vehicle delays.

Park et al., (1999) have presented traffic signal optimization programme for over saturated condition applying Genetic Algorithm approach. Though, these programs have been used widely among transportation professionals, none of the existing computer programs can optimize all four traffic control parameters (i.e., cycle length, green split, offset, and phase sequence) simultaneously, even for under saturated conditions. They have demonstrated that in the case of medium demand volume level, the signal timing plan obtained from the GA-based program produced statistically equivalent queue time compared with TRANSYT-7F.

Liu et al., (2002) introduced an adaptive signal control system utilizing an on-line signal performance measure. Unlike conventional signal control systems, the proposed method employs real-time delay estimation and an on-line signal timing update algorithm. They have used the method of algorithm and conclusion was that the proposed adaptive controller was sought in terms of maximizing the combined performance of all of the controllers. As addressed in the paper, the performance of the system can be improved by employing more complicated control logics.

Li et al., (2005) have discussed that it is an important aspect in urban traffic control to make the arterial traffic operate under good condition. The prevalent method is by coordinating the traffic signal lamps to obtain the progression on a corridor. They used Minimizing performance index (PI), Maximizing the bandwidth of progression, Practical methods and Theoretical methods. This paper is for pre-timed arterial traffic control. It can enhance the practical methods, and make traffic signal transition speedy and smooth to a certain extent.

Arasan and Koshy (2005) have described a modeling methodology adopted to simulate the flow of heterogeneous traffic with vehicles of wide ranging static and dynamic

characteristics. They have given simulation framework for the traffic flow in mixed traffic flow conditions where lane discipline is absent. They have also discussed the procedures adopted for validation of the proposed model and the outcomes.

Perrone (2006) discusses that the traffic signal is one of the most common facilities being operated by traffic engineers to control traffic in an orderly manner. Traffic signal timing optimization has been recognized as one of the most cost-effective methods for improving accessibility and mobility at signalized urban transportation networks. In this paper author has presented an application of stochastic optimization. The signal timings optimized by the GA-based method consistently reduced both control delay and queue time in those average values method for a large-scale signalized corridor.

Aboudolas et al., (2007) have deliberated the problem of designing real-time traffic signal control strategies for large-scale congested urban road networks via suitable application of control and optimization methods. Three alternative methodologies are proposed, all based on the store-and-forward modeling (SFM) paradigm. Simulation-based investigation of the signal control problem for a large-scale urban network using these methodologies is presented. Results demonstrate the efficiency and real-time feasibility of the developed generic control method in planning new transit routes, introducing tolls in city centers, or imposing traffic restrictions are important ingredients for combating traffic congestion in urban road networks. Practical methods make traffic signal transition speedy and smooth to a certain extent.

Andreas et al., (2008) have carried out survey to cover the research in the area of adaptive traffic control with emphasis on the applied optimization methods. Method uses Bi-level formulation, and dynamic for the online. There are several models for trace networks, which are not based on the periodic behavior of online systems to perform coordination. Instead they assign green time to phases in some order, which is optimal given the detected and predicted trace.

Yin and Chen (2009) have considered the optimal traffic signal setting for an urban arterial road by introducing the concepts of synchronization rate and non-synchronization degree. Method uses a mathematical model. They have developed algorithm to solve this optimal traffic control signal setting problem to each lane.

Kadiya and Varia (2010) have first presented a methodology to coordinate the signals for four arm intersections in two-way directions on the busy urban corridor for typical Indian

condition. They have suggested phase plan A and B comparing their suitability for the satisfactory coordination in odd and even phase differences between two 4 arm intersections. A strategy is fixed to select suitable phase plan, according to odd or even phase difference for better two-way coordination. This strategy is useful for pre-timed signal control and reduces about 30% travel time without any cost for sensors and software.

Patel K. M. (2011) has presented a methodology to coordinate the signals in four- way direction on the busy urban corridors preparing time-space diagram in AutoCAD. They have proposed phase plan A, B and H for better coordination of the selected network. Using actual road network traffic data for the four-way signal coordination, Patel K. M. (2011) has calculated delay in AutoCAD and MS Excel, compared these two delays and found that there may be considerable reduction in overall delay by four-way coordination.

Shah et al., (2013) have discussed that in the studies of Kadiya and Varia (2010) and Patel (2011) on site implementation of optimization of phase offset or virtual implementation of developed method were not performed through simulation software. In the proposed phase plan B of Kadiya and Varia (2010) having random phase movement in which right turning and through movement of particular approach is separated, which results in considerable delay in backward direction

Varia et al., (2013) in their paper (received most cited paper award in December 2016 by Elsevier) used the genetic algorithms technique for joint optimization of signal setting parameters and dynamic user equilibrium (DUE) traffic assignment for the congested urban road network. The proposed method is applied to the real network data of Fort Area of Mumbai city (India) comprising of 17 nodes and 56 unidirectional links with 72 Origin–Destination pairs, where all the 17 nodes are signalized intersections. The traffic flow condition for the optimized signal setting parameters is considerably improved compared to the existing signal settings.

Maitra et al., (2015) have studied effect of providing Queue Jump Lane (QJL) for bus in heterogeneous traffic condition. The paper documents an experience of using micro-simulation software VISSIM© to evaluate the impact of providing Queue Jump Lane (QJL) as a bus priority strategy for three representative four-arm isolated signalized intersections in Kolkata, India. The study shows that QJL is expected to be beneficial even in a heterogeneous traffic environment that is prevalent in Indian scenario. The effectiveness of

QJL is found to be influenced by various factors such as traffic volume, vehicle composition and road geometry.

Rao and Rao (2016) have identified traffic congestion factors for heterogeneous traffic in urban area. They have presented some insight on how to identify the traffic congestion and establish the congestion thresholds on urban arterial. Stream speed has emerged as one of the candidate in identifying congestion on urban arterials. Speed studies conducted on an interrupted heterogeneous mix of vehicles plying on Delhi urban arterials is also explained in the paper.

Roshandeh, et al. (2016) have elaborated that the vehicle-to-vehicle and vehicle-to-pedestrian crashes at intersections are decreased in different crash severity levels and types, especially for angle and rear-end ones after signal timing optimization. Similar results are found for multi-vehicle rear-end crashes on street segments. These indicate that intersection signal timing optimization in dense urban street networks has a potential for improving traffic mobility, vehicle and pedestrian safety at intersections, and vehicle safety on street segments.

Janrao et al., (2017) have given necessity to efficiently manage the traffic flow by completely utilizing the existing capacity of the road. Modern Cities are facing a lot of trouble due to the traffic congestion. Increasing population results in subsequent increase in the vehicles causing congestion. Traffic jams and congestion create several issues like wastage of time, excess fuel consumption. Apart from these, it directly affects routine life and sometimes may result in loss of life, e.g. in emergency cases, the ambulance boarding a critical patient cannot reach the destined hospital on time due to the congestion, where every second counts.

Cui et al., (2017) in their paper explained signal coordination method by offset control using cellular automata (CA) model. In the proposed coordinated traffic signal control method, splits of each cycle and common cycle length are calculated using a modified stochastic optimal control method, and then the offset is calculated using an estimation method based on a modified CA traffic model at intervals. In online signal optimization, the signal offset cannot be reset frequently, because the adjustment of the offset requires several cycles. The signal offset and common cycle length can be updated per 10 min in the proposed method.

Maryam et al., (2017) have incorporated urban mobility dynamics of the developing countries in a micro simulator for real-time decision making. A pilot-scale research was conducted in Peshawar, one of the ten largest cities of Pakistan. Traffic volume, travel time, vehicle specifications and geometry parameters were determined in the field as input

for the microsimulation model. Calibration and validation of the model were done, and results of traffic volume and travel time were compared with actual field data and statistical similarity confirmed through paired t-test. The analysis of queue delay, travel time and traffic flow indicates that in most of the cases congestion was found near the intersections and U-turns which was verified from the field data as well. The options of a roundabout and a flyover were assumed in the analysis; resultant queue delay and travel time decreased. With the provision of the roundabout and flyover, the capacity of traffic flow was also improved.

Fluren (2017) has compared signal coordination strategy of pre-timed control versus adaptive control for high traffic conditions. When the intersection cannot (or can just barely) handle the amount of traffic arriving at it, it becomes of extreme importance to use the intersection as effectively as possible; for some phase sequences (order of green intervals) the intersection can be used more efficiently than for others because for these sequences the time wasted on clearance times is smaller and, therefore, more time is available for green intervals. Due to the high computational complexity, it is impossible for an actuated controller to oversee all possible orders in which these signal groups can be served (in real time). Therefore, he expects an (appropriate) pre-timed controller (or an actuated controller based on this pre-timed controller) to outperform the truly adaptive controller in these high traffic situations. In fact, the truly adaptive controller can even fail completely in high traffic situations due to, for example, dynamic instabilities. An additional advantage, of keeping the order of the green intervals fixed is that road users know what to expect; if this order changes, road users might anticipate on the traffic light switching to a green indication resulting in an unsafe situation. When serving the signal groups in a pre-determined order, it is common to visualize this order in a phase diagram, which repeats indefinitely.

Lu et al., (2017) provided case study of Cebu city of Philippines and found that deployment of an adaptive area traffic control system is expensive; physical sensors require installation, calibration, and regular maintenance. Because of low budgets and technical capacity in resource-constrained economies, area traffic control systems found minimally functioning; throughout in World Bank partner countries.

Williamson M. R. et al., (2018) has studied safety impact of signal coordination along urban arterial at Mount Vernon, Illinois in the United States. The study identified the safety benefit from traffic signal coordination projects on major arterial roadways through urban areas using a before and after study with a comparison groups approach and a meta-analysis

method. The findings suggest that traffic signal coordination could decrease total crashes by 21 percent, injury crashes by 52 percent and property-damage-only crashes by 21 percent.

3.2 Previous study on Saturation Flow Rate and Passenger Car Unit

Sarna and Malhotra (1967) have conducted studies on saturation flow at a number of different intersections with varying approach road widths. They developed the relationship between the saturation flow and the approach road width at signalized intersections. Also, the effect of approach volume and increasing percentage of bicycles on the saturation flow was studied. The study has shown that the saturation flow increases with the increase in approach volume.

Maini & Khan (2000) found that the intersection clearing speed is relatively constant for different vehicle types at an intersection. They have carried out survey in Vadodara and New Delhi. This study has shown that the clearing speed does not vary significantly by vehicle type. As a result, the queue discharges in a platoon at a uniform clearing speed. The intersection clearing speed of a vehicle does not significantly vary based on the position in the queue.

Chandra and Kumar (2003) have established new concept to estimate the passenger car unit (PCU) of different types of vehicles under mixed traffic conditions in India. They utilize the area, as opposed to only the length, and speed of a vehicle. Data were collected at ten sections of two-lane roads in different parts of India. All vehicles were divided into nine different categories and their PCUs were estimated at each road section. They found that the PCU for a vehicle type increases linearly with the width of carriageway. This is attributed to the greater freedom of movement on wider roads and therefore a greater speed differential between a car and a vehicle type. The capacity of a two-lane road also increases with total width of the carriageway and the relationship between the two follows a second-degree curve.

Arasan and Vedagiri (2007) have estimated Saturation Flow Rate (SFR) of heterogeneous traffic using computer simulation and given a range of saturation flow rate from 610 to 660 PCU/meter for width of approach 3.5 to 14 meter. They have conducted study in the Chennai city under heterogeneous traffic condition. They have established significant increase in SFR with increase in width of approach road.

Prasanna Kumar and Dhinakaran (2012) have given estimation of delay at signalized intersections for mixed traffic conditions considering five locations in Tiruchirapalli, Tamilnadu, India. They have suggested that saturation flow measurement for Indian condition can be started after 3 second of initiation of green. However, HCM 2010 assumes that the SFR begins after the passing of fourth vehicle in the queue. (This is approximately equal to 10 seconds of displayed green time). They have re-established the finding of Teply (1989) that delay cannot be precisely measured and a perfect correlation between observed and predicted delay could not be obtained easily.

Chang-qiao-Shao and Xiao-ming-liu (2012) have suggested a method to estimate SFR and observed that when saturation headway distribution is symmetrical the proposed method gives consistent result with the traditional approach.

Verma et al., (2013) have shown that SFR does not depend only on width of approach, therefore empirical formula suggested for Indian condition in IRC (SP)-41(IRC 1994) is inappropriate for obtaining SFR. The study was conducted at two intersections of the CBD area of Bangalore city. While measuring SFR in their study they have assumed start-up lost time 5 second for Indian condition based on past studies. However, from the field observation carried out in their study, practically no initial start-up lost time is observed.

Dey et al., (2014) have demonstrated queue discharge characteristics at signalized intersections under mixed traffic conditions. In this paper the speed and acceleration characteristics at which the vehicles move during queue discharge for three different categories of vehicles (2-wheeler, 3-wheeler, and car) are analysed.

Chakraborty et al., (2014) have demonstrated queue discharge characteristics at signalized intersections under mixed traffic conditions. In this paper the speed and acceleration characteristics at which the vehicles move during queue discharge for three categories of vehicles (2-wheeler, 3-wheeler, and car) were analyzed. Contrary to the traditional concept of saturation flow, the discharge rates do not become stable after the fourth queuing vehicle enters the intersection. The average discharge headways become stable from position 6th (sixth) under mixed traffic conditions and the saturation headway is 1.57 s.

Savitha et al., (2017) have discussed influence of geometric factors on the saturation flow rate under mixed traffic condition of CBD area Bangalore city, India. Studies were carried out at 15 signalized intersections in the city of Bangalore with varying geometric factors such as width of road (w), gradient of the road (g), and turning radius (r) for right turning

vehicles. The geometric factors, which affect the saturation flow, have been considered in this study and accordingly a new model has been proposed for determining saturation flow. They have shown that by the introduction of the suggested adjustment factors in this paper, the saturation flow rate can give better picture of the field conditions, especially under heterogeneous traffic conditions of an urban area.

Researchers who have worked in the area of heterogeneous traffic condition defined PCU in different forms. PCU value depends on the factors such as vehicle characteristics, roadway characteristics, environmental conditions, climatic conditions, control conditions etc. In India, many researchers/ organizations have worked out the PCU values at urban as well as for rural roads and intersections. Most of the research works, already done have determined the PCU values of vehicles on midblock sections. Only a few literatures are there for determining the PCU values of vehicles at signalized intersection.

Dynamic PCU model concept developed by Chandra and Sikdar (1993) considered the intersection clearing speed of each category of vehicles. Arasan and Jagadeesh (1995) estimated the PCU for different categories of vehicles using the multiple linear regression procedure, where the saturated green time was regressed against the number of each category of vehicles crossing the stop line, during the green time, assuming a linear relationship between the variables. PCU values obtained were for bus in the range of 2.11 to 2.83, for two wheelers 0.30 to 0.38 and for three wheelers 0.58 to 0.64.

Rahman et al., (2004) presented a procedure for estimating PCE (Passenger Car Equivalent) of rickshaws and auto rickshaws at signalized intersections and suggested that the PCE values for rickshaws and auto rickshaws varies from 0.75 to 1.0 and 0.35 to 1.0 respectively depending on the proportion of vehicles in mixed traffic flow.

Patil et al., (2007) studied the influence of area type in the PCU values and estimated that the PCU for two wheeler ranges from 0.09 to 1.23, three wheelers from 0.23 to 6.14 and that of bus from 1.02 to 3.78.

Arasan and Krishnamurthy (2008) conducted a study on the effect of traffic volume and road width on PCU values of vehicles using microscopic simulation at mid-block sections of urban roads. The results showed that the PCU value of a vehicle significantly changes with change in traffic volume.

Arasan and Arkatkar (2010) used the simulation model HETEROSIM, to derive the PCU values for different types of vehicles in urban roads. The results showed that the PCU value of a vehicle significantly changes with change in traffic volume and width of roadway.

Radhakrishnan and Mathew (2011) in their study have proposed an optimization technique for the computation of dynamic PCU values. PCU values obtained were 0.34 for two wheelers, 1.88 for three wheelers and 3.90 for heavy vehicles considering eight intersection approaches and 0.24 for two wheelers, 0.6 for three wheelers and 2.26 for heavy vehicles considering eleven intersection approaches.

Joshi and Vagadia (2013), Praveen and Arasan (2013) have derived the vehicle equivalency factors for urban roads in India. It was found that under heterogeneous traffic conditions, for a given roadway and traffic composition, the PCU value of vehicles vary significantly with change in traffic volume. Hence it is desirable to consider PCU as a dynamic quantity instead of assigning fixed PCU values for the different vehicle categories of road traffic.

Sheela and Isaac (2015) have focused on Dynamic PCU values on signalized intersection and recommended variation of PCU values with flow ratio using the output obtained from the micro simulation model, TRAFFICSIM. The scope of this work was limited to four legged signalized intersections of carriageway widths varying from 3.5 to 10.5 m on level stretches The study showed that the PCU values are highly sensitive to the given traffic conditions such as approach width, traffic composition, stream speed as well as flow ratio. In this paper for nearly saturated condition having flow ratio 0.8 to 1.0 the variation in PCU values of two wheelers, three wheelers, car and bus is negligible.

3.4 Inference from literature review

- It is very difficult to include traffic heterogeneity and limited lane discipline into control model for field implementation.

- The existing offset optimization model always assumes that the travel speed is constant which does not reflect actual field observations. Intersection signal timing optimization in dense urban street networks has a potential for improving traffic mobility, vehicle and pedestrian safety at intersections, and vehicle safety on street segments.
- All available mathematical model and simulation software of signal optimization requires extensive computational efforts, licensing issues, technical manpower, sensors, huge data collection and proves costly. The applicability of these models and software to replicate Indian mixed traffic condition is a debatable issue.
- The Highway Capacity and Quality of Service Committee of the Transportation Research Board (the Official Creator of the HCM) “does not” review software nor

make any statement concerning the degree to which it faithfully replicates the HCM 2010.

- The HCM model for signalized intersection analysis is extremely complex and includes many iterative elements. As a result, there are traffic agencies, even in most advanced county like United States still use the methodology of the Interim Materials (Published by Transportation Research Board in 1980) to analyze the signalized intersections including the California Department of Transportation.
- The longest cycle length, in general, should, be used as common cycle length for optimization along corridor.
- As per authors' knowledge a few literature is available pertaining to pre-timed two-way traffic signal coordination in mixed traffic condition having Right Hand Drive (RHD) with Left Hand Traffic (LHT). This research is an attempt to address a gap in the existing body of knowledge.
- IRC 93-1985 gives guideline about signal cycle time determination based on Webster's methodology which fails to give optimum signal cycle time for saturated condition. The Australian Road Research Board (ARRB) and Highway Capacity Manual (HCM) method for calculating signal cycle time also have serious limitations to replicate highly heterogeneous and saturated Indian traffic condition. This is because no straightforward signal design and timing process can hope to include and fully address all of the potential complexities that may exist in any given situation.
- It is very difficult for any optimization software to simultaneously optimize all four signal optimization parameters in highly heterogeneous over saturated conditions.
- Saturation Flow Rate (SFR) is perhaps the most important parameter in signalized intersection control strategy. Despite its central importance it is extremely difficult to accurately measure SFR at current stage of research.
- Even the HCM 2010 suggests eleven different adjustment factors to estimate *SFR* and gives default value of 1900 pcu/hr/ln which is not relevant in Indian condition. Indian Road Congress (IRC) in its Special Publication (SP)-41(IRC 1994) suggest empirical formula 525*w* to estimate SFR.
- It is found that, under heterogeneous traffic conditions, for a given roadway condition and traffic composition, the PCU value of vehicles varies significantly with change in traffic volume and composition. Hence, it is desirable to treat PCU as dynamic quantity for the different vehicle categories.

- Indian Roads Congress (IRC), the professional organization responsible for development of codes and guidelines related to road transportation in India, has provided a single set of constant PCU values for urban roads for different vehicle categories (IRC: 106-1990), which are based on limited field observed data. IRC – SP-41-1994 suggests PCU values for estimating capacity of four lagged intersections in rural and urban roads.

3.5 Concluding Remark

The researcher after thorough literature review believes that it might be better to control a network of intersections with a pre-timed control because of its predictability. Today, traffic lights cause unpredictable delays when navigating through a network of signalized intersections. When using pre-timed control, the signal group diagram of each intersection can be communicated to the driver. This information can be used to visualize the future state of the upcoming traffic light to the road user, who can use this info to adjust its travel speed, reduce its waiting time at traffic lights, and save fuel. Furthermore, this information on the future state of the traffic lights can be used to obtain better estimates for travel times; these travel time estimates include the estimated waiting time at the traffic lights. Car navigation systems can use these travel time estimates to calculate a smart route through a network of signalized intersections controlled by a pre-timed controller.

3.6 Summary

The chapter covers exhaustive review of the on line and offline signal coordination techniques, saturation flow rate and passenger car unit values and inferences derived from the literature review. Next chapter describes basic methodology derived by considering key variables responsible for signal coordination.

CHAPTER: 4

Methodology

4.1 General

For urban arterial having four arm signalized intersections in series with constant spacing, two-way coordination of pre-timed traffic signals can be possible by traditional system like alternative or double alternative or simultaneously progressive system. But when the busy urban arterial is having signalized intersections of diverse number of approaches and different spacing between each other, then the methodology other than traditional progressive system shall be worked out. The methodology for signal coordination is derived considering holistic aspect of the signal synchronization for all possibilities. The Indian traffic is itself peculiar in nature which operates with Left Hand Traffic (LHT) having Right Hand Drive (RHD). Majority of the Indian signal systems run with Left Turn on Red (LTOR) condition. Design alternatives should be exercised to convert more than five arm intersection into rotary intersection. This chapter presents the methodology developed for the two-way coordination of pre-timed signal for 3 arm, 4 arm and 5 arm intersections for all probable six cases considering five important signal control parameters i.e. travel time, link distance, Saturation Flow Rate (SFR), Demand Flow Rate (DFR) and width of approach. After scrutinizing several permutations and combinations of phase plan and phase sequence on graph paper, nine different phase plans (five for 4 arm, and two each for 3 arm and 5 arm intersection) have been prepared to solve two- way traffic signal coordination problem with different travel time situation. Then after main emphasis of the research is devoted to evolve robust signal coordination model for commonly available four arm signalized intersections (explained in following chapter).

Optimization of signal control plan includes decision of optimal cycle length, green split (phase length), phase sequence and phase offset. After deciding phase sequence and phase plan, for other three signal control parameters, robust methodological approach is developed to calculate cycle length, green split (phase length) and phase offset for suitable two-way coordination. Traffic researchers across the world assume common travel time in both forward and backward directions to arrive at common cycle length is mandatory for signal coordination. The model (Methodology 1) described here also works on the same

assumption. Figure 4.1 elaborates flow chart of sequential approach explained in this chapter.

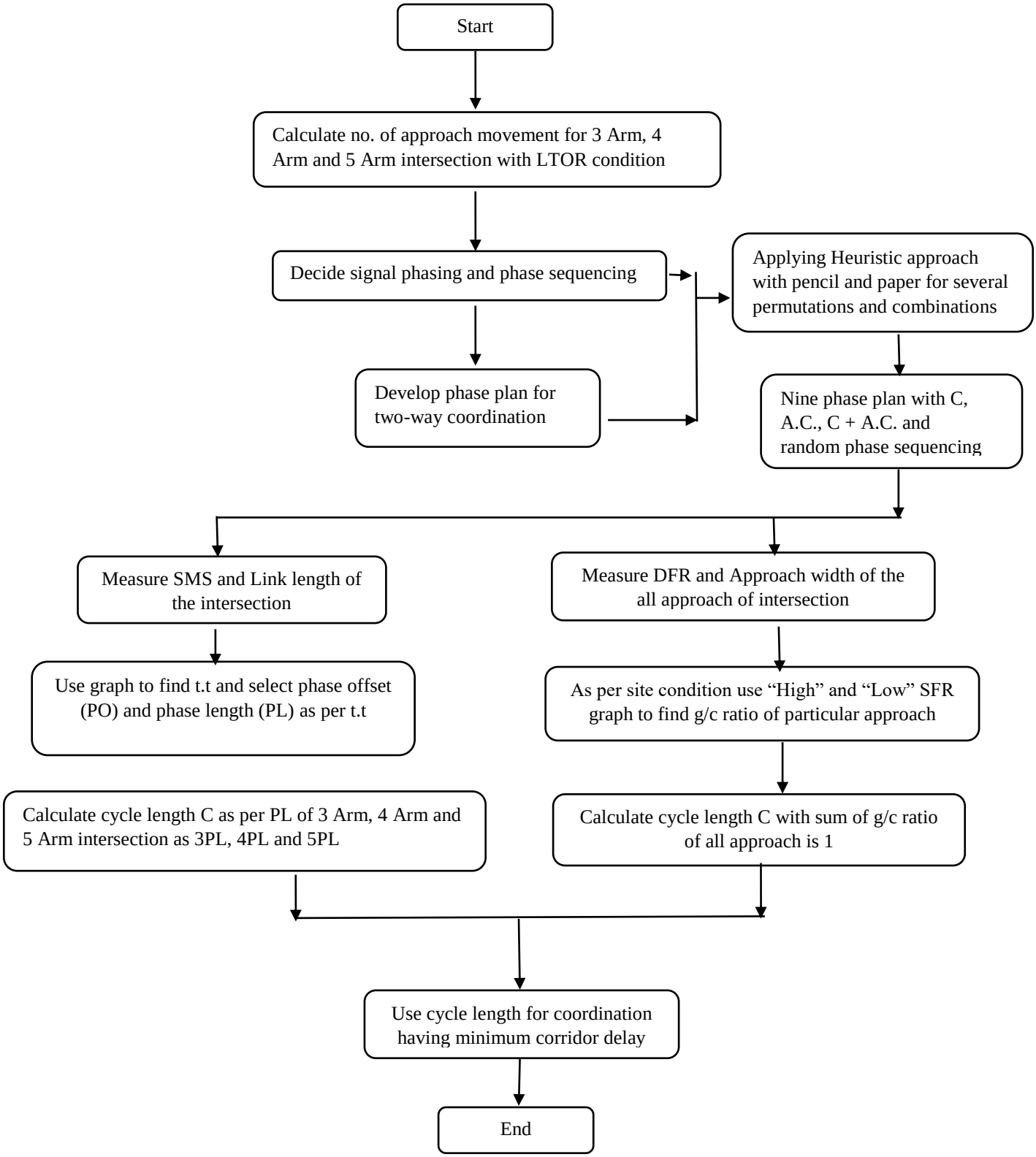


Figure 4.1: Flow chart of signal coordination approach

Signal cycle length for two way coordination depends on various key factors like travel time, link length, SFR, DFR and width of approach. Accordingly the methodology presented here for optimum cycle length calculation is divided in two parts. Initially cycle time calculation method is developed considering travel time and link length between intersections (Methodology 1). Then for deriving appropriate g/c (green/cycle) ratio and cycle time, other three key variables namely SFR, DFR and width of approach are considered (Methodology 2).

4.2 Development of phase plan and phase sequence

One of the most important aspects of signal design is the development of an appropriate phase plan for a given situation. While such aspects of signal design as the determination of cycle length and the splitting of available green time among critical movements may be formulated analytically, there are no such simple approaches to developing a phase plan. The objective of phase design is to separate the conflicting movements of an intersection into various phases, so that movements in a phase should have no conflicts. If all the movements to be separated with no conflicts, then a large number of phases are required which results in the sub optimal performance of the corridor. In such a situation, the objective is to design phases with minimum conflicts or with less severe conflicts. There is no precise methodology for the design of phases. The development of phase plan involves more professional judgment than the determination of timing parameters.

For each intersection number of possible movements on each approach will always be, n-1 where n is the number of approaches at the intersection

$$\therefore \text{ Total number of movements at intersection} = n \times (n-1) \qquad \ldots\ldots\ldots (4.1)$$

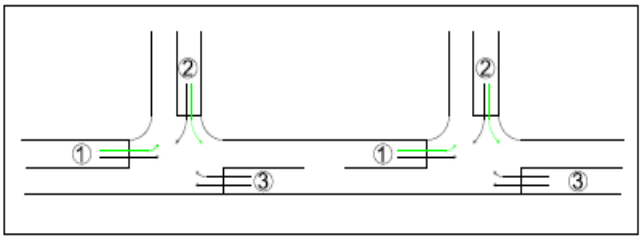
Here, for peculiar driving condition in India assuming LTOR condition in which left turners are considered having continuous flow.

$$\therefore \text{ Total number of movements to be considered for coordination} = n \times (n-1-1)$$

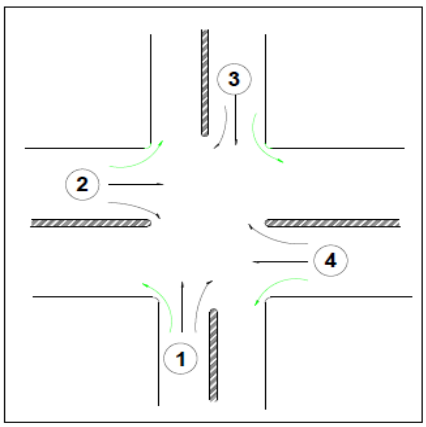
$$= n \times (n-2) \ldots. (4.2)$$

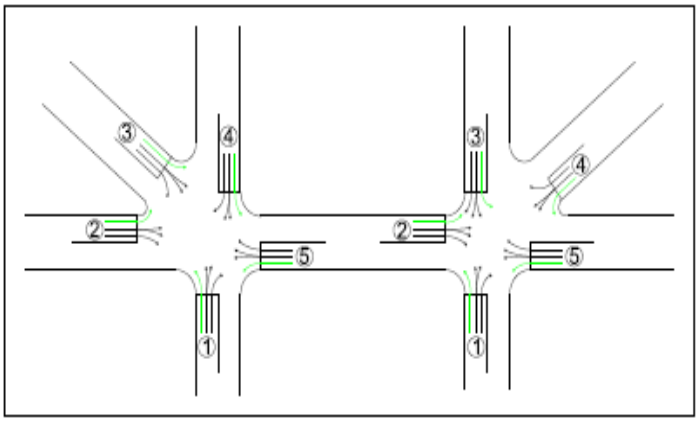
The number of movements found by this equation is to be grouped accordingly to develop efficient phase plan for coordination. Every legal movement should receive green in each cycle so for four-arm intersection as per equation (4.2) there are eight different movements

of the four approaches to be accommodated in each cycle. These eight movements are to be grouped accordingly which results in two phase, three phase, four phase or more than four phase signal plan. Two phase signal plan is adopted in the through corridor having negligible right turning traffic with permitted right turn. More than four phase signal plans have philosophical importance and are rarely used in practice for two way coordination. When the turning movements are having sizable amount of flow, four phase signal plan is adopted for achieving efficient two- way coordination. The eight movements of the intersection are suitably grouped in the four groups and these movement groups are coded with 1, 2, 3 and 4. Here figure 4.2 a, 4.2b and 4.2c depicts the movements and group selected to develop phase plan for two- way coordination of three, four and five arm intersection. After scrutinizing several permutations and combinations of phase plan, phase group and phase sequence on graph paper, nine different phase plans (five for 4 arm, and two each for 3 arm and 5 arm intersection) have been prepared to solve two- way traffic signal coordination problem (Figures 4.3, 4.4a and 4.4b).



(a)





(b) (c)

Figure 4.2: Adopted phase movement group for (a) 3 arm intersection, (b) 4 arm intersection and (c) 5 arm intersection

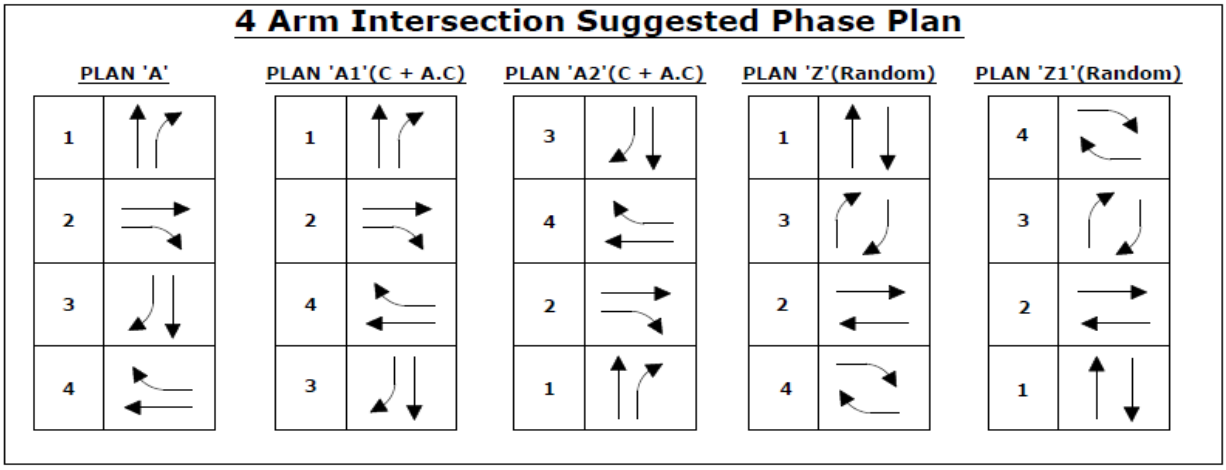
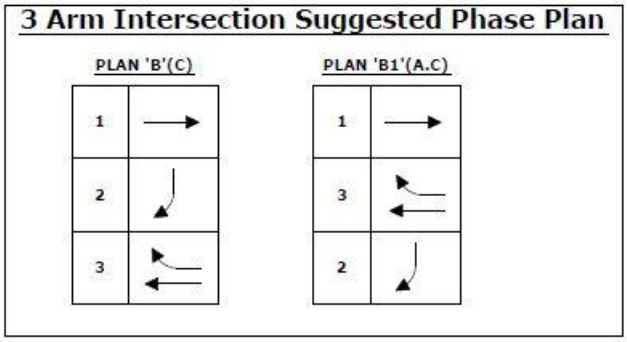
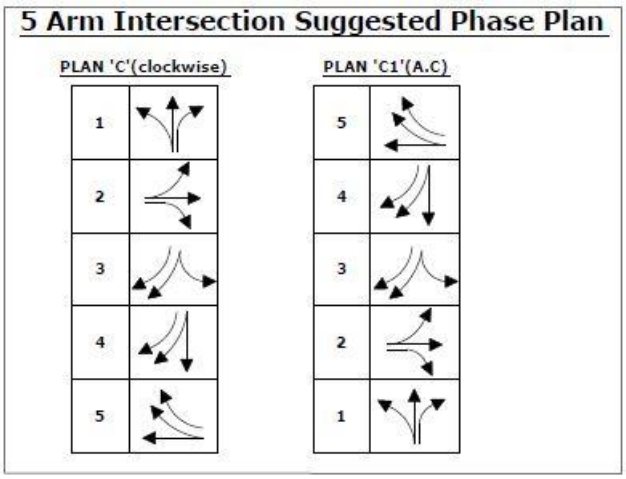


Figure 4.3: Suggested phase plans for 4 arm intersection



(a)



(b)

Figure 4.4: Proposed phase plans (a) for 3 arm intersection and (b) for 5 arm intersection

Figure 4.3 shows suggested phase plans for 4 arm intersection and figures 4.4a & 4.4b present details of different phase plans developed in this research for effective two-way

coordination for 3 arm and 5 arm intersections. Table 4.1 elaborates combination of phase sequence for odd and even phase difference situations for all six considered cases of coordination. Judicious arrangements of phase plans and phase offsets with all probable groupings of 3 arm, 4 arm and 5 arm intersections have been worked out and presented in tabular format (Table 4.2 in methodology 1).

4.2.1 Offset selection

When traffic signals are located in close proximity, the presence of the upstream traffic signals alters the arrival pattern of traffic at the downstream traffic signals from random arrivals to arrivals in platoons. This means that improved traffic flow can be achieved if the green signal at the downstream traffic signal is arranged to coincide with the arrival of the platoon. To achieve this, traffic signals are coordinated, sometimes called “linked”. This improves the level of service on a road network where the spacing of traffic signals is such that intersection operation causes excessive delays.

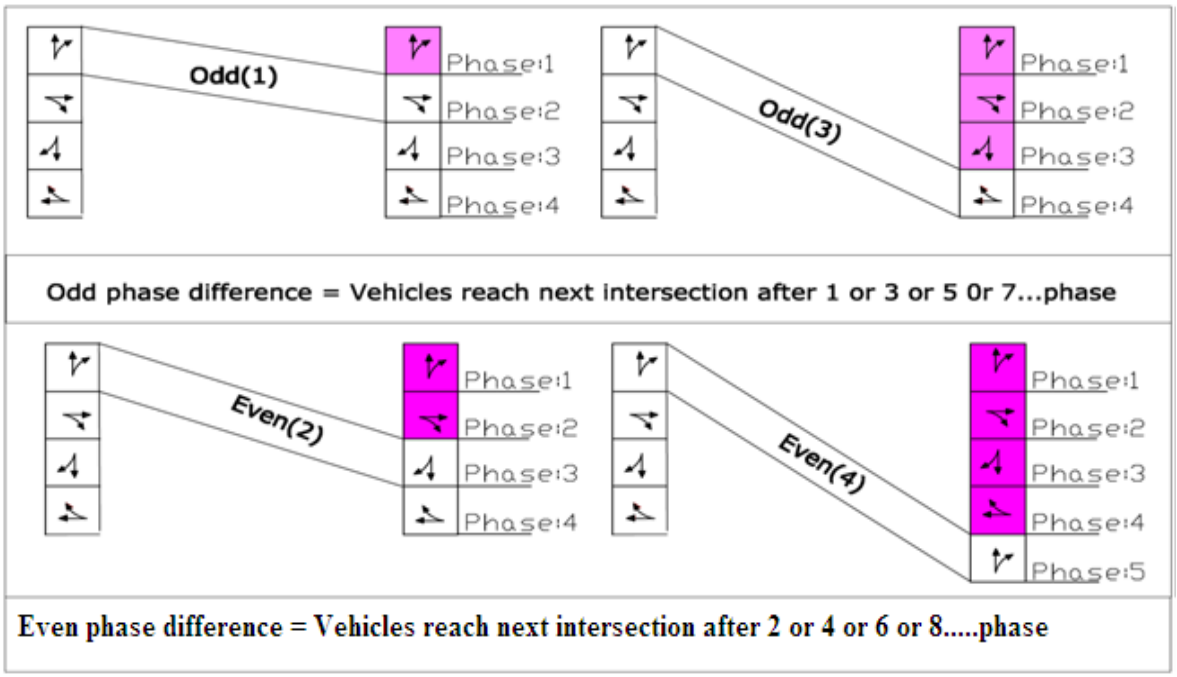


Figure 4.5: Odd and even phase difference

Appropriate phase sequences coupled with suitable phase offsets are also very important to avoid delay. In this study four arm junctions with four phases are considered, which are generally found in Indian cities. The phase sequence shall be adjusted such that right turners entering to main corridor and straight movers (through traffic) entering to main corridor on same direction should have minimum time difference to reach at the next intersection. If straight movers are more (i.e. through traffic flow is higher) than right turners, straight flow

should be given second priority to get minimum delay on next intersection for clearance. Thus, through traffic and right turners should get quicker clearance on major corridor in all the directions. Unequal phase timing and inappropriate phase sequence on two consecutive four-arm junctions responsible for more delay to the vehicles. Phase difference between intersections depends on distance, average speed of traffic stream, geometrics of the link, vehicle composition and other reasons. Phase difference means average travel time of vehicles from upstream intersection to the next downstream intersection in terms of phase length. According to phase difference, strategy of phase plan shall be adopted (table 4.2). Figure 4.5 shows clear understanding of odd and even phase differences.

Now, it is necessary to find out which phase plan strategy is better for odd and even phase difference. Accodingly Two Way Traffic Signal Coordination Strategy 1 (TW_TSCS1) is derived.

