GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES

December, 2017

REPUBLIC OF TURKEY
YILDIZ TECHNICAL UNIVERSITY
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EVALUATION OF TRAVEL TIME FOR DIFFERENT TYPE OF VEHICLES IN TRAFFIC SIMULATION

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MSc. THESIS
DEPARTMENT OF CIVIL ENGINEERING
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İSTANBUL, 2017

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A thesis submitted by Sura Hussein Mijdim AL- HAMEEDAWI in partial fulfillment of the requirements for the degree of **MASTER IN CIVIL ENGINEERING** is approved by the committee on 18.12.2017 in Department of Civil Engineering, Transportation Program.

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ACKNOWLEDGEMENTS

Consequent to expressing profound gratitude to God for giving the strength and capacity to finish this research, i might need to show the truthful to fathers and family who have conferred in me the drive and support to complete this work.

The completion of this study would not have been possible without the assistance and incentives from some deferential individuals to whom I might want to appreciate:

I am grateful to thesis supervisor and adviser Assoc.Prof Halit OZEN for his invaluable guidance, encouragement, availability and continuous support throughout the course of Master program.

Special thanks to parents, brothers and sisters.

The endeavors of the greater part of the teachers and the college instructors who liberally shared their insight in various phases of my past training are additionally appreciatively valued.

December, 2017

Sura Hussein Mijdim AL- HAMEEDAWI

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LIST OF SYMBOLS

α Alpha - indicate the probability
δ delta - the approximate probability distribution
τ Correlation coefficient

LIST OF ABBREVIATIONS

CCO	Average Standstill Distance
AADT	Annual Average Daily Traffic
ADT	Average Daily Traffic
PHV	Peak Hourly Volume
DHV	Design Hourly Volume
PTV VISSIM	Planning Transport Verkehr AG in Karlsruhe
RVs	Recreational Vehicle
HOV-Lane	High Occupancy Vehicle –lane
OD-demand	Origin Destination- demand
YTU	Yildiz Technical University
ITC	Information Technology Center
SA	Sensitivity Analysis

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ABSTRACT

EVALUATION OF TRAVEL TIME FOR DIFFERENT TYPE OF VEHICLES IN TRAFFIC SIMULATION

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MSc. Thesis

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Microscopic simulation models have been widely used in both transportation operations and management analyses. This is because simulation is safer, less expensive and faster than field implementation and testing. However, for the simulation model the most important issues when running traffic simulation models are distinguished calibration inputs among model inputs (e.g., headway, desired speed, driving behavior etc.). Such inputs that explain driver behaviors and vehicle characteristics are difficult to collect from the field. Therefore, the simulation model user needs to fine-tune all inputs related to driving behavior and vehicle characteristics by comparing and adjusting some absolute measures and this overall process is called calibration.

Traffic simulation is an indispensable instrument for transport planners and traffic engineers. VISSIM is a microscopic, behavior-based multi-purpose traffic simulation model designed to analyze and optimize the traffic flows. It offers a wide variety of urban and highway applications and integrates both public and private transportation. Complex traffic conditions are visualized in high level of detail supported by realistic traffic models.

This study attempts to fill the gap in knowledge by providing further understanding of the traffic calibration resulting from two intersections and the associated field data. Two consecutive intersections with 4 legs were selected in Malatya for calibration. Data includes travel time, desired speed, hourly volume and headway. Traffic count data was collected during morning periods starting from 7:00 am to 9:00 am and video streaming was analyzed at the YTU ITS Lab. Signal timing and travel time data between the intersections was also collected in addition to headway.

Headway(s) data was statistically analyzed to determine whether different lanes had the same headway. The t-test hypothesis was applied to detect the gap between simulated and field results. The results of this study show that different lanes had the same headway(s).
Travel time data was also statistically analyzed to find out whether different vehicle types had the same travel time. The results of this study show that different vehicle types had different travel times. Therefore, different vehicle types should have different speed parameters including desired speed, acceleration and deceleration.
Calibration was necessary to minimize the errors between the model and field results. Parameters such as CCO, headway(s), and headway (m) were used during the calibration where one parameter was varied independently and taken as variable while the other two parameters were taken as stable until reasonable results were found by applying RMSE (Root mean square error). This same process was repeated for other parameters.

Key words: travel time, desired speed, calibration, PTV VISSIM, traffic count, traffic simulation

FARKLI TAŞITLAR TÜRLERİ İÇİN SEYAHAT SÜRELERİNİN BENZETİM MODELİNDE İNCELENMESİ

Sura Hussein Mijdim AL- HAMEEDAWI

İnşaat Mühendisliği Anabilim Dali
Yüksek Lisans Tezi

Danışman: Doç. Dr. Halit ÖZEN

Mikroskopik benzetim modelleri, değerlendirme aşamasında arazi uygulamaları ve geleneksel yöntemlere göre daha güvenli, hızlı ve maliyetinin düşük olmasından dolayı ulaştırma projelerinin değerlendirilmesinde yaygın olarak kullanılmaktadır. Bununla birlikte, trafik benzetim modellerinin kalibrasyon çalışmaları aşamasında model girdileri (taşıt üzleme mesafesi, hız dağılımı, şerit değiştirme karakteristiğigibi.) büyük önem arz etmektedir. Bu tip verilerin arazi çalışmaları ile sağlıklı bir şekilde edilmesi, zorluklar içermeketedir. Bu nedenle, sürücü davranışları ve araç özellikleri gibi verilerin benzetim modelininden geröek durmu temsil edecek şekilde doğru çıktı olarak alamak için araziden elde edilen veriler ile benzetim modeli çıktıları karşılaştırılmak suretiyle parametrelerin değiştirilmesine ve uygun değerin bulunmasına gerek duyulmaktadır. Bu genel süreç Kalibrasyon olarak tanımlanmaktadır.Ulaşım plancıları ve trafik mühendisleri için benzetim modeli önemli bir araç olup; bu çalışma kapsamında da kullanılan PTV tarafından geliştirilen VISSIM ise trafik akımının analzi ve optimize edilmesi amacıyla kullanılan sürücü davranışı tabanlı, çok amaçlı bir benzetim modeli yazılımdır. Toplu ve birieysel ulaşımı birlikte değerlediren,şehriçi ve otoyollar için kapsamlı uygulamaları içermektedir.Benzetim modelleri ile karmaşık trafik koşulları gerçekçi trafik modelleri ile detaylı olarak incelenebilmekte ve sunulmaktadır.

Bu çalışma, benzetim modelinde kalibrasyon kavramının anlaşılmasındaki boşluğu doldurmak amacıyla birbirini izleyen iki adet dört kollu kavşak için arazi çalışmaları ile veri toplamayıda kapsayan bir benzetim modeli kalibrasyonunu çalışmasını içermektedir. Çalışmad kullanılmak üzer gerekli verilerin toplandığı aynı güzergah üzerinde bir birini izleyen dört kollu kavşaklar Malataya ilinden seçilmiştir. Kavşaklara ilişkin olarak, saatlik trafik hacimleri ve video görüntüleri, Malatya Belediyesinden temin edilmiştir. Ayrıca, güzergah üzerindeki seyahat süresi, hız, doygun akım verileri ise Yıldız Teknik Üniversitesi AUS (Akıllı Ulaşım Sistemleri) Laboratuarı'nda görüntilerin çözümlemesi ile elde edilmiştir. Sinyal sürelerinide içeren veriler sabah zirve saat olan 7:00 - 09:00 aralığı için toplanmıştır.

Çlaışma kapsamında, doygun akım değerlerinin şeritler için (sol,orta veya sağ) ve model girdisi olan taşıt hızlarının taşıt tiplerine göre farklılık gösterip güstermediği istatistiksel yöntemlerle incelenmiş ve benzetim modeli kalibrasyonu gerçekleştirilmiştir. Çalışmanın sonucunda, taşıt hızlarının taşıt tipine göre farklılık gösterdiği ancak doygun akım değerlerinin şerit konumuna göre farklılaşmadığı ve benzetim modelinin modelde kullanılan yazılım varsayılan değerleri ile çalıştırılması durumunda farklı sonuçlar verdiği bu ndenele benzetim modelinin kalibrasyonun büyük öenm arz ettiği sonuçları elde edilmiştir.

Anahtar Kelimeler : trafik akımı, benzetim modeli, seyahat süresi, arzulanan hız, kalibrasyon.

CHAPTER 1

INTRODUCTION

1.1 Literature Review

Tom V. Mathew and Padmakumar Radhakrishnan mentioned that most countries in the world, especially Asian countries have heterogeneous traffic characterized by diverse vehicles, changing composition, lack of lane discipline etc., resulting in a very complex behavior. A methodology for representing non-lane-based driving behavior and calibrating a micro simulation model for highly heterogeneous traffic at the signalized intersection is hence proposed. Calibration parameters are identified using sensitivity analysis and the best values for these parameters obtained by minimizing the error between the simulated and field delay using a genetic algorithm. The proposed methodology is illustrated using Verkehr in Staedten simulation; signalized intersections with diverse traffic and geometric characteristics of two cities in India are considered as a case study [1].