4.3 Two -Way Traffic Signal Coordination Strategy 1 (TW_TSCS1)

In two-way signal coordination, it is preferable to adopt an average cycle time of the intersections of the corridor. An average travel time between intersections generally depends on distance, average speed of traffic stream, geometrics of the link, traffic composition and other reasons. According to average travel time, strategy of phase plan shall be adopted. Following assumptions and considerations are made in this strategy:

- Link joining two consecutive intersections has uniform geometrics throughout.
- Vehicles travelling in the band and arriving at approaches as well as other vehicles in the queue on an approach will clear the intersection during the green phase.
- Phase length (*Pl*) includes green time (*G*) + amber time (*A*) + all red time (*AR*) if any for a given phase
- All approaches have same traffic demand and all movements on each approach proceed simultaneously during green phase
- As far as possible, right turners from minor street will get first green phase and then straight movers from major street will get green phase, to reduce the waiting delay of major street flow on the next intersection
- Analysis is carried out using time-space diagram. Acceleration and deceleration of vehicles are neglected and band is considered with straight line boundaries having uniform width.

Kadiya (2011) has concluded that for the four arm junctions, if the ratio of travel time (*tt*) to the phase length (*Pl*) is kept even number (i.e. *2n*, where *n* = 1, 2, 3 …), then clockwise progression of equal phases on both intersections will give good coordination in both directions with minimum delay. This is depicted in Figure 4.6. A phase sequence 1-2-3-4 in clockwise progression of approach number 1, 2, 3 and 4 respectively is denoted by 'Phase Plan A'.

Here, travel time (*tt*) can be obtained as,

$$tt = d/v \qquad \text{….. (4.3)}$$

Where, *tt* = average travel time of vehicles in sec to reach the next intersection

d = distance in meter between two consecutive intersections

v = space mean speed of traffic stream in m/s

For the two-way coordination on four arm junctions,

$$\text{Phase offset } (Po) = \text{travel time } (tt) = Pl \times 2n \qquad \text{….. (4.4)}$$

$$\text{Therefore, phase length } (Pl) = tt/2n \qquad (\text{where } n = 1, 2, 3 \ldots) \qquad \text{….. (4.5)}$$

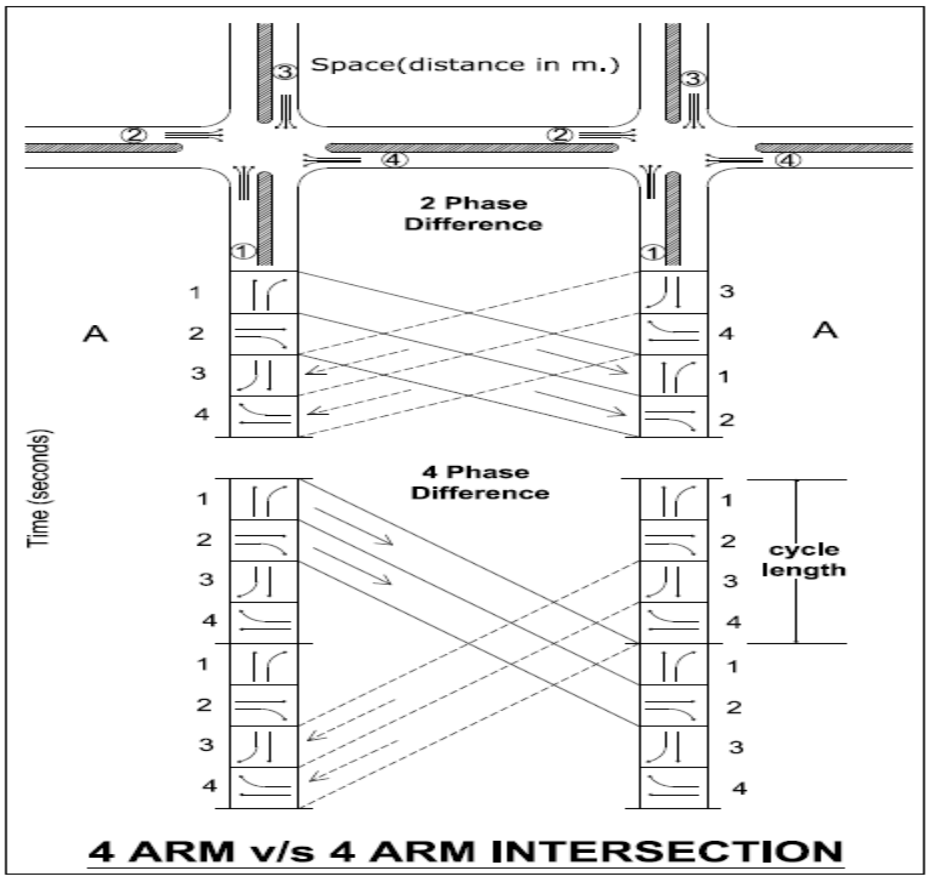


Figure 4.6: Two- way coordination for even phase difference

The above relationship (4.5) remains valid until the $Pl \geq$ minimum green requirement (g_{min}). As per Indian Road Congress (IRC 93-1985) recommendations g_{min} should be 16 sec. However, considering the actual traffic flow demand on both the intersections, minimum green time (g_{min}) shall be adopted which may be greater than or equal to 16 sec. If this $g_{min} > tt$, then proper two-way coordination cannot be carried out. But, when $g_{min} \leq tt < 2\, g_{min}$ condition satisfies, then $Pl = tt$ can be considered. These situations give two criteria for two-way coordination shown as follows:

1. $Pl = tt/2n$ (when, $tt \geq 2\, g_{min}$) ….. (4.5)

2. $Pl = tt$ (when, $g_{min} \leq tt < 2\, g_{min}$) ….. (4.6)

Considering 4 equal phases for 4 arm intersections, total cycle time C in sec will be,

$$C = 4Pl \qquad \text{….. (4.7)}$$

The above situation 1 (equation 4.5) can be named as ‘even phase difference’ situation, whereas situation 2 (equation 4.6) can be named as ‘odd phase difference’ situation. The figure 4.7 shows the two-way coordination for the odd phase difference between two four arm intersections having equal phase lengths. It seems that two-way coordination can be obtained by changing the phase offset and phase sequence in proper way. Let the left side phase plan named as ‘Phase Plan A1’ which is having phase sequence 1-2-4-3 and right side phase plan named as ‘Phase Plan A2’ which is having phase sequence 3-4-2-1. It is observed that in the upstream of Phase Plan A1, if the four arm intersection is situated at even phase difference, Phase Plan A will be appropriate, whereas for odd phase difference, Phase Plan A1 will be appropriate. Similarly, in the downstream of Phase Plan A2, for the even phase difference Phase Plan A and for the odd phase difference Phase Plan A2 will be appropriate. This is shown in figure 4.8.

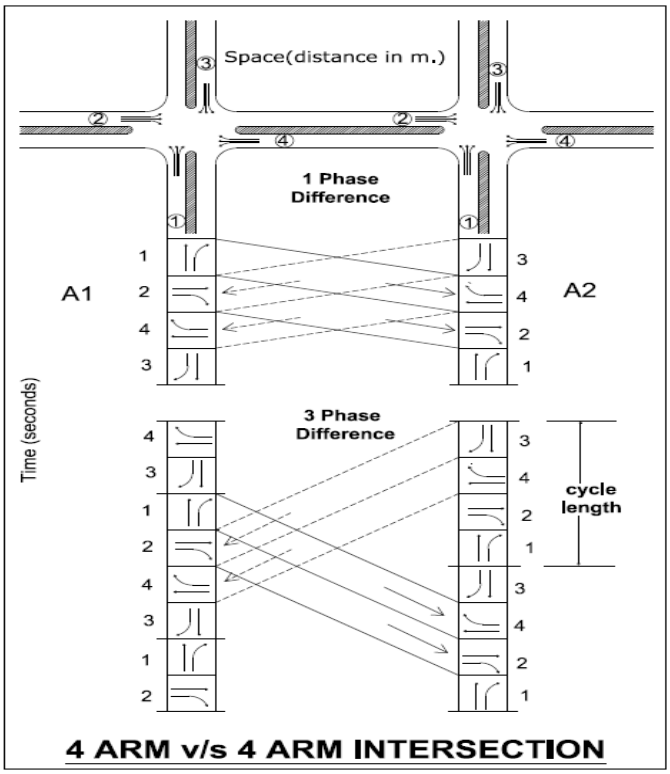


Figure 4.7: Two- way coordination for odd phase difference

The developed methodology is checked by trial and error method using virtual network adopting 120 second cycle time with equal phase length of 30 second. The average delay for right turners (R) and straight through (S) in forward and backward directions obtained as in figures 4.6, 4.7 and 4.8, presented in table 4.1. Sample delay calculation is presented here. A detail of delay calculation procedure is given in Annexure II.

1 Right Turner delay (For fig. 4.6)

Phase time =30 second

From figure it is visible that Right turning movement band (movement 1) of upstream intersection intersects exactly above the available green band of downstream intersection.

∴ Average delay for all vehicles of the movement =30/2= 15 second

2 Straight through delay (for fig. 4.6)

From figure it is clear that straight movement band (movement 2) of upstream intersection exactly coincide with the available green band of downstream intersection.

∴ Average delay for all vehicles of the movement = 0

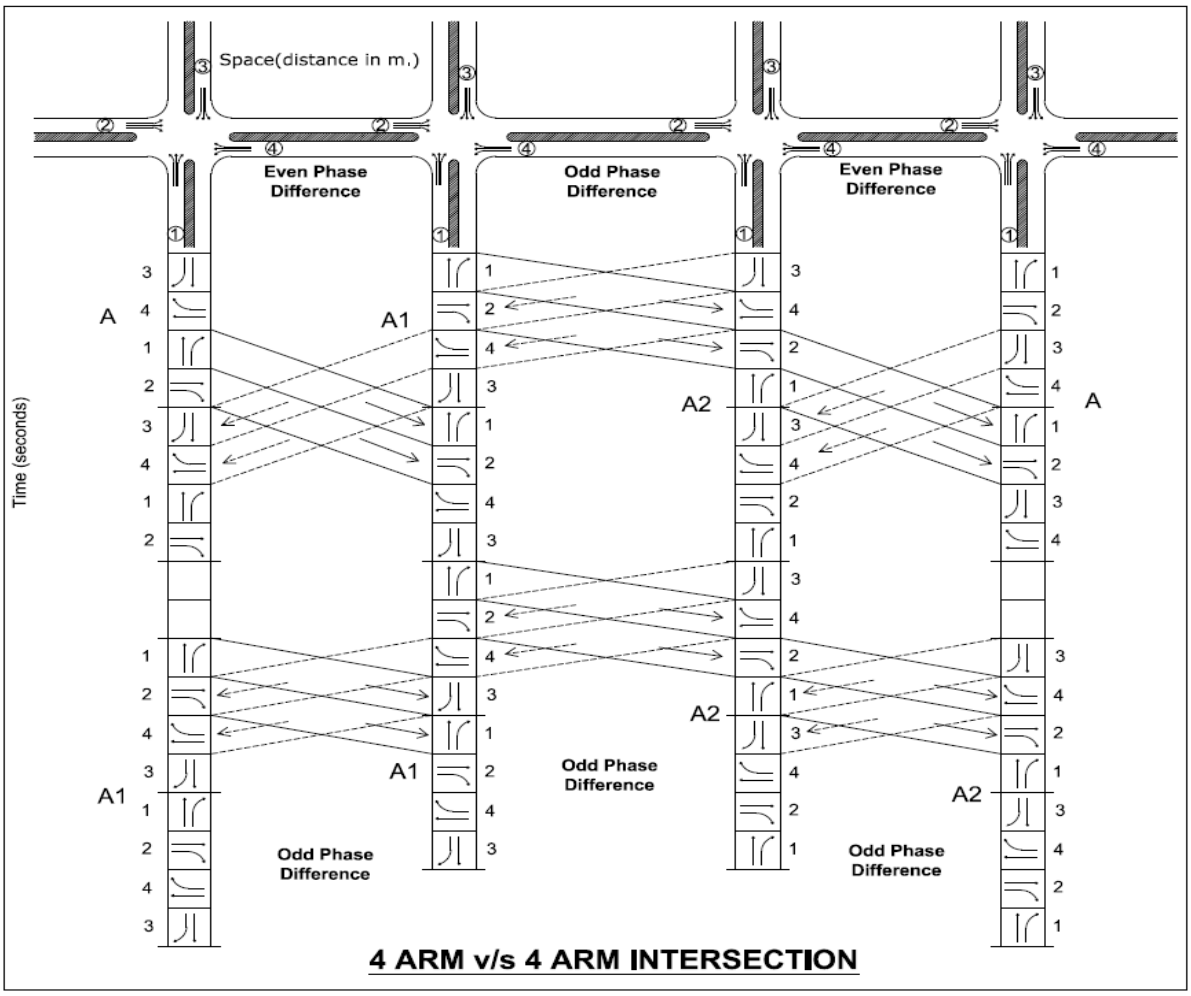


Figure 4.8: Continuous two- way coordination

Table 4.1: Average delay calculation for even and odd phase difference (Second)

No	Direction of Movement	Plan A To A (Fig 3.6)		Plan A1 To A2 (Fig 3.7)		Plan A1-A1-A2-A2 (Fig 3.8)		Plan A-A1-A2-A (Fig 3.8)	
		Forward	Backward	Forward	Backward	Forward	Backward	Forward	Backward
1	R to R & R to S	15	15	15	15	60	60	30	30
2	S to S & S to R	0	0	0	0	30	30	15	15
Total delay		15	15	15	15	90	90	45	45
Combined delay		30		30		180		90	

In the continuation of 4 arm intersections on the busy urban corridor, sometimes 3 arm or 5 arm or more than 5 arm intersections may exist. In this situation, it is challenging to decide a strategy for phase plans and phase sequences. Generally more than 5 arm intersections are converted into Rotary Intersection and it is difficult to set two-way coordination for that. So, in this study following combinations of intersections are considered for developing the methodology.

1. 3 arm –vs- 3 arm
2. 3 arm –vs- 4 arm
3. 3 arm –vs- 5 arm
4. 4 arm –vs- 5 arm
5. 5 arm –vs- 5 arm

4.3.1 3 arm -vs- 3 arm

Figure 4.9 shows the phase plans required for even and odd phase differences for 3 arm –vs- 3 arm signalized intersections. It is observed that for even phase difference, clockwise progression on both intersections will work properly. This is named as 'Phase Plan B' with 1-2-3 phase sequence. For the odd phase difference, anti-clockwise progression on both intersections will work properly. This is named as 'Phase Plan B1' with 1-3-2 phase sequence. Considering 3 equal phases for 3 arm intersection, total cycle time *C* (in sec) will be,

$$C = 3Pl \qquad \text{….. (4.8)}$$

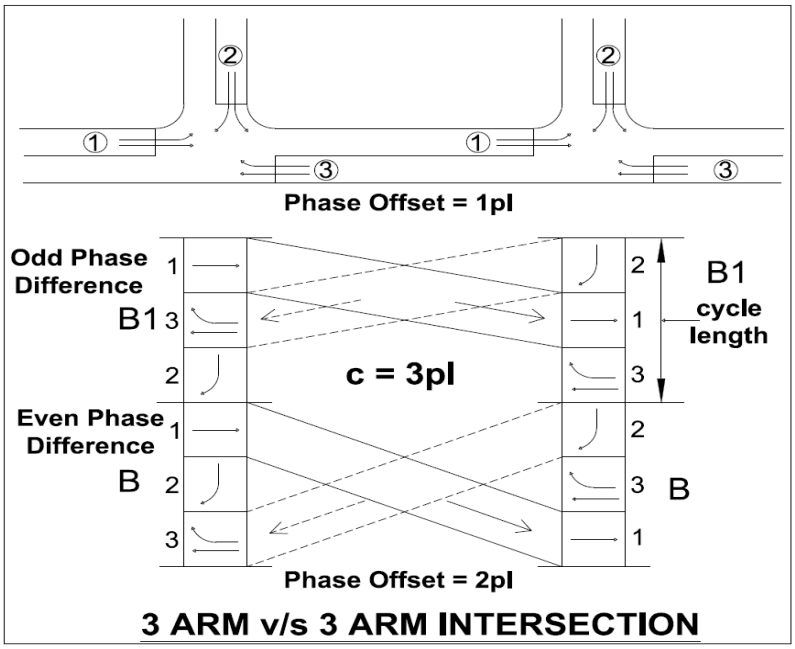


Figure 4.9: Two- way coordination for 3 arm –vs- 3 arm intersection

4.3.2. 3 arm -vs- 4 arm

Figure 4.10 shows the phase plans required for even and odd phase differences for 3 arm –vs- 4 arm signalized intersections.

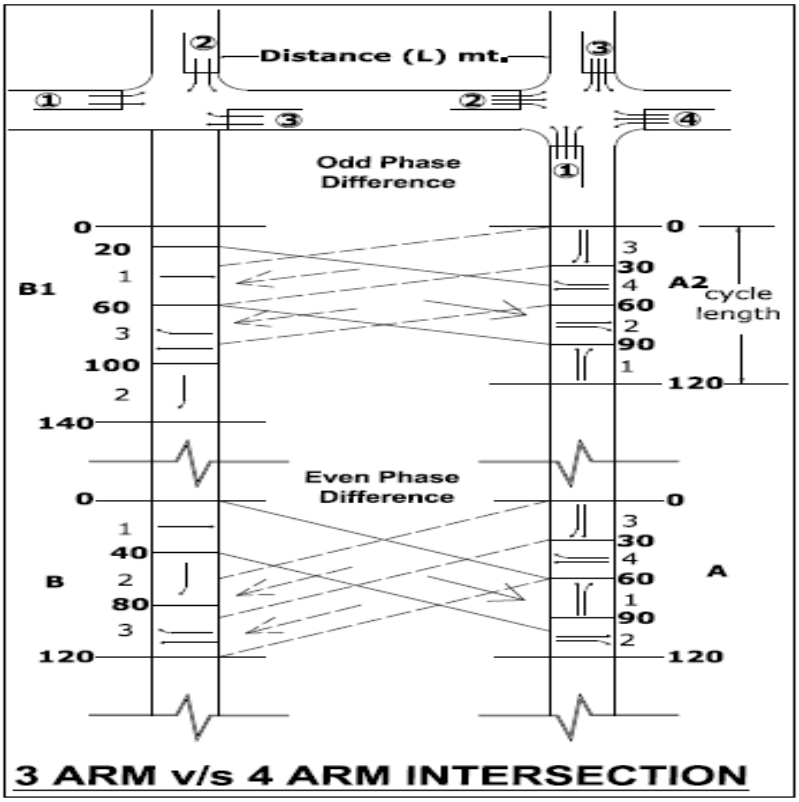


Figure 4.10: Two- way coordination for 3 arm –vs- 4 arm intersection

It is observed that for even phase difference, clockwise progression on both intersections, i.e. Phase Plan B –vs- Phase Plan A will work properly with suitable phase offset. For the odd phase difference, Phase Plan B1 –vs- Phase Plan A2 will work properly with suitable phase offset. Calculate cycle time for each intersection as $C_{3\,arm} = 3Pl_{3\,arm}$ and $C_{4\,arm} = 4Pl_{4\,arm}$, considering the equation (4.5) and (4.6). It is advisable to keep equal cycle on both intersections. So, consider C = Max $\{C_{3\,arm}, C_{4\,arm}\}$. Therefore,

$$Pl_{3\,arm} = C/3 \quad \text{(for 3 arm intersection)} \qquad \text{….. (4.9)}$$

$$Pl_{4\,arm} = C/4 \quad \text{(for 4 arm intersection)} \qquad \text{….. (4.10)}$$

4.3.3 3 arm -vs- 5 arm

Figure 4.11 shows the phase plans required for even and odd phase differences for 3 arm –vs- 5 arm signalized intersections.

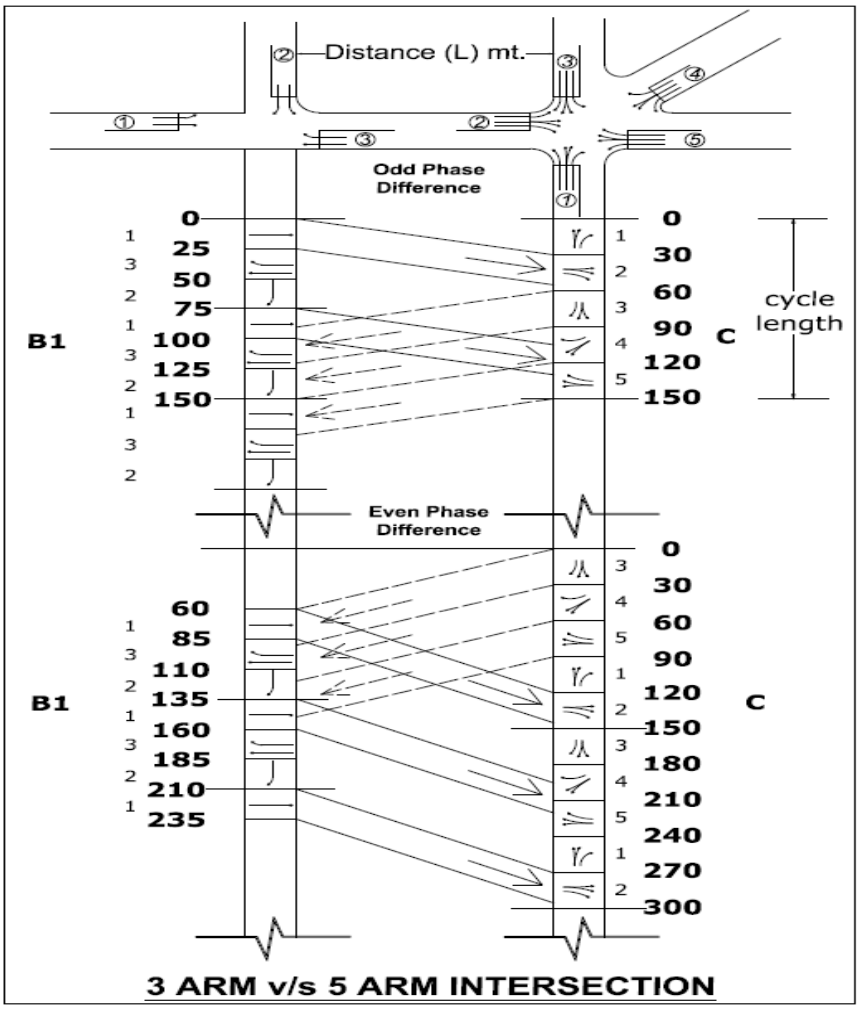


Figure 4.11: Two- way coordination for 3 arm –vs- 5 arm intersection

It is observed that for even and odd phase difference, Phase Plan B1 –vs- Phase Plan C of 1-2-3-4-5 phase sequence will work properly with suitable phase offset. Calculate cycle time for 5 arm intersection $C_{5\ arm} = 5Pl_{5\ arm}$, considering the equation (4.5) and (4.6). It is advisable to keep half of this cycle *(i.e.* $C_{5\ arm}/2$*)* on 3 arm intersection, if $(C_{5\ arm}/6) \geq g_{min}$ of 3 arm intersection otherwise $C_{5\ arm} = 6\ g_{min}$ of 3 arm intersection. Therefore,

$$Pl_{3\ arm} = C_{5\ arm}/6 \quad \text{(for 3 arm intersection)} \qquad \text{….. (4.11)}$$

$$Pl_{5\ arm} = C_{5\ arm}/5 \quad \text{(for 5 arm intersection)} \qquad \text{….. (4.12)}$$

4.3.4 4 arm -vs- 5 arm

Figure 4.12 shows the phase plans required for even and odd phase differences for 4 arm –vs- 5 arm signalized intersections.

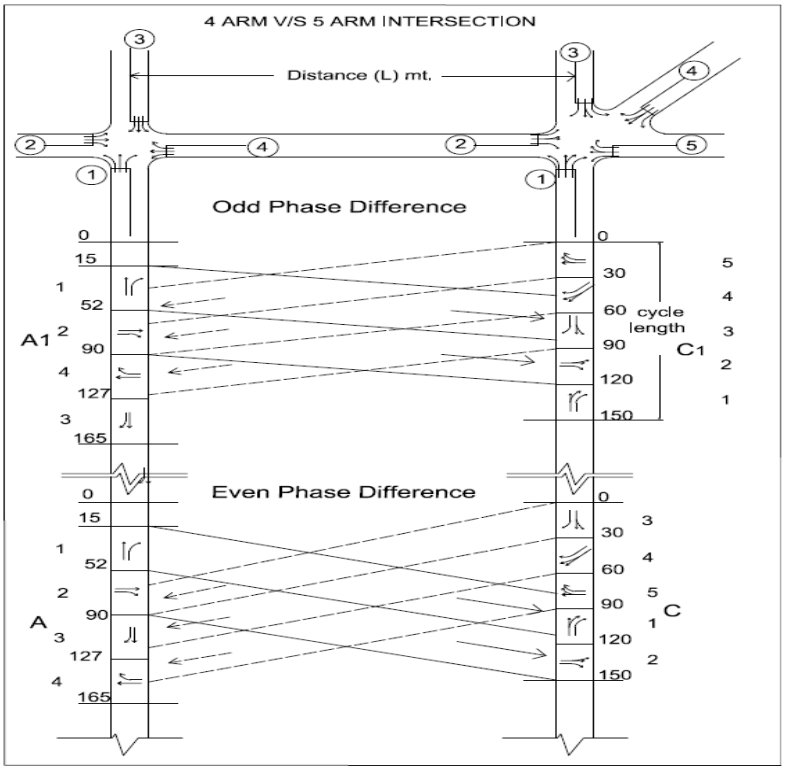


Figure 4.12: Two- way coordination between 4 arms –vs- 5 arm intersection

It is observed that for even phase difference, Phase Plan A –vs- Phase Plan C of 1-2-3-4-5 phase sequence will work properly with suitable phase offset. Whereas, for the odd phase difference, Phase Plan A1 –vs- Phase Plan C1 of 1-5-4-3-2 phase sequence will work properly with suitable phase offset. Calculate cycle time for 5 arm intersection $C_{5\,arm} = 5Pl_{5\,arm}$, considering the equation (4.5) and (4.6). It is advisable to keep the same cycle on 4 arm intersection, if $(C_{5\,arm}/4) \geq g_{min}$ of 4 arm intersection, otherwise $C_{5\,arm} = 4\,g_{min}$ of 4 arm intersection. Therefore,

$$Pl_{4\,arm} = C_{5\,arm}/4 \quad \text{(for 4 arm intersection)} \qquad \text{….. (4.13)}$$

$$Pl_{5\,arm} = C_{5\,arm}/5 \quad \text{(for 5 arm intersection)} \qquad \text{….. (4.14)}$$

4.3.5 5 arm -vs- 5 arm

Figure 4.13 shows the phase plans required for even and odd phase differences for 5 arm –vs- 5 arm signalized intersections. It is observed that for even phase difference, Phase Plan C –vs- Phase Plan C of 1-2-3-4-5 phase sequence will work properly with suitable phase offset. Whereas, for the odd phase difference, Phase Plan C1 –vs- Phase Plan C1 of 1-5-4-3-2 phase sequence will work properly with suitable phase offset. Calculate cycle time for 5

arm intersection $C_{5\,arm} = 5Pl_{5\,arm}$, considering the equation (4.5) and (4.6). It is advisable to keep the same cycle on both intersections. Therefore,

$$Pl_{5\,arm} = C_{5\,arm}/5 \quad \text{(for 5 arm intersection)} \qquad \text{….. (4.15)}$$

From the time-space diagrams it is clear that due to 5 arms, vehicles experience more delay. Some of the findings from the above figures are given in Table 4.2.

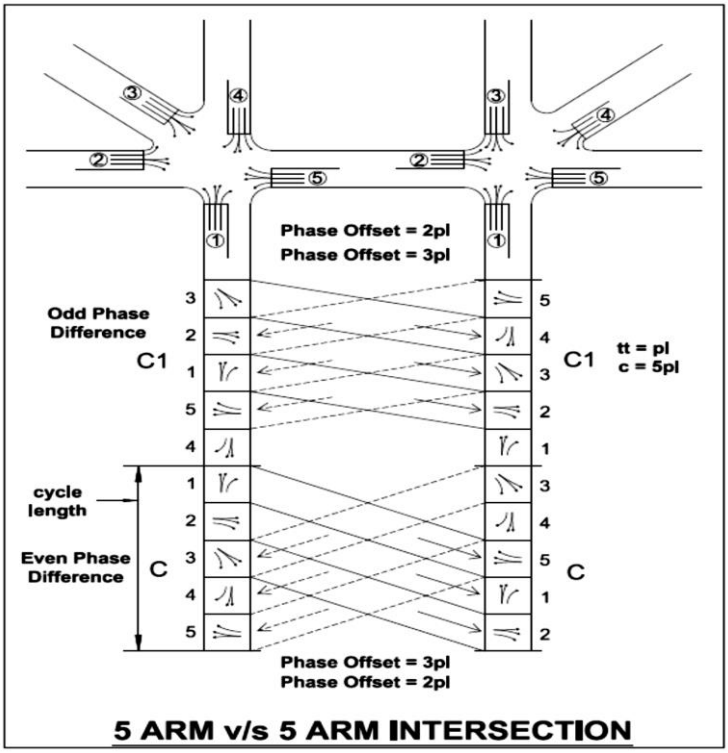


Figure 4.13: Two- way coordination between 5 arms –vs- 5 arm intersection

Table 4.2: Details of two-way coordination for the different combinations

Combination	Situation (Phase difference)	Type of Phase Plan		Cycle time		Required phase offset for major street flow	
(Left-vs-Right)		Left	Right	Left	Right	Forward direction	Backward direction
4arm-vs-4arm	Even	A	A	*4Pl*	*4Pl*	*2Pl*	*2Pl*
	Odd	A1	A2	*4Pl*	*4Pl*	*Pl*	*Pl*
3arm-vs-3arm	Even	B	B	*3Pl*	*3Pl*	*2Pl*	*2Pl*
	Odd	B1	B1	*3Pl*	*3Pl*	*Pl*	*Pl*
3arm-vs-4arm	Even	B	A	*4Pl*	*4Pl*	*3Pl*	*2Pl*
	Odd	B1	A2	*4Pl*	*4Pl*	*Pl*	*Pl*
3arm-vs-5arm	Even	B1	C	*2.5Pl*	*5Pl*	*2Pl&4 Pl*	*3Pl*
	Odd	B1	C	*2.5Pl*	*5Pl*	*Pl& 3 Pl*	*2Pl*
4arm-vs-5arm	Even	A	C	*5Pl*	*5Pl*	*2Pl*	*2Pl*
	Odd	A1	C1	*5Pl*	*5Pl*	*Pl*	*3Pl*
5 arm-vs-5arm	Even	C	C	*5Pl*	*5Pl*	*3Pl*	*2Pl*
	Odd	C1	C1	*5Pl*	*5Pl*	*2Pl*	*3Pl*

Table 4.3: Combination of movements suggested for odd and even phase difference

Type of Coordination	Odd Phase Difference (Travel Time < 32 sec)		Even Phase Difference (Travel Time > 32 sec)	
Three arm V/S Three arm	Anti-Clockwise	Anti-Clockwise	Clockwise	Clockwise
Four arm V/S Four arm	Clockwise + Anti Clockwise	Clockwise + Anti Clockwise	Clockwise	Clockwise
Five arm V/S Five arm	Anti-Clockwise	Anti-Clockwise	Clockwise	Clockwise
Three arm V/S Four arm	Anti-Clockwise	Clockwise + Anti Clockwise	Clockwise	Clockwise
Three arm V/S Five arm	Anti-Clockwise	Clockwise	Anti-Clockwise	Clockwise
Four arm V/S Five arm	Clockwise + Anti Clockwise	Anti-Clockwise	Clockwise	Clockwise

After developing the methodology of calculating phase length and signal cycle time (depending on the above conditions given in Table 4.2), the graph (Fig. 4.14) is prepared to determine the feasible phase length (s) depending on distance between intersection (m) and the range of SMS (kmph). It gives $Pl = tt/2n$ (Equation 4.5). As per prevailing Indian condition, link length between two adjacent intersections can be considered ranging from 100 meter to 1000 meter and space mean speed range of 15 kmph to 40 kmph. According to practical considerations and optimal cycle lengths, minimum phase length of 16 sec and maximum phase length of 60 sec is considered. For example, if the spacing between two 4 arm intersections is 500 m and SMS is 30 kmph, then using Fig. 4.14 and Table 4.2; phase length will be 30 sec ($tt/2$). Considering equal length of 4 phases, cycle length on both intersections will be 4 x 30 = 120 sec. Phase Plan A can be adopted on both intersections with phase offset of 2 x 30 = 60 sec, which will give proper two-way coordination. In another case, if the spacing between two 4 arm intersections is 300 m and SMS is 36 kmph, then phase length comes out 15 sec ($tt/2$) (Fig. 4.14) i.e. tt will be 30 sec. Suppose, the required g_{min} is 20 sec. Then, as per equation 4.5, $Pl = tt = 30$ sec and cycle length on both intersections shall be 4 x 30 = 120 sec. This is a case of odd phase difference, therefore Phase Plan A1 and A2 can be applied with 30 sec phase offset (i.e. Pl) in both directions.

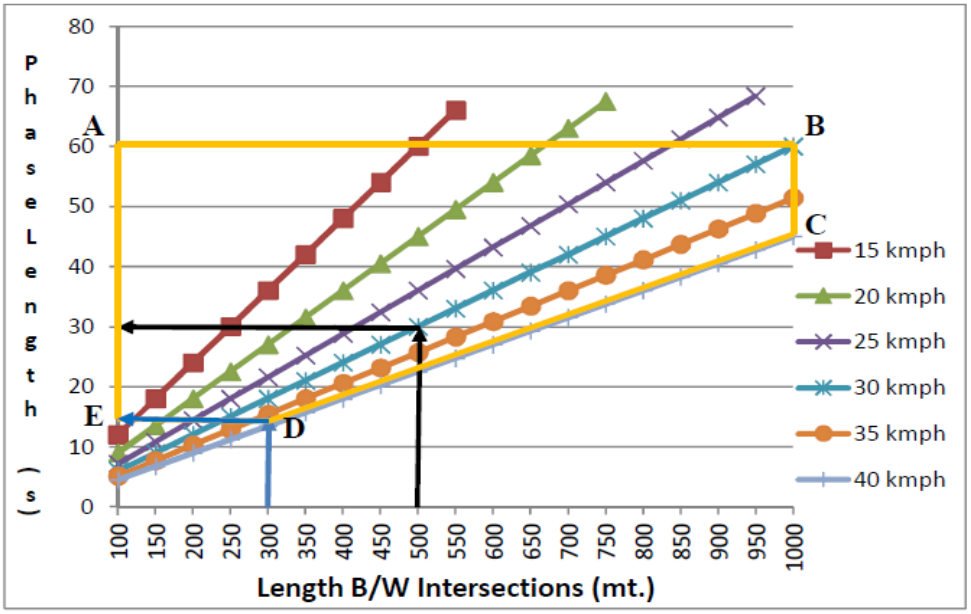


Figure 4.14: Relationship among spacing, SMS and phase length

4.4 Two-Way Traffic Signal Coordination Strategy 2 (TW_TSCS2)

The method proposes easy to comprehend green time to cycle time ratio (g/c) calculating methodology simply by measuring width of approach and DFR of concerned signalized intersection thus eliminating the need of gigantic task of measuring SFR. Here to devise the methodology following assumptions are made.

1. After exploring available past literature "Low" SFR is assumed as 650W i.e. 0.63 pcu/sec/lane (for 3.5 meter lane width)

2. "High" SFR is assumed 1028W i.e. 1pcu/sec/lane

3. Demand Flow Rate range is assumed varying from 0.1 to 0.95 percent of the saturation flow rate.

4. Width of approach is taken from 3.5 to 14 meter and "low" saturation flow rate for Indian condition is assumed as 650 pcu/hr/m for 3.5 meter to 705 pcu/hr/m for 14 meter width of approach.

The saturation flow rate is, in fact, the capacity of the approach lane or lanes if they were available for use all of the time (i.e., if the signals were always GREEN). The signal, of course, is not always GREEN for any given movement as well as practically it is not possible to find any signal with continuous green of one hour. Thus, some mechanism (or model) for

dealing with the cyclic starting and stopping of movements must be developed. Green time requirement for any approach is governed by the age old economics principle of demand and supply. Here demand is considered as product of demand flow rate and cycle time while supply is multiplication of effective green time and saturation flow rate. Putting this into equation form, degree of saturation for a given approach is,

$$X = \frac{qc}{gs} \qquad \text{……. (4.16)}$$

Where,

X= Degree of saturation

q = Demand Flow Rate (pcu/hour)

c = cycle time (second)

g = effective green time (second)

s = Saturation Flow Rate (pcu/hour)

From this equation (4.16) for X= 1 the g/c ratio would be,

$$\frac{g}{c} = \frac{q}{s} \qquad \text{…… (4.17)}$$

To achieve optimal efficiency and maximize vehicular throughput at the signalized intersection, traffic flow must be sustained at or near saturation flow rate on each approach. For deriving the methodology, two different cases (1) “Low” SFR condition and (2) “High” SFR condition have been considered.

4.4.1 “Low” SFR condition

Analysis of equation (4.17) will give g/c (green time/ cycle time) ratio required to satisfy demand of the particular approach. For fixing denominator values, past literature on the saturation flow rate measurement in mixed traffic condition has been explored and extensively studied. After studying all the manuals and codes globally, the key factor responsible for saturation flow rate is approach width and gradient. As shown in table 2.2 of chapter 2, all the global standard and guidelines are unanimously agreed that approach width and gradient will effect on the SFR. For developing methodology, uniform geometry throughout the corridor is considered. So for deriving SFR, approach width is considered. Depending on the lane width, the saturation headway value will differ slightly. Generally, increase in lane width will reduce the saturation headway which ultimately leads to greater

saturation flow rate for bigger lane width. Arasan and Vedagiri (2007) have estimated Saturation Flow Rate (SFR) of heterogeneous traffic using computer simulation and given a range of saturation flow rate from 610 to 660 PCU/meter for width of approach 3.5 to 14 meter. They have established significant increase in SFR with increase in width of approach road. After careful investigation and study of the SFR values for varying width of approach, here following values of low SFR for calculating g/c ratio are adopted, which are slightly higher side than obtained in Vedagiri and Arasan (2007) paper. The calculation carried out by adopting values is shown in Table 4.4.

Table 4.4: SFR values adopted for width of approach.

Width of approach (*w*) (m)	"Low" Saturation Flow Rate (pcu/hr/m)	Saturation Flow Rate (pcu/hr)	Saturation Flow Rate (pcu/sec)
3.5 (one lane)	***650***	***2275***	***0.632***
4.5	655	2947.5	0.819
5.5	660	3630	1.008
6.5	665	4322.5	1.201
7.5 (two lane)	***670***	***5025***	***1.396***
8.5	675	5737.5	1.594
9.5	680	6460	1.794
10.5 (three lane)	***685***	***7192.5***	***1.998***
11.5	690	7935	2.204
12.5	695	8687.5	2.413
13.5	700	9450	2.625
14.0(four lane)	***705***	***10222.5***	***2.840***

The values typified in bold letters is used as a "low" SFR for 3.5 meter (single lane), 7.0 meter (two lane), 10.5 meter (three lane) and 14.0 meter (four lane) width of approach for further calculation of g/c ratio. The numerator DFR values are used considering lean volume 0.1 pcu/sec to high volume 1.0 pcu/sec. Outliner values when g/c> 0.5 has been neglected to develop graph as the study considered four arm intersections and g/c> 0.5 for a single arm is rare occurrence. The graph prepared from data presented in fig 4.15.

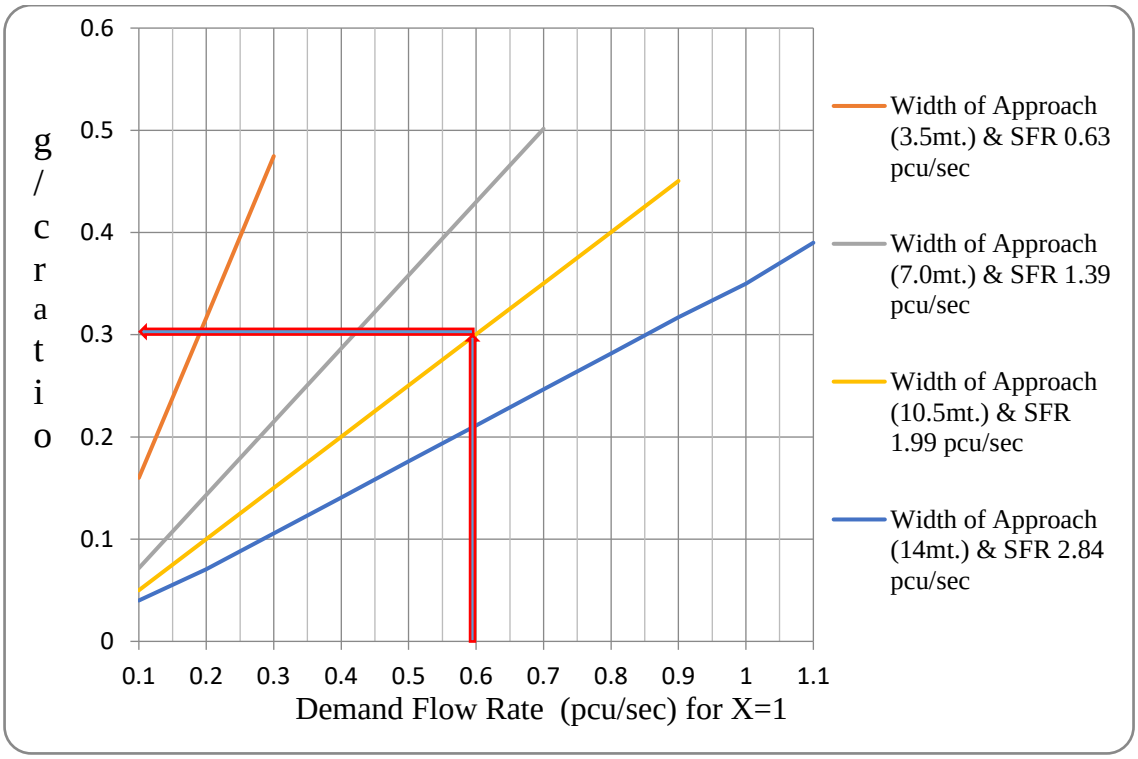


Figure 4.15: Relationship chart among variables (for "Low" SFR condition).

4.4.2 "High" SFR condition

Investigation of the past literature on SFR values for mixed traffic condition reveals wide range of SFR and value comes as high as 3600 pcu/hr/lane at some signalized intersections. As suggested in available literature Income Tax cross road in Ahmedabad city has maximum saturation flow rate observed in E-W direction is 3531 pcu/hr (Patel and Patel, 2012). After investigating Ahmedabad city traffic data, Gundaliya and Raval (2011) has given empirical formula to measure SFR as S= 628w + 268. Considering near saturated condition prevailing in the city, maximum SFR value 3600 pcu/hr/lane for 3.5-meter width of road is considered. Anticipating extreme SFR condition, "High" SFR for 3.5 m (single lane), 7.0 m (two lane), 10.5 m (three lane) and 14.0 m (four lane) width of approach is adopted as 1pcu/sec, 2 pcu/sec, 3 pcu/sec and 4 pcu/sec respectively for further calculation of g/c ratio. In this situation the DFR value is used between 0.1 pcu/sec to 2.8 pcu/sec for calculating g/c ratio. Outliner values when g/c> 0.5 has been neglected to develop graph as the study considered four arm intersections and g/c> 0.5 for a single arm is rare occurrence. The developed graph is shown in fig. 4.16.

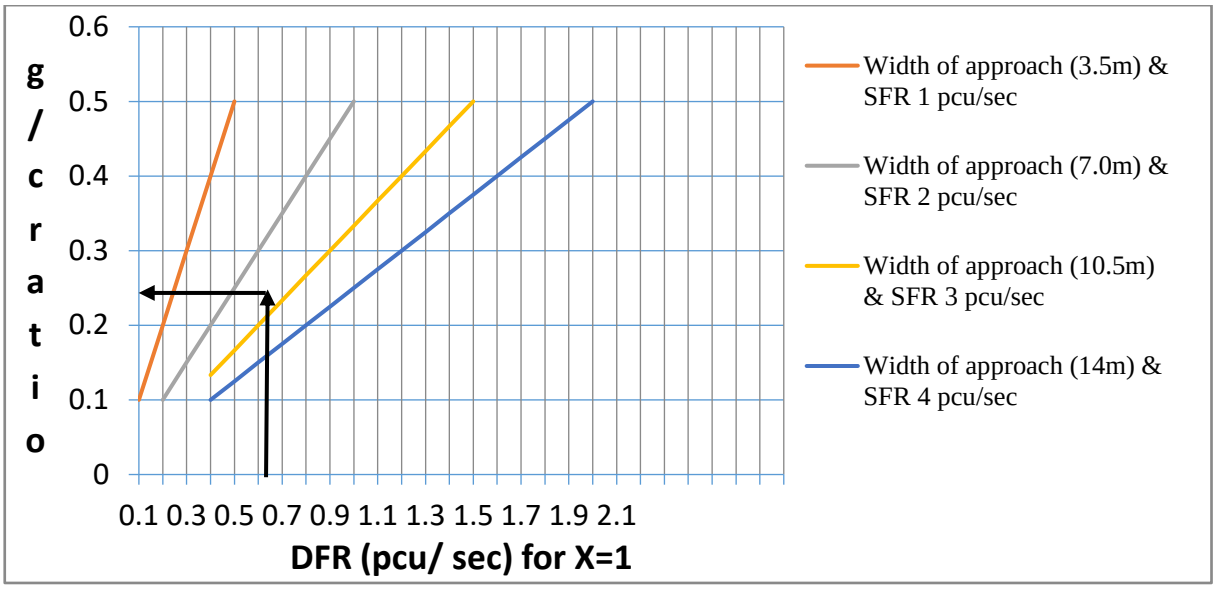


Figure 4.16: Relationship chart among variables (For "High" SFR Condition).

In signalized intersection, there are approaches having high demand compared to other approach because directional flow of each approach is different. The approach with high demand can be called as major street approach while the approach with low demand can be called as minor street approach. The requirement of the green time for major street approach and minor street approach is different and again governed by their DFR, SFR and width. For signal cycle design, target *v/c* (volume/capacity) ratios are generally in the range of 0.80 to 0.95. Very low values of *v/c* increase delays, due to underutilized green phase. Values of *v/c* > 0.95 indicate conditions in which frequent individual phase or cycle failures are possible, thereby increasing delay.

Consider a simple case of four arm intersection having cycle length of 120 second, high SFR 2 pcu/sec and width of approach 7.0 meter.

For equal demand on all four approaches,

$$g = 0.25c \qquad \text{..... (4.18)}$$

Now as per IRC recommendations consider g_{min} = 16 sec for two minor approaches.

∴ g available on main approach= 120-(16×2) = 88 sec.

∴ Major approach g/c = 44/120 = 0.37

Applying HCM equation of capacity, the capacity of major approach (C) is,

C = S× g/c of major approach

∴ C = 2× 0.37 = 0.74 pcu/sec

∴ DFR Satisfied by major approach at v/c= 0.8 is = 0.6 pcu/sec

∴ DFR Satisfied by major approach at v/c= 0.95 is = 0.71 pcu/sec

For the condition of effective signal coordination assuming lower limit of *v/c* as 0.8 and upper limit 0.95, lower boundary values and upper boundary values of DFR that can be satisfied in the condition stated above is described in the graph. The graphs (Figure 4.17, 4.18 & 4.19) have been developed for width of approach 7.0 meter (two lane), 10.5 meter (three lane) and 14 meter (four lane) for previously assumed "low" and "high" SFR values.

This graph can be used to calculate signal cycle length for any four arm signalized intersection which have observed DFR values for both "low" and "high" SFR condition. It can be applied for varying width of approach considering minimum and maximum green time requirement of minor and major approach of intersection. The graphs are helpful to derive cycle time for known DFR values of the approaches for better two way coordination. It is to be noted that after 160 sec signal cycle the relationship among DFR curves and g/c curve becomes flat. This happens because DFR satisfaction is depends on available g/c ratio of particular approach.

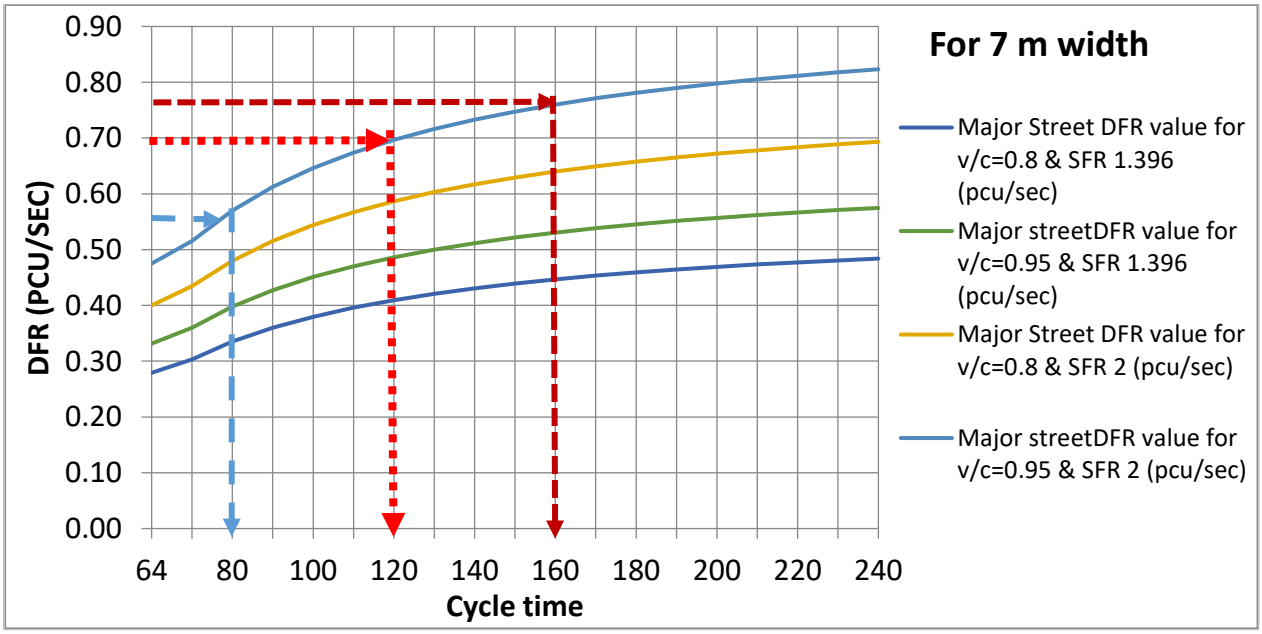


Figure 4.17: Cycle time (sec) from DFR (for coordination).

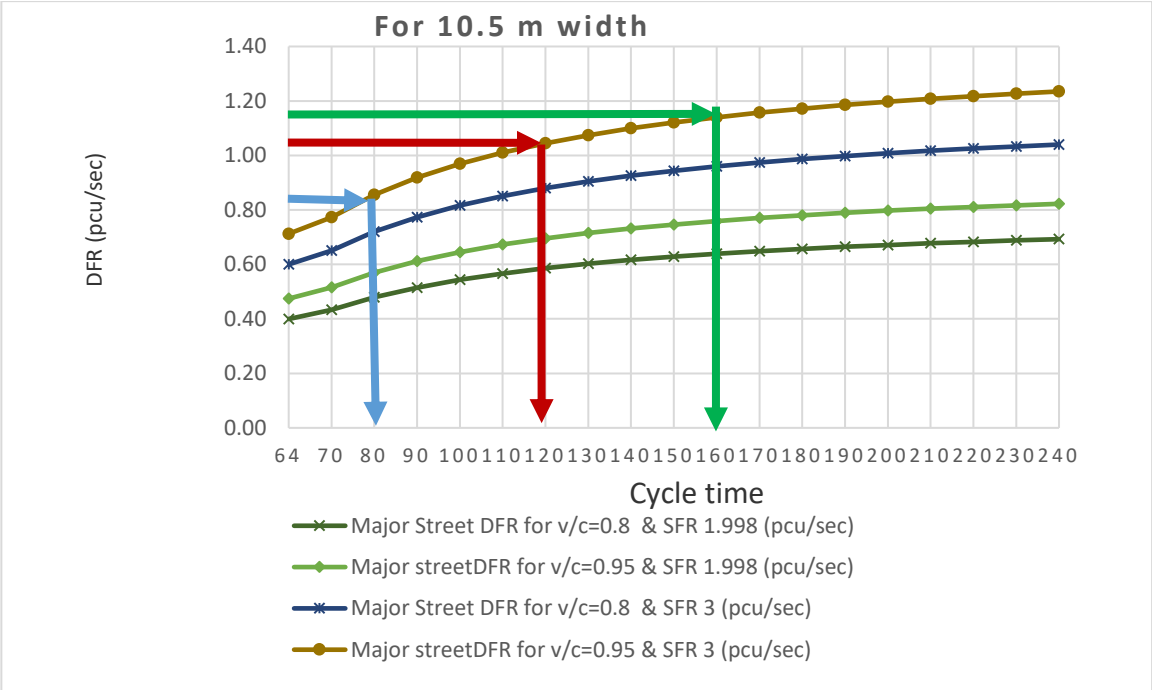


Figure 4.18: Cycle Time (sec) from DFR (for coordination)

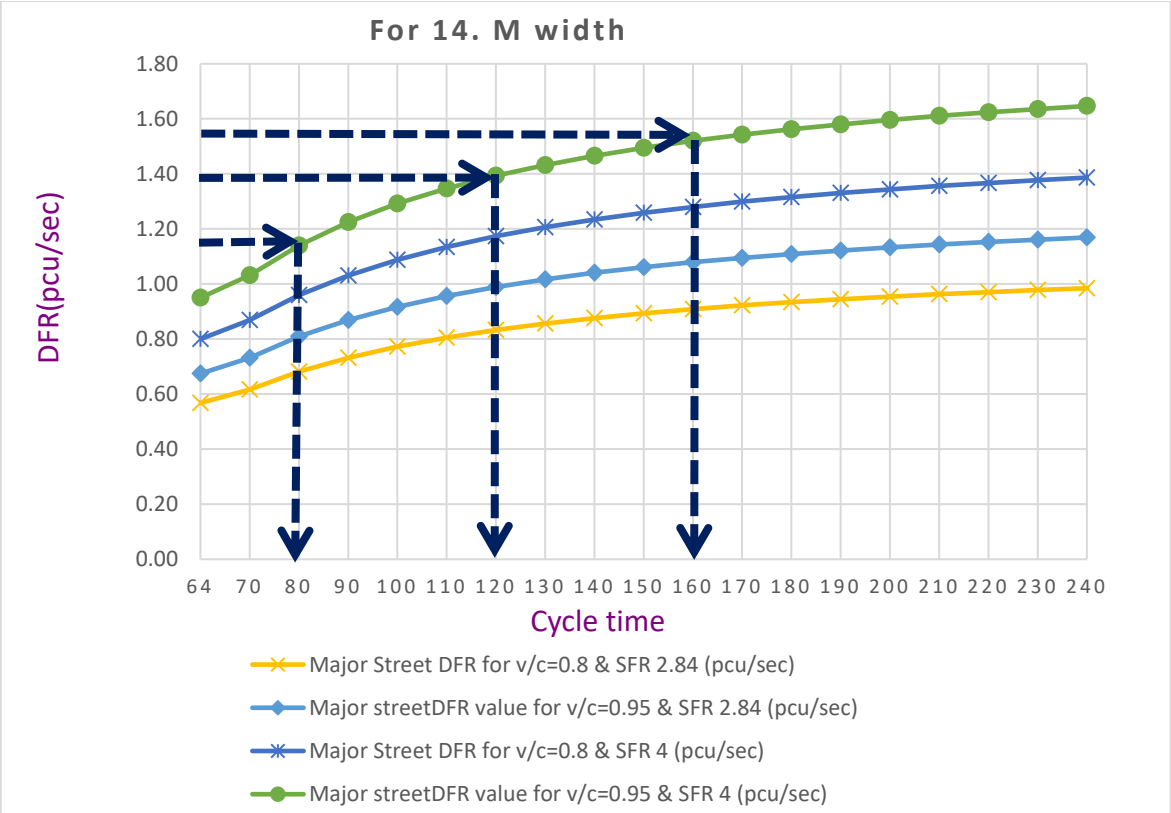


Figure 4.19: Cycle time (sec) from DFR (for coordination)

For the considered condition as shown in the graph, increase in major street g/c ratio for 80 s to 120 s cycles is 0.3 to 0.37 respectively while for 120 s to 160 s cycles increase in g/c ratio is 0.37 to 0.4. Again for 160 s to 210 s cycles the increase in g/c ratio is merely 0.4 to

0.42. It clearly implies that cycle length increment beyond 120 s would not proportionately increase the DFR satisfaction rate. Therefore, cycle length beyond 160 s is not advisable for signal coordination.

4.4.3 Validation of the method

The paper no. 685 presented on 18th December 2015 in the Conference of Transportation Research Group (CTRG 2015) includes validation of the developed TW_TSCS2. Vijay cross-road signalized intersection of Ahmedabad city is selected and data has been collected in the year 2015 for the purpose of measuring saturation flow rate. The selected intersection passes heavy traffic flow in the morning and evening peak hours. Long queues are found on the approaches during rush hours. Typical Indian traffic condition of mixed traffic without lane discipline is observed at the intersection. It has four approaches of varying width, University approach (9m), Darpan approach (10.2 m), Gurukul approach (8m) and Commerce college approach (7.2m). The intersection has pre-timed signal control system having signal cycle length of 155 second. The data for traffic flow was recorded and later analyzed.

The data for finding the saturation flow rate was collected using a videography and departures from stop lines are measured at every 3 second. By adopting Justo-Tuladhar (1984) PCU values, SFR and DFR of the approaches are calculated. This data has been used for validation of developed methodology. The results show that g/c ratio derived from the developed graph is nearer to the existing g/c ratio observed at selected intersection. This has been shown in table 4.5. For commerce college approach obtained g/c ratio (0.25) from figure 4.16 is shown on the graph for visualization.

Table 4.5: Comparison of g/c ratio of existing situation and developed graph

Approach name	Existing green time(sec)	Existing g/c ratio	Approach width (m)	Average Departure volume (pcu/hr/m) (q)	Observed maximum SFR (pcu/hr/m) (s)	q/s	g/c obtained by figure 3.16
University	36	0.23	9	372	637.33	0.58	0.21
Darpan	32	0.21	10.2	530	835.29	0.63	0.20
Gurukul	50	0.32	8	644	818.33	0.79	0.35
Commerce College	37	0.24	7.2	540	1109.86	0.49	0.24

4.5 Summary

The chapter covers methodology developed for two-way coordination which consider traditional practice adopted worldwide by traffic researchers for coordination. It elaborate developed methodologies i.e. TW_TSCS1 and TW_TSCS2 for two way coordination in typical Indian condition. Next chapter presents model for coordination by relaxing the assumptions made in this chapter.

CHAPTER: 5
Model Development

5. 1 General

Majority of the transport professionals adopt same travel speed in both directions for deriving strategy for signal coordination. The TW_TSCS1 narrated in previous chapter has been derived based on the assumption that average travel time in both (forward and backward) directions and phase time of all phases at intersection will remain equal. This implies that approach demand i.e. Demand Flow Rate (DFR) at all approaches is equal as well as travel speed of traffic stream in both forward and backward directions is same. In reality there is every possibility that DFR at various approaches of the intersection will vary significantly based on the several key factors of the traffic generation and mitigation. The average travel speed of the corridor in inbound and outbound direction is also governed by geometry of the road section, commercial activity, parking facility and width of road section among others. As per field observation on selected corridor the difference in DFR values of approaches was varying from 15% to 20% while speed variation in forward and backward direction of the selected link between intersections was varying between 10% and 15% (details given in next chapter). To overcome this difficulty in two-way coordination a simplistic model for deriving cycle time for effective two-way coordination has been developed. Apart from the identical situation considered in developing TW_TSCS1, this situation leads to two more cases for coordination.

1 When travel time in forward direction i.e. tt_{ij} and travel time in backward direction i.e. tt_{ji} is different but phase timings are same.

2. When travel time as well as phase time both are different.

5.2 Proposed Model

Let us consider the case when travel time in forward direction i.e. tt_{ij} and travel time in backward direction i.e. tt_{ji} are different.

Here, figure 5.1 shows optimum cycle for coordination with variable travel time.

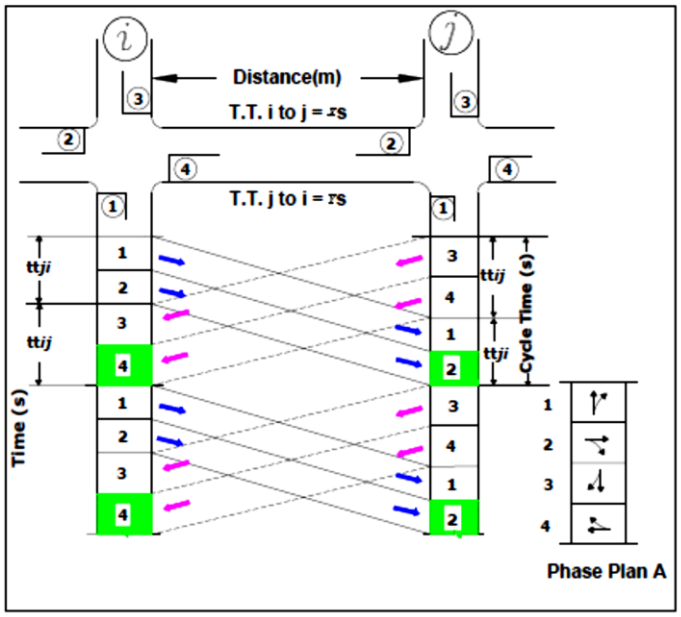


Figure 5.1: Optimum cycle for coordination with variable travel time

tt_{ij} = Travel time of traffic stream between intersection *i* to *j* (forward direction)

tt_{ji} = Travel time of traffic stream between intersection *j* to *i* (backward direction)

C_i = Cycle time for two-way coordination at intersection *i*

α_i, α_j = Start of green for phase at intersection *i* and *j*

βi, β_j = End of green for phase at intersection *i* and *j*

P_i, = Respective phase time at intersection *i*

P_j, = Respective phase time at intersection *j*

Here considering the figure and applying ideal offset concept used for coordination, starting time of phase 1 at intersection *j* would be,

$$\alpha_{1j} = \alpha_{1i} + tt_{ij} \qquad \text{..... (5.1)}$$

Similarly, starting time of phase 3 at intersection *i* would be

$$\alpha_{3i} = \alpha_{3j} + tt_{ji} \qquad \text{..... (5.2)}$$

Similarly, End time of phase 4 at intersection *i* would be,

$$\beta_{4i} = \beta_{4j} + tt_{ji} \qquad \text{.....(5.3)}$$

Considering initial offset of 0 second at both intersection, from equation (5.1),

$$\alpha_{1j} = tt_{ij} \qquad \text{..... (5.4)}$$

and from equation (5.2),

$$\alpha_{3i} = tt_{ji} \qquad \ldots\ldots (5.5)$$

Looking to the figure for proper two-way coordination between two four arm intersections in direction *i* to *j* if equation (5.4) is satisfied than first two phase movements (in this case P_{3j} and P_{4j}) at intersection *j* should be completed before the phase movement 1 starts at intersection *j*.

Here, as per phase plan first two phase movements at intersection *j* is P_{3j} andP_{4j}

$$\therefore\ P_{3j} + P_{4j} = \alpha_{1j} \qquad \ldots\ldots (5.6)$$

From equation (5.4),

$$\boxed{\therefore\ P_{3j} + P_{4j} = tt_{ij} \ldots\ldots\ldots (5.7)}$$

Similarly, as per the figure for proper two-way coordination between two four arm intersections in direction *j* to *i* if equation (5.5) is satisfied than first two phase movements (here P_{1i} and P_{2i}) should be completed before the phase movement P_{3i} starts at intersection *i*.

Here, as per phase plan first two phase movements at intersection *i* is P_{1i} and P_{2i}

$$\therefore\ P_{1i} + P_{2i} = \alpha_{3i} \qquad \ldots..(5.8)$$

From equation (5.5),

$$\boxed{\therefore\ P_{1i} + P_{2i} = tt_{ji} \quad \ldots..(5.9)}$$

Cycle time at intersection *i* will be,

$$C_i = P_{1i} + P_{2i} + P_{3i} + P_{4i} \qquad \ldots..(5.10)$$

Placing value of equation (5.9) in equation (5.10)

$$C_i = tt_{ji} + P_{3i} + P_{4i} \qquad \ldots..(5.11)$$

Now, for successful two-way coordination, end time of last phase 4 at intersection *i*

$$\beta_{4i} = tt_{ji} + P_{3i} + P_{4i} \qquad \ldots..(5.12)$$

Considering initial 0 second offset, from the equation (5.3), (5.6) & (5.7) phase completion time of phase 4 at intersection *j*

$$\beta_{4j} = \alpha_{1j} = tt_{ij} \qquad \text{.....(5.13)}$$

For effective two-way coordination, applying value of equation (5.13) in to equation (5.3) the phase completion time of last phase 4 at intersection *i*

$$\therefore \beta_{4i} = \alpha_{1j} + tt_{ji} = tt_{ij} + tt_{ji} \qquad \text{.....(5.14)}$$

∴ Cycle time for two-way coordination of 4 arm intersection with 4 phases is,

$$\boxed{\therefore\ C_i = tt_{ij} + tt_{ji} \ \text{..... (5.15)}}$$

This relationship will hold good for any four arm intersection having two phase difference (when, $tt \geq 2\ g_{min}$). In other words, this model will work when travel time between any two links is ≥ 32 second with two phase offset. Coordination between adjacent signals is advisable when distance between intersections is 300 m to 800 m (U.S. DOT-MUTCD 2012 and FHWA 2009). Signal optimization software synchro also uses this criteria of distance to calculate coordinatability factor. For odd phase difference condition stated in equation 4.6 chapter 4, i.e. $Pl = tt$ (when, $g_{min} \leq tt < 2\ g_{min}$) is to be satisfied the travel time should be < 32 second. To cover minimum 300 m distance in less than 32 second required speed of vehicle should be >36 km/h. Traffic Index published by "Numbeo" site for 2016 suggest average commuting speed of Indian cities is 23 km/h. With the lowest speed for six big mega cities of India ranging from 13 to 17 km/h. In Indian heterogeneous traffic condition having traffic congestion in urban area it seems impracticable for traffic stream to achieve speed >36 km/h between two signalized intersections. Therefore, for signal coordination possibility of having odd phase difference is minimal.

As speed and travel time are inversely proportional, increase in traffic stream speed will reduce the travel time and vice versa. For successful coordination between intersections along corridor using developed model, one has to be cautious while adopting the mean stream speed in forward and backward direction. As it is clearly visible from the figure 5.1 that in actual field condition due to some reason, if speed of the traffic platoon is reduced than increase in travel time in both directions will have monumental impact on delay as straight through traffic has to wait for whole cycle to clear the intersection. On the other hand, incase speed of the traffic platoon increases on the field because of any reason

whatsoever than reduction in travel time will not have much effect on the delay. As stated the bandwidth impact of "underestimating" the platoon speed is not as severe as the consequences of "overestimating" the platoon speed (Roess et al., 2010). Considering this it is advisable to adopt lower speed to calculate travel time while designing the required cycle length for coordination in developed model. Therefore, while selecting tt_{ij} (travel time in forward direction) and tt_{ji} (travel time in backward direction) for common cycle time calculation of the selected corridor having more than two intersections it is advisable to select maximum or average value of tt_{ij} and tt_{ji} among the intersections for deriving cycle time C_i. This derived model presents following rule for successful two-way coordination of four arm intersection even when the travel time in forward and backward direction is different.

Rule 1: $P_1 + P_2 = tt_{ji}$

Rule 2: $P_3 + P_4 = tt_{ij}$

Rule 3: $C_i = tt_{ij} + tt_{ji}$

5.2.1 Model generalization

It is attempted to generalize the developed model, so that it can be used to decide the required cycle length for proper two-way coordination. Now, let consider travel time tt_{ij} and tt_{ji} will create angle θ_1 and θ_2 depending on the speed of the traffic stream.

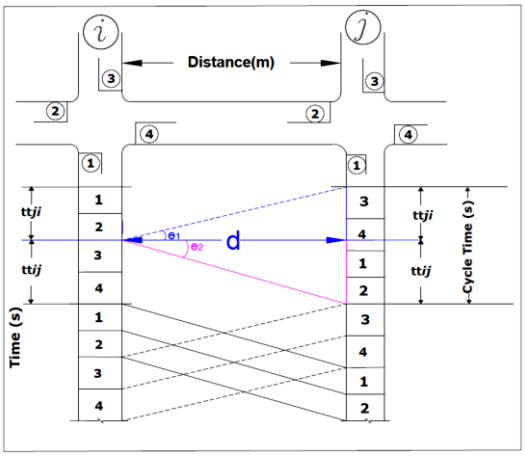


Figure 5.2: Optimum cycle for coordination with varying SMS

If speed is more than angle is less and vice versa. The distance at the intersection facility will remain same. Figure 5.2 shows this feature for varying SMS (Space Mean Speed).

Here from the figure 5.2, applying trigonometric function

$$\tan\theta_1 = \frac{tt_{ji}}{d} \qquad \ldots\ldots (5.16)$$

$$\& \ \tan\theta_2 = \frac{tt_{ij}}{d} \qquad \ldots\ldots (5.17)$$

Where,

d= distance between two intersections *i* and *j*

tt_{ji}=Travel time between intersection *j* to *i*

tt_{ij}= Travel time between intersection *i* to *j*

θ_1= Angle between band width and link direction (*j* to *i*)

θ_2= Angle between band width and link direction (*i* to *j*)

$\therefore\ d\ \tan\theta_1 + d\ \tan\theta_2 = n.c$

$\therefore\ \tan\theta_1 + \tan\theta_2 = \frac{n}{d}.c$

$$\boxed{\therefore\ c = \frac{d}{n}(\tan\theta_1 + \tan\theta_2) \quad \ldots\ldots\ldots (5.18)}$$

Where,

n = 1, 2, 3 … integer value

All other variables are previously defined.

As per equation (5.18) $c = \frac{d}{n}(\tan\theta_1 + \tan\theta_2)$, using Microsoft Excel, two graphs are developed to calculate cycle time. Considering SMS 10kmph to 50kmph and distance between intersections from 100m to 1000m, graphs have been developed (Fig 5.3 and 5.4).

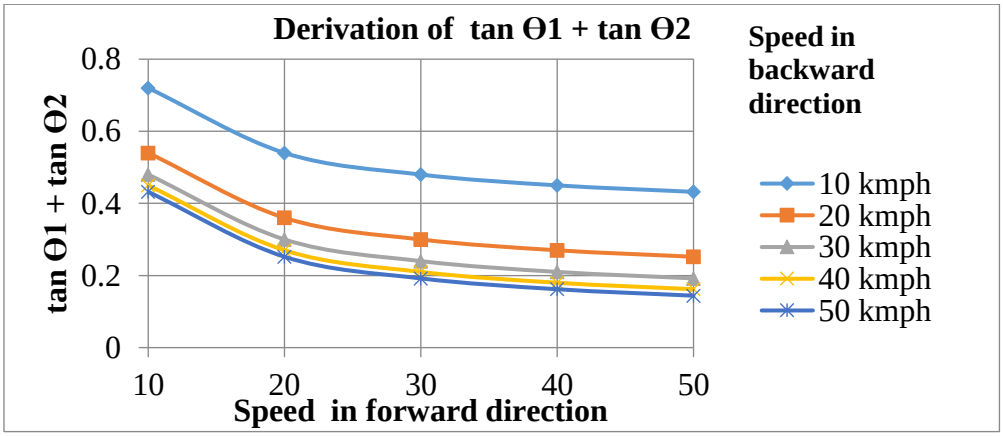


Figure 5.3: Derivation of $\tan\theta_1 + \tan\theta_2$ from SMS

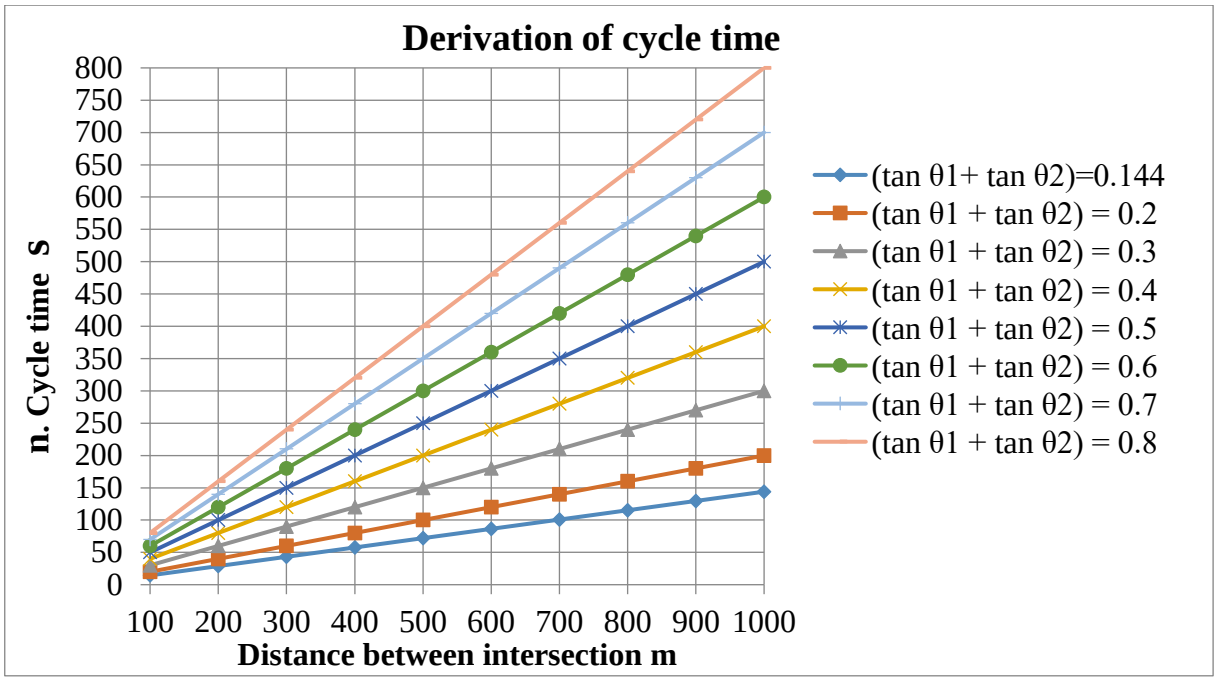


Figure 5.4: Derivation of optimum cycle time for distance

These graphs are handy tools to select the optimum cycle length necessary for two- way coordination. The integer *n* in equation 5.18 reveals that the developed graphs are useful also when the sum of travel time in forward and backward direction is greater than the feasible cycle length. When the speed of traffic stream is less and length is more between two signalized intersections in downstream or upstream or both direction the sum of tt_{ji} and tt_{ij} will be more than the feasible cycle length. Two-way coordination by developed methodology will be possible in this condition by providing double or triple cycle instead of one cycle. It is elaborated in the following figure 5.5

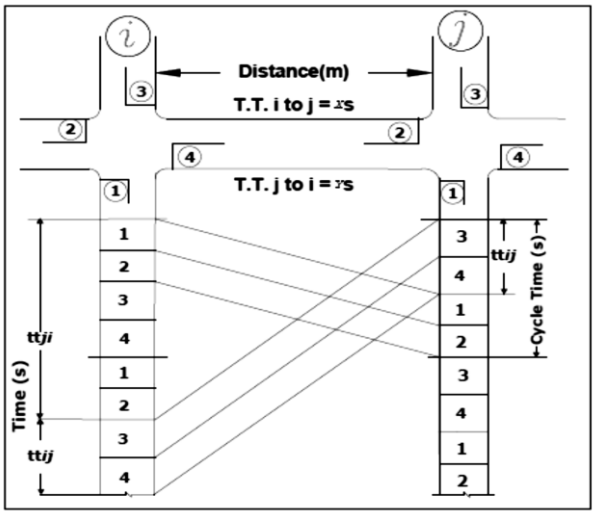


Figure 5.5: Coordination condition for 2.C and variable travel time

The above figure demonstrate the condition when travel time in forward direction (tt_{ij}) is less and travel time in backward direction(tt_{ji}) is more in such condition if we provide double cycle (n=2) where 2.C = tt_{ij}+tt_{ji} than for this condition also two-way coordination is possible.

5.3 Analytical validation of developed model

Validation of the model is performed using hypothetical data with two cases.

5.3.1 Case- I

When the phase lengths of two phases i.e. P1 and P2 from i to j and two phases i.e. P3 and P4 from j to i are equal (i.e. P_{1i}=P_{2i} and P_{3j}=P_{4j}). The figure 5.6 is shown with the different

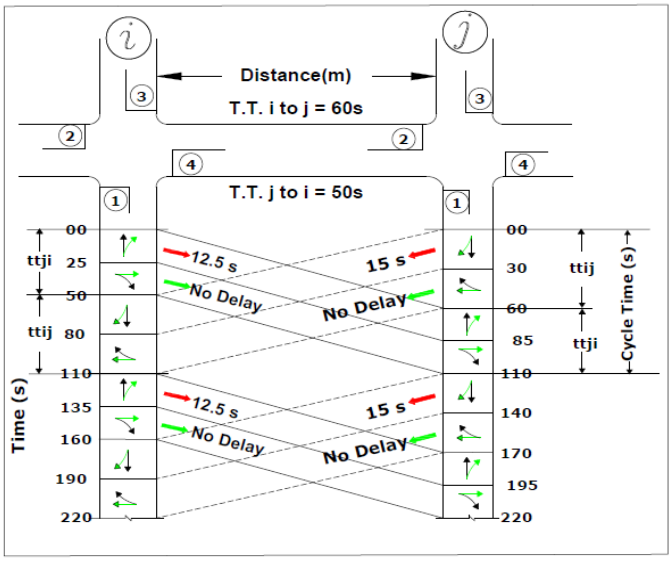


Figure 5.6: Validation of developed model (Case –I)

travel time in both forward and backward directions. In this case as depicted in the figure 4.5 tt_{ij} = 60 second while tt_{ji} = 50 second. Now for four arm intersection with four phases at both intersections, cycle time for two-way coordination can be obtained by applying rule 3.

$C_i = tt_{ij} + tt_{ji}$

$C_i = 60 + 50$

∴ C_i = 110 second.

Here P_{1i}, P_{2i}, P_{3i} and P_{4i} which is the respective phase length at intersection i

Similarly, P_{1j}, P_{2j}, P_{3j} and P4j are the corresponding phase lengths at intersection *j*

Applying rule 1

tt_{ji} = $P_1 + P_2$ at intersection *i* and *j*

Phase length of $P_1 + P_2$ = 50 second at both intersections.

Applying rule 2

tt_{ij} = $P_3 + P_4$

Phase length of $P_3 + P_4$ = 60 second at both intersections.