Gabriel Gomes, Adolf May Roberto Horowitz presented a procedure for constructing and calibrating a detailed model of a freeway using VISSIM and applied it to a 15-mile stretch of I-210 West in Pasadena, California. The model construction procedure involved: 1) identification of important geometric features, 2) collection and processing of traffic data, 3) analysis of the mainline data to show recurring bottlenecks, 4) VISSIM coding, and 5) calibration based on observations from 3). A qualitative set of goals was established for the calibration. These were met with relatively few modifications to VISSIM's driver behavior parameters (CC-parameters) [2].

The researchers Sandeep Mennen and Carlos Sun presented a micro simulation calibration method based on matching speed-flow graphs from field and simulation Pattern recognition was used for Evaluation and automation of matching speed-flow graphs and the method was applied to US101 freeway network in San Francisco, California using an Evolutionary Algorithm. This method was compared to traditional methods of calibration based on capacity and was shown to do better. In addition, a small scale test network simulation model was developed to aid in the implementation of this method for large simulation models. The performance of the test network simulation model was comparable to the US101 simulation model [3].

Qiao Ge and, Monica Menendez of ETH Zurich Institute used sensitivity analysis for calibrating VISSIM for modeling the inner city of Zurich. The complex network connection of the inner city makes the computational cost of running the simulation very expensive. Therefore, it does not allow using a brute force approach to do the SA. An improved SA method, which is based on the Elementary Effects method, was proposed and applied to this project. This method reduced the computation time required for the SA from 77 days to 2 days for this specific case. The results show that the proposed method is correct and efficient, especially for dealing with the SA of complex VISSIM networks [4].

According to the researchers Byungkyu (Brian) Park and J. D. Schneeberger, microscopic simulation models have been widely used in both transportation operations and management analyses because simulation is safer, less expensive and faster than field implementation and testing. While these simulation models can be helpful to engineers, a procedure was proposed for microscopic simulation model calibration and validation and an example case study is presented with real-world traffic data from Route 50 on Lee Jackson Highway in Fairfax, Virginia [5]. The proposed procedure consisted of nine steps: (a) measurement of effectiveness choice, (b) data collection, (c) identification of calibration parameter, (d) experimental design, (e) running of simulation, (f) surface function development, (g) generation of candidate parameter, (h) evaluation and (i) validation through new data collection. The case study indicates that the proposed procedure appears to properly calibrate and validate the VISSIM simulation model for the test-bed network [5].

According to Pruthvi Manjunatha and Peter Vortisch mixed traffic, characterized by diverse vehicles, changing composition, lack of lane discipline, etc. are best modeled by micro simulation. However, the majority of the leading micro simulation packages and their calibration process have been progressing with less complex homogeneous traffic. Hence, a methodology for calibrating a micro simulation model for mixed traffic is proposed, with multi-parameter sensitivity analysis being used to identify calibration parameters, and the greatest values for these parameters obtained by minimizing the error between the simulated and field delay using a genetic algorithm. Multiple criteria included in the optimization formularization by constraint insertion. The proposed methodology is illustrated using VISSIM; the most widely used micro simulation software. Signalized intersections with different traffic characteristics from Mumbai are taken as the case study [6].

Li-Hong CHEN1， Da-Wei HU and Hong-Guo ZHU proposed a method of bus transit network optimization based on microscopic simulation software. The researchers took the Lin He district of Bayannur as a case study using surveyed data. They used the microscopic simulation software VISSIM to simulate and analyze the current situation through the evaluation file of the software then optimized the network. By analyzing the simulation results before and after optimization, results show that after the transit network optimization, there was an obvious decrease in the buses' average queue length and load because of each section achieves a good level. Using microscopic traffic simulation software to simulate can intuitively and vividly show the optimization effect. It not only provides technical means for transit network optimization but it is also providing an important for mass transit network optimization in addition to solving the problem of traffic congestion [7].

The researchers Byungkyu (Brian) Park and Hongtu (Maggie) Qi found that microscopic simulation models have been widely accepted for evaluation of various transportation system design and traffic operations and management strategies. In that regard, the calibration and validation of simulation model are crucial for proper decision-making process. The study done by Byungkyu (Brian) Park and Hongtu (Maggie) Qi presents an application of previously developed microscopic simulation model calibration and validation procedure for a freeway work zone network. Multiple days of field data were collected to consider variability and these days were divided into calibration and validation datasets. The study results showed that the procedure was effective in the calibration and validation of a freeway work zone network [8].

1.2 Objective of the Thesis

The main objective of this study is to analyze traffic data in two intersections in the city of Malatya. In order to accomplish the main objective, the following specific objectives should be met:

- ✓ To analysis the traffic data of selected intersections for travel time, headway, and speed.
- ✓ To provide further understanding on the need for the calibration of traffic parameters in the traffic simulation model.

1.3 Hypothesis

This study attempts to analysis and providing a further understanding of calibration and evaluation of collected traffic data of (headway(s), travel time, and speed). In order to capture this aspect, the t-test hypothesis and calibration on VISSIM model was used to determine the impact of traffic parameters including travel time, speed and headway.

Calibration of the model will be done to minimize the errors between the simulated and fields' results based on three parameters which are driving behavior parameters (CCO), headway (m) and headway(s). During the calibration, one parameter is varied independently and taken as variable whereas the other two parameters are taken as stable until reasonable results are obtained by applying RMSE (Root mean square error).

Aim of the study is to prove that the headway does not effected by the different lanes of the roadway and travel time of the roadway segment have differentiate by the type of the vehicle like car, van or truck.

CHAPTER 2

HIGHWAY GEOMETRY AND INTERSECTION

The geometric design is defined as the design or proportioning of the visible elements of the street or highway. The goals of geometric design are to maximize the comfort, safety, economy of facilities, and provide efficiency in traffic operation with provide maximum safety at a reasonable cost and minimize the environmental impacts [9]. Physical dimensions of geometric design elements are:

- Cross-section
- Horizontal alignment
- Vertical alignment
- Sight distance
- Intersections etc. [10].

Physical dimensions of geometric design elements are determined by:

- Characteristics of driver
- Characteristics of vehicle
- Characteristics of road [11].

In geometric design of highways, should be considered for consistent and compatibility (the standards proposed for the different elements should be compatible with one another) and include all aspects of the geometry of the road (signs, markings, proper lighting, intersections etc.), the highway should enable all the road users to use the facility and humans (the physical, mental and psychological characteristics of the driver and pedestrians like the reaction time) [12].

2.1 Factors Influencing Highway Design

- Topography
- Volume and composition of traffic
- Traffic
- speed
- Design vehicle.

The first step in the design process is to define the function that the facility is to serve. The level of service required to fulfill this function for the anticipated volume and composition of traffic provides a rational and cost-effective basis for the selection of design speed and geometric criteria within the range of values available to the designer (for the specified functional classification). The use of functional classification as a design type should appropriately integrate the highway planning and design process.

Road can be classified in many ways for instance

- Traffic volume
- Surface condition
- Level of service etc.
- Mobility: The ability to move goods and passengers to their destination. (in a reasonable time) [10].
- Accessibility: the ability to reach desired destination functional classification of the road like in Figure 2.1. Below so the functional classification of highway being designed as:
- Principal arterials
- Minor arterials
- Collectors
- Local roads [11].

However, classification based on the speed and accessibility is the most generic one.

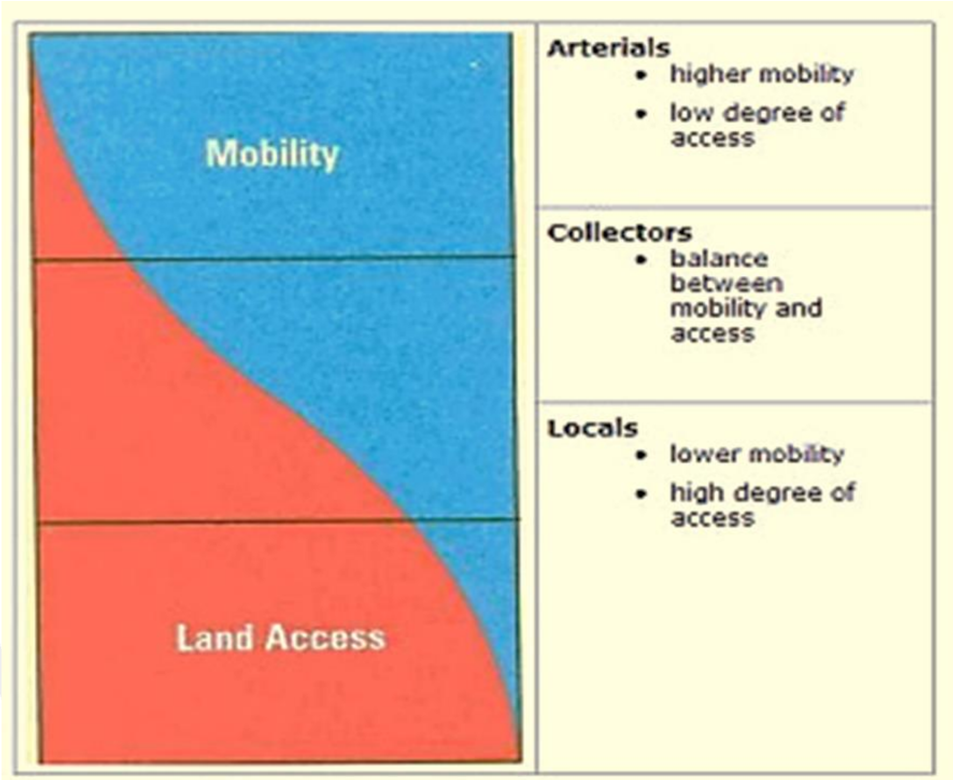


Figure 2. 1 Classification of Road

From the Figure2.1 above show there are only three levels in a functional road classification (arterial, collector, local) functional road classification can be defined in terms of two topics. One is “land access”, the other is “mobility” The graph is a continuous curve – it does not show 3 discrete steps. This implies that there can be any number of levels in a road hierarchy, and just 3. Indeed,

2.1.1 Topography

The geometric design elements of a road depend on the traverse terrain through which the road passes. Traverse terrain properties are categorized into four classes as follows:

- Flat or gently rolling terrain
- Rolling terrain
- Mountainous terrain
- Escarpment [10].