Here, average delay can be calculated as,

1. **Right turner's delay**

Phase time of right turner movement from intersection *i* to *j* = 25 second

∴ Average phase time =25/2= 12.5 second

∴Total Right Turner delay= Average stopped delay + delay up to getting green phase
= 25/2 + 0
= 12.5 + 0
= 12.5 second

5.3.2 Case- II

When the phase length of two phases i.e. P1 and P2 from *i* to *j* and two phases i.e. P3 and P4 from *j* to *i* are unequal having minimum green for right turning movement. Above case I is based on the assumption that the pair of phase at intersection *i* and intersection *j* have equal demand as the phase lengths given to right turner and straight movers are equal. There are all possibilities that directional flow ratio as well as DFR and SFR of each approach may differ which necessitate altering phase time of the pair of phase. It is interesting to understand that depending on the DFR and SFR of particular approach, phase time of the pair of phases (1, 2 & 3, 4) at intersection can vary depending on the minimum green criteria and travel time in both directions. Here extreme condition is considered by allocating only minimum green time as per IRC criteria of pedestrian requirement to the right turning movement of the both intersections. The graphical validation of the developed model for case II is carried out with time space diagram in AutoCAD software (Figure 5.7) and obtained result is presented here.

1 Right turners' delay

Phase time of right turners =16 second (Min green as per IRC)

∴ Average stopped delay time =16/2= 8 second

∴Total Right Turner delay= Average stopped delay + delay up to getting green phase

= 16/2 + 0

= 8 + 0

= 8 second

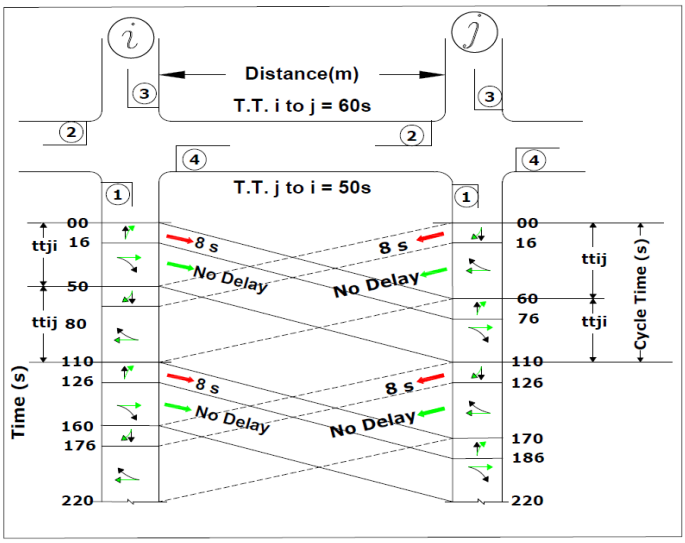


Figure 5.7: Validation of developed model (Case –II)

From above calculation it is to be noted that right turners will have some delay while straight moving traffic will receive through band of green wave having no delay.

5.4 Development of Algorithm for optimization

For optimization of the delay by developed methodology and model a programme is developed in C language. The programme also provides check to find out *coordinatability factor* based on adopted criteria. It optimizes the delay based on four iterations and gives the best suitable phase plan, phase sequence, phase length and cycle length required to be adopted for better two- way coordination having four arm intersections along the corridor.

Coordination Criteria:

1. Distance among two intersections = 300 to 800 m (As per FHWA and MUTCD 2012)
2. Mean Stream Speed = 10 to 50 km/h
3. Volume to Capacity (v/c) ratio from 0.8 to 1.0

The flow chart of the optimization programme is presented in figure 5.8.

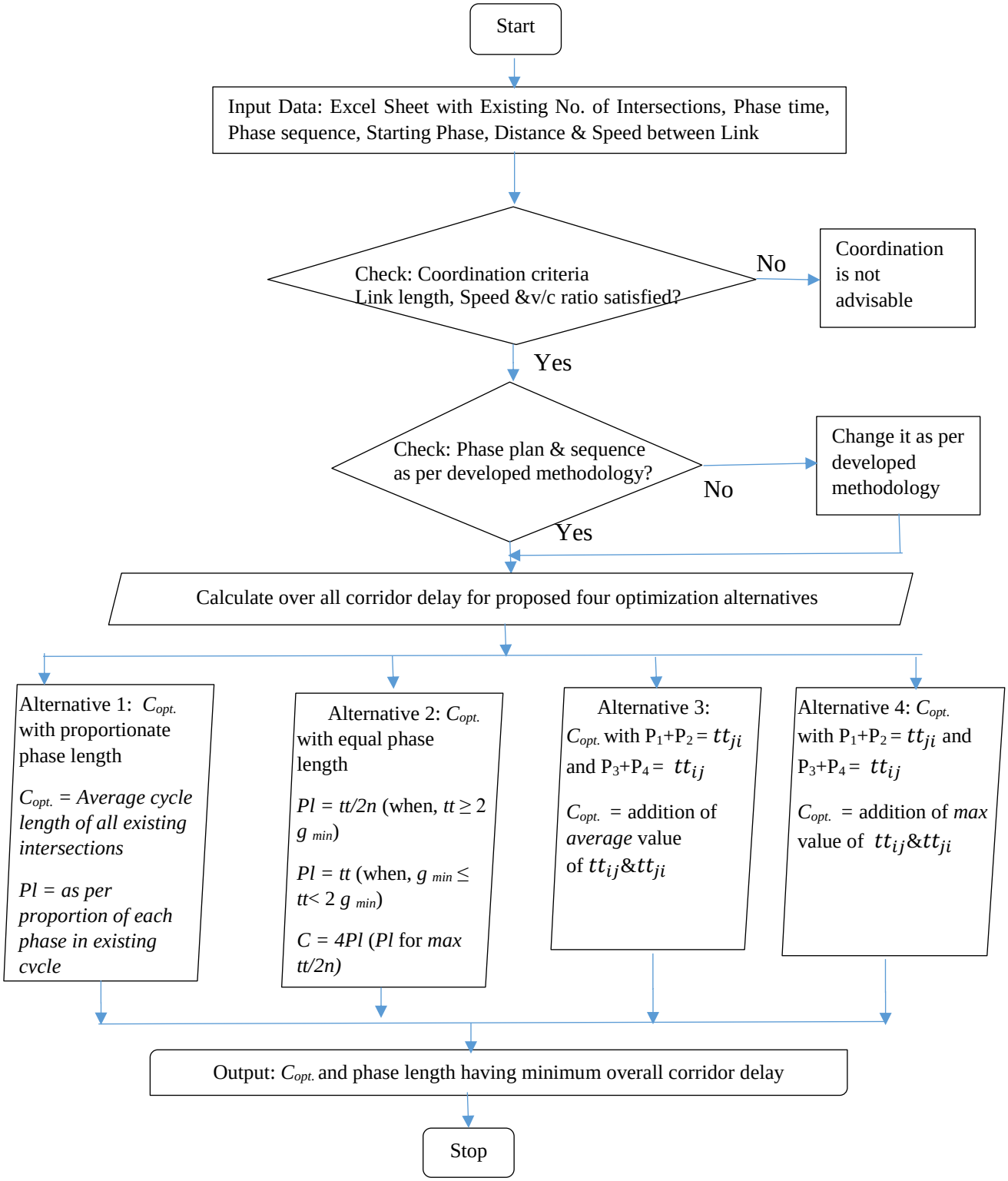


Figure 5.8: Flow Chart of the algorithm for optimization

5.5 Genetic Algorithm approach

Delay is perhaps the most important parameter in traffic signal optimization and estimation of level of service at signalized intersection approaches. Transport professionals across the world use delay as a primary criterion to evaluate the performance of signalized intersection. For example, delay minimization is frequently used as a primary optimization criterion when

determining the operating parameters of traffic signals at isolated and coordinated intersections. The Highway Capacity Manual (HCM) further uses the average control delay incurred by vehicles at intersection approaches as a base for determining the level of service provided by the traffic signals located at the downstream end of these approaches (TRB, 2010).

The popularity of delay as an optimization and evaluation criterion is attributed to its direct relation to what motorists experience while attempting to cross an intersection. However, delay is also a parameter that is not easily determined. Teply (1989) for instance, indicated that a perfect match between field-measured delay and analytical formulae could not be expected. The difficulty in estimating vehicle delay at signalized intersections is also demonstrated by the variety of delay models for signalized intersections that have been proposed over the years. Average delay and control delay is the same according to HCM for saturated signalized intersections.

Inspired by Darwin's theory of survival of fittest during evaluation the Genetic Algorithm (GA) is the iterative procedure that maintains a population of candidate solution to the objective function. Accordingly, GA is employed to check whether change in phase time can be useful to further reduce the overall corridor delay. To achieve the objective of the model to minimize overall total delay, constraint of minimum green time is applied as per IRC 93-1985 to the right turning (odd phase) movement of the both intersections.

Let, tt_{ij} = travel time in forward direction on given link

tt_{ji} = travel time in backward direction on given link

Now for four arm intersection with four phases at both intersections, cycle time for two-way coordination can be obtained by applying developed rule 3

$\therefore C_i = tt_{ij} + tt_{ji}$

Where, $tt_{ji} = P_1 + P_2$

$tt_{ij} = P_3 + P_4$

Let,

$tt_{ji} = P_1 + P_2 = 50$ s

$tt_{ij} = P_3 + P_4 = 60$ s

If applying minimum green constraint to the right turning movement of $P_{1\ and}$ P_3 =16 s then for corridor delay minimization in both directions having unequal travel time and variable phase length, phase P_2 may vary between 16 s to 34 s and phase P_4 may vary between 16 s to 44s. The other possibility that $P_{1\ and}$ P_3 may require extra amount of green time other than minimum green criteria to satisfy DFR of approach. Considering these to derive appropriate phase time and cycle time to accommodate existing DFR and minimizing the overall delay is an optimization problem which can be solved using different techniques. The condition leads to large decision space when combining all possible signal timings with four-phase signal plan. For example if we consider corridor of only three four arm intersections having four-phase signal plan with 200 seconds cycle time than as per IRC guideline minimum 16 seconds green is required at each phase. If we divide available 136 seconds to all phases than every phase have 18 different values (assuming minimum of 16 seconds and maximum of 34 seconds). The combinations of these solutions yields $(18\times18\times18\times18)^3$, or 1.15×10^{15}. Therefore, traditional optimization methods either do not find the optimal solution (since the objective function does not have a closed-form formulation), or they need an extraordinary amount of time to find an optimal solution. On the other hand, meta-heuristic approaches such as Genetic Algorithms (GA) could be used to effectively determine optimal or near-optimal signal timing parameters in a transportation network.

It is well known that solving the problem of finding optimal signal timings for a corridor, particularly in oversaturated conditions, is very challenging. This is the case because the signal timing at one intersection influences the state of other intersections, and also because no closed-form expressions are available for corridor delay and throughput based on signal timing parameters.

In the optimization problem, for the signal setting parameters, optimum cycle length, green time splits according to flows on the approaches, phasing and phase sequences, offsets between consecutive signalized intersections etc. are required to be set in such a way that delays due to signalized intersections in the corridor shall be minimum. To obtain the optimum cycle length and green splits, any of the available methods like, Webster's formula (1958), Australian Road Capacity Guide Method (ITE, 1982), Highway Capacity Manual method (2010) is being used generally. These are good enough for isolated intersections.

Whereas, for the congested network having number of signalized intersections, phase sequences, offsets and management of turning movements are also required to be considered. For this purpose, software like TRANSYT, SCOOT for static assignment and CONTRAM, DYNASMART for DTA can be used. In the proposed study genetic algorithm approach is used to select the optimum cycle, green splits and phase sequence simultaneously to solve optimization problem. The GAs are stochastic algorithms and can find close to optimal solution of the noisy, discontinuous or complex objective functions faster than the conventional optimization methods.

The selection of various parameters is discussed here after.

(1) Signal cycle times: Maximum and minimum cycle times are decided as per maximum and minimum travel time of all links in forward and backward direction.

(2) Green times: It is decided as per existing green time. Minimum green time as per IRC recommendation is assumed to be 16 s.

(3) Phase sequence number: Traffic movement in existing phase is kept as it is. Phase sequence of a given link is to be kept clockwise.

Only phase offsets are changed, by varying phase length of right turning phase in forward and backward direction starting from minimum 16 second up to convergence of function when delay is minimized.

Different GA parameters can be decided from the optimal solutions obtained for the simulation of existing traffic control parameters. The types of GA parameters like, uniform crossover, simple random mutation, roulette selection, seed value and crossover rate can be selected. The values of fitness functions for the different mutation rates are obtained and minimum values of fitness functions can be obtained for the particular mutation rate. These values are compared with the optimum values obtained from model 2 of existing signal settings. Average travel time of platoons and total travel cost can be reduced after adopting optimal signal settings.

Here,

G_{ip}^{min} = min. green time for phase *p* of intersection *i*

G_{ip} = green time for phase *p* of intersection i for the given time period *t*

g/c_{ip} = $\frac{g}{c}$ ratio for the phase *p* of intersection *i*

$Req.\, g/c_{ip}$ = required $\frac{g}{c}$ ratio for the phase *p* of intersection *i* to satisfy DFR of approach

P_{ik} = Number of phase *k* at intersection *i* (*k*=1, 2… n)

tt_{ij} = travel time from *i* to *j*

tt_{ji} = travel time from *j* to *i*

φ_{ij} = Delay at *j* to the right turner coming from *i*

φ_{ji} = Delay at *i* to the right turner coming from *j*

ψ_{ij} = Delay at *j* to the straight movers coming from *i*

ψ_{ji} = Delay at *i* to the straight movers coming from *j*

The general form of optimization model with GA would be,

$$\min Z(f) = \sum(\varphi_{ij} + \varphi_{ji} + \psi_{ij} + \psi_{ji}) \quad \text{….. (5.19)}$$

Subject to the constraints

$$C_{min} \geq \sum_{p \in Pn} G_{ip}^{min} \quad \text{……. (5.20)}$$

$$C_{\min} \geq \left\{tt_{ij}^{min} + tt_{ji}^{min}\right\} \quad \text{……. (5.21)}$$

$$C_{\max} \leq \left\{tt_{ij}^{max} + tt_{ji}^{max}\right\} \quad \text{…... (5.22)}$$

$$G_{ip} \geq G_{ip}^{min} \quad \text{……. (5.23)}$$

$$g/c_{ip} \geq Req.\, g/c_{ip} \quad \text{…….. (5.24)}$$

$$P_1 + P_2 = tt_{ji} \quad \text{…… (5.25)}$$

$$P_3 + P_4 = tt_{ij} \quad \text{…….(5.26)}$$

The objective function, (5.19) of this optimization formulation is to minimize the total average control delay. Constraint (5.20) ensures that the cycle time is more than the sum of minimum green time for all phases, constraint (5.21) puts bounds on the minimum cycle time and constraint (5.22) ensures that the cycle time should not be more than maximum values of travel time between any two links in outbound and inbound directions. Constraint (5.23) ensures that each phase time obtained for the cycle should follow minimum green requirement of the IRC. Constraint (5.24) will ensure that obtained g/c ratio is sufficient to satisfy DFR of particular approach. Constraint (5.25) and (5.26) puts check on the sum of the phase time.

The Lib GA software (version 1.00, developed at the Massachusetts Institute of Technology, USA) can be used for GA application. Green times and phase sequences of signalized intersections can be decided by GA optimizer to minimize the values of objective functions. The figure 5.9 explains overall methodology in flow chart.

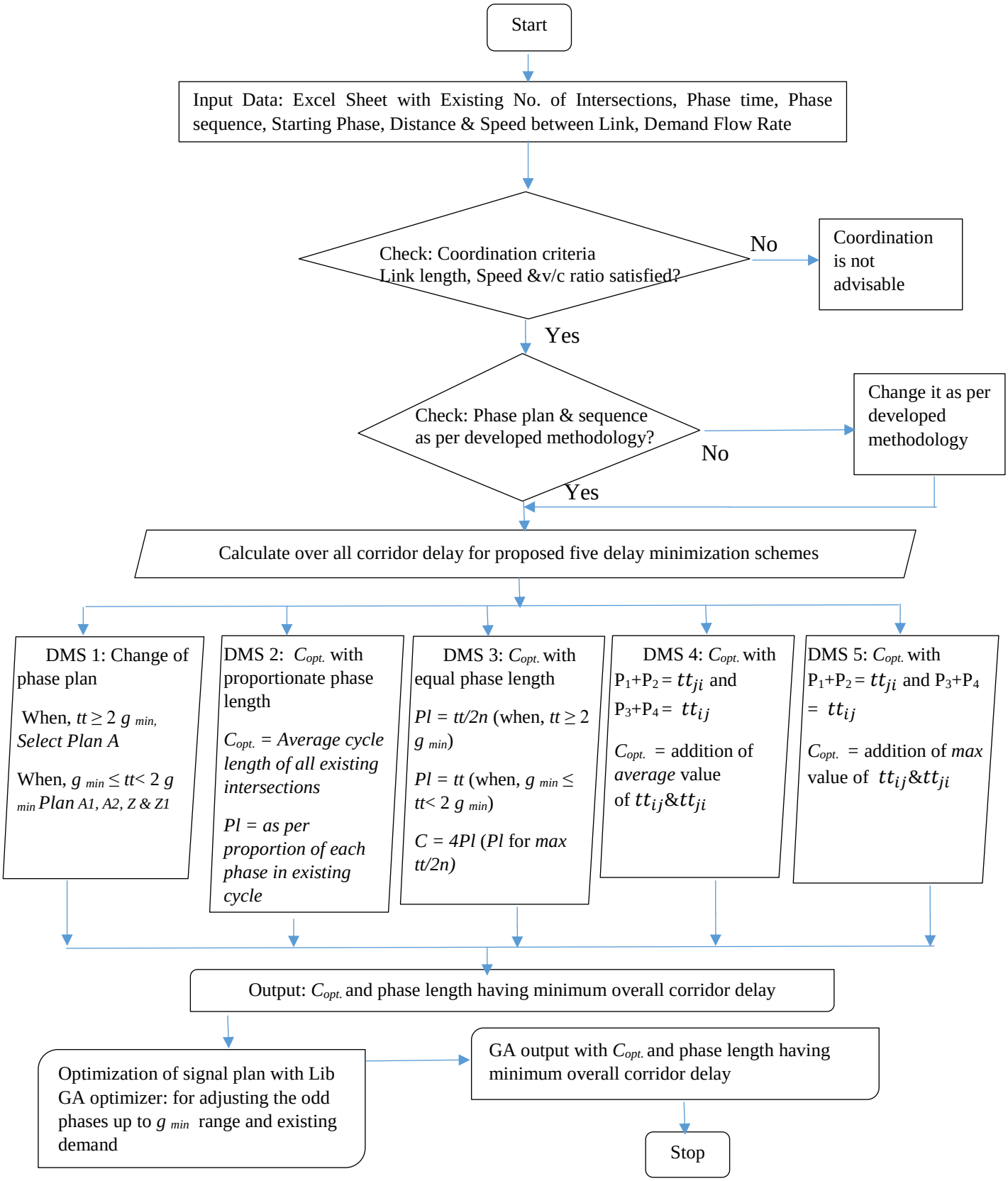


Figure 5.9: Flow chart of signal coordination computational model for minimum corridor delay

5.6 Summary

This chapter discusses computational model developed in this study for providing signal coordination along the urban road corridor. Following chapter is devoted to explain technique adopted in selecting study area to achieve the goal, data to be collected from selected site and its required analysis to get insight of various traffic control parameters.

CHAPTER: 6

Study Area, Data Collection and Analysis

6.1 General

In the previous chapter a computational model has been framed for the recent study of two-way traffic signal coordination on busy urban road corridor. The present study targets to develop the model for two-way traffic signal coordination in Indian urban traffic context. Selection of the appropriate road stretch from the urban road network is to be carried out carefully to meet the study objectives. Proposed field implementation of derived methodology requires collection of several data at different intersections in a peak hour period. This includes field surveys with respect to road geometry, traffic volume, speed of the vehicle, and composition of traffic to build necessary database along with requisite approval from the competent authority. A brief review of selected Ahmedabad city and its present traffic scenario with collected and analyzed data is discussed in the following paragraphs.

6.2 Ahmedabad city- An overview

Ahmadabad lies at 23.03°N, 72.58°E in western India at 53 meters (174 ft.) above sea level on the banks of the Sabarmati River, in north-central Gujarat. It covers an area of 464 km^2 (179 sq. mi). According to the Bureau of Indian Standards, the city falls under seismic zone 3, in a scale of 2 to 5. The city, known as Ashapalli or Ashaval in ancient times, was founded by King Karnadeva Vaghela as Karnavati in 11th Century as capital of his kingdom. Later on Sultan Ahmed Shah of Gujarat Sultanate shifted his capital from Patan to Karnavati and renamed it as Ahmedabad in 1411 AD. A number of monuments built during his era are spread over the old city area. The walled city was also built during this era and its 12 gates still exist though most of the wall can't be seen anymore. The city thrived as the capital of strong kingdom but later became part of the Moghul Sultanate in 1573. Shahjahan spent the prime of his life in this city and developed the present Shahi Baug area. The city was invaded by the Marathas in the year 1707 and ruled by them from 1753 AD to 1817 AD, when the city was taken over by the Britishers. The city is the administrative centre of Ahmedabad district and was the capital of Gujarat from 1960 to 4th June 1970; the capital was shifted to Gandhinagar thereafter. The High court and many central government offices still exist in

the city. In colloquial Gujarati, the city is commonly called as "Amdavad". Recently in July 2017 the city has been declared as 1st "Heritage city" of the country by UNESCO (United Nations Educational Scientific and Cultural Organisation).

6.3 Study area selection

The city is having circular and radial pattern road system having many signalized intersections. Study stretch requires pre timed signal with successive four arm intersections having no major approach road crossing main corridor between intersections. The other important feature to be considered for selection of site to achieve research objective are:

- Significant data sample available especially for saturated cycles
- No queue spillback from the downstream signals
- Feasibility of observations
- Pre-Timed fixed signal cycle with four arm junction
- Distance between junctions should be 300 m to 800 m.

After reconnaissance survey 805m long corridor on Chimanlal Girdharlal (C. G.) road in Ahmedabad city has been selected for the said purpose. Figure 6.1 shows selected study area with location.

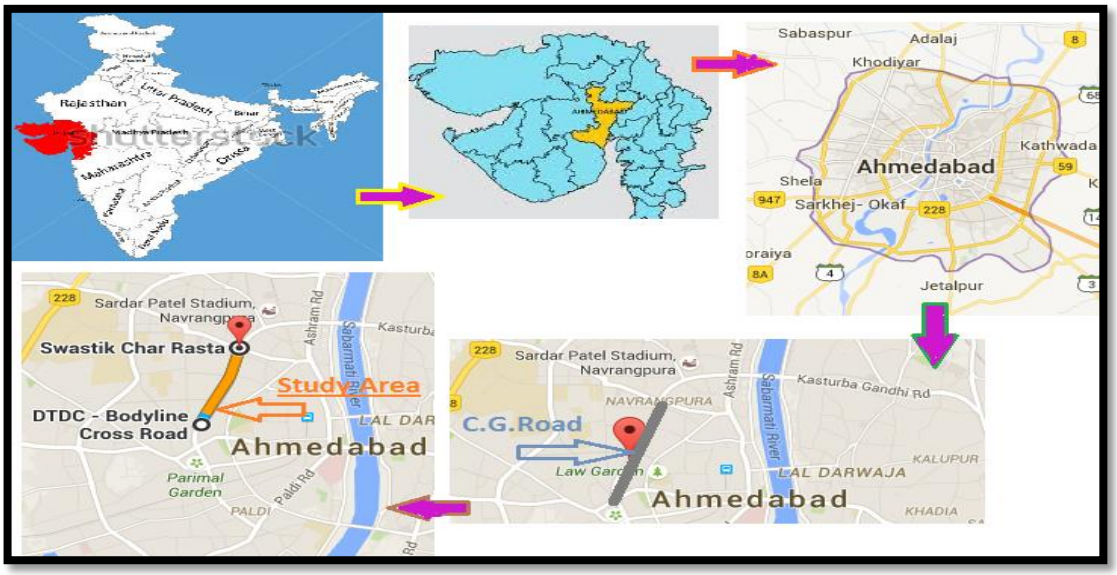


Figure 6.1: Location of study area

The C. G. Road of the 1st "Heritage City" of India is one of the most developed and well maintained roads with paid parking facility by Municipal Corporation. Recently the road

was proposed to be developed as a “Smart Road” under “Smart City” project of Ministry of Urban Development, Government of India. Ahmedabad is the biggest city of Gujarat and sixth largest city as well as seventh largest Metropolitan area of India as per population (Census-2011) having 270 signalized intersections spanning across the fourteen division of city.

6.4 Data collection and analysis

To arrive at a meaningful conclusion requisite data is indispensable. Looking to the fact physical inspection of the site was first carried out. Besides several visit of the corridor during the entire span of the research, videography data and manual data with trained enumerators for the complete stretch was collected four times on 27/09/2015, 08/02/2016, 06/07/2017 and 11/07/2017. In 2015 and 2016 raw data of the corridor was collected while in 2017 field implementation of the derived methodology was successfully executed with due permission of ACP Traffic- Admin Brach, Ahmedabad and Deputy City Engineer Traffic of AMC (explained in Chapter 8).

The whole link is continuous which interacts with three intersections respectively Swastik char Rasta, Girish cold drink and Swagat Intersection as shown in figure 6.2.

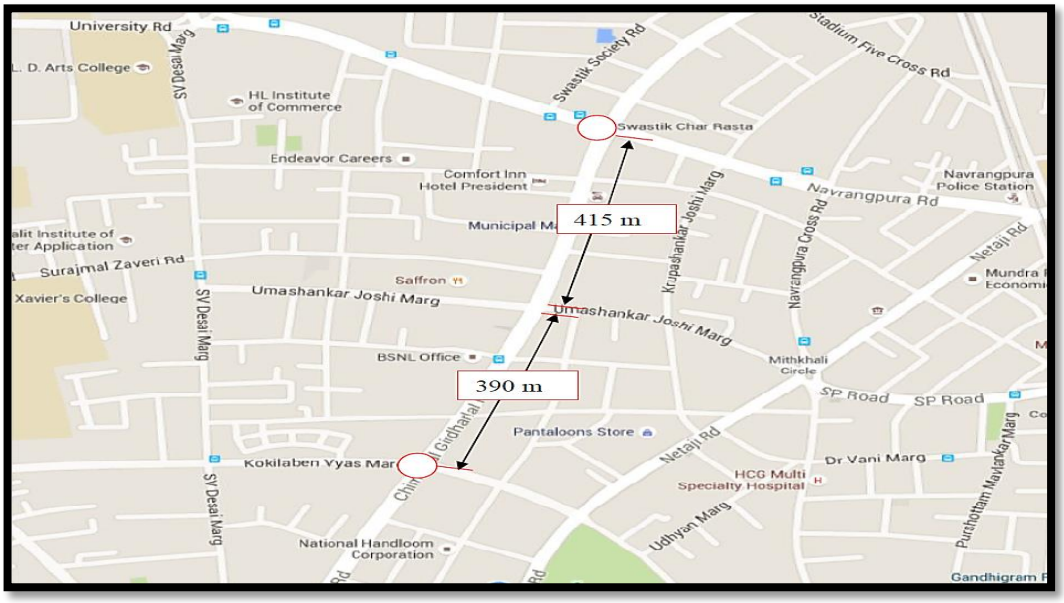


Figure 6.2: Study links between three continuous intersections named Swastik, Girish and Swagat (805 m)

Field survey of the whole stretch has been carried out during the research work to collect the road geometry, intersection geometry, road furniture, parking space inventory, details of

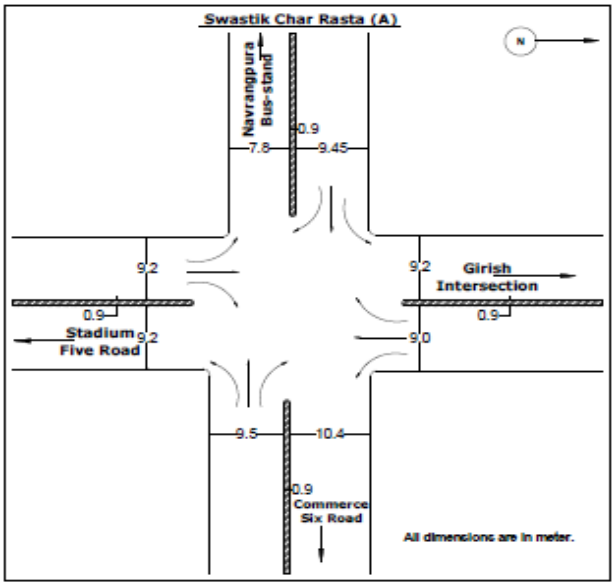
high rise building and its owner. Data of geometrical features were collected using Odometer (Figure 6.3a and 6.3b) and Google Earth Software.

The pilot survey was carried out for data collection by high resolution full HD video camera on 24/09/2015 for whole day to cover traffic fluctuation for morning peak, evening peak as well as off peak hours. For the cross authentication of the videography data, concurrently manual counts were also conducted for obtaining phase time, cycle time and spot speed data with the help of trained students. Collected data is subsequently analysed to extract necessary information's like actual travel time, space mean speed, traffic volume, Dynamic Passenger Car Unit (DPCU) values, signal cycle time, phase time, capacity of approach, saturation flow rate, demand flow rate, queue length, delay at intersection as well as acceleration and deceleration behaviour at signalized intersection by vehicle. After initial examination of the recording it has been kept as pilot survey and decided to repeat the whole process with utmost care and accuracy.

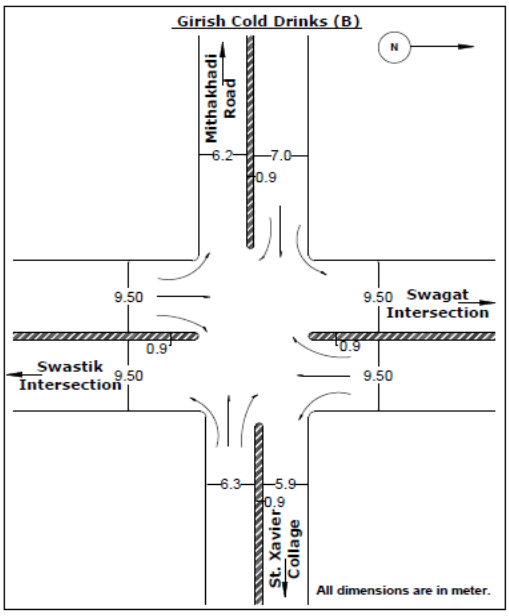
Figure 6.3a and 6.3b: Geometrical measurement of the link

Two multistoried buildings have been identified after investigation and field observations for the purpose of videography. After obtaining requisite permission from the competent authority, survey of the selected stretch was successfully conducted on 08/02/2016 (normal working day-Tuesday) for full day to capture traffic fluctuation for morning peak, evening peak as well as off peak hours with four ultra-high resolution full HD camera. Simultaneously, extra three low resolution cameras have been put up at three intersections on the nearby previously selected high rise buildings to record classified traffic volume, phase time, signal cycle time and vehicle travel time for measuring DPCU. For the cross verification of the videography data, concurrently manual counts for obtaining phase time,

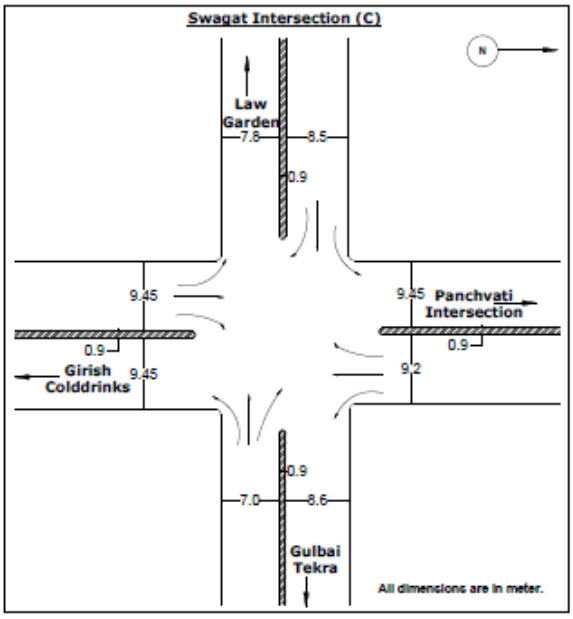
cycle time and spot speed data were also conducted with trained students. All the clocks of enumerators, videographers, students and video camera have been set with an accuracy of less than one second before starting the survey work. Collected geometrical detail with all measurements is shown in the following figure 6.4 a, 6.4b and 6.4c.



(a)



(b)



(c)

Figure 6.4a, b, & c: Geometrical details of selected three intersections

6.4.1 Signal control parameters at corridor

Collected data has been analyzed on 2m x 2m wide projector screen connected to desktop/laptop with latest windows 10 pro version operating system and updated VLC media player. Big screen was particularly selected to run multiple video at the same time with split screen facility. For the simplicity of the analysis of the three intersections Swastik Char Rasta, Girish Cold Drink intersection and Swagat intersection have been respectively coded by A, B and C intersection. Four lane divided roads in Ahmedabad city are selected as a study area. For coordinating the traffic signal, the stretches between two intersections are used for space mean speed (SMS) calculations which are mentioned below:

1. Stretch between Swastik char rasta and Girish cold drink intersection (415 m).
2. Stretch between Girish cold drink and Swagat intersection (390 m).

Details of the signal cycle, phase length. Phase sequence, SMS and travel time between the links are presented in the figure 6.5. The details of the individual phase length with phase group for morning peak and evening peak are shown in table 6.1 and 6.2.

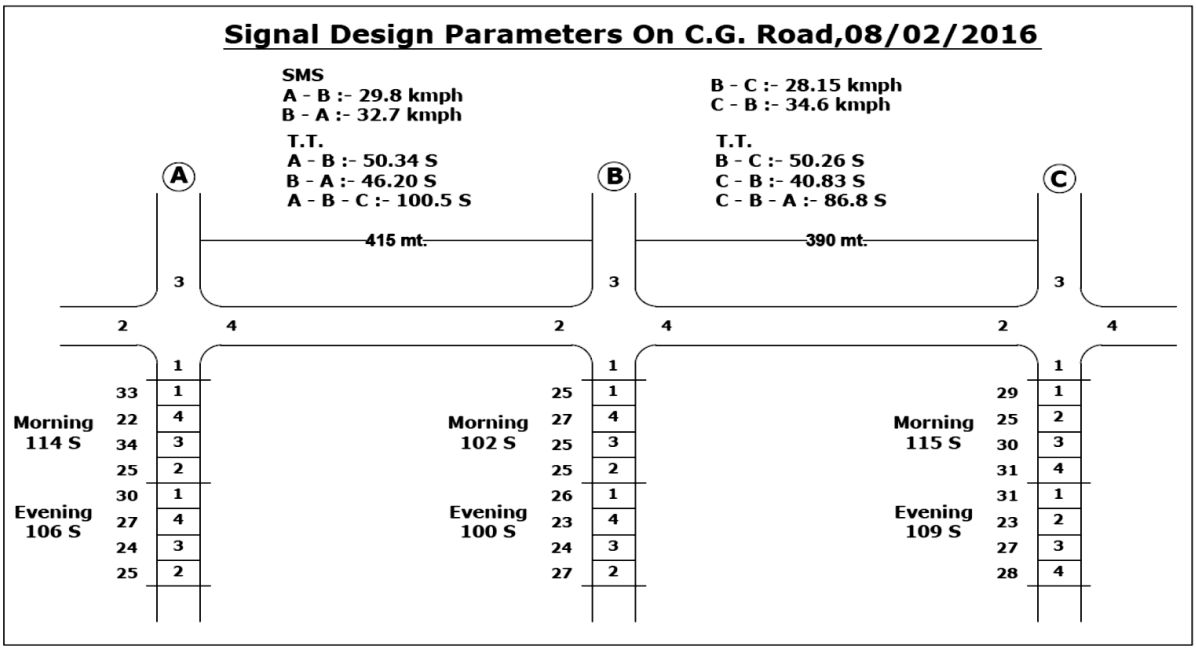


Figure 6.5: Signal control parameters at selected corridor

Table 6.1: Existing phase plans at C.G. road area of Ahmedabad City (Morning peak).

Swastik Intersection (A)			Girish Intersection (B)			Swagat Intersection (C)		
Intersection From	Green Time (s)	Phase Sequence	Intersection From	Green Time (s)	Phase Sequence	Intersection From	Green Time(s)	Phase Sequence
Commerce Six Road	33		St. Xavier college	25		Gulbai Tekra	29	
Girish Intersection	22		Swagat Intersection	27		Girish Intersection	25	
Navrangpura Bus Stand	34		Mithakhali Road	25		Law Garden	30	
Stadium Five Road	25		Swastik Intersection	25		Panchvati Intersection	31	

Table 6.2: Existing phase plans at C.G. road area of Ahmedabad City (Evening peak).

Swastik Intersection (A)			Girish Intersection (B)			Swagat Intersection (C)		
Intersection From	Green Time (s)	Phase Sequence	Intersection From	Green Time(s)	Phase Sequence	Intersection From	Green Time(s)	Phase Sequence
Commerce Six Road	30		St. Xavier college	26		Gulbai Tekra	31	
Girish Intersection	27		Swagat Intersection	23		Girish Intersection	23	
Navrangpura Bus Stand	24		Mithakhali Road	24		Law Garden	27	
Stadium Five Road	25		Swastik Intersection	27		Panchvati Intersection	28	

6.4.2 Speed measurements and analysis

Speed is the most important parameter defining the state of a given traffic stream. Speed is defined as the rate of motion, in distance per unit of time. In moving traffic stream, each vehicle travels at a different speed. Thus, the traffic stream does not have a single characteristic speed but rather distribution of individual vehicle speeds. From a distribution of a discrete vehicle speeds, a number of average or typical values may be used to characterize the traffic stream as a whole. Average or mean speeds can be computed in two different ways, Time Mean Speed (TMS) and Space Mean Speed (SMS), yielding two different values with differing physical significance.

Speed data of the all four sections A-B, B-A, B-C and C-B of selected corridor A-B-C was collected manually during data collection. For manual collection of the data, pavement marking method with observer was used. In this method, markings of pavement are placed across the road at each end of trap. Observer start and stops the watch as vehicle passes lines. In this method, minimum two observers required to collect the data, of which one is stand at the starting point to start and stop the stop watch and other one is stand at end point to give indication to stop the watch when vehicle passes the end line. Advantages of this method are that after the initial installation no set-up time is required, markings are easily renewed, and disadvantage of this is that substantial error can be introduced, and magnitude of error may change for substitute studies and this method is only applicable for low traffic conditions.

The spot speed can also be directly collected by tracking the vehicle movement on the big video screen. The vehicle speed collected through videography technique is found better than the collected from the field in the traditional method. The base line marking is created on the video screen such that it can identify the vehicle when it crosses the base line, as shown in figure 6.6.

Figure 6.6: Spot Speed measurement by videography

Generally, chi square test is carried out to check whether the Null hypothesis (H_0) is true or not. Chi- Square (So called "goodness- of- fit") test were performed on the spot speed data collected through videography technique and manual counting method. It is performed to check whether the observed speed data and estimated speed data for different categories of vehicles following normal distribution. Speed is the important variable which decides

operational quality of the signal coordination scheme. This data was analyzed and sample graph for two wheelers plying between sections A (Swastik) to Section B (Girish) for morning peak data is presented in following figures 6.7 and 6.8. Similar graphs of three categories of vehicles viz. two wheelers, three wheelers and cars for all four sections had been developed.

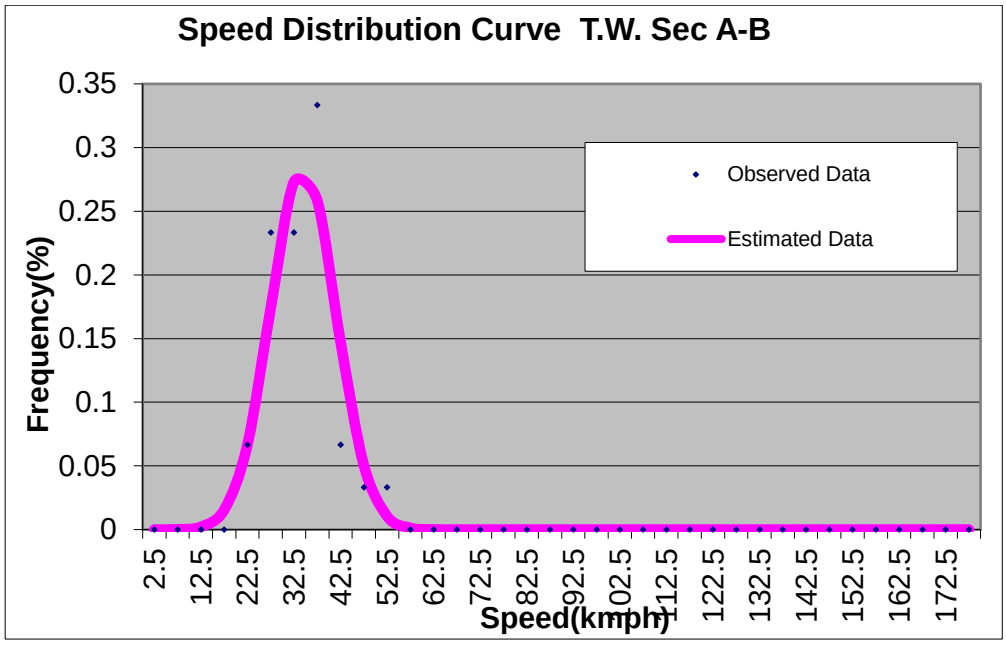


Figure 6.7: Speed data analysis by chi square test.

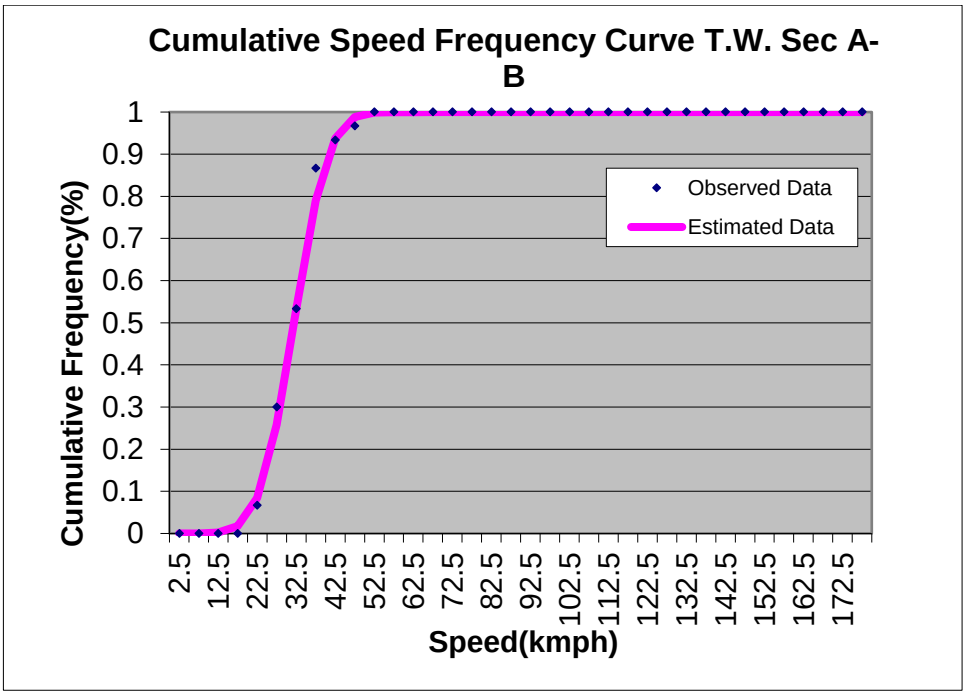


Figure 6.8: Speed data analysis by chi square test.

The test results obtained for the analysis of the two wheeler speed data for the noon peak period is displayed in table 6.3

Table 6.3: Chi square test result of A-B section for two wheelers

Chi Square (χ^2)Observed	3.211
Degree of freedom	2
Chi Square (χ_c^2) Critical (from table)	5.99

For 5% significance level i.e. 95% confidence level, Chi square (χ^2) (observed) < Chi square (χ_c^2) (Critical) at p< 0.05. It clearly demonstrates significant correlation between observed data and estimated data.

6.4.3 Classified traffic volume count survey

Classified traffic volume count survey is considered as prerequisite for most of the traffic studies. Vehicle classification counts are used in establishing structural and geometric design criteria, computing expected highway user revenue, and computing capacity. It gives volume of traffic passing through a point in a specified period from highway or lane. It also provides count of left turning and right turning vehicles at intersection.

Table 6.4: Total vehicular volume observed at the selected corridor (30 cycles)

Sr. No.	Intersection	Approach Name	Vehicle Category					
			2W	3W	Car	L.C.V.	Bus/Truck	N. M.
1	Swastik Intersection	Commerce Six Road	1945	405	581	13	24	32
		Girish Intersection	1874	397	673	41	4	11
		Navrangpura Bus Stand	1828	534	540	10	53	34
		Stadium Five Road	2047	363	525	4	6	57
2	Girish Intersection	St. Xavier college	1971	368	605	6	12	39
		Swagat Intersection	1953	451	574	9	5	18
		Mithakhali Road	2108	435	426	9	4	18
		Swastik Intersection	1946	502	506	10	4	32
3	Swagat Intersection	Gulbai Tekra	1908	331	753	0	5	3
		Girish Intersection	1803	569	601	17	4	6
		Law Garden	1952	463	574	9	0	2
		Panchvati Intersection	1877	433	674	7	9	0

Here, videography survey technique is used to collect this information. The video of the selected intersection and stretch has been replayed several times to decode the data. The data has been collected for 30 signal cycles of morning and evening peak to have fair and objective assessment of the traffic plying on the all three intersections. Table 6.4 presents category wise traffic volume at different intersections.

Analysis of the volume data reveals that there are more than 60% two wheelers, around 20% car and average 15% three wheelers present in the traffic mix of the selected stretch. All other categories of vehicles constitutes less than 5% volume. The pictorial representation of the volume via pie chart at selected three intersections is shown in figure 6.9a, 6.9b and 6.9c.

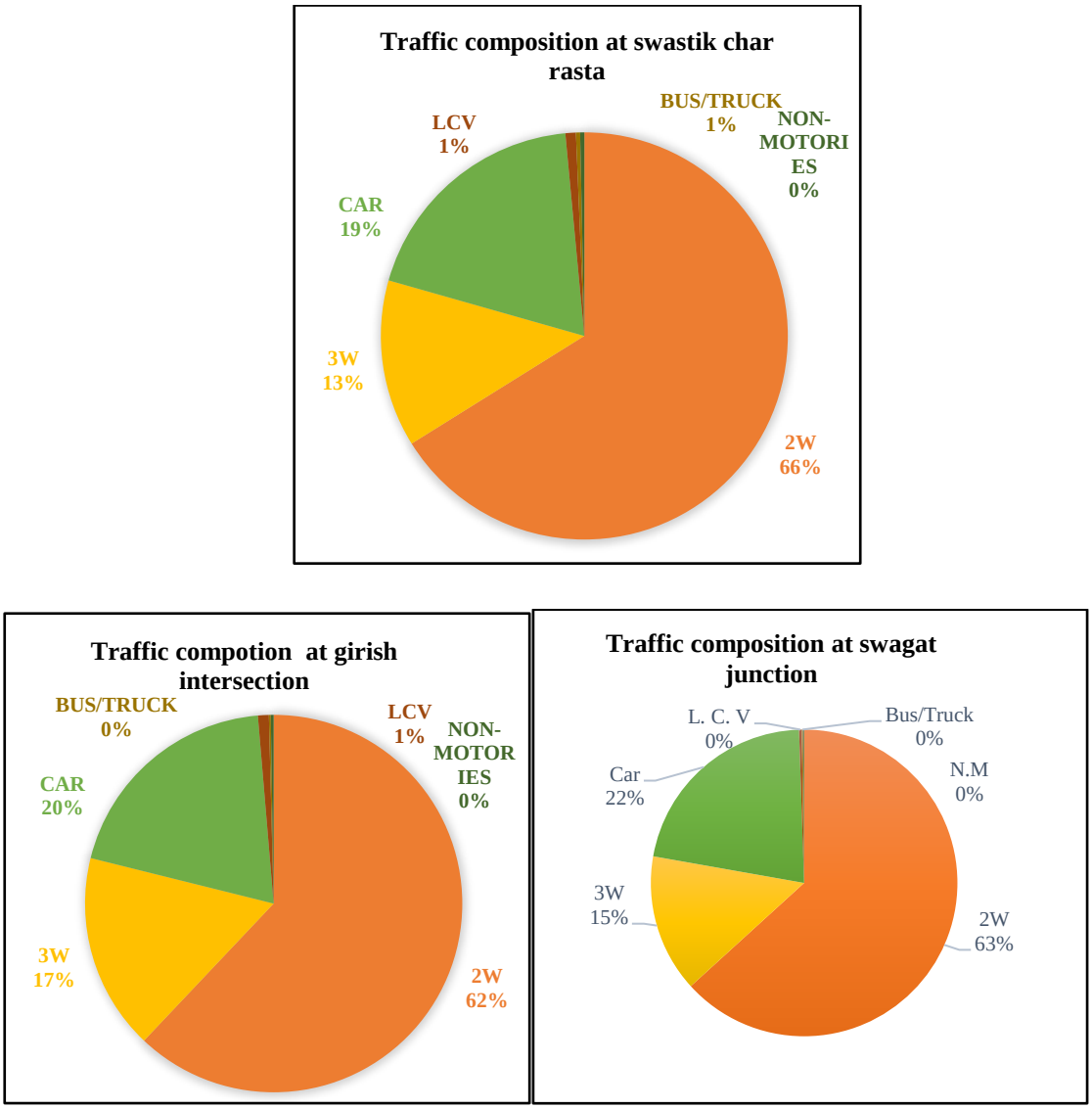


Figure 6.9a, b & c: Traffic composition at intersections of selected corridor

6.4.4 Derivation of dynamic PCU

Researchers who have worked in the area of heterogeneous traffic condition defined Passenger Car Unit (PCU) in different forms. PCU value depends on the factors such as vehicle characteristics, roadway characteristics, environmental conditions, climatic conditions, control conditions etc. As elaborated in literature review chapter 3 most of the research works have already been done to determine the PCU values of vehicles on midblock sections. Only a few literatures are there for determining the PCU values of vehicles at signalized intersection. In this research an attempt is made to develop Dynamic PCU (DPCU) values for selected three signalized intersections. For the purpose of collecting data specifically for DPCU the camera position was kept as shown in figure 6.10.

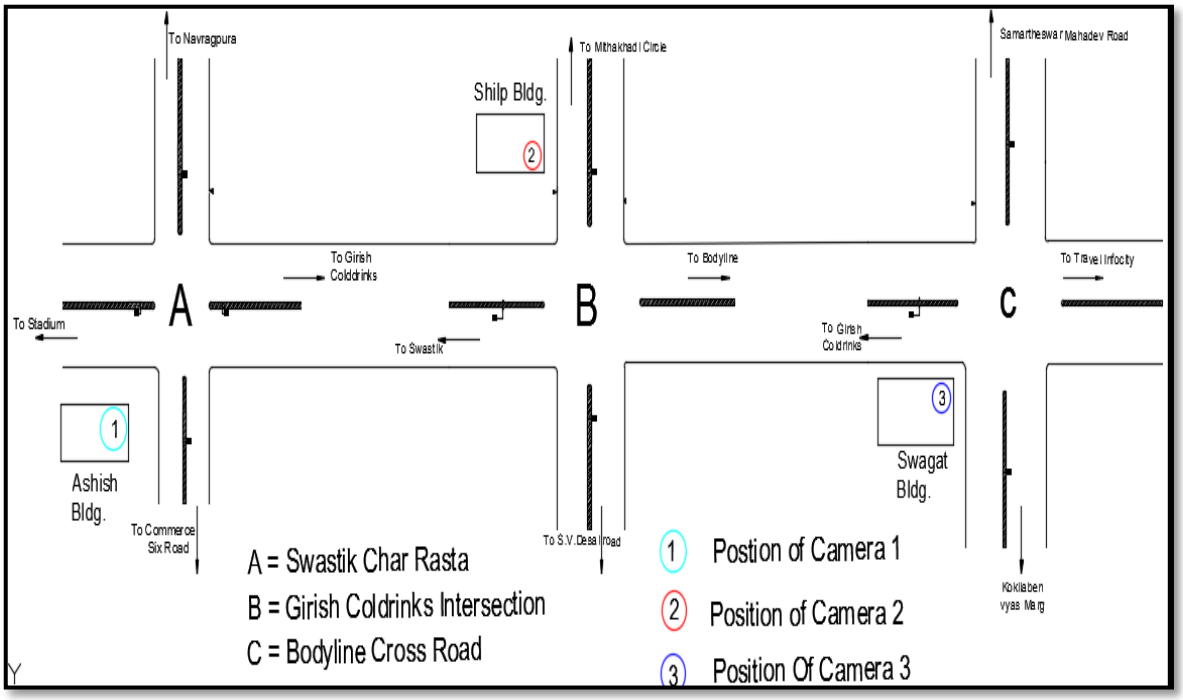


Figure 6.10: Camera position at study area (for DPCU)

Chandra and Kumar (2003) have developed a method for PCU values for different categories of vehicles at ten different sections of National Highway (NH) and State highway (SH). To estimate the PCU values, they have suggested that it is directly proportional to the ratio of clearing speed of vehicle, and inversely proportional to the space occupancy ratio of vehicle with respect to the standard area of vehicle, i.e. a car.

$$PCU = \frac{Vc}{Vi} \times \frac{Ai}{Ac} \qquad \text{..... (6.1)}$$

Where,

PCU = passenger car unit value of ith type vehicle

Vc / Vi = Speed ratio of the car to the ith vehicle

Ac / Ai = Space ratio of the car to the ith vehicle

Vc = speed of car (km/h)

Vi = speed of ith type vehicle (km/h)

Ac = projected rectangular area (length × width) on road of a car (m^2)

Ai = projected rectangular area (length × width) on road of ith type of vehicle (m^2)

To estimate the DPCU values for the signalized intersection in this study, travel time of vehicle to cross the intersection is considered instead of its speed. Because travel time is the most important functioning parameter which includes vehicle's deceleration, acceleration, turning/overtaking time etc. Also, it is easier to measure the total travel time taken by vehicle to cross the intersection. To derive the DPCU values in this study 30 cycles of the selected intersections for morning peak period were analyzed. The video was replayed several times to extract the desired information. Travel time taken to clear the intersection by each category of vehicles in each direction was extracted meticulously. DPCU is considered proportional to the ratio of travel time of vehicle with respect to the travel time of standard vehicle i.e. car and the space occupancy ratio of vehicle with respect to the standard area of vehicle, i.e. a car.

$$\text{Dynamic PCU} = \frac{\text{T.Ti}}{\text{T.Tc}} \times \frac{\text{Ai}}{\text{Ac}} \qquad \text{..... (6.2)}$$

Where,

PCU = passenger car unit value of ith type vehicle

T.Ti = travel time of ith vehicle (s)

T.Tc = travel time of car (s)

Ac = projected rectangular area (length × width) on road of a car (m^2)

Ai = projected rectangular area (length × width) on road of ith type of vehicle (m^2)

Applying this equation, DPCU values of all categories of vehicles with straight, right and left movement at all three intersections are derived. To derive the values, extracted travel time from data analysis and area table proposed in Chandra and Kumar (2003) paper was adopted. The DPCU values for all four approaches of the selected three intersections were derived and average DPCU values of the Swastik char Rasta intersection is presented in table

6.5. DPCU values derived for other two intersections with sample calculation sheet is given in the Appendix III.

Table 6.5: Average dynamic PCU values at swastik char rasta

Vehicle category	Direction	Approach			
		Stadium	Commerce Six Road	Girish Cold drinks	Navrangpura
2W	S	0.23	0.23	0.21	0.21
	R	0.25	0.22	0.21	0.19
	L	0.17	0.16	0.16	0.17
3W	S	0.72	0.65	0.62	0.6
	R	0.65	0.7	0.63	0.63
	L	0.4	0.89	0.62	0.53
Car	S	1	1	1	1
	R	1	1	1	1
	L	1	1	1	1
LCV	S	2.67	2.94	2.47	2.33
	R	-	3.39	2.27	-
	L	-	2.13	2.34	3.37
Bus/Truck	S	7.33	5.28	6.04	5.73
	R	-	-	-	-
	L	-	-	-	-
NM	S	0.26	0.22	0.25	0.2
	R	0.22	0.18	-	0.16
	L	-	0.37	-	-

In view of the limited research and standardized guidelines for the PCU values at signalized intersection having heterogeneous Left hand traffic with RHD, the developed PCU values are compared with the previously available standards of Static PCU (SPCU) values. The comparison is displayed in table 6.6. The table 6.7 present comparisons of developed DPCU values derived by the equation 6.2 and DPCU values derived by the equation 6.1. It is clearly observed that the DPCU values by developed method and that of Chandra and Kumar's (2003) method have nearly identical values.

Table 6.6: Comparison of derived DPCU values with available standards.

Vehicle Type	SPCU		DPCU values					
	As per IRC SP-41-1994	As per Justo & Tuladhar (1984)	Swastik Char Rasta Intersection		Girish Cold drinks Intersection		Swagat Intersection	
			Straight	Right	Straight	Right	Straight	Right
2W	0.5	0.3	0.22	0.22	0.24	0.23	0.24	0.24
3W	1.0	0.4	0.65	0.65	0.63	0.63	0.62	0.68
LCV	1.5	-	2.6	2.71	2.71	3.16	2.65	2.73
Bus/ Truck	3.0	2.8	6.1	-	4.9	5.75	4.8	6.78
NM	0.5	0.4	0.23	0.19	0.25	0.19	0.18	-

Table 6.7: Comparison of derived DPCU values with Chandra & Kumar's method

Vehicle Type	DPCU values											
	Swastik Char Rasta				Girish Cold drink				Swagat Junction			
	By Adopted Method		By Chandra's Method		By Adopted Method		By Chandra's Method		By Adopted Method		By Chandra's Method	
	S	R	S	R	S	R	S	R	S	R	S	R
2W	0.22	0.22	0.22	0.24	0.24	0.23	0.22	0.28	0.24	0.24	0.24	0.27
3W	0.65	0.65	0.62	0.59	0.63	0.63	0.51	0.59	0.62	0.68	0.66	0.89
LCV	2.6	2.71	3.25	3.39	2.71	3.16	2.96	2.81	2.65	2.73	2.52	2.34
Bus/ Truck	6.1	-	5.54	-	4.9	5.75	5.42	5.75	4.8	6.78	5.42	6.78
NM	0.23	0.19	0.19	0.20	0.25	0.19	0.28	0.16	0.18	-	0.20	-

Recent study of Sheela, A., Isaac, K.P. (2015) focuses on DPCU values on signalized intersection in Indian heterogeneous traffic condition and recommended variation of PCU values with flow ratio using the output obtained from the micro simulation model, TRAFFICSIM. They have worked on four legged signalized intersections of carriageway widths varying from 3.5 to 10.5 m on level stretches. This is also the basic assumptions adopted in this research to develop methodology. They have concluded that for nearly saturated condition having flow ratio 0.8 to 1.0 (required criteria for coordination) the variation in PCU values of two wheelers, three wheelers, car and bus is negligible. Accordingly, the obtained volume of 30 cycles presented in table 6.4 are converted into derived DPCU values and presented in the following table 6.8, 6.9 and 6.10.

Table 6.8: Existing approach volume (Intersection A- swastik intersection)

Approach Name	Existing Green Time	Approach Width (M)	Approach Volume (PCU/hr)	
			By Developed DPCU	By IRC SPCU
Commerce College	33	9.5	1460	2066
Girish Intersection	22	9	1479	2086
Navarangpura Bus Stand	34	9.45	1646	2179
Stadium Five Road	25	9.2	1271	1964

Table 6.9: Existing approach volume (Intersection B- girish cold drink)

Approach Name	Existing Green Time	Approach Width (M)	Approach Volume (PCU/hr)	
			By Developed DPCU	By IRC SPCU
St. Xavier college	25	8.3	1400	2023
Swagat Intersection	27	9.5	1384	2039
Mithakhali Road	25	8	1258	1939
Swastik Intersection	25	9.5	1347	2024

Table 6.10: Existing approach volume (Intersection C- swagat intersection)

Approach Name	Existing Green Time	Approach Width (M)	Approach Volume (PCU/hr)	
			By Developed DPCU	By IRC SPCU
Gulbai Tekra	29	8	1452	2054
Girish Intersection	25	9.2	1468	2112
Law Garden	30	8.5	1364	2027
Panchvati Intersection	31	9.45	1473	2083

6.4.5 Relationship among actual travel time and travel time by SMS

As discussed in the chapter 4, to calculate ideal offset for signal co-ordination, average speed of traffic stream shall be considered. Average speed of traffic stream is obtained by spot speed study of mid-block section. Spot speed study gives Space Mean Speed (SMS) which is considered for signal co-ordination. Travel time by space mean speed and actual travel time of the vehicle may be different. Figure 6.11 presents fundamental details of vehicle passing through signalized intersection and travel time of vehicle.

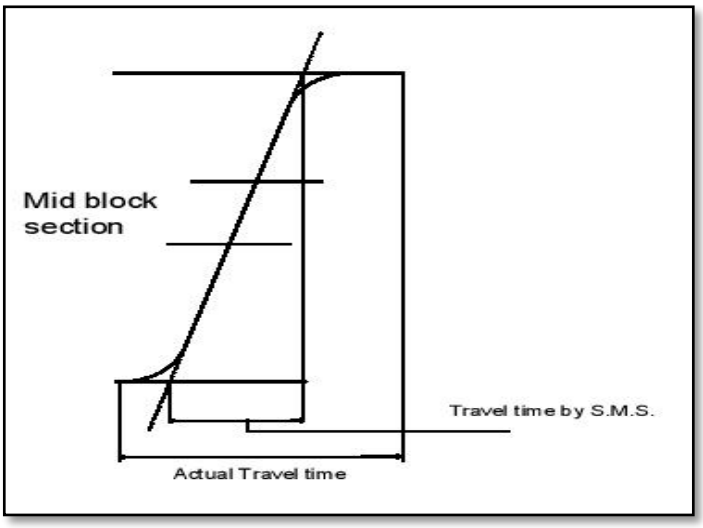


Figure 6.11: Track of vehicle at signalized intersection

SMS of the vehicle is taken at the mid-block of link and if slope of SMS is extended than it will give us travel time by SMS (tt_s). In other hand actual track of vehicle is different and it gives actual travel time of vehicle (tt_a) for that stretch. Travel time by space mean speed might be less than actual travel time of the vehicle. So, relationship between travel time by SMS and actual travel time of a vehicle is to be found out. The equation 6.3 will give SMS

$$\text{Space Mean Speed} = \frac{\text{Mid block length (m)} * \text{No.of vehicles (n)}}{\sum_{i=1}^{n} \text{ti (mid block)}} \quad \text{..... (6.3)}$$

Now equation 6.4 can be used to calculate travel time by SMS.

$$Travel\ time\ by\ S.M.S\ (tts) = \frac{Total\ link\ length\ (m)}{S.M.S\ of\ that\ vehicles} \quad \text{..... (6.4)}$$

The actual travel time of the vehicle can be obtained by tracking individual vehicle by constant replaying the videography data and average actual travel time of traffic stream can be obtained using following equation 6.5

$$Actual\ Travel\ time\ (tta) = \frac{\sum_{i=1}^{n} ti\ \text{(Observed travel time for whole link)}}{No.of\ vehicles\ (n)} \quad \text{..... (6.5)}$$

From the videos different vehicles were tracked from A to B and B to C. Camera 1 covered the A intersection, Camera 2 covered the vehicles coming from A intersection and B intersection too. Camera 3 covered only vehicle coming from B and Camera 4 covered vehicles coming from camera 3 and vehicle going to C intersection.

In data analysis fixed location was taken as the reference and that fixed locations were electric pole (street light-average 28 m apart). 15 fixed points were taken from A to B and B to C, so vehicle tracked with respect to fixed reference points. Time space diagram of tracked vehicle from intersection A to C is displayed in figure 6.12.

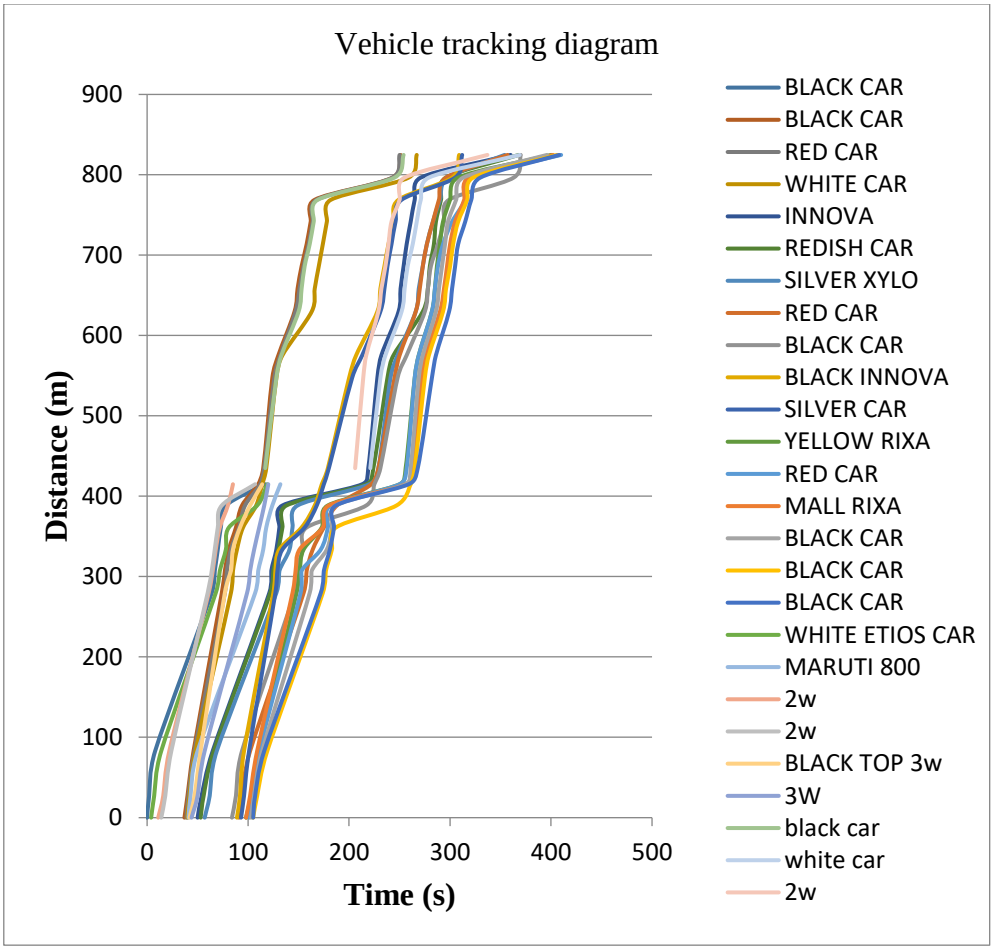


Figure 6.12: Time space diagram of individual vehicle along corridor

From the analysis for corridor of Swastik Char rasta (A) to Swagat intersection (C), the ratio of actual travel time and travel time by SMS for non-stopped vehicle is observed around 1.7, while it is calculated 2.6 for the stopped vehicle condition which includes stopped delay at red light as well as delay due to acceleration and deceleration behavior of the vehicle. For average condition, value of this ratio is observed as 2.3.

6.5 Summary

This chapter explains details regarding study area, data collection techniques adopted as well as data analysis performed to ascertain different signal control parameters value at the selected corridor. The next chapter covers analytical approach applying time space diagram to validate developed methodology discussed in chapter 4 and 5.

CHAPTER: 7
Model Validation- Analytical Approach

7.1 General

The previous chapter has elaborated different procedures performed to analyze and find various signal control parameter values at selected corridor. Depending on the analyzed data and methodology of signal coordination described in previous chapters, analytical validation of developed methodology and computational model is performed for noon peak as well as evening peak data applying Time Space Diagram and Genetic Algorithm.

The objective of the model is to minimize total delay of the corridor under consideration in forward and backward direction for right turning and straight movement. To accomplish the objectives; the collected actual data is validated for noon peak and evening peak real time condition extracted from videography data. For both analyses the validation process is suitably divided in to three different parts.

1. Validation of phase sequence and phase plan. (DMS1)
2. Validation of TW_TSCS1 with proportionate demand phase time (DMS2)
3. Validation of TW_TSCS1 with equal phase time. (DMS3)

The assumption made to derive TW_TSCS1 is relaxed and for the condition of different travel time in forward and backward direction as well as dissimilar phase time model as explained in chapter 5 is developed. The validation of this model for hypothetical data considering two different cases is described in chapter 5. Further validation of model is attempted in the following section with actual collected field data and obtained results are discussed.

7.2. Validation of TW_TSCS1 (noon peak)

Analysis of the data has been performed by accessing manually collected data on site as well as data extracted from the videography. Figure 7.1 reveals prevailing phase sequence, phase timing, space mean speed and existing delay of the selected corridor during noon peak period starting from 12:22:01 onwards. This figure is drawn based on factual assessment of the videography data of the analysis period. Extensive and through analysis of the data exposes

absence of coordination at the selected corridor which will be clear from the following points.

- At observed time it is detected that at intersections A, B & C, phase no. 1 is about to start which exhibits that the ideal offset between the intersections for proper two way coordination are not set properly.
- As average travel time by SMS between intersections is around 50 second, there should be two phase offsets with equal signal cycle time to establish coordination between intersections.
- From analysis of the space mean speed data and observed SMS in links of the corridor elaborated in previous chapter, vehicle has to spend actually 100.5 second (Figure 6.5) in A-B-C- direction to cover 805m length of the corridor as per prevailing space mean speed.
- As per analysis explained in previous chapter the ratio of actual travel time and travel time by SMS for average condition is 2.3. It implies that vehicle requires average 230 second to travel the 805 m length of the corridor which is more than double time considering the speed of the vehicle at mid-block section.
- As per observation prevailing anticlockwise progression of phase sequence at intersection A and B, and clockwise progression of phase sequence at intersection C are also responsible for lack of coordination through corridor which results in excessive delay to right turning and straight moving movement negotiating the corridor in both directions.
- Common cycle length is the mandatory requirement for the coordination which is also not observed at the intersections.

The condition demands immediate intervention from the authority to improve traffic condition on the corridor.

7.2.1 Delay Minimization Scheme (DMS) 1

First alternative is slight modification of the so called “do nothing” approach. Keeping cycle time and phase time as it is just by changing phase sequence as per the developed phase plan, delay calculation of noon peak period for forward direction and backward direction are done applying Auto CAD software using time space diagram. This phase prioritization technique is coded as Delay Minimization Scheme (DMS) 1.

Sample weighted average delay calculation for the figure 7.1 is presented here. Appendix II gives detail about the delay calculation procedure adopted for calculating delay from time space diagram.

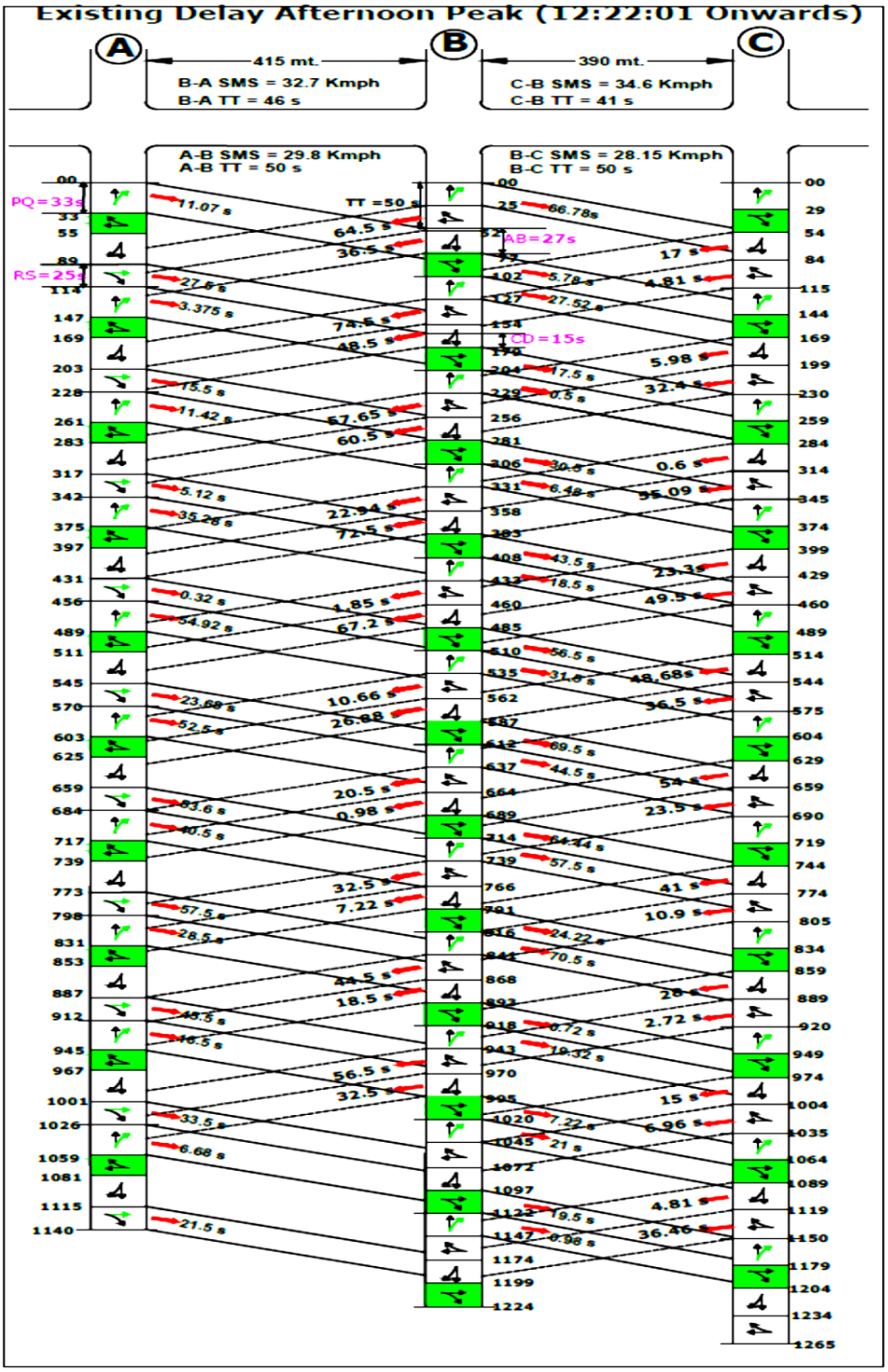


Figure 7.1: Time space diagram of existing signal timings and delay (noon peak)

TT= Travel time between Intersection A to B and B to C

PQ= Phase time of right turner

RS= Phase time of straight movers

AB= Right turner delay up to getting green phase

CD= Straight movers delay up to getting green phase

1 Travel Time $=\frac{(\text{Distance between Intersection A \& B})\times 3.6}{\text{Space Mean Speed}}$

$= \frac{415 \times 3.6}{29.8}$

=50.13

≅ 50 second

2 Right Turner Delay (For A to B in 1st Cycle)

Step -I

PQ= Phase time of right mover = 33 second

AB= Total waiting time of right turning phase = 27 second

∴ BC = Total Clearance time available = PQ –AB = 33-27 = 6 second

Step -II

Clearance % = (Clearance time / Phase time at Intersection A) × 100

= (6/33) × 100

= 18 %

Waiting % = (Waiting time/ Phase time at Intersection A) × 100

= (27/33) × 100

= 82%

Step- III

Weighted Average

= (Clearance percentage × clearance time) + [Waiting percentage × (Waiting time/2 + delay up to getting green phase)]

= (0.18× 0) + [0.82 × (27/2 +0)]

= 0 + [0.82× 13.5]

=11.07 second

3 Straight Movers delay (For A to B in 1st Cycle)

RS= phase time =25 second

∴ Average phase time =25/2= 12.5 second

CD= Delay up to getting green phase = 15 second

∴Total Straight movers delay= Average stopped delay + delay up to getting green phase

= RS/2 + CD

= 12.5 + 15

= 27.5 second

Similarly delay calculations for all cases have been carried out. Figure 7.2 shows the time space diagram of the same position with change of phase plan as per developed methodology DMS1.

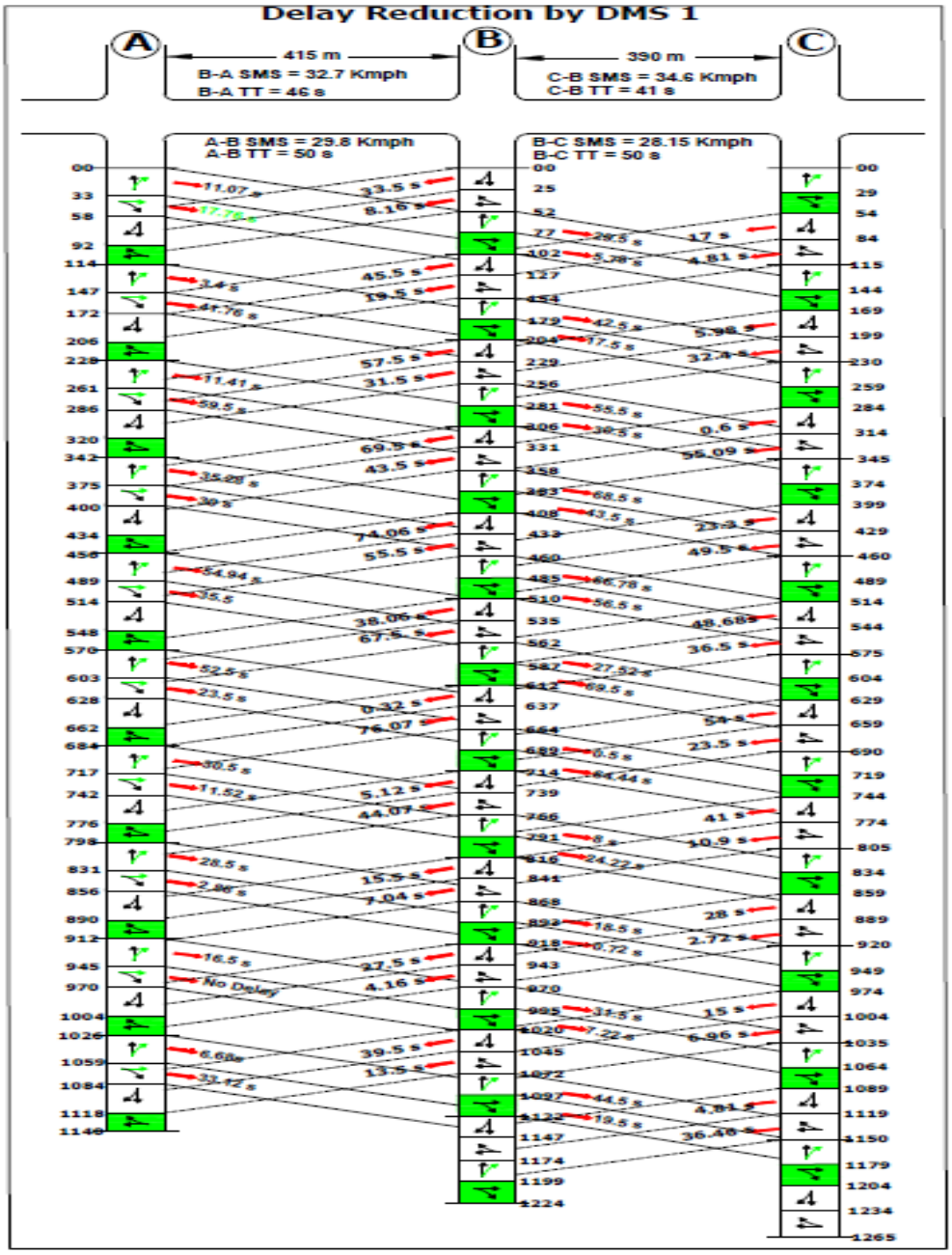


Figure 7.2: Delay reduction by DMS 1 (noon peak)

For the fair and objective assessment of the actual delay, it is calculated continuously up to the point where repetition of the starting 1st cycle is achieved. As the cycle times of all intersections as well as phase time of all the phases are different, the near repetition of existing situation at the starting time of analysis is achieved only at 10th cycle. Figure 7.1 shows that after 946 second the actual position of the phases are similar to the starting time of the analysis. During this analysis time, right turner and straight mover movement attain high and low values of the delay in both forward as well as backward directions. Reduction of delay in both directions is observed in DMS1, which is reflected in figure 7.2.