2.1.2 Volume and Composition Traffic

Traffic data indicates the service for which the road is being planned and directly affects the geometric elements such as width, alignment, etc.

- Traffic volume – AADT, ADT, PHV, DHV
- Directional distribution – the percentage of traffic volume
- Traffic composition – the percentage of different types of vehicles in the traffic stream – different types of vehicles are converted into passenger car unit to design a road width
- Traffic projection – using the design period of a road (5-20 years).

2.1.3 Traffic

It has many of the items below:

- Current traffic – currently using the existing road
- Normal traffic growth – anticipated growth due to population growth or change in land use
- Diverted traffic – traffic that switches to a new facility from nearby roads
- Converted traffic – traffic resulting from changes of mode
- Change of destination traffic – traffic that has changed to different destination due to new or improved transport and not changes in land use
- Development traffic – traffic due to improvement on adjacent land development that would have taken place had the new or improved road not been constructed [10].

2.1.4 Speed (v)

Speed is defined as the distance it travels per unit of time. Each vehicle on the roadway will have a speed that is dissimilar from other vehicles around it. In calculating the average speed and traffic flow is the important variable. The average speed (space mean speed) can be found by averaging the individual speeds of all of the vehicles in the study area [12].

$$v = \frac{d}{t} \tag{2.1}$$

Where v is speed, d is distance, and t is time

The following are explainable:

1- Design Speed
2- Operating Speed

3- Running Speed when new or improved transport facilities are provided

1- Design Speed

It was a selected rate of travel used to determine the various geometric features of the roadway. Since many critical design features (e.g., sight distance and curvature) are predicated upon design speed. The selected design speed should be a logical one with respect to the anticipated operating speed, topography, the adjacent land use, and the functional classification of the highway.

2- Operating Speed

Operating speed is the speed at which drivers are observed operating their vehicles during free-flow conditions. The 85th percentile of the distribution of observed speeds is the most frequently used measure of the operating speed associated with a particular location or geometric feature.

3- Running Speed when new or improved transport facilities are provided

The speed at which an individual vehicle travels over a highway section is known as its running speed.

$$RS = \frac{L_{highway}}{RT_{vehicle}} \tag{2.2}$$

Where:

RS is Running Speed

L is Length

RT is Running Time [9].

2.1.5 Design vehicle

A selected motor vehicle that weight, dimensions, and operating characteristics. Those are important used to establish highway design controls to accommodate vehicles of a designated type as show in Figure 2.2. And include [9]:

- Passenger cars, buses, trucks, RVs
- Physical characteristics: weight, dimensions

- Establish intersection radius, pavement markings
- Vehicle Performance
- Operating characteristics: acceleration/acceleration
- Impacts air quality, noise, and land use [12].

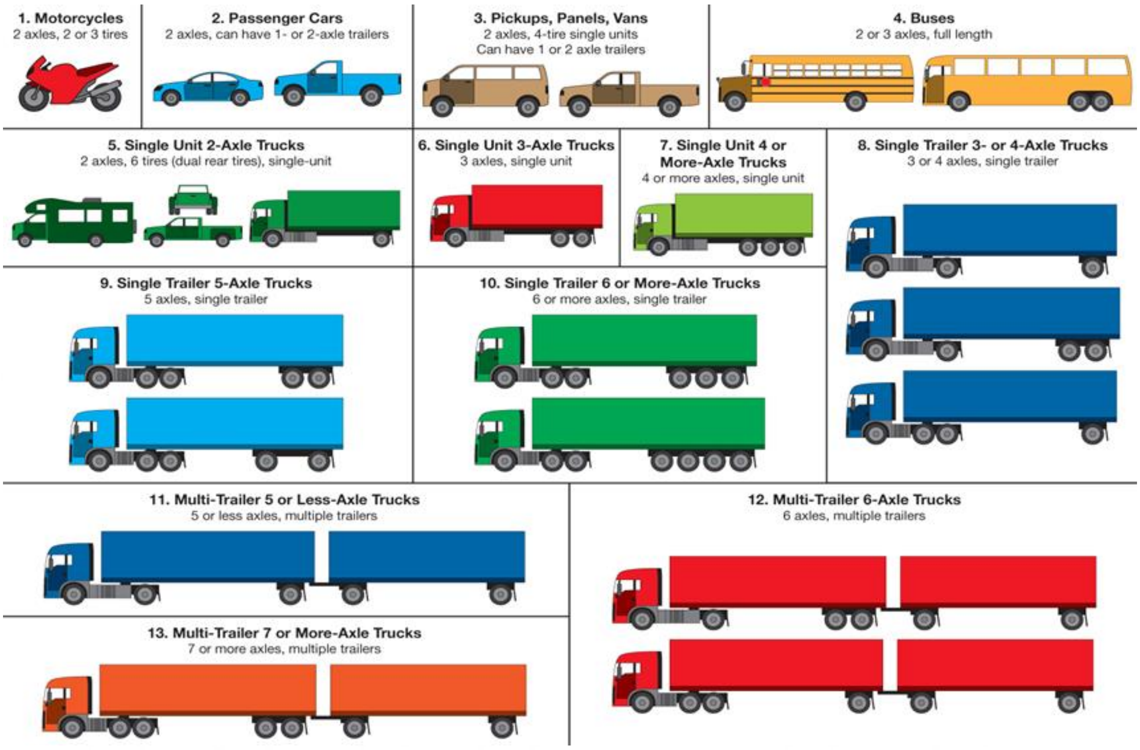


Figure 2. 2 Design Vehicle

2.2 Cross-Section Elements of Highways

Cross sections were the configuration of a proposed roadway at right angles to the centerline. Typical sections show the width, thickness and descriptions of the surfacing courses as well as the geometrics of the graded roadbed, side ditches and side slopes as like show in Figure 2.3. Cross-section includes [10]:

1- **Right-of-Way**

It is the width of land secured and preserved to the public for road purposes, should be adequate to accommodate all the elements that make up the cross-section of the highway and may reasonably provide for future development.

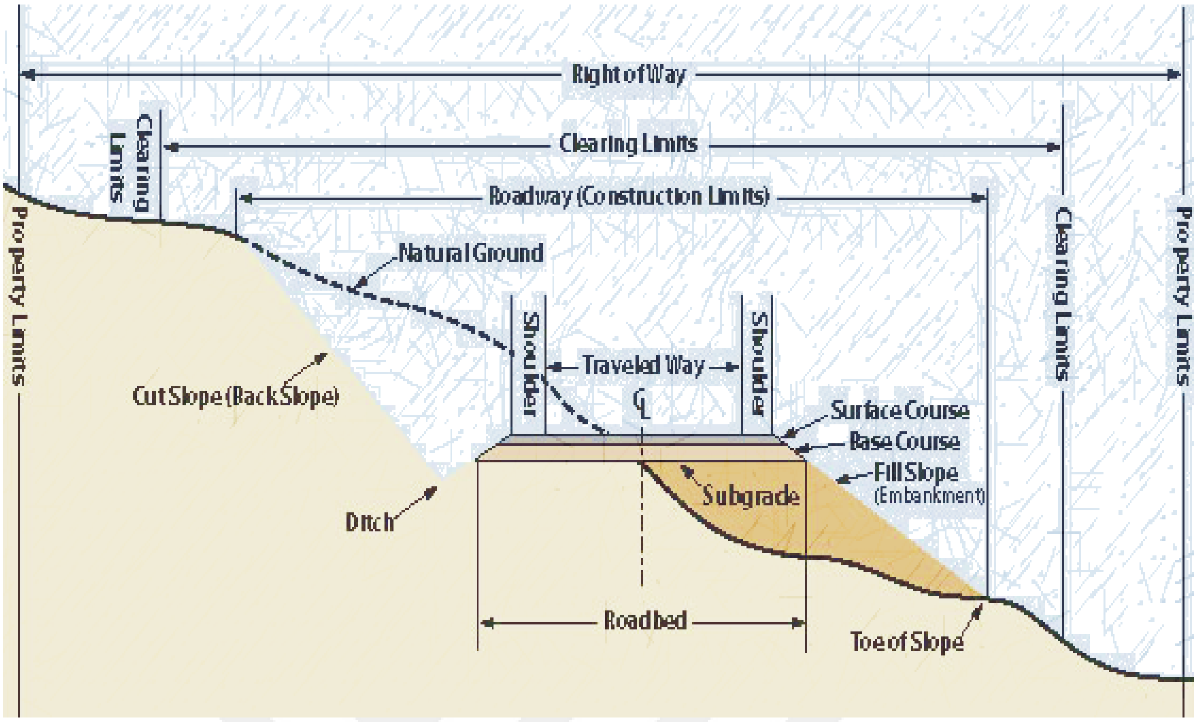


Figure 2. 3 Cross-Section of Road

2- Carriageway

The part of the road constructed for use by moving traffic, including traffic lanes, auxiliary lanes such as acceleration and deceleration lanes, climbing lanes, width of median strip and passing lanes.

3- Shoulders

The portion of the roadway adjacent to the carriageway,

4- **Roadway**

It consists of the carriageway and the shoulders, parking lanes and viewing areas.

5- Median

It was Section of divided road that separates lanes in the opposite directions.

6- Crown Slope/Camber

It's the cross slope provided to raise middle of the road surface in the transverse direction to drain rain water from the road surface but not being so great as to make steering difficult. Normal cross fall should be 3% on paved roads and 4-6% on unpaved roads

7- Side Slopes & Back Slopes

It was the graded area immediately adjacent to the graded roadway shoulder and side slope toe or ditch. Side slopes should be designed to insure the stability of the roadway and to provide a reasonable opportunity for recovery of an out of control vehicle.

Three regions of the roadside are important when evaluating the safety aspects of side slope

- the top of the slope (hinge point),
- the side slope, and
- the toe of the slope

Embankment or fill slopes parallel to the flow of traffic maybe defined as recoverable, non-recoverable, or critical.

8- Curbs

Raised structures used mainly on urban roads to delineate pavement edge and pedestrian walkways.

9- Side Walks

It was provided on urban or sub urban roads when pedestrian traffic is high along main or high speed roads. In urban areas, sidewalks are provided along both sides of streets to serve pedestrians access to schools, parks, shopping centers, and transit stops.

10- Drainage Ditches

Function of collecting and conveying surface water from the highway right-of-way. The depth of channel should be sufficient to remove surface water without saturation of the sub grade also should be located and shaped to avoid creating a hazard to traffic safety [10].

2.3 Horizontal Alignment

The horizontal alignment of a road is usually a series of straights (tangents) and circular curves that may or may not be connected by transition curves as show in Figure 2.4. It should be designed to provide motorists with a facility for driving in a safe and comfortable manner [12].

The designer should adhere to several general controls for horizontal alignment. These are based on aesthetic and safety considerations. They include:

Figure 2. 4 Horizontal Alignment

1- Horizontal alignment should be as directional as possible. Where feasible, minimum radii should be avoided. Flatter curvature with shorter tangents is generally preferable to sharp curves connected by long tangents.
2- Curves with small deflection angles should be long enough to avoid the appearance of a kink.
3- Very small deflection angles may not require a horizontal curve; i.e., the roadway may be designed with an angular break, because the evaluation on the use of an angle point will be based on urban/rural location, aesthetics, construction costs and the visibility of the kink.
4- Broken back curvature should be avoided.
5- Sharp horizontal curves should not be introduced near crest or sag vertical curves. The combination of horizontal and vertical curves can greatly reduce sight distance, and the likelihood of accidents is increased.
6- Horizontal curves and superelevation transitions should be avoided on bridges. These cause design, construction and operational problems when snow and ice are present. Where a curve is necessary on a bridge, a simple curve

7- The crossover line will often be a control for setting the rates of superelevation and radius and profile where two roadways converge. Freeway gores are an example [9].

Type of horizontal curve:

1- Simple Curve (Circular)
2- Compound Curve
3- Reveres Curve
4- Broken Back Curve
5- Spiral Curve.

Horizontal Alignment Considerations:

- Radius
- Superelevation
  - ✓ Runoff
  - ✓ Run out [13].

2.3.1 Minimum radius of curve

The design of radius of roadway curves should be based on an appropriate relationship between design speeds, curvature with relationship the superelevation (roadway banking) and side friction. The minimum curve radius must be selected very well for safe and comfortable driving in the alignment design, the equation can find the r-minimum is:

$$R\_min = \frac{V^2}{127(0.01e_{max} + f\,max)} \tag{2.3}$$

Where:

R_min is minimum radius of curve, fit, and m

e is superelevation rate

F is side friction factor

V is vehicle speed, mph [13].

2.3.2 Superelevation

Superelevation is tilting the roadway to help offset centripetal forces developed as the vehicle goes around a curve. Along with friction, appear in Figure 2.5. They are what keep a vehicle from going off the road. It must be done gradually over a distance without noticeable reduction in speed or safety [13]. Superelevation is the raising of the outer edge of the road along a curve in order to counteract the effect of radial centrifugal force in combination with the friction between the surface and tires developed in the lateral direction. To provide safety and comfort while navigating through curves at higher speeds [10].

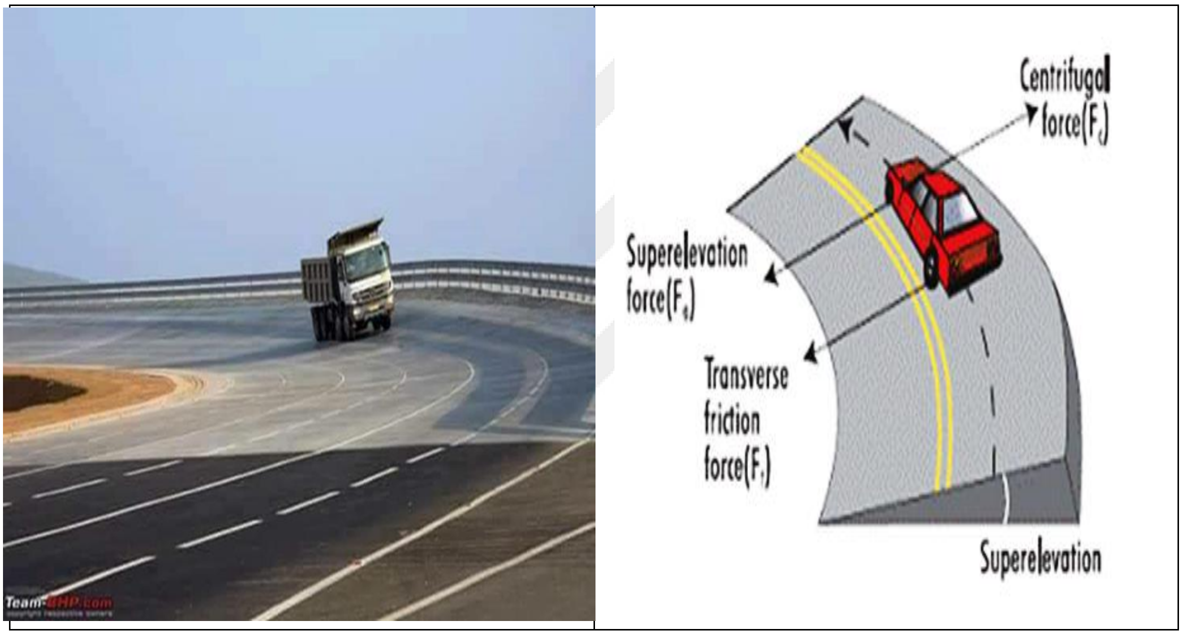


Figure 2. 5 Superelevation

Side friction coefficients are dependent on

- Vehicle speed
- Type, condition and texture of roadway surface;
- Weather conditions; and
- Type and condition of tires [13].

2.4 Vertical Alignment

Vertical Curves are used to provide a gradual change from one tangent grade to another as shown in Figure 2.6. So that vehicles may smoothly navigate changes in grade as they travel the highway, so the vertical alignment consists of grade tangents connected with parabolic curves and desirable maximum grades with gradient changes depend on the facility type and vehicle characteristics. The desirable grade as function of facility type:

- 2% for freeways
- 6% for Local Street
- Higher grades are unavoidable at location with difficult topography [14].

Figure 2. 6 Vertical Alignment

Vertical alignment contains from:

- Grades
- Vertical curve

2.4.1 Grades

Roadway grades have a direct correlation to the uniform operation of vehicles. Vehicle weight and the steepness of the roadway grade have a direct relationship on the ability maximum grade recommendations are presented for the area types depending upon the terrain in which the facility is located, the level or rolling terrain category is applicable, of the driver to maintain uniform speed.

Grades – vertical slope from reference station

- Upgrade – positive
- Downgrade - negative **[**12].

Critical Lengths of Grade for Design

To establish design values for critical lengths of grade for which grade ability of trucks is the determining factor, data or assumptions are needed for the following:

1- Size and power of a representative truck or truck combination to be used as a design vehicle along with the grade ability data
2- Speed at entrance to critical length of grade.
3- Minimum speed on the grade below in which interference to following vehicles is considered unreasonable [12].