7.2.2 Delay Minimization Scheme (DMS) 2

The results obtained by DMS 1 necessitated rethinking the strategy and accordingly looking to the situation, it is interesting to know whether implementation of proposed two-way coordination of signal system with identical cycle elaborated in chapter 3 at all intersections can improve the traffic condition or not. As per the obtained SMS and existing distances between the intersections, average travel time between intersections Swastik (A) to Girish (B) is 50 sec, Girish (B) to Swastik (A) is 46 sec, Girish (B) to Swagat (C) is 50 sec, and Swagat (C) to Girish (B) is 41 sec. Considering the same average travel time in both directions as per the equation (3), even phase difference with phase length for higher travel time 50/2= 25 second is selected. According to existing traffic volume on three signalized intersections, as well as travel time criterion 25+3 = 28 second phase length (green +amber) is selected. As per equation 3.7, C=4*Pl*, i.e. cycles of 112 sec are selected. Alternatively, as per prevailing situation at intersections A, B & C current cycle time is 114, 102 and 115 second. Considering demand supply scenario as well as prevalent signal cycle times at all intersections, average 112 second cycle with 28 second phase length looks appropriate.

For DMS 2, instead of providing equal phase of 28 second for all phases of all intersections, the cycle time of 112 second is proportionately divided among phases to cater the existing demand (g/c ratio). Likewise, all phase lengths are apportioned to 112 second cycle time for their current demand. Figure 7.3 shows delay reduction by applying DMS 2 where phase sequence and equal cycle time are adopted and existing g/c ratio of phase is unaltered.

7.2.3 Delay Minimization Scheme (DMS) 3

In this third scheme after applying phase prioritization and equal cycle with existing g/c ratio on selected three signalized intersections of corridor, now it is interesting to know

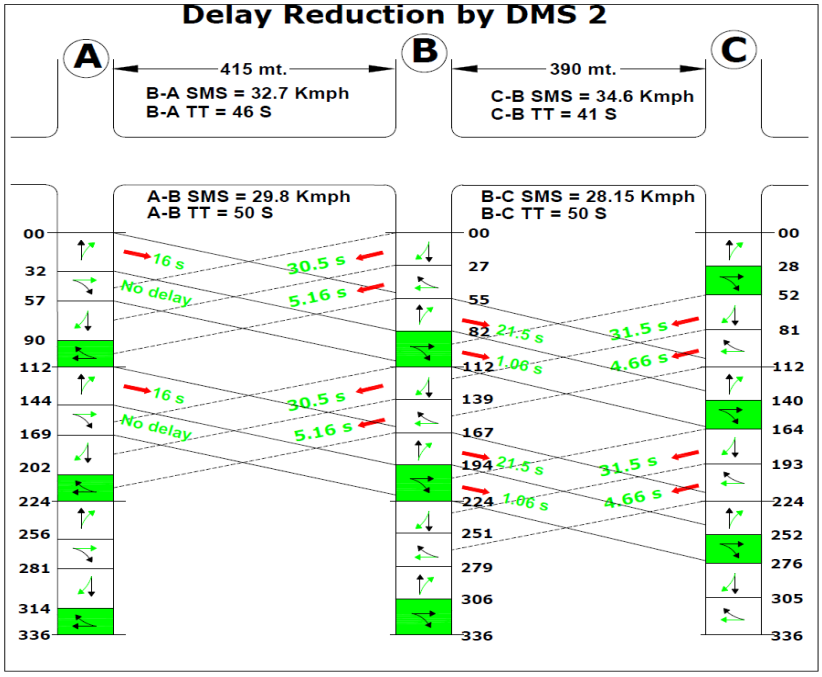


Figure 7.3: Delay reduction by DMS 2 (noon peak)

whether implementation of proposed two-way coordination of signal system with identical cycle and equal phases at all intersections as described in section 4.3 (TW_TSCS 1) can improve the traffic condition or not. Accordingly, DMS 3 is applied with identical cycle and equal phases at all intersections. Figure 7.4 illustrates time space diagram and reduced delay situation after implementation of DMS 3.

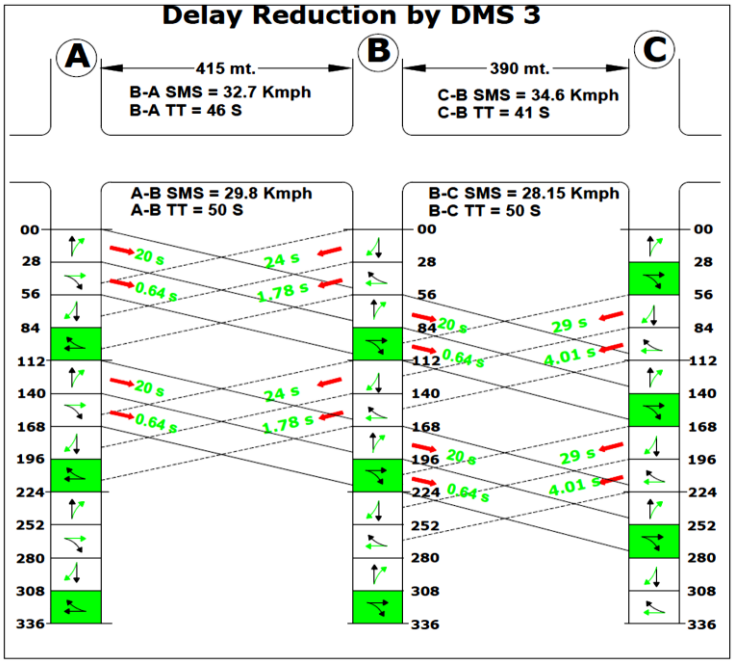


Figure 7.4: Delay reduction by DMS 3 (noon peak)

Table 7.1 illustrates comparison amongst existing delay and reduction in delay achieved by applying DMS 1, 2 and 3 in forward and backward directions respectively. From the table it is observed that methodology proposed in this research is capable to reduce delay and can improve the existing situation.

Table 7.1: Comparison of existing delay and delay by DMS 1, 2 & 3 (noon peak in sec)

Direction of Progress	Type of Movement	Total Delay in Existing Condition (s)	Delay by Applying DMS1 (s)	% Reduction by DMS1	Delay by Applying DMS2 (s)	% Reduction by DMS2	Delay by Applying DM (s)	% Reduction by DMS 3
Forward	R	580.005	570.66	1.61	170.6	70.58	206.4	64.31
	S	603.06	575.42	4.57	10.6	98.24	12.8	97.87
Backward	R	624.47	595.36	4.66	366.60	41.29	307.8	50.71
	S	644.94	615.83	4.51	98.20	84.77	57.9	91.02
Total Cumulative Delay		2452.475	2357.27	3.83	646	73.72	584.9	75.97

(R-Right turner, S-Straight through)

7.3 Validation of TW_TSCS1 (evening peak)

Analysis of the data has been performed by assessing manually collected data on site as well as data extracted from the videography. Figure 7.5 reveals prevailing phase sequence, phase timing, space mean speed and existing delay of the selected corridor during evening peak period starting from 17:51:01 onwards. These figures are drawn based on factual assessment of the videography data of the analysis period. Form these figures it is observed that during analysis period at Intersection A phase number 3, at Intersection B phase number 4 & at Intersection C phase number 1 is about to start. As explained in the preceding section the condition demands immediate intervention from the authority to improve traffic condition on the selected corridor.

Sample delay calculation for the 6th cycle for figure 7.5 is presented here.

TT= Travel time between Intersection A to B & B to C

PQ= Phase time of straight movers from figure i.e. 49-24=25 second

RS= Phase time of right turner from figure i.e. 609-579=30 second

AB= Straight Movers' delay up to getting green phase

CD= Right turners' delay up to getting green phase

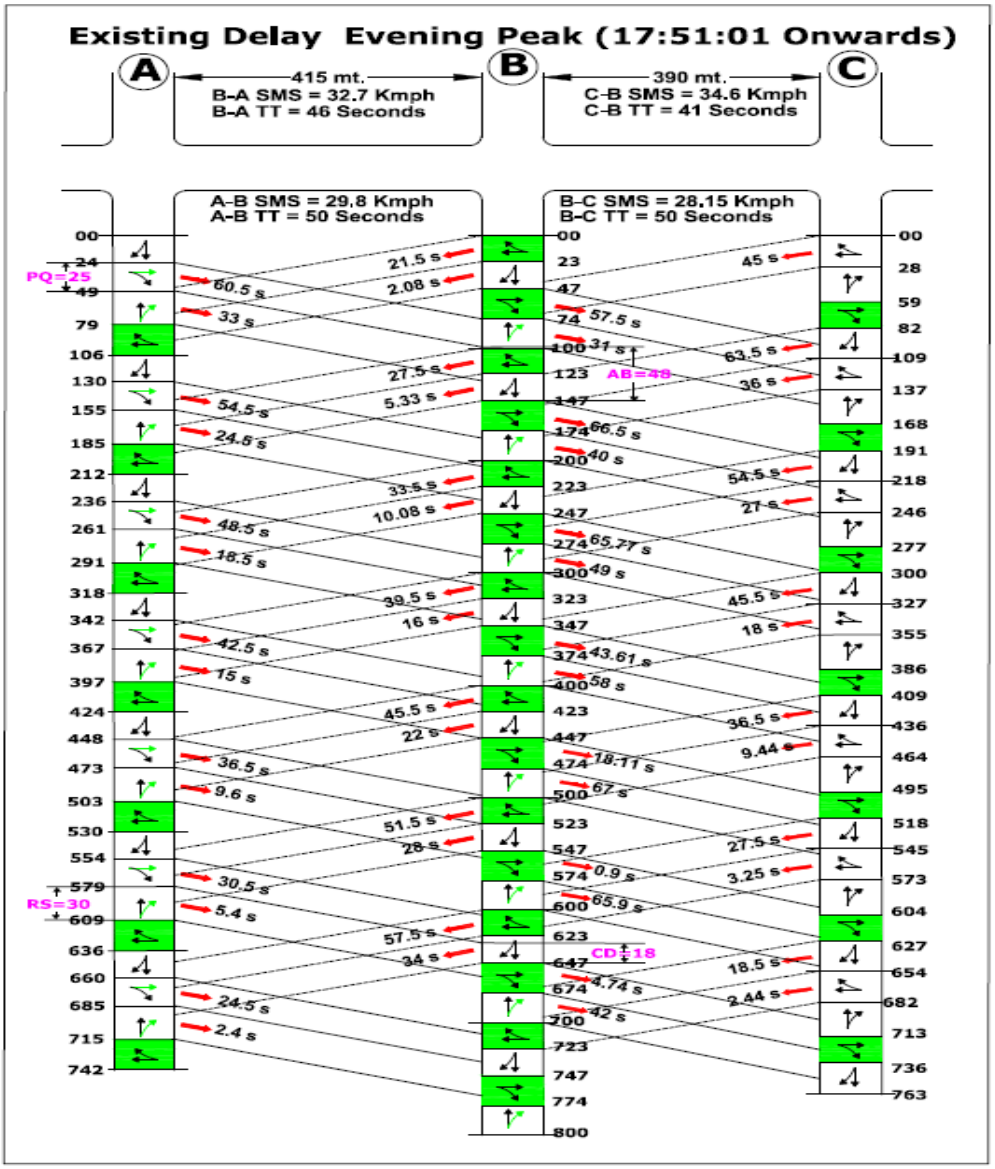


Figure 7.5: Time space diagram of existing signal timing and delay (evening peak)

1. Travel Time $= \frac{(Distance\ between\ Intersection\ A\ \&\ B)\times 3.6}{Space\ Mean\ Speed}$

$$= \frac{415 \times 3.6}{29.8}$$

$$= 50.13$$

$$\cong 50 \text{ second}$$

2. Straight Movers delay (For A to B 1st Cycle):

PQ = phase time =25 second

∴ Average phase time =25/2= 12.5 second

AB= Delay up to getting green phase = 48 second

∴ Total Straight movers delay= Average stopped delay + delay up to getting green phase

= PQ/2 + AB

= 12.5 + 48

= 60.5 second

3. Right Turner Delay (For A to B 6th Cycle):

Step -I

RS= Phase time of right mover = 30 second

CD= Total waiting time of right turning phase = 18 second

∴ Total Clearance time available = RS –CD = 30-18 = 12 second

Step -II

Clearance % = (Clearance time / Phase time at Intersection A) × 100

= (12/30) × 100

= 40 %

Waiting % = (Waiting time/ Phase time at Intersection A) × 100

= (18/30) × 100

= 60%

Step- III

Weighted Average

= (Clearance percentage × clearance time) + [Waiting percentage × (Waiting time/2 + delay up to getting green phase)]

= (0.40× 0) + [0.60 × (18/2 +0)]

= 0 + [0.63× 9]

=5.4 second

Similarly delay calculation for right turning and straight through movement in forward and backward direction for all three schemes discussed hereafter is carried out and result received from the same is presented.

7.3.1 Delay Minimization Scheme (DMS) 1

First alternative is slight modification of the so called "do nothing" approach. Keeping cycle time and phase time as it is just by changing phase sequence as per the developed phase plan, delay calculations are carried out applying Auto CAD software using time space diagram. This phase prioritization technique is coded as DMS 1. As the cycle times of all intersections as well as phase times of all the phases are different the delay calculation is continued up to 6th cycle to collect information of actual delay. Figure 7.6 shows the time space diagram of the same position with change of phase plan as per developed methodology. Reduction of delay in both directions is observed in DMS 1, which is reflected in said figure.

As depicted in the figure 7.5 the value of delay for straight movers in forward direction is 121 second in 2nd cycle. Excessive delay in the cycle leads to accumulation of not served vehicle platoon at intersection which results into queue spillback and spillover effect at upstream intersection that ultimately leads to individual cycle failure as observed during peak hours. The condition becomes more aggravated due to paid parking facility, nearby Municipal Market, BSNL Head office and unauthorized parking of vehicles in eating free space of road width. This could be the reason for high ratio of travel time by space mean speed and actual travel time at selected corridor as found in data analysis in previous chapter.

7.3.2 Delay Minimization Scheme (DMS) 2

The results obtained by DMS 1 necessitated rethinking the strategy and accordingly looking to the situation, it is interesting to know whether implementation of proposed two-way coordination of signal system with identical cycle at all intersection can improve the traffic condition or not. As per the obtained SMS and existing distances between the intersections, average travel time between intersections Swastik (A) to Girish (B) is 50 sec, Girish (B) to Swastik (A) is 46 sec, Girish (B) to Swagat (C) is 50 sec, and Swagat (C) to Girish (B) is 41 sec. Considering the same average travel time in both directions as per the equation (3) even phase difference with phase length for higher travel time 50/2= 25 second is selected.

According to existing traffic volume on three signalized intersections, as well as travel time criterion 25+2 = 27 second phase length (green +amber) is selected. As per equation 4.7 $C=4Pl$, cycles of 108 sec is selected. Alternatively, as per prevailing situation at intersections A, B and C current cycle time is 106, 100 and 109 second. Considering demand supply

scenario as well as prevalent signal cycle time at all intersection average 108 second cycle with 27 second phase length looks appropriate.

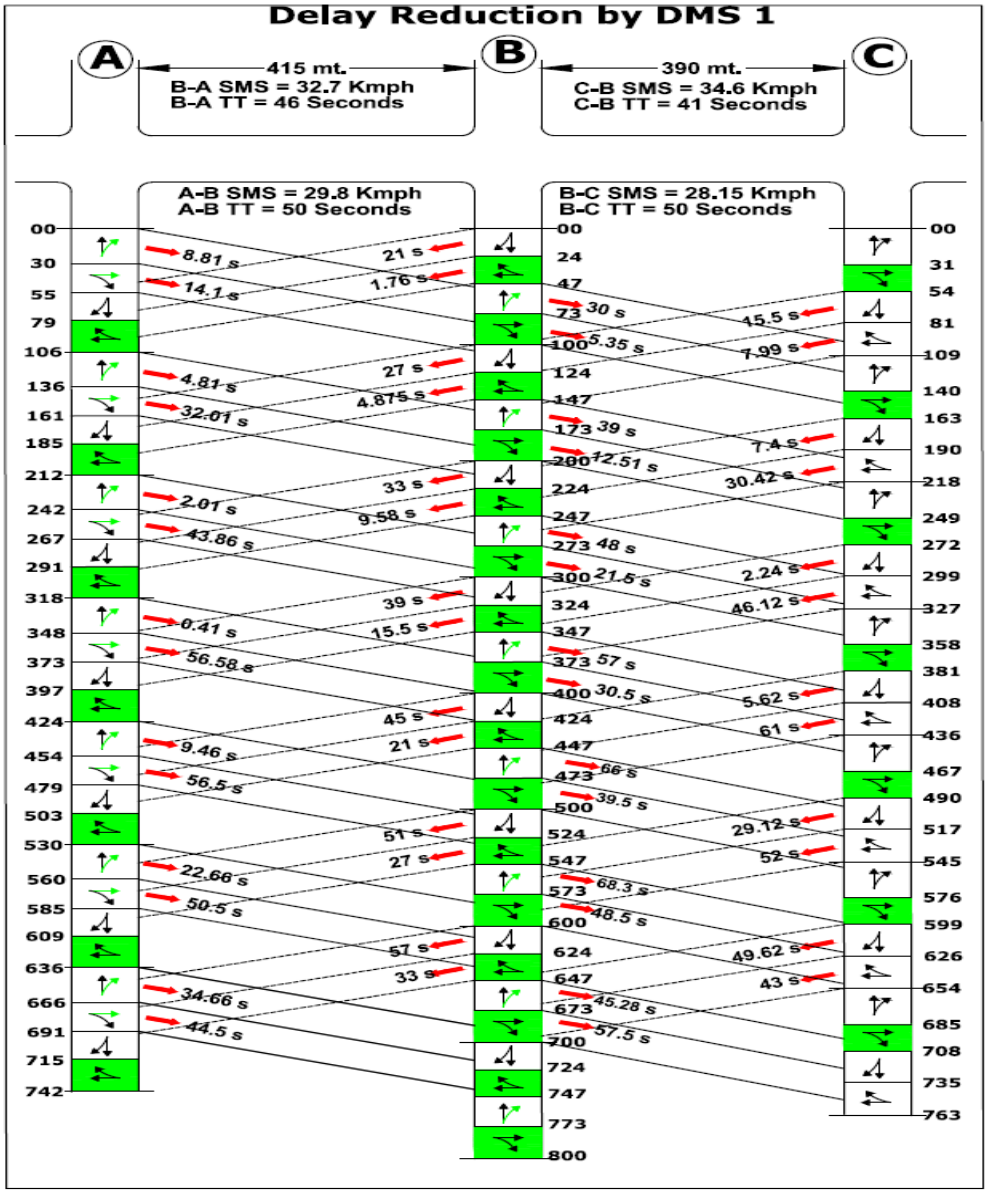


Figure 7.6: Delay reduction by DMS 1 (evening peak)

For DMS 2 instead of providing equal phase of 27 second for all phases of all intersections, the cycle time of 108 second is proportionately divided to phases to cater the existing demand (g/c ratio). Likewise, all phase lengths are apportioned to 108 second cycle time for their current demand. Figure 7.7 shows delay reduction by applying DMS 2 where phase sequence and equal cycle time is adopted while g/c ratio of existing phase time remains unchanged. The following chapter demonstrates field testing and evaluation of developed model in real traffic scenario for evening peak on same corridor where also 108 second cycle length with varying phase timing as per approach demand is implemented.

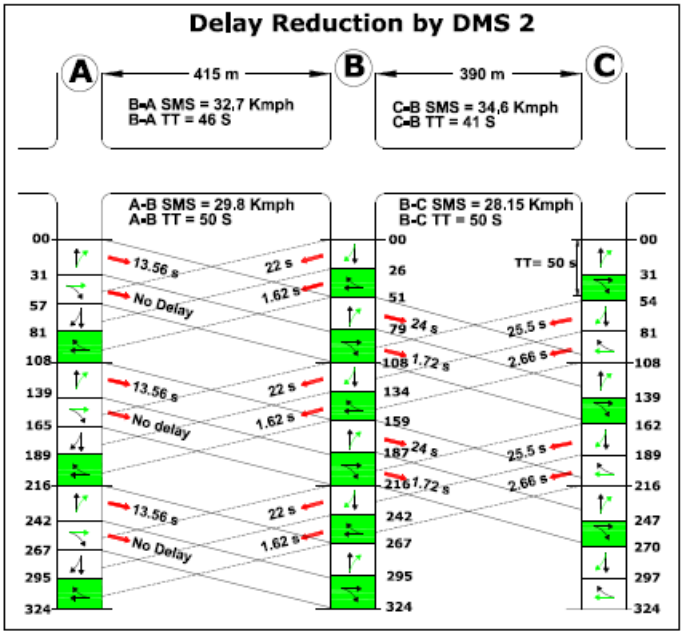


Figure 7.7: Delay reduction by DMS 2 (evening peak)

7.3.3 Delay Minimization Scheme (DMS) 3

In this third scheme after applying phase prioritization and equal cycle with existing g/c ratio on three signalized intersections, it is interesting to know whether implementation of proposed two-way coordination of signal system with identical cycle and equal phase at all intersections can improve the traffic condition or not. Accordingly developed methodology as discussed in section 4.3 (TW_TSCS1) is applied with identical cycle and equal phases at all intersections. Figure 7.8 illustrates time space diagram and reduced delay situation after implementation of DMS 3.

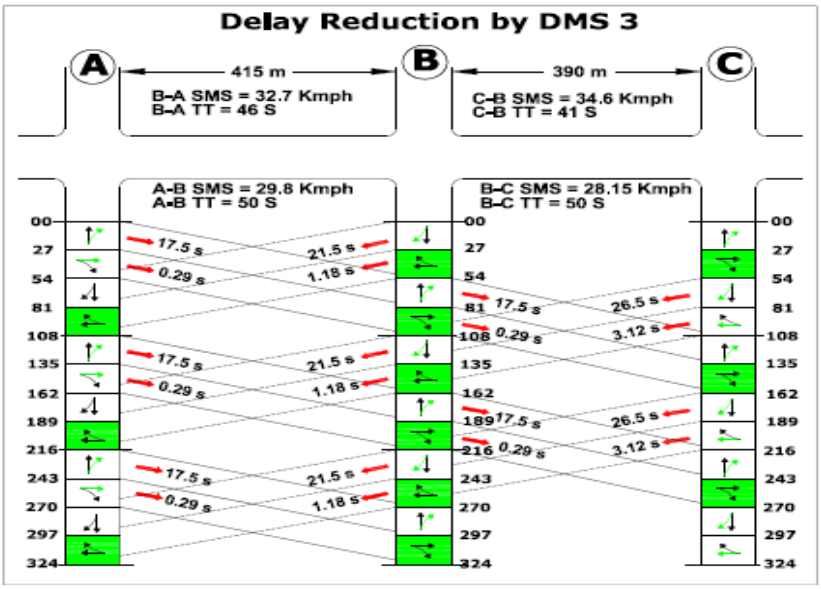


Figure 7.8: Delay reduction by DMS 3 (evening peak)

Table 7.2 illustrates comparison amongst existing delay position and reduction in delay achieved by applying delay minimization schemes 1, 2 and 3 in forward and backward directions respectively.

Table 7.2: Comparison of existing delay and delay by DMS 1, 2 & 3 (evening peak in sec)

Direction of Progress	Type of Movement	Total Delay in Existing Condition (s)	Delay by Applying DMS1 (s)	% Reduction by DMS1	Delay by Applying DMS2 (s)	% Reduction by DMS2	Delay by Applying DMS3 (s)	% Reduction by DMS3
Forward	R	416.9	356.46	14.49	91.68	78	106.74	74.39
	S	525.29	411.41	21.67	10.32	98.01	3.48	99.33
Backward	R	329.29	325.5	0.01	162.72	50.58	166.08	49.56
	S	357.69	320.245	10.46	25.68	92.82	25.8	92.78
Total Cumulative Delay		1629.17	1413.615	11.65	290.4	79.85	302.1	79.01

(R-Right turner, S-Straight through)

From the above table it is observed that methodology proposed in this research is capable to reduce delay and can improve the existing situation. As shown in the table 6.2, DMS 3 with equal cycle time at all intersections, it is possible to reduce delay up to 74.39 % for right turner traffic while for straight movers i.e. main through movement, it produces unhindered progress of the vehicle platoon with no delay because methodology is capable to minimize delay by more than 92%. As far as total cumulative delay reduction is thought of both scheme i.e. DMS 2 and DMS3 reduce delay by more than 79%. The results provide more flexibility to transport professional while deciding individual phase timing of the selected intersection to achieve two way coordination.

7.4 Validation of TW_TSCS2 with developed model

Even when the travel time in forward and backward direction is fluctuating than also coordination can be achieved, which was explained in the model for the two-way coordination in chapter 5. In section 5.3 analytical validation of developed model using hypothetical data for two different cases were also described. Here, it is tried to ascertain the validity of the model with actually collected data from the field. As per the actual calculated SMS and extracted SMS through videography data and distances between the intersections, average travel time between intersections Swastik (A) to Girish (B) is 50 secs, Girish (B) to

Swastik (A) is 46 secs, Girish (B) to Swagat(C) is 50 secs, and Swagat (C) to Girish (B) is 41 sec. Considering the logic described in chapter 5 section 5.2, and applying DMS 4 and DMS 5 presented in the figure 5.8 here,

For DMS 4

tt_{ij} = *avg.tt* out of all intersection in forward direction, here A to B and B to C = 50 sec

tt_{ji} = *avg.tt* out of all intersection in backward direction, here B to A and C to B = 43.5 sec

For DMS 5

tt_{ij} = *max.tt* out of all intersection in forward direction, here A to B and B to C = 50 sec

tt_{ji} = *max.tt* out of all intersection in backward direction, here B to A and C to B = 46 sec

Looking to the minimal difference of travel time for the observed site condition applying DMS 4 and DMS 5 and adopting DMS 5

Here P_{1i}, P_{2i}, P_{3i} and P_{4i} which is the respective phase length at intersection i

For two way coordination sum of phase length for phase 1 and 2 at all intersections can be obtained by applying rule 1 explained in chapter 4,

$$\therefore P_{1i} + P_{2i} = tt_{ji} = 46 \text{ s} \qquad \ldots\ldots (7.1)$$

Similarly, sum of phase length for phase 3 and 4 at all intersections can be found by using rule 2 described in chapter 4,

$$\therefore P_{3i} + P_{4i} = tt_{ij} = 50 \text{ s} \qquad \ldots\ldots..(7.2)$$

Now as per developed model cycle time for two-way coordination, from rule 3

$$\therefore C_i = tt_{ij} + tt_{ji}$$

$$C_i = 50 + 46$$

$$\therefore C_i = 96 \text{ second.} \qquad \ldots\ldots (7.3)$$

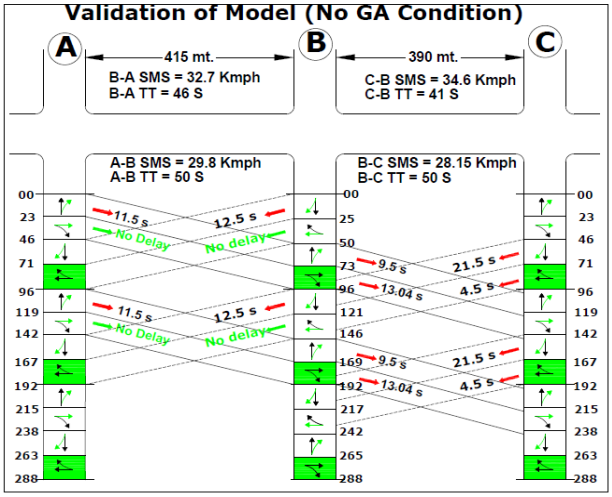


Figure 7.9: Model validation (no GA condition)

For derived 96 sec optimized cycle and applying TW_TSCS1 methodology equal phase time i.e. 23 second for all four phases at selected three intersections is considered. Delay calculation by applying delay calculation procedure shown in previous para and elaborated in Annexure 2 using time space diagram is carried out as shown in figure 7.9. The delay obtained from selected signal cycle time and phase time for model validation is presented as well as compared with actual delay of noon peak and evening peak for one cycle as depicted in table 7.3.

Table 6.3: Reduction of delay by model (without GA application in sec)

Direction of progress	Type of movement	Average delay in existing condition (noon Peak) (s)	Total delay in existing condition (Evening Peak) (s)	Total delay by model (Without GA application) (s)	% Reduction in delay (noon peak)	% Reduction in delay(Evening peak)
Forward	R	58	69.48	21	63.8	69.8
	S	60.3	87.54	13.04	78.4	85.1
Backward	R	62.44	54.88	34	54.5	38.05
	S	64.4	59.61	4.5	93	92.45
Total Cumulative Delay		245.14	271.51	71.54	70.8	73.65

(R-Right turner, S-Straight movers)

7.4.1 Optimization of delay with application of Genetic Algorithm

Herein application of the model coupled with the TW_TSCS1 methodology is capable for reduction in considerable amount of delay. To attain further improvement in total delay, hereafter Genetic Algorithm is applied, nested with the developed model and incorporating TW_TSCS1 with TW_TSCS2.

As explained in chapter 6, 30 cycles of the collected data is analyzed and volume data has been extracted from videography by manual counting. The one-hour flow values presented in table 6.8, 6.9 and 6.10 reveals that as per IRC SPCU flow values ranges from min 1939 pcu/hr/total width of approach from Mithakhali six road at Girish intersection to max 2179 pcu/hr/total width of approach from Navrangpura bus stand at Swastik intersection. Similarly, one-hour flow values as per developed DPCU derived specifically for selected corridor ranges from min 1258 pcu/hr/total width of approach from Mithakhali six road at Girish intersection to max 1646 pcu/hr/total width of approach from Navrangpura bus stand at Swastik intersection. The variation in flow is about 20% as per IRC PCU values but it is more than 30% as per Dynamic PCU values. The observed flow values for major and minor approaches have inspired to rethink the strategy of equal phase timings for all approaches of intersection. As per the derived model and rule 1 and 2 the phase time of phase 3, 4 and phase time of phase 1, 2 are fixed according to the travel time in forward and backward direction. When the traffic flow at major and minor approaches are varying then after applying minimum green constraint to the minor approaches, the increase in green time of major approaches will help to minimize the overall delay of intersection provided the g/c ratio available at minor street is sufficient to satisfy DFR of that approach.

Accordingly, GA is applied to the above dataset and after 14 iterations programme converges to the minimum delay value with optimum signal cycle and phase timing. The detailed output generated by the Lib GA software is given in Annexure IV. Figure 7.10 shows delay calculation with time space diagram for 95 second “optimized” signal cycle for minimum corridor delay.

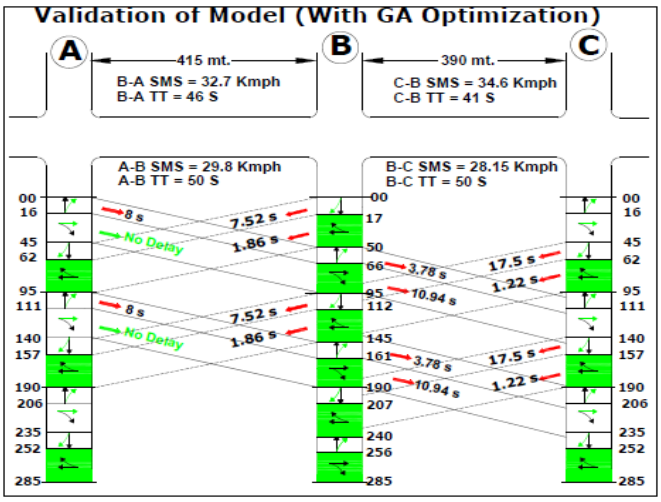


Figure 7.10: Model validation (with GA optimization)

The GA gives 16 second phase time for phase 1 and 17 second phase time for phase 3 this time is also cross verified for the existing DFR values of the approaches which are served by the phase 1 and phase 3. The programme has the inbuilt facility to verify it for the assumed SFR values as discussed in chapter 4 in methodology TW_TSCS2. For the purpose of calculating volume, DPCU values which are developed and explained in chapter 6 were used. Table 7.4 shows comparison of the delay values for one cycle, whereas table 7.5 reveals comparison of delay using developed model for no GA condition and with GA condition. It discloses that Genetic Algorithm is successful in further reduction of delay which is 29%.

Table 7.4: Delay comparison for one cycle by developed model (in sec)

Direction of Progress	Type of Movement	Total Delay in (noon Peak) (s)	Total Delay in (Evening Peak) (s)	Total delay by Model (Non GA) (s)	Total delay by Model (With GA) (s)
Forward	R	58	69.48	21	11.78
	S	60.3	87.54	13.04	10.94
Backward	R	62.44	54.88	34	25.02
	S	64.4	59.61	4.5	3.08
Total Cumulative Delay		245.14	271.51	71.54	50.82

(R-Right turner, S-Straight through)

Table 7.5: Reduction of delay by model (no GA and with GA in sec)

Direction of Progress	Type of Movement	Total delay by Model (Non GA) (s)	Total delay by Model (With GA) (s)	% Reduction in Delay(With GA)
Forward	R	21	11.78	43.9
	S	13.04	10.94	16.1
Backward	R	34	25.02	26.41
	S	4.5	3.08	31.55
Total Cumulative Delay		71.54	50.82	28.96

(R-Right turner, S-Straight through)

7.5 Model validation results

To validate the above developed methodology TW_TSCS1 and TW_TSCS2, noon peak data and evening peak data have been analyzed. For noon peak the delay calculation is performed by time space diagram till the near repetition condition of the starting position of the phases are attained. Reduction in delay for three Delay Minimization Schemes (DMS) based on TW_TSCS1 is observed and it is presented in fig. 7.11. Here, reduction of around 4% total delay is observed in DMS 1 (which validates the developed phase plan and phase sequence). Similarly DMS 2 (which validates the application of average cycle length with “existing”

proportionate demand phase timings), and DMS 3 (which validates the application of TW_TSCS1) reduces delay by 73.7% and 75.9 % respectively. For evening peak data delay is calculated for 6 cycle and results are presented in fig 7.12. Here, reduction of delay is observed by 11.6% for the DMS1 while it is 79% for DMS 2 and 3.

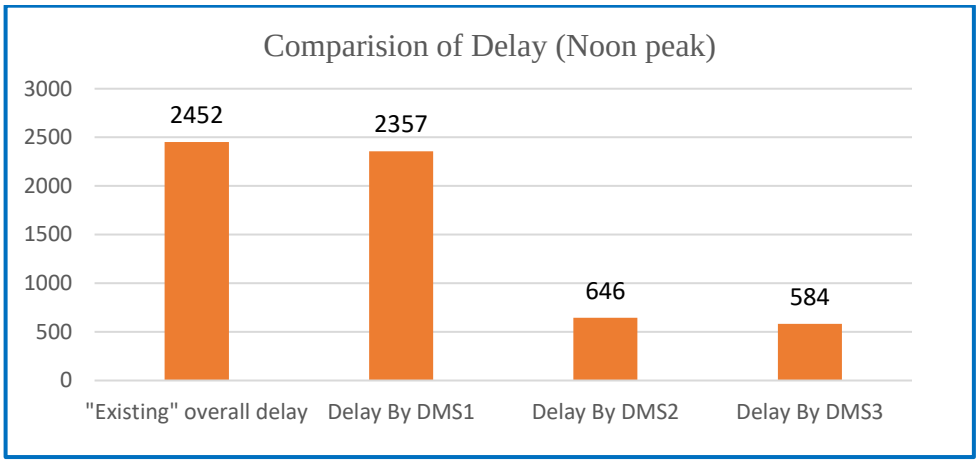


Figure 7.11: Validation of TW_TSCS1 (delay in sec for 10 cycles)

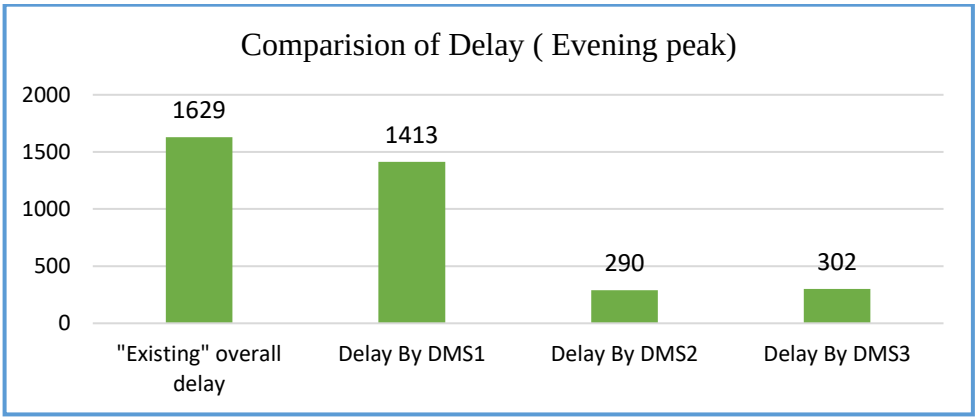


Figure 7.12: Validation of TW_TSCS1 (delay in sec for 6 cycles)

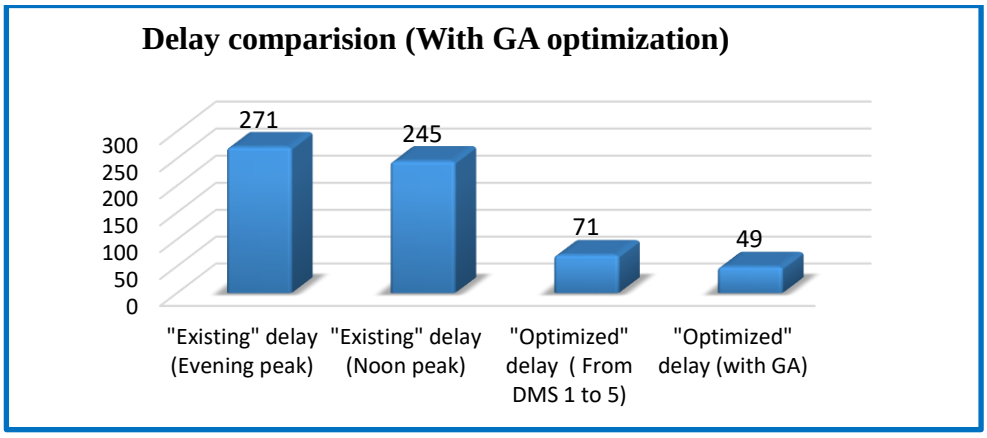


Figure 7.13: Validation of model (delay in sec for 1 cycle)

Figure 7.13 shows model validation and comparison of delay which is calculated applying “optimized” signal setting from DMS 4, 5 and “optimized” signal setting with GA for one cycle. The GA is effective for reducing delay up to 29% for cycle time 95. Table 7.6 is presented with the comparison “existing” and “optimized” signal timing parameters for the selected intersections of C. G. Road of Ahmedabad, India.