2.4.2 Vertical curve

A parabolic curve is the most common type used to connect two vertical tangents as shown in Figure 2.7. And the type of vertical curve:

1- Crest – stopping, or passing sight distance controls
2- Sag – headlight/stopping sight distance, comfort, drainage and appearance control Green Book vertical curves defined by K = L/A = length of vertical curve/difference in grades (in percent) = length to change one percent in grade.

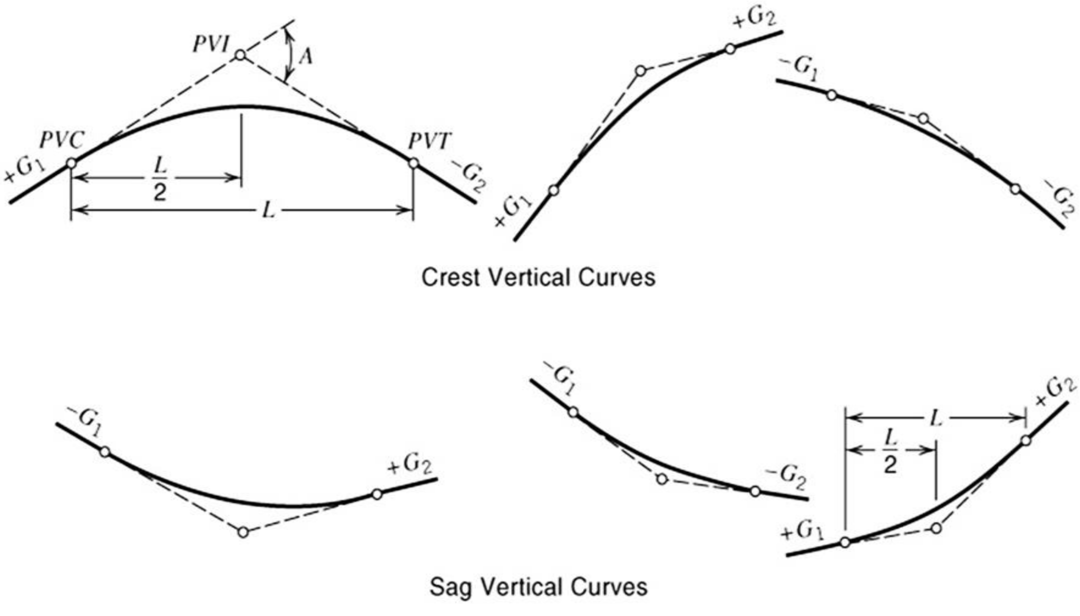


Figure 2. 7 Type of Vertical Alignment

- G1 is initial roadway grade also referred to as initial tangent grade.
- G2 is final roadway (tangent) grade a is the absolute value of the difference in grades (generally expressed in percent).

$$A = |G2 - G1| \tag{2.4}$$

L is the length of the vertical curve measured in a horizontal plane (not along curve center line, like horizontal curves) and should be in vertical curve being careful for:

- Controlling factor: sight distance
- Stopping sight distance should be provided as a minimum
- Drainage of sag curves are important consideration, grades not less than 0.5% needed for drainage to outer edge of roadway.

2.5 Sight Distance

The provision for adequate horizontal and vertical sight distance is an essential factor in the development of a safe street or highway an unobstructed view of the upcoming roadway is necessary to allow time and space for the safe execution of passing, stopping, intersection movements, and other normal and emergency maneuvers and its type is:

1- Stopping Sight Distance
2- Head Light Sight Distance
3- Passing Sight Distance
4- Decision Sight Distance
5- Intersection Sight Distance

1- Stopping Sight Distance (SSD)

Stopping sight distance is the distance required to see an object 0.15m high on the roadway and stop the vehicle before crushing the object.

The greater sight distances required to provide safe overtaking opportunities are not easily provided on crest curves. If full overtaking sight distance cannot be obtained, the design should aim to reduce the length of crest curves to provide the minimum stopping sight distance, thus increasing overtaking opportunities on the gradients on either side of the curve two conditions exist when considering minimum sight distance criteria on vertical curves. The first is where sight distance is less than the length of the vertical curve, and the second is where sight distance extends beyond the vertical curve. Sight distances have been based on the characteristics of car drivers as, although braking distances are greater with trucks, they will usually be travelling more slowly and the eye height of truck drivers is about 1.0 meter higher. Requirements are related to rates of deceleration available with an emergency stop [9].

2- Head Light Sight Distance

At night, the portion of highway that is visible to the driver is dependent on the position of the headlights and the direction of the light beam.

3- Passing Sight Distance

Passing sight distance is the distance required to see an oncoming vehicle of a certain minimum size and pass through it safely. There is no need to consider passing sight distance on highways or streets that have two or more traffic lanes in each direction of travel, most roads and many streets are two-lane, two-way highways on which vehicles frequently overtake slower moving vehicles show in Figure 2.8. Passing maneuvers in which faster vehicles move ahead of slower vehicles are accomplished on lanes regularly used by opposing traffic the overtaking maneuvers has a large number of variables such as:

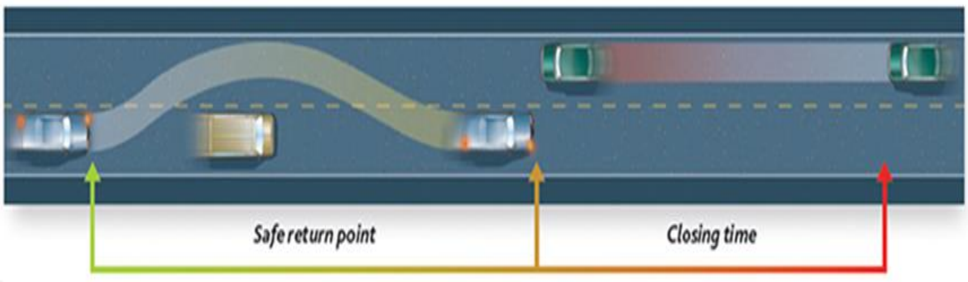


Figure 2. 8 Passing Sight Distance

- The judgment of the overtaking driver and the risks the driver is prepared to take.
- The speed and size of vehicles to be overtaken.
- The speed of the overtaking vehicle.
- The speed of a potential oncoming vehicle.
- The evasive action or braking undertaken by the vehicle or the overtaken vehicle.

4- Decision Sight Distance

Decision sight distance is the distance needed for a driver to detect an unexpected or otherwise difficult to perceive information source or condition in a roadway environment that may be visually cluttered, recognize the condition or its potential threat, select an appropriate speed and path, and initiate and complete complex maneuvers.

5- Intersection Sight Distance

A motorist attempting to enter or cross a highway from a stopped condition should be able to observe traffic at a distance that will allow safe movement. The methods described in the following paragraphs produce distances that provide sufficient sight distance for the stopped driver to make a safe crossing or turning maneuver. If these distances cannot be obtained, the minimum sight distance provided should not be less than the stopping sight distance for the through roadway [9].

2.6 Intersection

It was the general area where two or more highways join or cross, including the roadway and roadside facilities for traffic movements within the area. Each highway radiating from an intersection and forming part of it is an intersection leg.

Five basic elements should be considered in intersection design.

- Human Factors consist of: driving habits, driver expectancy, decision and reaction time, pedestrian use and habits, ability of drivers to make decisions, conformance to natural paths of movement and bicycle traffic use and habits
- Traffic Considerations contain from: design and actual capacities, design-hour turning movements, vehicle speeds, transit involvement, crash experience, bicycle movements, pedestrian movements, size and operating characteristics of vehicle, variety of movements (diverging, merging, weaving, and crossing)
- Physical Elements: sight distance, angle of the intersection, conflict area, speed-change lanes, geometric design features, traffic control devices, lighting equipment, safety features, bicycle traffic, environmental factors, cross walks, character and use of abutting property, vertical alignments at the intersection

- Economic Factors have of: cost of improvements, effects of controlling or limiting rights-of-way on abutting residential or commercial properties where channelization restricts or prohibits vehicular movements, energy consumption
- Functional Intersection Area [12].

Classification of road intersections on the basis of number of roads intersecting:

Three-way intersection: Also known as T or Y Junction is the linking of three roads as in above Figure 2.9.

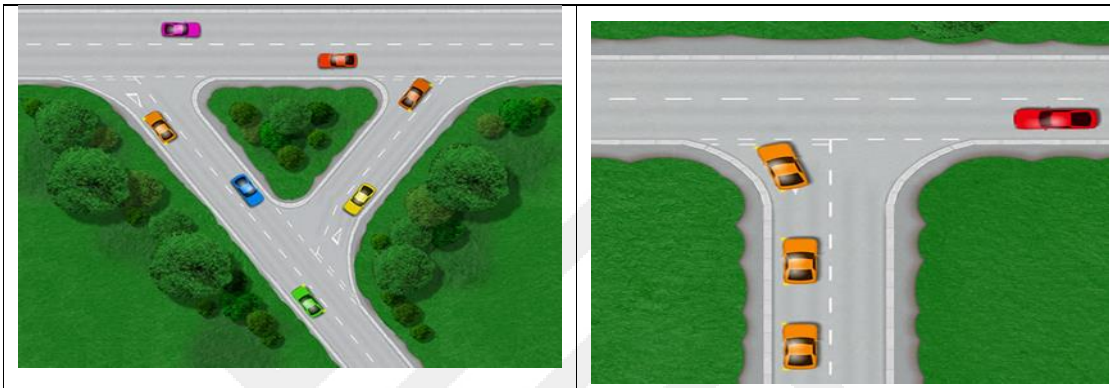


Figure 2. 9 T and Y Intersection

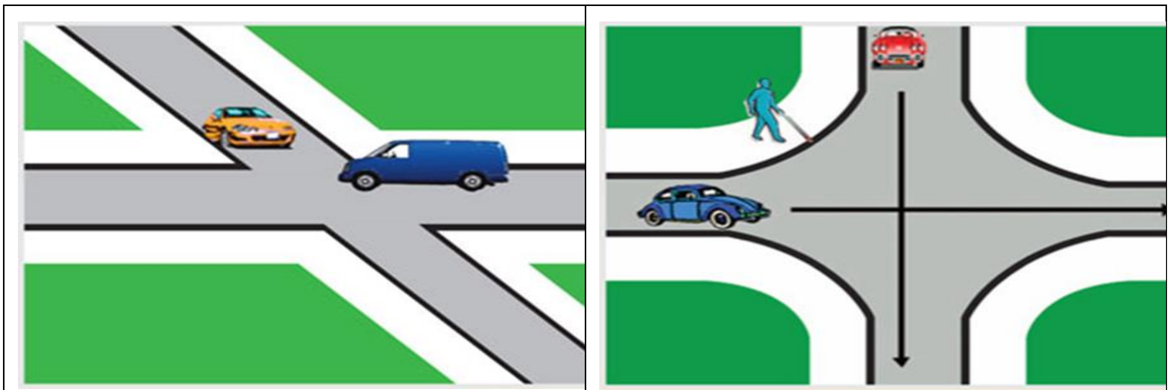


Figure 2. 10 Four Intersection

Four-way intersection: Figure 2.10.

These are the most common intersections where crossing over of two roads is involved. It is further divided into two categories depending on the angle by which the two roads intersect each other.

When the two joining roads intersect each other perpendicularly, it is termed as a regular intersection. When the two roads cross at a different angle the junction is called a Skewed Intersection.

- Five-way intersection: crossing over of five roads as show in Figure 2.11.

Figure 2. 11 Five -Ways Intersection

- Six-way intersection:

It involves the crossing of three streets; most often a crossing of two perpendicular streets and a diagonal street. More than six roads coming together at a single intersection are rare [15].

2.6.1 Intersection Types and Configurations

Intersections can be categorized two main types of intersection of roads. As illustrated in below intersection types.

- At-grade intersections.
- Grade-separated intersections or interchanges.

2.6.1.1 At-Grade Intersections

At-grade intersections are in which all the exchanges between the roads take place on the same plane.

These are of four main types:

A. Simple Intersections.
B. Flared Intersections.

C. Channelized intersections.
D. Roundabouts.

A. Simple Intersections

Simple intersections maintain the street's typical cross section and number of lanes throughout the intersection, on both the major and minor streets. As show in Figure 2.12.

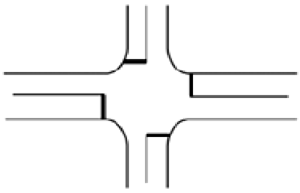


Figure 2. 12 Type A Intersection

B. Flared Intersections

Flared intersections expand the cross-section of the street (main, cross or both) as show in Figure 2.13. The flaring is often done to accommodate a left-turn lane, so that left-turning bicycles and motor vehicles are removed from the through-traffic stream to increase capacity at high-volume locations, and safety on higher speed streets. Right-turn lanes, less frequently used than left turn lanes, are usually a response to large volumes of right turns.

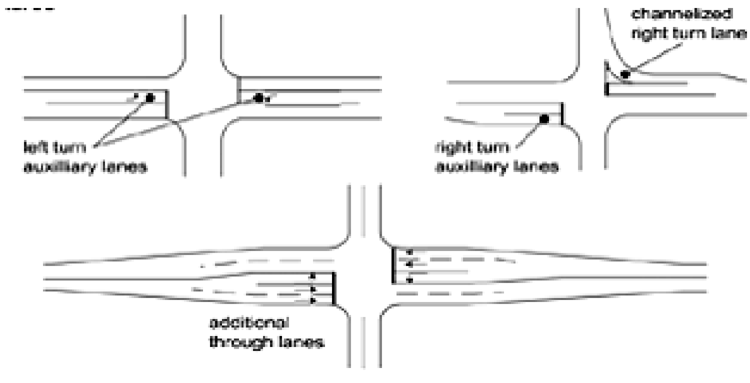


Figure 2. 13 Type B Intersection

C. Channelized Intersections

It was use pavement markings or raised islands to designate the intended vehicle paths. Show in Figure 2.14. The most frequent use is for right turns, particularly when accompanied by an auxiliary right-turn lane. At skewed intersections, channelization islands are often used to delineate right turns, even in the absence of auxiliary right turn lanes. At intersections located on a curve, divisional islands can help direct drivers to and through the intersection.

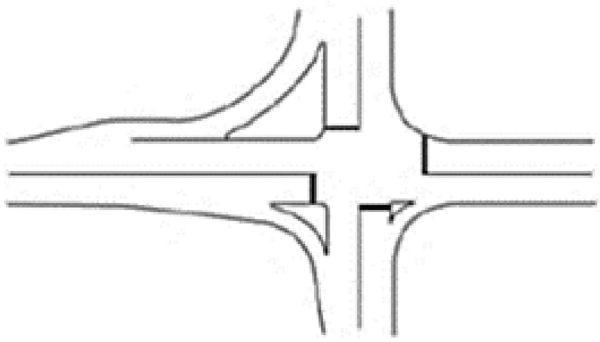


Figure 2. 14 Type C Intersection

D. Roundabouts

The roundabout is a channelized intersection with one-way traffic flow circulating around a central island as show in Figure 2.15. All traffic—through as well as turning—enters this one-way flow. Although usually circular in shape, the central island of a roundabout can be oval or irregularly shaped.

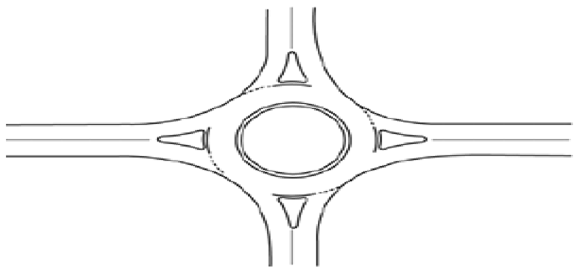


Figure 2. 15 Type D Intersection

Roundabouts differ from "rotaries" in the following respects:

- Size — Single lane roundabouts have an outside diameter between 80 and 140 feet, whereas, rotaries are typically much larger with diameters as large as 650 feet.

- Speed — the small diameter of roundabouts limits circulating vehicle speeds to

10 to 25 miles per hour, whereas, circulating speeds at rotaries is typically 30 to 40 miles per hour.

- Capacity — the slower circulating speeds at roundabouts allow entering vehicles to accept smaller gaps in the circulating traffic flow, meaning more gaps are available, increasing the volume of traffic processed. At rotaries, vehicles need larger gaps in the circulating traffic flow reducing the volume of traffic processed.
- Safety — the slower speeds at roundabouts not only reduce the severity of crashes, but minimizes the total number of all crashes, whereas, rotaries typically see high numbers of crashes with a greater severity [9].

2.6.1.2 Grade-separated intersections or interchanges

It is a bridge that eliminates crossing conflicts at intersections by vertical separation of roadways in space. Grade separated intersection are otherwise known as Interchanges. It causes less hazard and delay than grade intersections. The ultimate objective of grade separated intersections is to eliminate all grade crossing conflicts and to accommodate other intersecting maneuvers by merging, diverging and weaving at low relative speed**.** One of the distinctions made in type of Interchange is between the directional and the non-directional interchange.

- Directional interchanges are those having ramps that tend to follow the natural direction of movement.
- Non directional interchanges require a change in the natural path of traffic flow.

And major interchanges as show below:

- Underpass
- Overpass
- Trumpet Interchange
- Diamond Interchange
- Cloverleaf Interchange
- Partial Cloverleaf Interchange

- Directional Interchange
- Bridged Rotary

- **Underpass**

An underpass or a tunnel is an underground passageway, completely enclosed except for openings for ingress and egress, commonly at each end. A tunnel may be for foot or vehicular road traffic, for rail traffic as show in Figure2.16.

Figure 2. 16 Underpass Intersection

- Overpass

An overpass also known as a flyover is a bridge, road, railway or similar structure that crosses over another road or railway as show in Figure 2.17. A pedestrian overpass allows pedestrians safe crossing over busy roads without impacting traffic [16].