Table 7.6: “Existing” and “Optimized” signal parameters (for noon peak)

"Existing" signal setting					" Optimized" Signal setting														
					By DMS 1 to 3					By DMS 4, 5					By Applying GA				
Z	C	P	T	M	Z	C	P	T	M	Z	C	P	T	M	Z	C	P	T	M
A	114	1	33	Anti-Clockwise	A	112	1	28	Clockwise	A	96	1	23	Clock wise	A	95	1	16	Clockwise
		4	22				2	28				2	23				2	29	
		3	34				3	28				3	23				3	17	
		2	25				4	28				4	23				4	33	
B	102	1	25	Anti-Clockwise	B	112	1	28	Clockwise	B	96	1	23	Clock wise	B	95	1	16	Clockwise
		4	27				2	28				2	23				2	29	
		3	25				3	28				3	23				3	17	
		2	25				4	28				4	23				4	33	
C	115	1	29	Clockwise	C	112	1	28	Clockwise	C	96	1	23	Clock wise	C	95	1	16	Clockwise
		2	25				2	28				2	23				2	29	
		3	30				3	28				3	23				3	17	
		4	31				4	28				4	23				4	33	

[Where, Z= Intersection code, C= Cycle time(sec) , P= phase number, T= Time of phase (sec) , M=phase sequence]

7.6 Summary

This chapter covers detailed description of analytical approach applied to validate the developed methodology (TW_TSCS1 and TW_TSCS2) and developed model. It elaborates procedure used for its validation with collected field data for both noon peak and evening peak periods. This chapter summarizes use of Genetic Algorithm technique which gives optimized signal cycle and phase time to reduce overall corridor delay. Traffic researchers working in the field of signal optimization and different software of the signal optimization across the world use either delay minimization approach or bandwidth maximization approach. Here, programme developed in this research is capable to augment benefits of both the techniques to minimize total corridor delay. The next chapter presents detailed case study with field implementation and evaluation of the developed methodology and result received with actual operation.

Chapter 8
Case Study-Field Implementation

8.1 General

In the previous chapter, analytical approach was applied to validate the developed methodology (TW_TSCS1 and TW_TSCS2) and developed model. Now, it is imperative to know that, how far this methodology is applicable to the real corridor traffic conditions? Is it possible to apply derived methodology in real time deployment? Whether traffic is really following the analytically derived delay conditions? Whether the developed methodology is capable to reduce widely accepted traffic performance parameters for common man, i.e. travel time during field testing? What will be the behaviour of traffic after field application? Such types of questions are still to be answered. To get the clue to these answers, factual data from corridor after field implementation is collected and analysed. This chapter mainly discusses the selection of urban road corridor, a procedure followed for the collection of required data, comparison of different signal control parameters before implementation and after implementation with observed data and possible improvement of traffic conditions with two-way signal coordination.

8.2 Study area

The same C.G. Road corridor of Ahmedabad city which was selected for collection of raw data is picked for the field testing of the developed methodology. The land use distribution of Ahmedabad, which reveals that predominant areas are residential (35%) and followed by open. This indicates further development in city has good scope. Industrial area, which covers 15%, is mainly concentrated in eastern periphery of the city. Commercial activities are concentrated within walled city area. The major water bodies are Kankaria Lake and Sabarmati River. The river divides city in two parts. Major traffic movements within the city take place between eastern and western part. There has been modest shift of commercial activities to the Ashram Road, Chimanlal Girdharlal (C.G.) road and Sarkhej Gandhinagar (S.G.) highway after the construction of Nehru Bridge in 1960. The residential colonies have come up outside the walled city in eastern part of Sabarmati River and in western part such as Navrangpura, Ellis Bridge, Naranpura, Vastrapur, Paldi, Ambavadi, and Usmanpura.

The industrial estates are located in Naroda, Odhav, Vatva, Rakhial, Asarwa and Bapunagar. After construction of 76 km circular Sardar Patel (S.P.) ring road there has been phenomenal increase in the all-round development of city. Many residential and commercial pockets are developed around the ring road. Figure 8.1 presents map of Ahmedabad city.

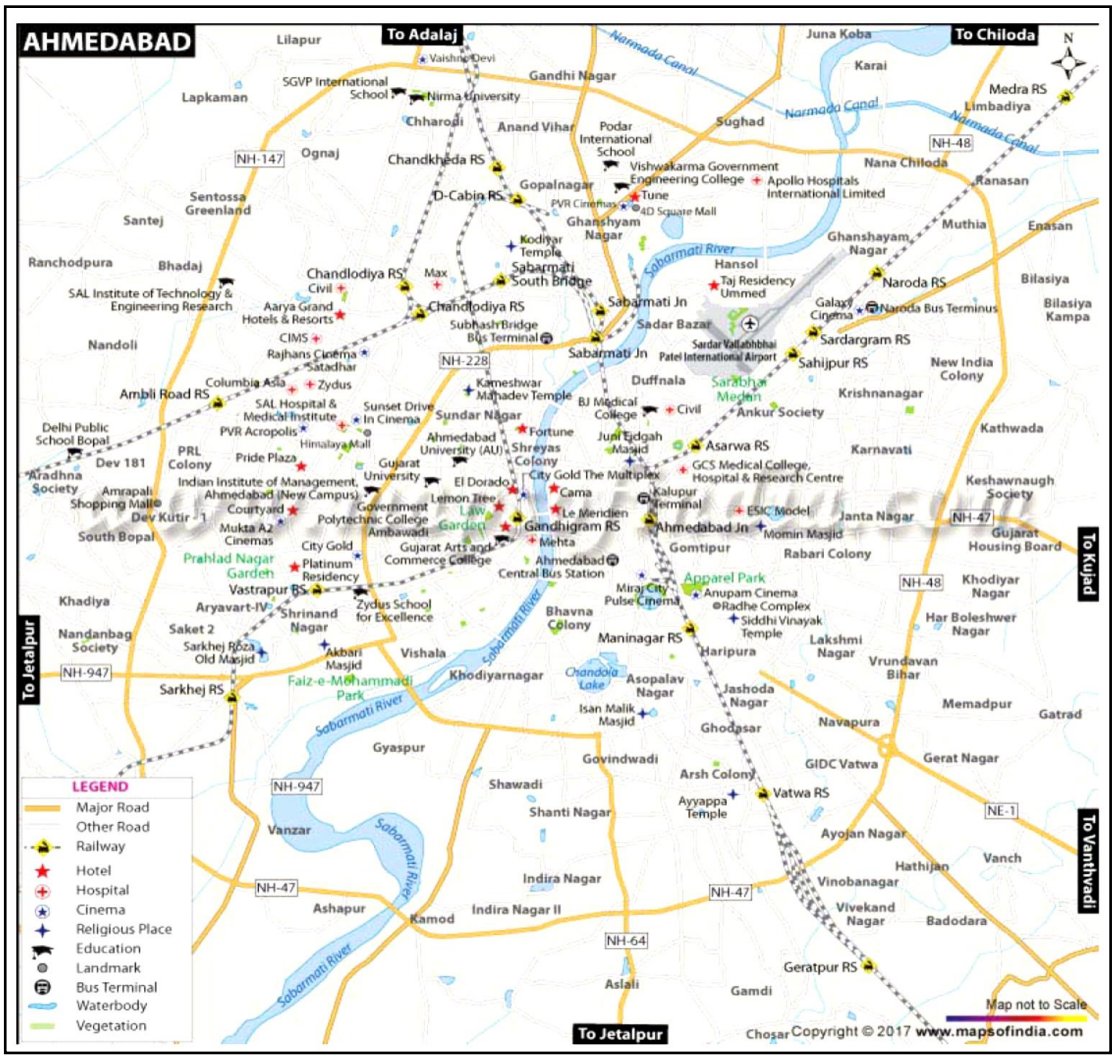


Figure 8.1: The map of Ahmedabad city

As per the Smart City project proposal submitted by Amdavad Municipal Corporation (AMC) to the Ministry of Urban Development, Government of India, the Ahmedabad Municipal Transport Service (AMTS) provides 88% coverage of AMC area with daily ridership of 0.7 Million passengers and fleet of over 750 buses. The AMTS covers 780 km of city area. Ahmedabad Janmarg Limited (AJL) runs Bus rapid transit (BRT) system which runs with Dedicated Bus Lane (DBL) with peak hour speed of 20-25 km/h against average traffic speeds of 9 -17 km/h. BRTS has daily ridership of 0.12 Million passengers and fleet of 235 buses. BRTS covers 130 km of city area and 76 km of S.P. Ring road. Average speed of trip is 20.65 km/h. The city is having population of over 60,000 CNG Auto-rickshaw.

Presently construction work of another mass transportation system- Metro rail- constructed by Metro link express for Gandhinagar and Ahmedabad (MEGA) is in progress as per schedule.

8.3 Field testing procedure

Field evaluation is a bit difficult task to be performed for testing of the developed methodology. The procedure itself involves coordination among different departments like traffic branch of AMC, Ahmedabad city traffic police and agency responsible for maintenance, monitoring and setting of signal control parameters of the city. To implement the developed methodology of TW_TSCS1, formal approval from the concerned authority is to be obtained. For obtaining the permission, the "Signal Retiming Proposal" was prepared and presented to all concerned authority for its implementation. After getting formal approval of the same proposal, its objective was explained in a lucid way to other stake holders such as on duty traffic personnel and technician of the signal operating and maintaining company Delhi Integrated Multi Modal Transit System Ltd. (DIMTS). Figure 8.2 shows study area.

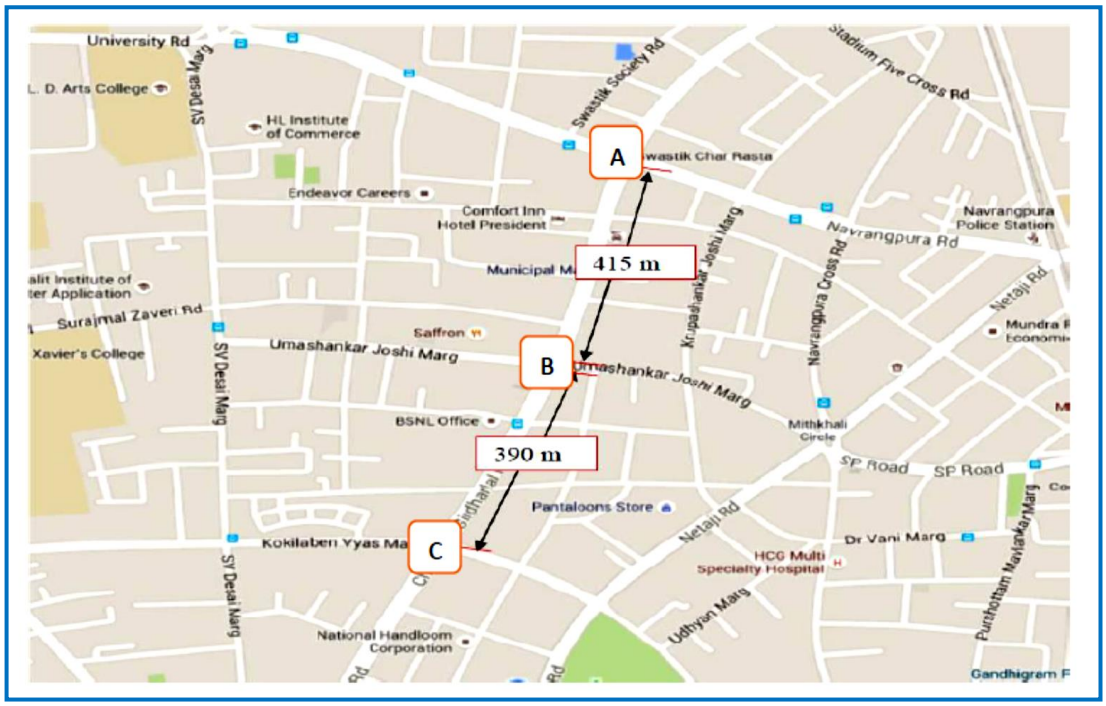


Figure 8.2: Study area of C.G. road Ahmedabad for field evaluation

As per the NCHRP 409 reports the signal retiming in US is followed by every 3 to 5 years. In this study, to verify whether any changes in the signal timing is implemented by the authorities in the earlier collected data in 2016, the pilot survey of the stretch was conducted

on 06/07/2017. This survey is conducted on working day Thursday so that minimum variation in traffic flow is observed. The collected data was thoroughly analysed to check various parameters such as speed, travel time, phase time, phase sequence, delay and signal cycle time. The analysis revealed small variation in space mean speed while phase sequence, phase time and signal cycle time remains unaffected as observed during previous data collection in 2016. The Red Time Countdown Timers (RTCT) was observed working nicely at all approaches of all selected intersections. Traffic police equipped with android mobile application specifically developed for Ahmedabad city traffic police for instantly generating e-challan of traffic rules violators was also present at selected intersections. Required fine tuning of the implementation plan based on the examination of pilot survey data was conducted jointly with all concerned agencies during the weekend time. After the examination of pilot survey data and necessary fine tuning, finally normal working day Tuesday, date 11/07/2017 is selected for the field testing of the developed methodology in this research. For evaluation 108 second signal cycle was selected which is the same cycle length adopted for analytical validation of developed strategy for evening peak time (explained in previous chapter). With the help of signal time controller, finally the proposed signal setting plan (table 8.1) with phase time and phase sequence was set at each control cabin situated at every intersection (Figure 8.3a, b).

Table 8.1: Adopted signal cycle plan (for implementation):

Phase sequence (Clock-wise) & Phase Time for 108 s cycle					
Phase Sequence at A	Phase Time (s) (with amber)	Phase Sequence at B	Phase Time (s) (with amber)	Phase Sequence at C	Phase Time (s) (with amber)
1	30	3	28	1	27
2	24	4	29	2	24
3	27	1	24	3	30
4	27	2	27	4	27

Figure 8.3a & 8.3b: Signal retiming with controller and discussion with on duty traffic police

The inserted signal cycle, phase time and phase sequence was instantly checked by adopting actual run for one cycle so that it can run effectively for selected time period. The whole task was completed before noon and experimental time was selected from 3:00 p.m. to 9:00 p.m. meticulous care was exercised for inserting two phase offsets in signal controller for forward and backward direction, so that from 3:00 p.m. it can work properly. At 3:00 p.m. when system started with developed methodology and plan, full one hour has been devoted to physically verify the timing, sequence and offset with the help of trained enumerators. The one-hour time was also useful to set the existing traffic in new environment. Then after from 4:00 p.m. to 8:00 p.m. required data was collected from newly set system. The same procedure explained in the chapter 5 was followed to collect required information of signal control parameters of the newly set system. Travel time data was also supplemented with manual collection by Licence Plate Method. For rationalization of the travel time data, android application “Speedometer GPS” available in the google play store was used.

8.4 Data collection and analysis

To validate the developed methodology, different Measures of Effectiveness (MOE’s) of the operational quality of signal parameters are derived by analysing collected data. The obtained post implementation MOE’s are compared with the derived pre implementation MOE’s to ascertain the efficacy of the developed methodology. The various MOE’s which are compared in the analysis are

- Stopped time delay
- Corridor travel time

8.4.1 Stopped time delay

Delay is perhaps the most important parameter to measure operational quality of signal coordination as it is directly related with the perception of the driver. Delay is a complex variable which is affected by many parameters including the cycle length. Stopped time delay for the vehicle includes only the time spent stopped at the signal. It begins when vehicle is fully stopped and ends when vehicle start to accelerate. Almost all delay models start with the Webster’s uniform delay model. All delay models perform well when the composition of traffic is homogeneous and flow of traffic is under saturated. Present delay models are incompetent to accurately measure the delay when traffic composition is heterogeneous in nature and traffic flow is at or near saturation level. Traffic signal coordination is justified when the volume to capacity ratio (degree of saturation) is from 0.85

to 1.0 at every intersection. The selected corridor is having mixed traffic condition with near saturation level traffic volume. IRC- the topmost and authentic technical body- responsible for developing the different codes related to traffic and transportation is silent on how to measure delay at coordinated signalized intersections.

The field measurement technique for intersection control delay given in Chapter 31 of HCM 2010 (8) to calculate stopped delay. The Stopped delay was calculated for a lane or a lane group of each approach of the intersections. The stopped delay was used (as traditionally it has been used) in the comparison instead of control delay because it is directly computed from field data and is not affected by adjustments that are applied to get the control delay. The following procedure is adopted to calculate stopped time delay at the coordinated intersections of the selected corridor. Video recording of the pre implementation and post implementation data was replayed several times to collect required information of stopped time delay. The count interval of 15 seconds was selected for stopped delay calculation because it is an integral divisor of the duration of survey period (1 hour) as required by the HCM 2010 procedure (8).

The observed stopped delay on the corridor was calculated on the four different points on the corridor. The figure 8.4 shown below depicts the location of the points on corridor where the delay values are calculated.

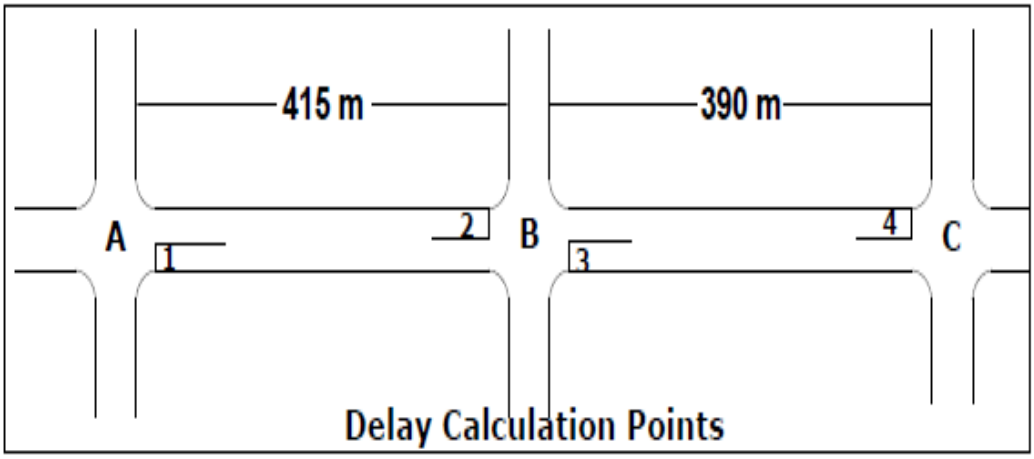


Figure 8.4: Location of observed stopped delay calculation

Evening peak data was analysed for deriving the delay values. Sample delay calculation of the observed stopped delay before implementation for evening peak from intersection B to C on point 4 shown in figure is given here. Table 8.2 represents delay calculation sheet of one link for 10 cycles.

Table 8.2: Calculation sheet of observed stopped delay from B to C at point 4

Cycle No.	Cumulative stopped Vehicles						Cumulative stopped vehicles	Total Stopped Vehicles/cycle	Delay (sec)/Cycle	Delay (sec) /vehicle per Cycle
	0-15 sec	15-30 sec	30-45 sec	45-60 sec	60-75 sec	75-90 sec				
1	8	15	24	33	36	48	116	48	1740	36.25
2	7	14	22	28	32	39	103	39	1545	39.62
3	10	18	24	27	35	43	114	43	1710	39.77
4	8	15	23	29	33	44	108	44	1620	36.82
5	6	14	21	24	29	34	94	34	1410	41.47
6	9	13	19	25	29	33	95	33	1425	43.18
7	11	16	21	29	35	42	112	42	1680	40.00
8	12	22	27	33	38	45	132	45	1980	44.00
9	10	21	30	32	35	40	128	40	1920	48.00
10	9	16	19	25	31	38	100	38	1500	39.47

Only first cycle's first row calculation is presented here:

Total stopped delay = Total waiting time/ Total stopped vehicles

= (Cumulative stopped vehicles x Time interval of counting)/ Total stopped vehicles

= (116 x 15)/48 = 1740/48

= 36.25 sec/vehicle.

Total stopped delay = 36.25 sec/vehicle

Similarly delay calculation from link A to B (pt. 2), B to A (pt. 1), B to C (pt.4) and C to B (pt.3) on the approaches shown in the figure was carried out for ten signal cycles. For all cases based on cumulative stopped vehicles average delay for each vehicle per cycle was calculated. Table 8.3 presents pre implementation delay derived for all links at selected approaches. Table 8.4 shows post implementation delay derived for all links at selected approaches.

Table 8.3: Observed stopped delay in second/veh for ten cycles

Observed Stopped Delay (before implementation-evening peak) in sec/veh					
Cycle No.	Girish to Swastik (Pt. 1)	Swastik to Girish (Pt. 2)	Swagat to Girish (Pt. 3)	Girish to Swagat (Pt. 4)	Total Delay
1	44.2	32.7	41.3	36.3	154.3
2	46.2	36.6	43.6	39.6	166.1
3	49.5	33.4	45.7	39.8	168.4
4	43.9	37.9	39.2	36.8	157.8
5	50.5	45.8	52.5	41.5	190.2
6	48.3	37.5	46.1	43.2	175.0
7	41.3	39.0	37.1	40.0	157.4
8	49.4	39.6	47.8	44.0	180.8
9	46.0	36.8	48.1	48.0	178.8
10	46.9	40.8	46.8	39.5	173.9

Table 8.4: Observed stopped delay in second/veh for ten cycles

Observed Stopped Delay (after implementation-evening peak) in sec/veh					
Cycle No.	Girish to Swastik (Pt. 1)	Swastik to Girish (Pt. 2)	Swagat to Girish (Pt. 3)	Girish to Swagat (Pt. 4)	Total Delay
1	36.6	29.1	30.0	29.0	124.7
2	36.5	32.4	33.1	33.6	135.6
3	34.9	29.6	34.4	34.0	132.9
4	32.9	31.7	32.2	31.3	128.1
5	40.5	34.0	39.5	31.6	145.5
6	32.8	34.7	30.0	38.6	136.2
7	38.8	30.0	26.7	31.9	127.4
8	35.0	38.0	29.5	36.0	138.5
9	33.6	30.8	33.8	36.1	134.3
10	40.2	29.7	32.9	29.4	132.2

After analyzing the table, it is evident that the developed methodology is successful in reducing the stopped delay for vehicles. The remarkable observation during field testing is that the residual queue of the unserved vehicles during green time is disappeared. The clockwise sequence, offset allocation and green time of particular phase were adequate to serve stopped vehicles of red light within allocated green time. Figure 8.5 (a, b, c) shown here reveals traffic congestion and queue spillback condition which was observed during data collection in 2016.

Figure 8.5 (a, b, c): Observed vehicle queue due to absence of coordination.

The figure 8.6 (a to d) shows improved traffic condition at the central Girish intersection (B) observed during field testing of the developed methodology. From this figure it can be seen that the residual queue was not there as all the vehicles are served during the allocated green time of each phase at all approaches.

Figure 8.6(a to d): Improvement at girish intersection (B) after coordination among all four approaches.

Comparison of the stopped delay values of the all four links from A to B, B to A, B to C and C to B is represented by the graph shown in figure 8.7 (a to d), whereas figure 8.8 gives graphical comparison of combined stopped delay values of before and after implementation along the corridor.

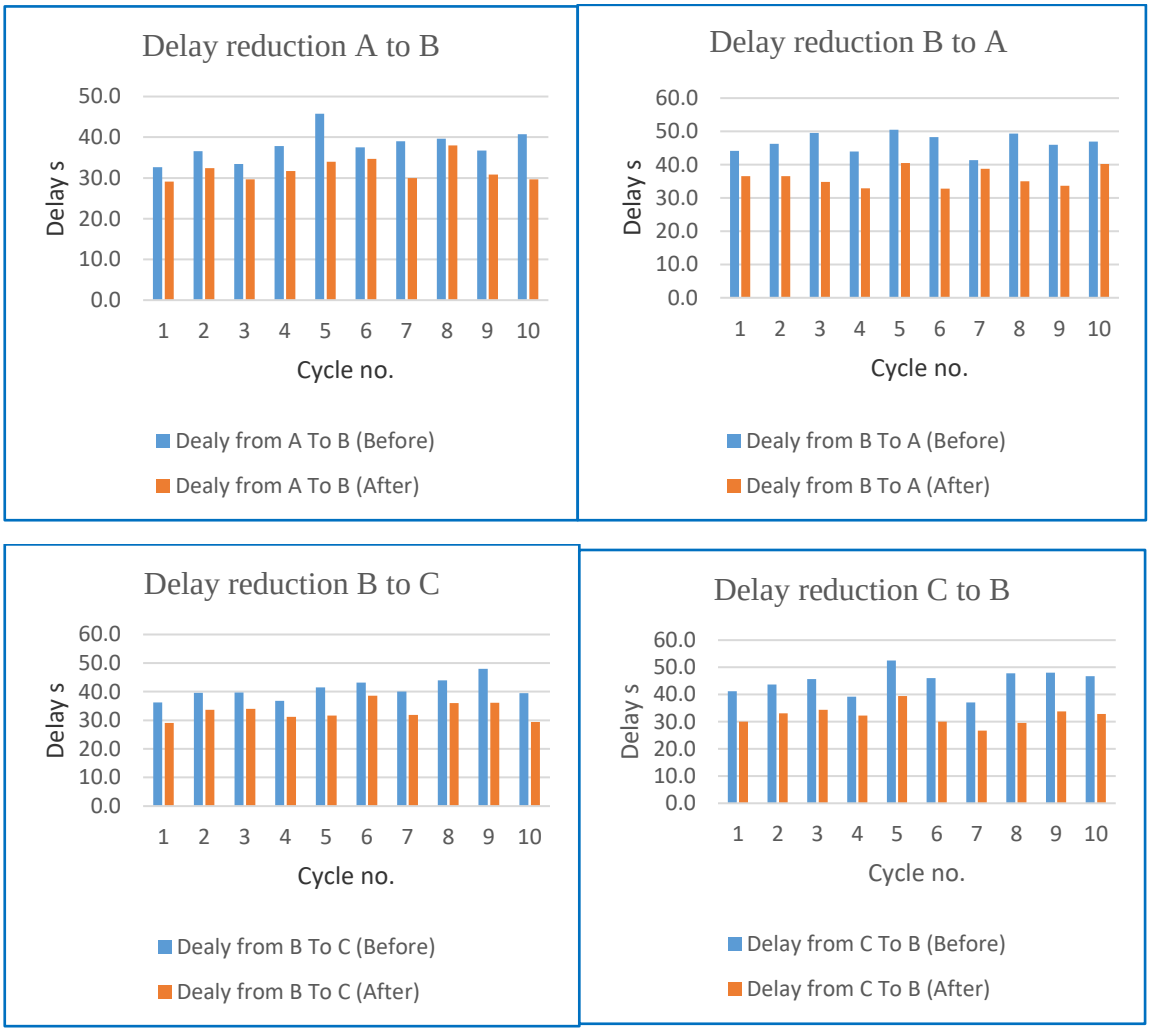


Figure 8.7 (a to d): Stopped delay reduction at individual link (both directions)

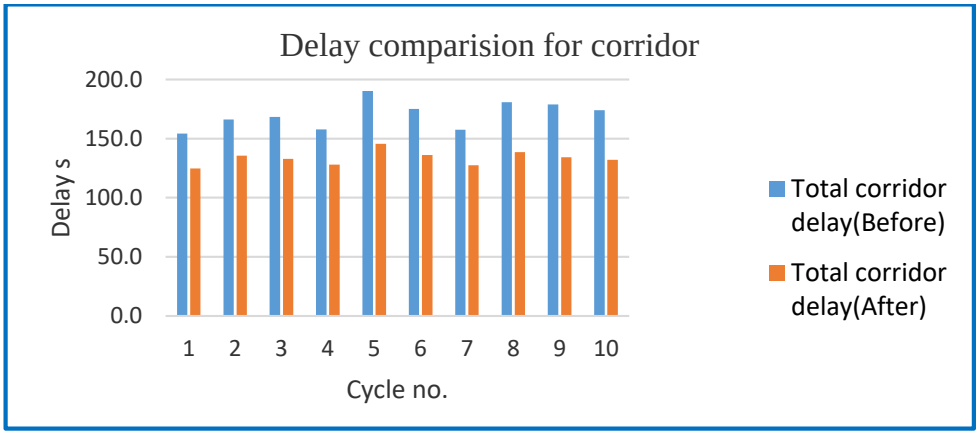


Figure 8.8: Total stopped delay reduction at corridor

From this graph it is evident that implemented signal cycle time, phase time and phase sequence is capable to reduce delay in all four links of the selected corridor. For two-way coordination, reduction in total corridor delay was observed. For evening peak period reduction in total stopped delay was observed from 1702.7 second to 1335.4 second which represents 21.5% delay reduction. Table 8.5 represents comparison.

Table 8.5: Comparison of observed stopped delay in second for ten cycles

Cycle No.	Total corridor delay(Before)	Total corridor delay(After)
1	154.3	124.7
2	166.1	135.6
3	168.4	132.9
4	157.8	128.1
5	190.2	145.5
6	175.0	136.2
7	157.4	127.4
8	180.8	138.5
9	178.8	134.3
10	173.9	132.2
Total	**1702.7**	**1335.4**

The reduction in stopped delay values after experiment was observed on the field during analysis period. Delay and queue length are considered as a significant parameter for inspection of the operation quality of the signal coordination strategy. Delay is a measure that most directly relates driver's experience and it is measure of excess time consumed in traversing the intersection. It is the most frequently used measure of effectiveness for signalized intersections as it is directly perceived by a driver (Mathew, 2014). It depends largely on the width of roadway, traffic composition, geometry of the corridor, traffic volume, type of signal control and environmental condition. The implementation and data collection were carried out in sunny day and to derive methodology uniform geometry throughout the corridor was assumed. Width of roadway before implementation and during implementation was remaining unchanged. Hereafter to validate the delay reduction, traffic composition and vehicular volume before testing and during testing was derived for evening peak period. The four locations depicted in figure 8.3 on link A to B, B to A, B to C and C to B on the approaches of corridor were selected to receive necessary evidence for ten signal cycles. Videography data was extracted to acquire required information of volume count and it is converted into derived DPCU values and IRC SPCU values. Figure 8.9a and 8.9b represent traffic composition values of before and during implementation collected for 10

cycles of evening peak period. Table 8.6 discloses traffic volume before and during implementation.

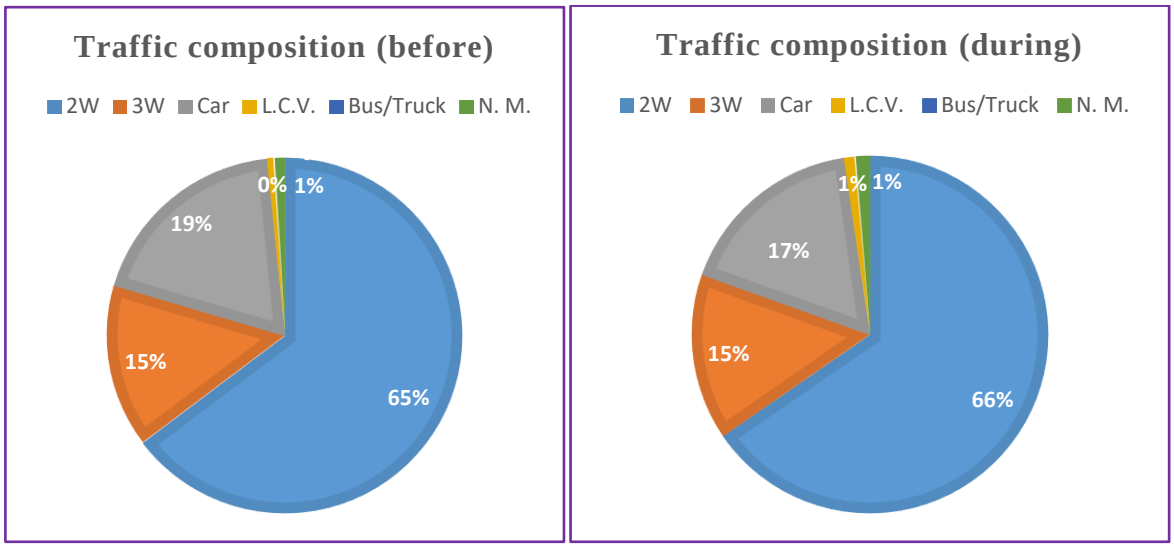


Figure 8.9a & 8.9b: Traffic composition on corridor

Table 8.6: Comparison of traffic volume for ten cycles (Evening peak)

Location	Before and During Implementation			Before and During Implementation		
	Volume in DPCU (Before)	Volume in DPCU (During)	% Difference	Volume in IRC SPCU (Before)	Volume in IRC SPCU (During)	% Difference
Girish to Swastik (Pt.1)	425.88	450	5.66	656	682.5	4.04
Swastik to Girish (Pt.2)	449.36	451.53	0.48	669.5	677	1.12
Swagat to Girish (Pt.3)	449.83	461.07	2.50	653.5	681.5	4.28
Girish to Swagat (Pt.4)	428.97	430.29	0.31	651	657.5	1.00
Total Volume on Corridor	1754.04	1792.89	2.21	2630	2698.5	2.60

Detecting to the pie chart of the traffic composition on the corridor, it clearly explicates that the traffic composition observed through the data collection and at the time of implementation witnessing the similar trend of the vehicular mix. At both the time of analysis more than 97% of the traffic mix constitute car, three wheelers and two wheelers. Percentage difference for the category wise vehicle is also only 1 to 3 percentages which is negligible. Similarly, traffic volume before implementation and after implementation has not

reported drastic changes and in fact exposes 2.21% increase in the volume during testing period by derived DPCU values.

8.4.2 Statistical analysis

The analysis of the traffic volume data before implementation and after implementation is checked by statistical validity of the derived volume. The paired sample t-test, also called the dependent sample t-test, is a statistical procedure used to compare the null hypothesis whether the mean difference between two sets of observations is zero. In a paired t-test, each variable is measured twice, resulting in pairs of observations. It is used when there is a natural pairing of observations in the samples, such as when a sample group is tested twice- before and after an experiment. The test is performed to determine whether traffic volume which are observed before implementation and traffic volume observed after implementation are likely to come distribution of equal traffic volume. For small sample size (N < 30) the normal approximation are no longer valid. The test is particularly useful when number of observations is less than 30. Microsoft Excel 2016 is used to perform the test at 95% confidence interval. The result derived from the test is presented in the table 7.7

Table 8.7: Paired T- test outcome

	Volume by DPCU (Before)	*Volume by DPCU (During)*
Mean	43.7305	43.9695
Variance	6.406089211	7.132573421
Observations	20	20
Pearson Correlation	0.847211069	
Hypothesized Mean Difference	0	
df	19	
t Stat	-0.740203627	
P(T<=t) one-tail	0.234112649	
t Critical one-tail	1.729132812	
P(T<=t) two-tail	0.468225299	
t Critical two-tail	2.093024054	

The test result indicates pearson correlation factor 0.84 which implies statical similarity among the traffic volume derived before testing and after testing. For one- sided test on the null hypothesis of “no increase” is verified in the above statically obtained result. The P < 0.05 (95% cofidence level) the value obtained by test result for degree of freedom 19 for t

critical one-tail is 1.72 is higher than the 0.05. The value falls between the critical values of t- distribution of two- tail 0.46 and 2.09. Given that this probability is greater than 0.05, the null hypothesis "H_0" (no statistical difference between the observed traffic volume before testing and during testing) is not rejected and there is not enough statistical evidence to demonstrate difference between field traffic volume before and during testing; the *P* value for the test is 0.23.

8.4.3 Regression analysis of observed stopped delay (Before implementation)

As delay calculated by time space diagram does not include the effect of road geometry (approach width) and vehicular flow values (arrival flow rate) which are responsible for actual delay, it is necessary to estimate actual stopped delay of vehicles in second per vehicle. For this purpose actual observed stopped delay values are measured for the existing signal settings (before implementation) and for the suggested coordinated signal settings (after implementation). These observed stopped delay values are regressed with width of approach, observed traffic flow and time space diagram delay values. This may be helpful in estimating stopped delay in sec/veh directly from approach width, flow values and time space diagram delay.

The regression analysis of the observed stopped delay before implementation was executed by applying Microsoft Excel. Observed stopped delay is a dependent variable that can be predicted by independent variables like, time space diagram delay of Auto CAD (which includes the effects of signal setting parameters; cycle length, phase length, phase sequence, phase offset), observed flow values (which include the effect of average speed of traffic) and width of approach (which includes the effect of road geometry). Relation between observed stopped delay and other variables for corridor is derived as per following equation 8.1

$$y = -8.32\,(x_1) + 0.21\,(x_2) + 0.025\,(x_3) + 108.92 \quad \text{..... (8.1)}$$

Where,

y = Observed stopped delay in sec/veh,

x_1 = Width of approach in m,

x_2 = Traffic volume in DPCU,

x_3 = Calculated time space diagram delay in sec.

The following table 8.8 presents multiplying values of different variables and table 8.9 gives derived regression statistics with Co-efficient of determination $r^2 = 0.66$

Table 8.8: Co-efficient of determination for corridor (before implementation)

	Coefficients	*Standard Error*	*t Stat*	*P-value*
Intercept	108.9263731	35.14671488	3.099190733	0.003754603
Width of Approach	-8.326775277	4.106124329	-2.027891659	0.050021671
Volume by DPCU	0.21753476	0.339112264	0.641483022	0.525272385
Time Space Delay	0.025056731	0.026067949	0.961208377	0.342859436

Table 8.9: Regression statistics for corridor (before implementation)

Regression Statistics	
Multiple R	0.80604596
R Square	0.66487332
Adjusted R square	0.59527943
Standard Error	4.70118523
Observations	40

8.4.4 Regression analysis of observed stopped delay (after implementation)

The regression analysis of the observed stopped delay after implementation was executed by applying Microsoft Excel. Observed stopped delay is a dependent variable that can be predicted by independent variables like time space diagram delay of Auto CAD, observed flow values and width of approach as mentioned above. Relation between observed delay and other variables for corridor is obtained as per following equation 8.2

$$y = -7.37\,(x_1) + 0.19\,(x_2) + 0.10\,(x_3) + 108.41 \quad \text{..... (8.2)}$$

Where,

y = Observed stopped delay in sec,

x_1 = Width of approach in m,

x_2 = Traffic volume in DPCU,

x_3 = Calculated time space diagram delay in sec,

The following table 8.10 presents multiplying values of different variables and table 8.11 gives derived regression statistics with Co-efficient of determination $r^2 = 0.73$

Table 8.10: Co-efficient of determination for corridor (after implementation)

	Coefficients	*Standard Error*	*t Stat*	*P-value*
Intercept	108.4124514	21.62605844	5.013047185	1.44366E-05
Width of Approach	-7.375721873	2.342060998	-3.14924414	0.003285679
Traffic Volume by DPCU	0.19285065	0.15390149	-1.253078513	0.218255231
Time Space Delay	0.100602936	0.104582483	0.961948244	0.342492724

Table 8.11: Regression statistics for corridor (after implementation)

Regression Statistics	
Multiple R	0.85083272
R Square	0.7383965
Adjusted R Square	0.69659621
Standard Error	3.04649272
Observations	40

It is observed that additive constant, multiplying constant of x1 and x2 are found almost identical for before and after implementation. The value of multiplying constant x3 for before implemetation is lower than after implementation values because calculated time space diagram values for before implementation are very high, whereas these are quite lower for after implemtation situation. It is also implied that stopped delay is reduced with increase in approach width and it increases with increase in flow values.