Figure 2. 17 Overpass Intersection

- **Trumpet interchange:**

Trumpet interchange is a popular form of three leg interchange. If one of the legs of the interchange meets a highway at some angle but does not cross it, then the interchange is called trumpet interchange. A typical layout of trumpet interchange is shown in Figure 2.18 below.

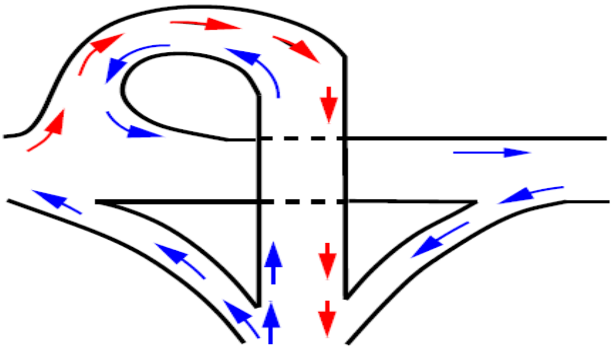


Figure 2. 18 Trumpet Intersection

- **Diamond interchange:**

Diamond interchange is a popular form of four-leg interchange found in the urban locations where major and minor roads crosses. The important feature of this interchange is that it can be designed even if the major road is relatively narrow. A typical layout of diamond interchange is shown in figure 2.19.

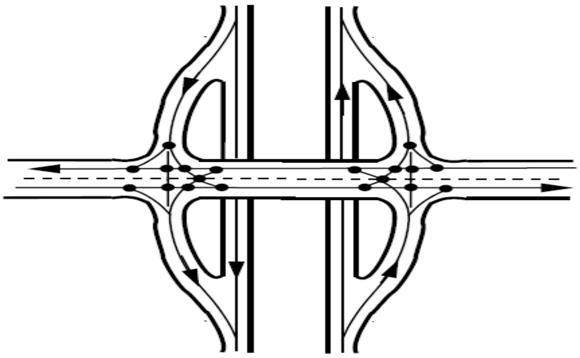


Figure 2. 19 Diamond Intersection

- **Clover leaf interchange:**

It is also a four road interchange and is used when two highways of high volume and speed intersect each other with considerable turning movements. The main advantage of cloverleaf intersection is that it provides complete separation of traffic in addition; high speed at intersections can be achieved. However, the disadvantage is that large area of land is required. Therefore, cloverleaf interchanges are provided mainly in rural areas [17].

- **Partial Cloverleaf Interchange**

Partial clover leaf is a modification that combines some elements of a diamond interchange with one or more loops of a cloverleaf to eliminate only the more critical turning conflicts. It provides more acceleration and deceleration space on the freeway. The atypical partial cloverleaf is show in Figure 2.20.

Figure 2. 20 Partial Cloverleaf Interchange

- **Directional Interchange**

A Directional interchange provides direct paths for left turns. These interchanges contain ramps for one or more direct or semi direct left turning movements. Interchanges of two freeways or interchanges with one or more very heavy turning movements usually warrant direct ramps, which have higher speeds of operation and higher capacities, compared to loop ramps as show in Figure 2.21 [16].

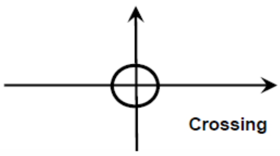
Figure 2. 21 Directional Interchange

2.6.2 Conflict point in Intersection

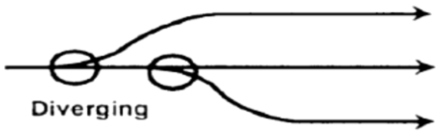
Intersections are more complicated areas for drivers than uninterrupted facilities. Drivers have to make split second decisions within intersections by considering their routes, intersection geometry, and speeds and directions of other vehicles etc. A small error in judgment can cause accidents. So that Main function of intersections is to provide change of direction. Therefore, direction changes within intersections define conflict points. Because. Maneuvers were within intersections because delays basic types of conflict points within intersections as show in Figure 2.22. Typical conflict points are:

- Crossing conflicts (through traffic, left turns with through traffic)
- Merging conflicts
- Diverging
- Weaving (sometime)

Crossings: may be direct, if the angle of skew is between 75 and 105 degrees, or oblique if the angle is in the range of below 75 or above 105degrees. (Oblique skews should be voided if at all possible).

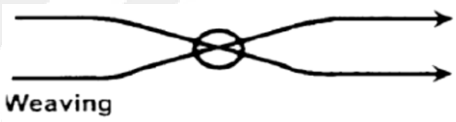


Diverging: is a traffic operation when the vehicles moving in one direction is separated into different streams according to their destinations.



Merging: is the opposite of diverging. Merging is referred to as the process of joining the traffic coming from different approaches and going to a common destination into a single stream.

Weaving: is the combined movement of both merging and diverging movements in the same direction [9].



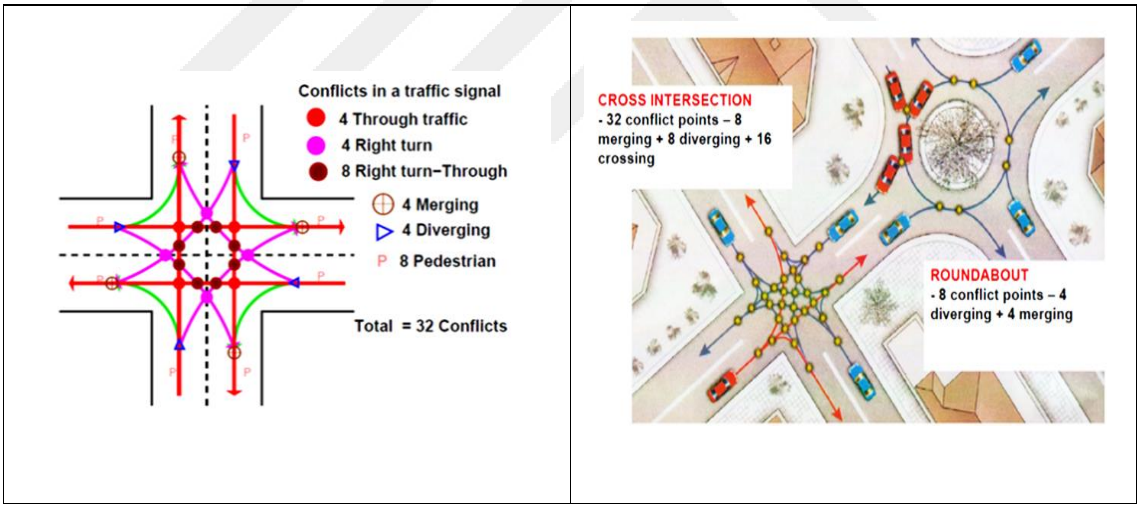


Figure 2. 22 Conflict Point in Intersection

2.7 Levels of intersection control

The control of an intersection can be exercised at different levels. They can be either passive control, semi control, or active control. In passive control, there is no explicit control on the driver. In semi control, some amount of control on the driver is there from the traffic agency. Active control means the movement of the traffic is fully controlled by the traffic agency and the drivers cannot simply maneuver the intersection according to his choice.

2.7.1 Passive Control

When the volume of traffic is less, no explicit control is required. Here the road users are required to obey the basic rules of the road.

No control: If the traffic coming to an intersection is low, then by applying the basic rules of the road like driver on the left side of the road must yield and that through movements will have priority than turning movements. The driver is expected to obey these basic rules of the road.

Traffic signs: With the help of warning signs, guide signs etc. it is able to provide some level of control at an intersection.

Traffic signs plus marking: In addition to the traffic signs, road markings also complement the traffic control at intersections.

2.7.2 Semi Control

In semi control or partial control, the drivers are gently guided to avoid conflicts. Channelization and traffic rotaries are two examples of this.

Channelization: The traffic is separated to flow through definite paths by raising a portion of the road in the middle usually called as islands distinguished by road markings. The conflicts in traffic movements are reduced to a great extent in such a case

Traffic rotaries: It is a form of intersection control in which the traffic is made to flow along one direction around a traffic island

2.7.3 Active Control

Active control implies that the road user will be forced to follow the path suggested by the traffic control agencies. He cannot maneuver according to his wish. Traffic signals and grade separated intersections come under this classification.

Control using traffic signal is based on time sharing approach. At a given time, with the help of appropriate signals, certain traffic movements are restricted where as certain other movements are permitted to pass through the intersection. The intersections are of two types. They are at-grade intersections and grade-separated intersections. In at-grade intersections, all roadways join or cross at the same vertical level. Grade separated intersections allows the traffic to cross at deferent vertical levels. Sometimes the topography itself may be helpful in constructing such intersections [17].

2.8 Control Type of Intersection

Intersections are an important part of highway design. They comprise only a small percentage of the overall highway system miles, yet they account for a high percentage of reported crashes and the majority of potential transportation conflict areas. Intersection control choice requires consideration of all potential users of the facility, including drivers of motorcycles, passenger cars, heavy vehicles of different classifications, public transit, and bicyclists and pedestrians. The intent of intersection control type analysis is not to design an intersection, but to evaluate the compatibility of different intersection control types with respect to context, modal priority, intersection design vehicle and the identified balance of performance needs [18].