8.4.5 Corridor travel time

The MOEs adopted for operational quality of the signal coordination like observed delay, stops and residual queues, which is limited to the performance and traffic state before the stop line. To overcome this limitation, travel time is another important measure, which can reflect the traffic performance beyond the stop line and the system as a whole. Therefore, in order to check the efficiency and adaptability of coordinated signal systems, travel time, delay, and queue length should be comprehensively considered in two-way signal coordination. The travel time after implementation was collected by two different techniques.

License plate matching was used as early as the 1950s for travel time studies which relied on observers to note the license plates of passing vehicles and corresponding times on paper or into a tape recorder. License plates were manually matched later in the office, and travel

times computed. The survey was conducted manually and registration number plates are matched later for deriving travel time for each category of the vehicle i.e. car, three wheelers and two wheelers. The survey was conducted from A to C direction and from C to A direction. During survey for the stipulated time period, only one category of vehicles is recorded at a time in both forward and backward direction. Coordination among the enumerators was established by mobile phone to locate as many vehicles possible to locate in either direction. This technique results in less manual error and easy data reduction. Table 8.12 presents observed travel time of different categories of vehicles during pilot survey before field testing and table 8.13 represents same values during field testing.

Table 8.12: Observed travel time difference before and after implementation (forward direction)

Vehicle Type	From Swastik(A) to Swagat (C) (Before)			From Swastik(A) to Swagat (C) (After)			Reduction in Travel Time (HH:MM:SS)
	Sample size	Total elapsed time	Average Travel Time	Sample size	Total elapsed time	Average Travel Time	
Car	24	1:29:05	00:03:43	27	01:11:15	00:02:38	00:01:04
Three wheeler	34	1:48:15	00:03:11	32	01:14:32	00:02:20	00:00:51
Two wheeler	32	1:33:22	00:02:55	34	01:13:12	00:02:09	00:00:46
Total Sum	90	4:50:42	0:09:49	93	3:38:59	0:07:07	0:02:42

Table 8.13: Observed travel time difference before and after implementation (backward direction)

Vehicle Type	From Swagat(C) to Swastik (A) (Before)			From Swagat(C) to Swastik (A) (After)			Reduction in Travel Time (HH:MM:SS)
	Sample size	Total elapsed time	Average Travel Time	Sample size	Total elapsed time	Average Travel Time	
Car	27	01:40:32	00:03:43	29	01:12:51	00:02:31	00:01:13
Three wheeler	29	01:30:08	00:03:06	29	01:03:04	00:02:10	00:00:56
Two wheeler	30	01:22:29	00:02:45	31	01:01:39	00:01:59	00:00:46
Total Sum	86	4:33:09	0:09:35	89	3:17:34	0:06:41	0:02:54

The tables exhibit reduction in travel time from direction Swastik intersection (A) to Swagat intersection (C) and in reverse direction i.e. Swagat intersection (C) to Swastik intersection (A). The reduction in travel time is observed in both direction with combined reduction of

travel time in forward direction for all traffic stream is 27.5% while it is 30.26% in backward direction However, the observation reveals comparatively less travel time is required to negotiate the stretch in backward direction compared to forward direction. As described in chapter 5 and 6 the travel time required to cover the corridor as per obtained SMS in forward direction i.e. from A to C is 100 second, compared to 87 second for C to A. Though geometry and the width of the road in both directions are more or less analogous in nature, presence of Municipal market, Bank and renowned brand showroom on the direction from A to C might have the impact on the speed of the traffic flow which resulted in excess travel time for traversing common distance.

Alternative method which was used to check the average travel time in both outbound and inbound direction is by applying open source android mobile application *"Speedometer GPS"* available in the Google play store. The application is connected with real time GPS data to track the vehicle speed, distance, time, location and can also get start time, time elapsed, average speed, maximum speed, altitude among others. The figures 8.10a, 8.10b and 8.11 represent the photographic images of travel time data collection process employed during field testing.

Figure 8.10a & 8.10b: Data collection for actual travel time

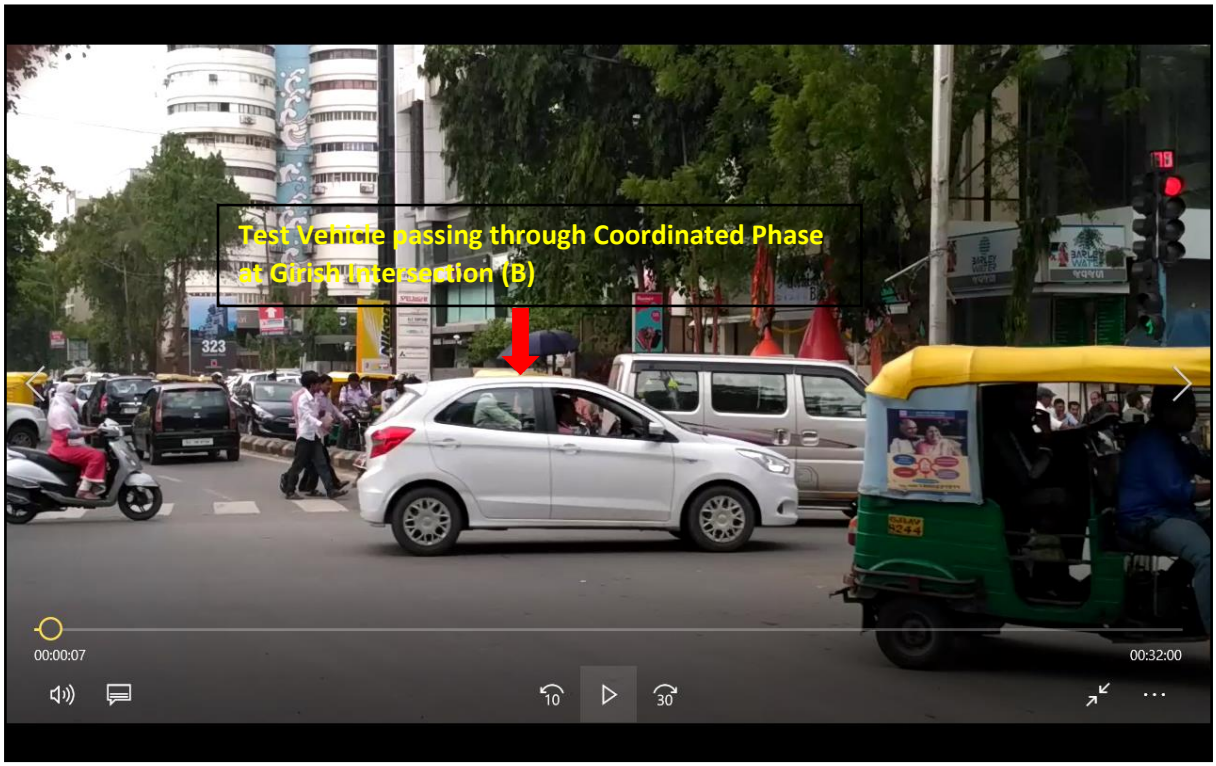


Figure 8.11: Data collection for actual travel time by GJ 31A 0965

The information collected of the travel time using floating car method by pre- timed control is more accurate as shown by Kringer (2013). For the purpose of collecting required data, available car of researcher, available two wheelers of on duty traffic police and hired auto were used. Starting from left to right the figure 8.12 reveals snap shot of sample result for two wheelers, three wheelers and car observed at the time of experiment.

Figure 8.12: **Observed travel speed by test vehicle during testing (evening peak).**

Bottom most portion of the figure 8.12 exhibits plot of the actual vehicle travel speed at the time of testing. Analysis of the figures expose that at the middle intersection B, the speed of the vehicles reduces but still all three categories of vehicles can cross the signalized intersection by maintaining the speed @ 20 km/h. It demonstrates that applied signal cycle plan with the developed phase plan and phase sequence is capable to coordinate the corridor for continuous travel of the vehicles. During the analysis period for the test case, strategy was able to provide 24 km/h average speed for traffic mix as depicted in the figure 8.12. Travel time reduction for car, three wheelers and two wheelers was observed by coordination of signals along the corridor. As per the report of the AMC submitted for smart city proposal the average traffic speed in the city is only 9-17 kmph. The improvement in the speed is observed during data collection. The analysis of the travel time data collected from the two different methods reveals that implemented methodology can reduce the travel time of the different categories of vehicles.

The figures 8.13 a and 8.13 b demostrate trajectories of individual vehicles observed before and after implementation in both downstream and upstream directions. Travel time reduction in both inbound and outbound direction is clearly visible in the vehicle tracking graph for all three categories of vehicles. For purpose of better visulization start time of all categories of vehicles are kept with minor variations.

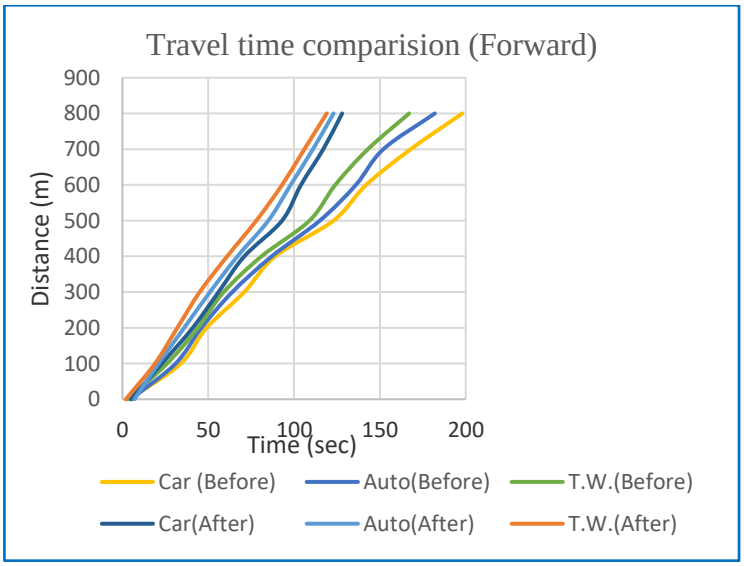


Figure 8.13(a)

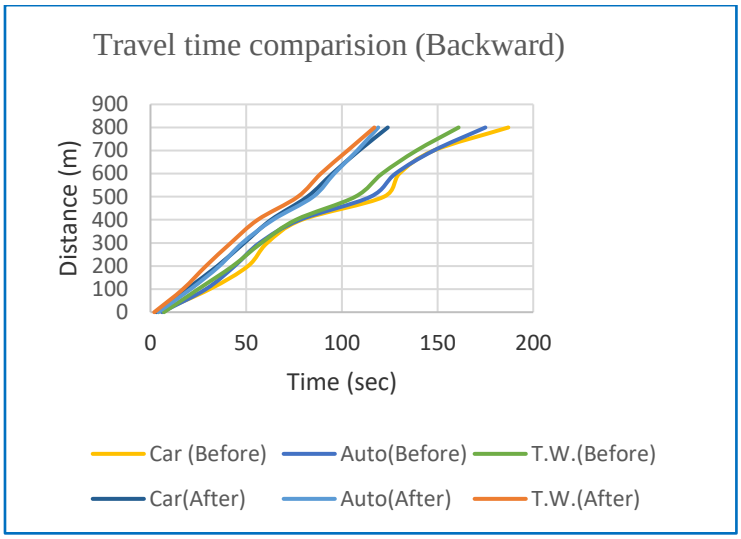


Figure 8.13(b)

Figure 8.13a & b: Time space diagram of vehicles (before and after implementation)

8.5 Summary

In this chapter, study area selection for actual implementation of the derived methodology was described. Different operational quality parameters of the signal coordination for before implementation and after implementation were derived and compared to verify developed methodology. Traffic volume and travel time values of pre implementation and post implementation were derived and compared to check improvement after implementation. Stastistical similarity by t-test was cofirmed for traffic volume, before and after implementation. Regression analysis was performed to develop equation for calculation of observed stopped delay values. Improvement in traveltime was confirmed through licence plate method and "Spedometer GPS". Next chapter summarizes conclusions derived by the developed methodology and models. At the end of next chapter limitations and future work required for generalization of developed model was presented.

CHAPTER: 9
Conclusions

9.1 Contribution of Research

The research has proposed robust technique for two-way coordination of the traffic signal along urban road corridor considering peculiar traffic condition prevailing in India having Right Hand Drive (RHD) with Left Hand Traffic (LHT). The complex nature of traffic flow dynamics, coupled with behavioural discrepancies among driving populations, has made this critical yet classical subject remain a challenging issue, and the challenge becomes insurmountable when traffic composition is of heterogeneous in nature. It is very difficult for any signal optimization software to simultaneously optimize four signal control parameters (cycle length, green split, phase sequence and phase length) in saturated heterogeneous traffic condition. This research has attempted to simultaneously optimize all four signal control parameters.

The current research has developed methodology, phase plan and phase sequence for 3 arm, 4 arm and 5 arm intersections to provide two-way coordination with all possible combinations. After scrutinizing numerous permutation and combination of phase plan, phase sequence and phase group on graph paper nine (five for 4 arm intersection and two each for 3 arm and 5 arm intersection) different phase plans have been prepared to optimize the phase sequence and phase grouping of the signal control. Considering equal travel time in both directions and equal phase length at all approaches, strategy is fixed for two-way coordination of pre-timed signal for 3 arm, 4 arm and 5 arm intersections. The strategy is developed for even phase difference condition (when, $tt \geq 2\ g_{min}$) and odd phase difference condition (when, $g_{min} \leq tt < 2\ g_{min}$). To calculate average delay by time space diagram, a methodology is developed for six different conditions. For deriving cycle length, the most important signal control parameter, the research has given two different models (TW_TSCS1 and TW_TSCS2) considering significant parameters which affect the cycle time; such as link length, travel time, saturation flow rate (SFR), demand flow rate (DFR) and width of approach. “For optimal use of available green time traffic flow at signalized intersection must be sustained at or near saturation level”, considering this premise the research has proposed method of deriving cycle time and g/c ratio depending on SFR, DFR

and approach width of particular approach of intersection. To cater the difficulty of measuring SFR for every approach, after exploring past literature particularly for Indian condition, the research has given two different conditions of “high” SFR and ‘low” SFR along with graphs to find SFR of the approach. “Coordination ability” factor proposed in the research is a vital tool for decision makers whether to opt for signal coordination or continue without it.

For two-way coordination, many researchers considered equal travel time of traffic platoon in forward and backward direction. In current research by relaxing this assumption, computational model is developed which gives optimum cycle time for different travel time and different phase time situation for two-way coordination along the corridor. The model is capable to provide coordination when condition demands single cycle or double cycle at any intersection of corridor due to demand variation. This model is first validated through hypothetical data for two different cases than after field validation by actual implementation of the developed model is performed on the selected corridor of Ahmedabad city. To solve the problem of total corridor delay minimization when travel time in both directions is varying and phase time is different. Genetic Algorithm (GA) optimization technique is used to obtain optimum cycle length and phase length which can optimize offset, gives wider bandwidth, thus minimizes overall corridor delay. Computer program in C language is developed which can be used to find best possible signal cycle plan, phase sequence and phase length for effective two- way coordination with minimum delay along the corridor. Output in Microsoft Excel is useful for analysis of all five alternatives of signal optimization.

9.2 Conclusions

- Clock wise movement of phase is beneficial for reduction in delay along corridor as well as equitable distribution of delay across the cycles, thus eliminating the possibilities of individual cycle failure.
- Pre-timed traffic signal coordination in two-way direction can be easily obtained simply by calculating travel time between links and adjusting phase sequence and phase plan developed in the TW_TSCS1 for 3arm, 4 arm and 5 arm intersections.
- As per IRC criteria of minimum green time at any phase of signalized intersection, possibility of odd phase difference condition (when, $g_{min} \leq tt < 2\ g_{min}$) for Indian

urban traffic is minimal, although developed tables are useful for two-way coordination in such odd situation.

- The calculation of delay in existing situation (designed by authority) for noon peak is carried out up to 10 cycles to replicate near repetition condition of cyclic pattern for delay. It endorses that developed phase plan and phase sequence of this study are successful in minimizing delay. The equitable distribution of delay among cycles by applying developed phase plan and phase sequence is capable to minimize the possibility of individual cycle failure and queue spillback condition at shorter links of the corridor.
- The DFR satisfaction rate of individual approach is govern by g/c ratio of the particular approach. For major and minor approaches, considering minimum green criteria given by IRC on minor approach the proportionate increase in g/c ratio of major approach is reducing sharply after 160 s cycle length which is reflected in developed graph. So, it is concluded that longer cycle length is not advisable for two-way signal coordination.
- Longer cycle length will increase the lost time at the approaches as well as leads to underutilization of the allotted green time, as coordination is justified for nearly saturated condition. Therefore, longer cycle length (more than 160 s) is not appropriate for two-way coordination.

- Validation efforts of developed TW_TSCS1 (for equal demand and equal travel time in both directions) on existing data reveals that from developed Delay Minimization Scheme (DMS), DMS 3 reduces 76% delay for noon peak and 79% delay for evening peak. Thus, it can be concluded that common cycle length with equal phase length at all signalized intersections of corridor may be selected for two-way signal coordination when demand at all approaches and travel time in both directions is nearly identical.
- Validation efforts for equal demand and variable travel time unveils that from proposed five DMS, DMS 5 is giving the minimum overall corridor delay. DMS 5 is capable to reduce delay by more than 70% for noon peak, evening peak and for no GA condition. So, it can be concluded that longest cycle length calculated by developed rule of the model along the corridor in general may be selected for two-way signal coordination.

- Validation efforts for GA based approach (for variable demand and variable travel time) exhibits that Genetic Algorithm (GA) based optimization technique outperform the traditional developed DMS. When DFR of major approach and minor approach is varying, the GA gives optimal phase length which gives wider bandwidth, thus minimizes overall corridor delay. By allocating wider bandwidth to through traffic while maintaining required green for minor approaches as per demand, GA is successful in further reduction of overall corridor delay by 29%.

- The developed phase plan, phase sequence and methodology (TW_TSCS1) are applied actually on the corridor of Ahmedabad city through changing the signal settings in the controller of three pre-timed signalized intersections. The analysis of post implementation on evening peak data reveals considerable improvement in existing condition with reduction in stopped delay is observed in both conditions. For nearly equal volume and composition of traffic, implementation of developed methodology is capable to reduce total combined delay on the corridor by 21.5%. It confirms applicability of developed methodology and model in field as well.
- Field implementation of the developed methodology reveals meaningful improvement in the travel time, the most visible and important parameter for judging operational quality of two-way signal coordination of traffic stream along the corridor in both inbound and outbound directions. The combined reduction of travel time in forward direction for the traffic stream is 27.5% whereas it is 30.26% in backward direction (measured by traditional license plate method). It is confirmed with most advanced “Speedometer GPS” application. It also endorses successful applicability of developed methodology and model in field as well.
- Using the static PCU values as per IRC on Vijay cross road intersection near the selected corridor, exhibits minimum SFR 637 pcu/hr/m on University approach and maximum 1110 pcu/hr/m on Commerce college approach. Thus, the value suggested by Webster SFR (525 pcu/hr/m for approach width more than 5.5m) is no longer valid for Indian traffic condition and confirm applicability of developed “low” and “high” SFR value graphs for Indian condition.
- The DPCU values at signalized intersection were derived by considering travel time (which includes the effect of geometry of intersection, acceleration and deceleration of vehicle) and projected area of vehicle with respect to car. The DPCU of two wheeler is obtained around 0.23 and for three wheeler it is around 0.60

- The Indian traffic mix has 60% to 70% combined composition of two wheelers and three wheelers. The notable variation obtained in developed DPCU values and IRC SPCU values particularly for three wheelers and two wheelers lead to conclude that for two-way traffic signal coordination, derived DPCU values should be used for computing volume.
- For the implementation of developed methodology of two-way coordination, there is no need of extra cost of soft-wares, sensors, technical manpower, huge data collection; it's processing and extensive computational efforts. This methodology and model can be easily adoptable to Indian mixed traffic conditions.
- Field implementation experience of the developed methodology suggest that there should be improved understanding and coordination is required among traffic police department, traffic branch of municipal corporation and agency responsible for maintenance and adjustment of traffic signal to achieve goal of two-way traffic signal coordination.
- The road traffic condition and behaviour of road user of every country is unique in nature, so recently published Indian Highway Capacity Manual (Indo-HCM) will prove as an indispensable resource for proper planning, design and operation of road traffic facilities in the country.
- The management of traffic control in urban area may be effectively and successfully implemented by single nodal agency.

9.3 Future scope

- Proposed phase plan and phase offset not only useful for corridor optimization but also it has potential application to cater the need of network level coordination. Further study to validate the method on the network may be carried out.
- The emphasis of the research is on four arm intersection. The methodology of two-way signal coordination developed for 3 arm and 5 arm intersection in the research can be extended and model for these cases may be developed.
- Presently on the selected corridor signals are running on the implemented phase plan, phase sequence and optimized cycle length. Further field implementation on the identical corridor will be helpful to ascertain sensitivity of the developed model.

9.4 Concluding Remark

No 'miracle' solution exists that applies in every situation, but applying a combination of measures to reduce demand for road transport and increase the supply of road infrastructure may significantly mitigate the congestion problem. With this thesis contributed to mitigation and the optimization of traffic signal control at intersections, which fits in the category "traffic management and control"; note that this topic mainly affects traffic in urban areas (and not highway traffic). An advantage of improving traffic signal control is that it requires no additional space to increase the capacity of road infrastructure. Since it requires no modification of the current infrastructure, it may also be a financially attractive solution.

Glossary of terms (Appendix-I)

Arterial – Signalized streets that serve primarily through traffic and provide access to abutting properties as a secondary function, having signal spacing's of 2 miles or less and turn movements at intersections that usually do not exceed 20 percent of total traffic.

Arterial segment – A one-way length of arterial from one signal to the next, including the downstream signalized intersection but not the upstream signalized intersection.

Average approach delay – Average stopped-time delay at a signalized intersection plus average time lost because of deceleration to and acceleration from a stop, generally estimated as 1.3 times the average stopped time delay.

Average running speed – The average speed of a traffic stream computed as the length of a highway segment divided by the average running time of vehicles traversing the segment, in kilometres per hour.

Average running time – The average time vehicles are in motion while traversing a highway segment of given length, excluding stopped-time delay, in seconds per vehicle or minutes per vehicle.

Average stopped-time delay – The total time vehicles are stopped in an intersection approach or lane group during a specified time interval divided by the volume departing from the approach or lane group during the same time period, in seconds per vehicle.

Average total delay – The total additional travel time experienced by drivers, passengers, or pedestrians as a result of control measures and interaction with other users of the facility divided by the volume departing from the corresponding cross section of the facility.

Average travel speed – The average speed of a traffic stream computed as the length of a highway segment divided by the average travel time of vehicles traversing the segment, in kilometres per hour.

Average travel time – The average time spent by vehicles traversing a highway segment of given length, including all stopped-time delay, in seconds per vehicle or minutes per vehicle.

Capacity – The maximum rate of flow at which persons or vehicles can be reasonably expected to traverse a point or uniform segment of a lane or roadway during a specified time period under prevailing roadway, traffic, and control conditions, usually expressed as vehicles per hour or persons per hour.

Critical density – The density at which capacity occurs for a given facility, usually expressed as vehicles per kilometre per lane.

Critical speed – The speed at which capacity occurs for a given facility, usually expressed as kilometres per hour.

Critical *v/c* ratio – The proportion of available intersection capacity used by vehicles in critical lane groups.

Cycle – Any complete sequence of signal indications.

Cycle length – The total time for a signal to complete one cycle.

Delay – Additional travel time experienced by a driver, passenger, or pedestrian beyond what would reasonably be desired for a given trip.

Demand volume – The traffic volume expected to desire service past a point or segment of the highway system at some future time, or the traffic currently arriving or desiring service past such a point, usually expressed as vehicles per hour.

Density – The number of vehicles occupying a given length of lane or roadway averaged over time, usually expressed as vehicles per kilometre or vehicles per kilometre per lane.

Effective green time – The time allocated for a given traffic movement (green plus yellow) at a signalized intersection less the start-up and clearance lost times for the movement.

Effective red time – The time during which a given traffic movement or set of movements is directed to stop; cycle length minus effective green times.

Free-flow speed – (1) the theoretical speed of traffic when density is zero, that is, when no vehicles are present; (2) the average speed of vehicles over an arterial segment not close to signalized intersections under conditions of low volume.

Green ratio – The ratio of the effective green time for a given movement at a signalized intersection to the cycle length.

Green time – The actual length of the green indication for a given movement at a signalized intersection.

Headway – The time between two successive vehicles in a traffic lane as they pass a point on the roadway, measured from front bumper to front bumper, in seconds.

Interrupted flow – A category of traffic facilities having traffic signals, STOP signs, or other fixed causes of periodic delay or interruption to the traffic stream; examples include intersections and arterials.

Interval – A period of time in a signal cycle during which all signal indications remain constant.

Jam density – The density at which congestion becomes so severe that all movement of persons or vehicles stops, usually expressed as vehicles per kilometre (per lane) or pedestrians per square meters.

Lane group – A set of lanes on an intersection approach that has been established for separate capacity and level-of-service analysis.

Level of service – A qualitative measure describing operational conditions within a traffic stream, generally described in terms of such factors as speed and travel time, freedom to manoeuvre, traffic interruptions, comfort and convenience, and safety.

Measures of effectiveness – Parameters describing the quality of service provided by a traffic facility to drivers, passengers, or pedestrians; examples include speed, density, delay, and similar measures.

Platoon – A group of vehicles or pedestrians traveling together as a group, either voluntarily or involuntarily because of signal control, geometrics, or other factors.

Platoon flow rate – The rate of flow of vehicles or pedestrians within a platoon.

Queue – A line of vehicles or persons waiting to be served by the system in which the rate of flow from the front of the queue determines the average speed within the queue. Slowly moving vehicles or people joining the rear of the queue are usually considered a part of the queue. The internal queue dynamics may involve a series of starts and stops. A faster moving line of vehicles is often referred to as a *moving queue* or a *platoon*.

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

Saturation flow rate – The equivalent hourly rate at which vehicles can traverse an intersection approach under prevailing conditions, assuming that the green signal is available at all times and no lost times are experienced, in vehicles per hour of green or vehicles per hour of green per lane.

Space mean speed – The average speed of the traffic stream computed as the length of the highway segment divided by the average travel time of vehicles to traverse the segment; average travel speed; in kilometres per hour.

Spacing – The distance between two successive vehicles in a traffic lane measured from front bumper to front bumper, in meters.

Speed – A rate of motion expressed as distance per unit time.

Time mean speed – The arithmetic average of individual vehicle speeds passing a point on a roadway or lane, in kilometres per hour.

Uninterrupted flow – A category of facilities having no fixed causes of delay or interruption external to the traffic stream; examples of such facilities include freeways and unsignalized sections of multilane and two-lane rural highways.

***v/c* ratio** – The ratio of demand flow rate to capacity for a traffic facility.

Volume – The number of persons or vehicles passing a point on a lane, roadway, or other traffic way during some time interval, often taken to be 1 hr, expressed in vehicles

Delay calculation methodology (Appendix-II)

Delay calculation is performed in forward and backward direction with Microsoft Excel software as well as by applying Time space diagram in Auto Cad software. The slope of bandwidth of any movement depend on travel speed in particular direction considering this six cases for delay calculation is obtained.

Case I Right turning (R.T.) or straight moving (S.M.) movement band of upstream intersection intersect above or below the available green band of downstream intersection.

Case II Bottom line of R.T. or S.M. Movement band of upstream intersection intersect within the available green band of downstream intersection.

Case III Bottom line of R.T. or S.M. Movement band of upstream intersection falls below the available green band of downstream intersection while top line is within green band.

Case IV Bottom line of R.T. or S.M. Movement band of upstream intersection falls below the available green band of downstream intersection while top line is above green band.

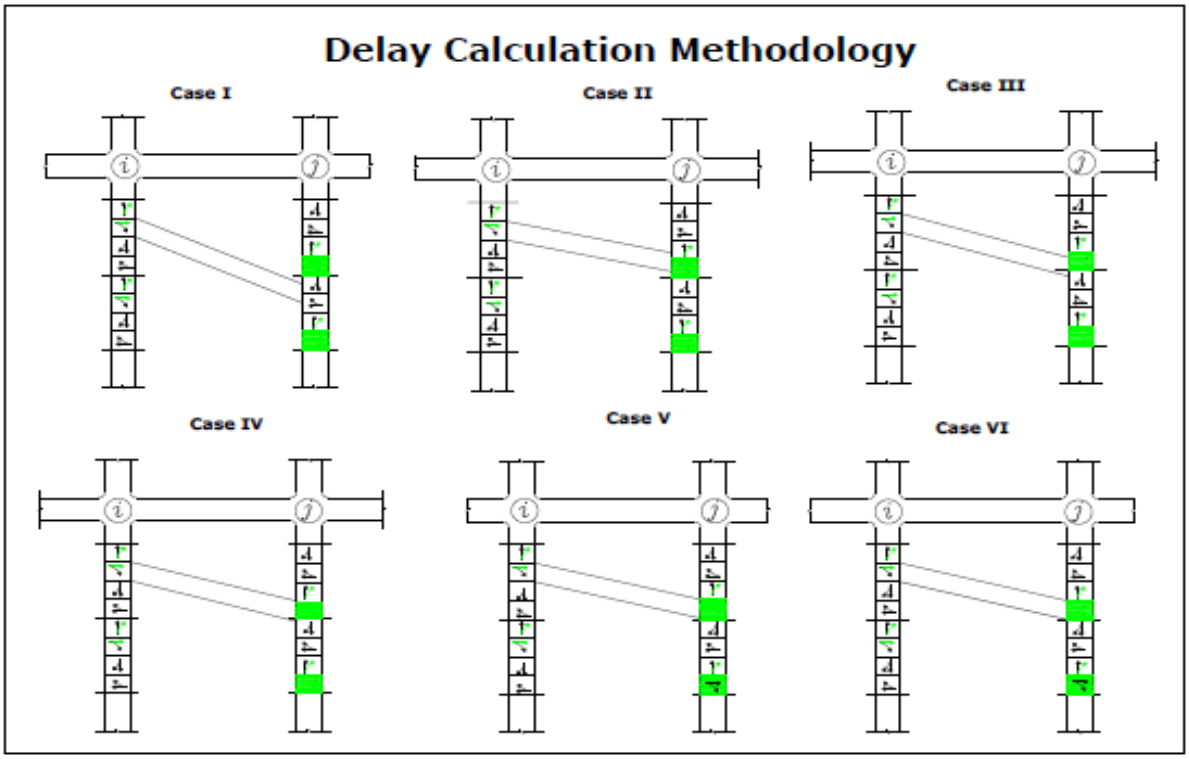


Figure 1: Delay calculation procedure

Let us consider,

αi = Phase time of R.T. or S.M. of upstream intersection.

βi = Unserved phase time of R.T. or S.M. At downstream intersection.

λi = Delay to get next green phase at downstream intersection. (Will be 0 for case II)

For Case I

Average delay = $\frac{\alpha i}{2} + \lambda i$ ---------- (i)

For Case II & III

∴ Clearance time available for downstream movement,

$\gamma i = \alpha i - \beta i$ ……..(ii)

Clearance % of vehicle $\delta i = \frac{\gamma_i}{\alpha i} * 100$ ---------- (iii)

Waiting % of vehicle $\eta i = \frac{\beta_i}{\alpha_i} * 100$ ---------- (iv)

∴ Weighted average delay,

$$\mu i = (\delta i * 0) + [\eta i * (\frac{\beta_i}{2} * \lambda i)]$$

$$\mu i = \eta i * (\frac{\beta_i}{2} * \lambda i)$$ ---------- (v)

Equation (i) is used to find delay for case I while equation (v) is used to find delay for case II and III.

For Case IV unserved phase time βi will have two value, Let upper value will be βiu and lower value will be βil.

∴ Weighted average delay for Case IV,

$$\mu i = \eta i * (\frac{\beta_{iu}}{2} * \lambda i) + \eta i * (\frac{\beta_{il}}{2} * \lambda i)$$ ---------- (vi)

Case V Right turning (R.T.) or straight moving (S.M.) movement band of upstream intersection intersect within the available green band of downstream intersection. (No Delay)

Case VI Right turning (R.T.) or straight moving (S.M.) movement band of upstream intersection coincide with the available green band of downstream intersection. (No Delay)

Derived DPCU values (Appendix- III)

Table 1: Average dynamic PCU values at swastik char rasta

Vehicle category	Direction	Approach			
		Stadium	Commerce Six Road	Girish Cold drinks	Navrangpura
2W	S	0.23	0.23	0.21	0.21
	R	0.25	0.22	0.21	0.19
	L	0.17	0.16	0.16	0.17
3W	S	0.72	0.65	0.62	0.6
	R	0.65	0.7	0.63	0.63
	L	0.4	0.89	0.62	0.53
Car	S	1	1	1	1
	R	1	1	1	1
	L	1	1	1	1
LCV	S	2.67	2.94	2.47	2.33
	R	-	3.39	2.27	-
	L	-	2.13	2.34	3.37
Bus/Truck	S	7.33	5.28	6.04	5.73
	R	-	-	-	-
	L	-	-	-	-
NM	S	0.26	0.22	0.25	0.2
	R	0.22	0.18	-	0.16
	L	-	0.37	-	-

Table 2: Average dynamic PCU values at girish cold drink

Vehicle category	Direction	Approach			
		Swastik Char rasta	St. xevier college	Swagat Junction	Mithakhali Six Road
2W	S	0.24	0.24	0.24	0.24
	R	0.22	0.22	0.24	0.23
	L	-	0.36	-	0.22
3W	S	0.65	0.63	0.64	0.58
	R	0.61	0.61	0.7	0.61
	L	-	0.69	0.74	0.58
Car	S	1	1	1	1
	R	1	1	1	1
	L	1	1	1	1
LCV	S	2.48	2.98	2.68	2.68
	R	-	-	-	3.16
	L	-	-	-	-
Bus/Truck	S	4.52	-	4.3	5.88
	R	5.75	-	-	-
	L	-	-	-	-
NM	S	0.26	0.27	0.22	0.25
	R	0.19	0.21	0.19	0.17
	L	-	-	-	-

Table3: Average dynamic PCU values at swagat junction

Vehicle category	Direction	Approach			
		Girish Cold drink	Gulbai Tekra	Panchvati Junction	Law Garden
2W	S	0.24	0.23	0.24	0.25
	R	0.25	0.24	0.22	0.25
	L	0.22	0.2	0.23	0.29
3W	S	0.62	0.64	0.6	0.62
	R	0.65	0.68	0.69	0.68
	L	0.58	0.83	0.44	0.79
Car	S	1	1	1	1
	R	1	1	1	1
	L	1	1	1	1
LCV	S	2.22	3.1	2.62	2.33
	R	2.45	2.63	3.12	-
	L	2.34	-	-	3.37
Bus/Truck	S	4.21	-	5.42	4.76
	R	-	-	-	6.78
	L	-	-	-	-
NM	S	0.2	-	-	0.15
	R	-	-	-	-
	L	-	-	-	-

GA output (Appendix IV)

GA output for considered Data:

```
Fitness= 48.502
Delay = 48.502
Penalty-1  0.00
Penalty-2  0.00

48.502
 no of intersaction = 3
Distance between 1 and 2 in meters    = 415.000
Speed between 1 and 2 in Kmph         = 29.800
Travel Time between 1 and 2 in sec = 50.134
Speed between 2 and 1 in Kmph         = 32.700
Travel Time between 2 and 1 in sec = 45.688
Distance between 2 and 3 in meters    = 390.000
Speed between 2 and 3 in Kmph         = 28.150
Travel Time between 2 and 3 in sec = 49.876
Speed between 3 and 2 in Kmph         = 34.600
Travel Time between 3 and 2 in sec = 40.578
Phase 1 = 16
Phase 2 = 29
Phase 3 = 17
Phase 4 = 33
Cycle Time = 95
```

GA output for field implementation data:

```
Fitness 63.38
 Delay = 63.38
 Penalty-1  0.00
 Penalty-2  0.00

 no of intersaction = 3
Distance between 1 and 2 in meters    = 415.000
Speed between 1 and 2 in Kmph         = 29.500
Travel Time between 1 and 2 in sec = 50.644
Speed between 2 and 1 in Kmph         = 32.400
Travel Time between 2 and 1 in sec = 46.111
Distance between 2 and 3 in meters    = 390.000
Speed between 2 and 3 in Kmph         = 25.600
Travel Time between 2 and 3 in sec = 54.844
Speed between 3 and 2 in Kmph         = 32.300
Travel Time between 3 and 2 in sec = 43.467
Phase 1 = 16
Phase 2 = 30
Phase 3 = 22
Phase 4 = 32
Cycle Time = 100

_[3J_[H_[2J
```

```
 Give User_Id :
 Give Key no :

GIVEN SELECT  1 = 1
GIVEN MINMUM  1 = 16.000000
GIVEN MAXIMUM 1 = 40.000000
GIVEN SELECT  2 = 1
GIVEN MINMUM  2 = 16.000000
GIVEN MAXIMUM 2 = 40.000000
GIVEN SELECT  3 = 1
GIVEN MINMUM  3 = 3.000000
GIVEN MAXIMUM 3 = 6.000000
LibGA Version 1.00
(c) Copyright Arthur L. Corcoran, 1992, 1993.  All rights
reserved.

GA Configuration Information:
-----------------------------
Basic Info
   Random Seed       : 1
   Data Type         : Bit
   Init Pool Entered : Randomly
   Chromosome Length : 15
   Pool Size         : 100
   cross-over(x_rate):  0.9
   Mutation rate     :  0.1
   Number of Trials  : Run until convergence
   Minimize          : Yes
   Elitism           : Yes
   Scale Factor      : 0

Functions
   GA          : generational (Gap = 0)
   Selection   : roulette
   Crossover   : uniform (Rate = 0.95)
   Mutation    : simple_random (Rate = 0.1)
   Replacement : append

Reports
   Type     : Short
   Interval : 1
-----------------------------

Gener    Min      Max      Ave    Variance   Std Dev    Tot Fit
Best
-----  -------  -------  -------  --------   -------    -------
-------
    0  71.0502  323.906      220  3.57E+03     59.7  22003.9
71.0502
    1  70.2972  287.947      168   4.5E+03     67.1  16785.5
70.2972
    2  68.1542  227.928      107  1.88E+03     43.4  10656.9
68.1542
    3  63.3755  196.427     82.3       569     23.9  8233.84
63.3755
```

```
    4  63.3755  170.798       77       335     18.3  7696.97
63.3755
    5  63.3755  151.415     72.9       165     12.9   7292.2
63.3755
    6  63.3755  87.6966       69      27.1      5.2  6896.65
63.3755
    7  63.3755  87.6966     67.2      28.3     5.32  6723.48
63.3755
    8  63.3755  86.7442     65.6      16.3     4.04  6558.51
63.3755
    9  63.3755  86.7442     64.2        11     3.31  6417.81
63.3755
   10  63.3755  85.4219     63.8      9.62      3.1  6383.55
63.3755
   11  63.3755  85.4219     63.6      4.86      2.2  6359.59
63.3755
   12  63.3755  85.4219     63.8      9.62      3.1  6381.64
63.3755
   13  63.3755  85.4219     63.6      4.86      2.2  6359.59
63.3755
   14  63.3755  63.3755     63.4         0        0  6337.55
63.3755

The GA has converged after 14 iterations.

Best: 1 0 0 0 0 0 0 0 0 1 0 0 0 0 0 (63.3755)
```

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