Common Types of Intersection Control:

- Intersections with no control
- Intersections with stop control on the minor road
  - Left turn from the minor road
  -Right turn from the minor road
  -Crossing maneuver from the minor road
- Intersections with yield control on the minor road
  -Crossing maneuver from the minor road
  -Left or right turn from the minor road
- Intersections with traffic signal control
- Intersections with all-way stop control
- Left turns from the major road [12].

- **Uncontrolled Intersections**

Uncontrolled intersections do not have signing, and the normal right of way rule applies. Most uncontrolled intersections are found on local roads and streets where the volumes of the intersecting roadways are low and roughly equal, speeds are low, and there is little to no crash history. Uncontrolled intersections are generally not appropriate for intersections with state routes [18].

- **Intersections with stop control on the minor road**

Departure sight triangles for intersections with stop control on the minor road should be considered for three situations:

- Left turns from the minor road;
- Right turns from the minor road; and
- Crossing the major road from a minor-road approach.

Intersection sight distance criteria for stop-controlled intersections are longer than stopping sight distance to ensure that the intersection operates smoothly. Minor-road vehicle operators can wait until they can proceed safely without forcing a major-road vehicle to stop

- **Intersections with yield control on the minor road**

Drivers approaching yield signs are permitted to enter or cross the major road without stopping, if there are no potentially conflicting vehicles on the major road. The sight distances needed by drivers on yield-controlled approaches exceed those for stop-controlled approaches.

- **Intersections with All-Way Stop Control**

At intersections with all-way stop control, the first stopped vehicle on one approach should be visible to the drivers of the first stopped vehicles on each of the other approaches. There are no other sight distance criteria applicable to intersections with all-way stop control and, indeed, all way stop control may be the best option at a limited number of intersections where sight distance for other control types cannot be attained.

- **Left Turns from the Major Road**

All locations along a major highway from which vehicles are permitted to turn left across opposing traffic, including intersections and driveways, should have sufficient sight distance to accommodate the left-turn maneuver. Left-turning drivers need sufficient sight distance to decide when it is safe to turn left across the lane(s) used by opposing traffic. Sight distance design should be based on a left turn by a stopped vehicle, since a vehicle that turns left without stopping would need less sight distance.

- **Intersections with Traffic Signal Control**

At signalized intersections, the first vehicle stopped on one approach should be visible to the driver of the first vehicle stopped on each of the other approaches. Left-turning vehicles should have sufficient sight distance to select gaps in oncoming traffic and complete left turns.

That benefit from the signalization intersection:

- Reductions in delay for the through vehicles.
- Reductions in delay for vehicles waiting to turn left.
- Reductions in delay for drivers of vehicles that have turned left and are traveling to the final crossing on the through approach.
- Reductions in the number of stops for all vehicles.
- Increase in efficiency for pedestrian crossings on all intersection roads [12].

CHAPTER 3

MICRO-SIMULATION

Micro simulation is a tool for analyzing the performance of roadways that have complicated geometric configurations and congested traffic. The core of a micro simulation model is a set of mathematical algorithms that evaluate the motion of each vehicle on a second-by-second basis as it interacts with the roadway network, the traffic control system, and other vehicles in the network [19]. Is also a term used in traffic modeling and is typified by software packages such as Trans Modeler, PTV VISSIM, TSIS-CORSIM, Cube Dynasim, LISA+, Quadstone Paramics, SiAS Paramics, Simtraffic, Aimsun, and MATSim. Analytical modelling software such as LINSIG, TRANSYT, TRANSYT-7F or SIDRA INTERSECTION represent a different class of models based on mathematical algorithms representing combinations of traffic model elements [20].

And in some time there so many models use in road micro simulation traffic elements to be more easy to use to road [21]. As seen in Table 3.1.

Table 3. 1 List of micro-simulation models

Model	Organisation	Country
AIMSUN 2	Universitat Politècnica de Catalunya, Barcelona	Spain
ANATOLL	ISIS and Centre d'Etudes Techniques de l'Equipement	France
AUTOBAHN	Benz Consult - GmbH	Germany
CASIMIR4	Institut National de Recherche sur les Transports et la Sécurité	France
CORSIM5	Federal Highway Administration	USA

Table 3.1 (cont'd)

DRACULA	Institute for Transport Studies, University of Leeds	UK
FLEXSYT II	Ministry of Transport	Netherlands
FREEVU	University of Waterloo, Department of Civil Engineering	Canada
FRESIM	Federal Highway Administration	USA
HUTSIM	Helsinki University of Technology	Finland
INTEGRATION	Queen's University, Transportation Research Group	Canada
MELROSE	Mitsubishi Electric Corporation	Japan
MICROSIM	Centre of parallel computing (ZPR), University of Cologne	Germany
MICSTRAN	National Research Institute of Police Science	Japan
MITSIM	Massachusetts Institute of Technology Netherlands	USA
MIXIC	Organisation for Applied Scientific Research - TNO	Netherlands
NEMIS	Mizar Automazione, Turin	Italy
PADSIM	Nottingham Trent University - NTU	UK
PARAMICS	The Edinburgh Parallel Computing Centre and Quadstone Ltd	UK
PHAROS	Institute for simulation and training	USA
PLANSIM-T	Centre of parallel computing (ZPR), University of Cologne	Germany
SHIVA	Robotics Institute - CMU	USA
SIGSIM	University of Newcastle	UK
SIMDAC	ONERA - Centre d'Etudes et de Recherche de Toulouse	France
SIMNET	Technical University Berlin	Germany
SISTM	Transport Research Laboratory, Crowthorne	UK
SITRA-B+	ONERA - Centre d'Etudes et de Recherche de Toulouse	France

Table 3.1 (cont'd)

SITRAS	University of New South Wales, School of Civil Engineering	Australia
TRANSIMS	Los Alamos National Laboratory	USA
THOREAU	The MITRE Corporation	USA
TRAF-NETSIM	Federal Highway Administration	USA
VISSIM PTV	System Software and Consulting GMBH	Germany

Micro-simulation models can be classified according to the traffic conditions they can be applied to as given in Table 3.2.

Table 3. 2 Type of four models

Urban	**Motorway**	**Combined**	**Other**
CASIMIR DRACULA HUTSIM MICSTRAN NEMIS NETSIM PADSIM SIGSIM SIMNET SITRA-B+ SITRAS	AUTOBAHN FREEVU FRESIM MIXIC SISTM	AIMSUN2 CORSIM FLEXSYT II INTEGRATION	ANATOLL PHAROS SHIVA SIMDAC MELROSE MICROSIM MITSIM PARAMICS PLANSIM-T TRANSIMS VISSIM

3.1 Simulation Tools

Macroscopic simulation models are based on the deterministic relationships of the flow, speed, and density of the traffic stream. The simulation in a macroscopic model takes place on a section-by-section basis rather than by tracking individual vehicles. Macroscopic simulation models were originally developed to model traffic in distinct transportation sub networks, such as freeways, corridors (including freeways and parallel arterials), surface-street grid networks, and rural highways.

Microscopic simulation models simulate the movement of individual vehicles based on car-following and lane-changing theories. These models are effective in evaluating heavily congested conditions, complex geometric configurations, and system-level impacts of proposed transportation improvements that are beyond the limitations of other tool types. However, these models are time consuming, costly, and can be difficult to calibrate [12].

3.2 Car Fallowing Models

Car following is the process by which drivers guide their vehicles when following another vehicle. Car-following decisions are more complex than road-following decisions because they involve speed-control modifications. In car following, drivers need to constantly modify their speed to maintain safe gaps between vehicles [12]. So it is a class of scientific models of vehicular traffic dynamics. In contrast to macroscopic models, microscopic traffic flow models simulate single vehicle-driver units, so the dynamic variables of the models represent microscopic properties like the position and velocity of single vehicles.

In VISSIM microsoft there were for car following model a Wiedemann car-following was constructed based on conceptual development and limited available data, and has to be calibrated to specific traffic stream data.

A driver follows another vehicle by judging**:**

a) Distance
b) Speed difference
c) Reaction time
d) Vehicle performance

The relatively simple and common driving task of one vehicle following another on a straight roadway where there is no passing (neglecting all other subsidiary tasks such as steering, routing, etc.) can be categorized in three specific subtasks:

Perception: The driver collects relevant information mainly through the visual channel. This information arises primarily from the motion of the lead vehicle and the driver's vehicle.

Decision Making: A driver interprets the information obtained by sampling and integrates it over time in order to provide adequate updating of inputs.

The microscopic approaches focused on describing the detailed manner in which one vehicle followed another. With such a description, the macroscopic behavior of single lane traffic flow can be approximated. Hence, car following models form a bridge between individual "car following" behavior and the macroscopic world of a line of vehicles and their corresponding flow and stability properties.

Control: The skilled driver can execute control commands with dexterity, smoothness, and coordination, constantly relying on feedback from his own responses which are superimposed on the dynamics of the system's counterparts (lead vehicle and roadway) [22].

Before going in to the details, various notations used in car-following models are discussed here with the help of Figure 3.1.

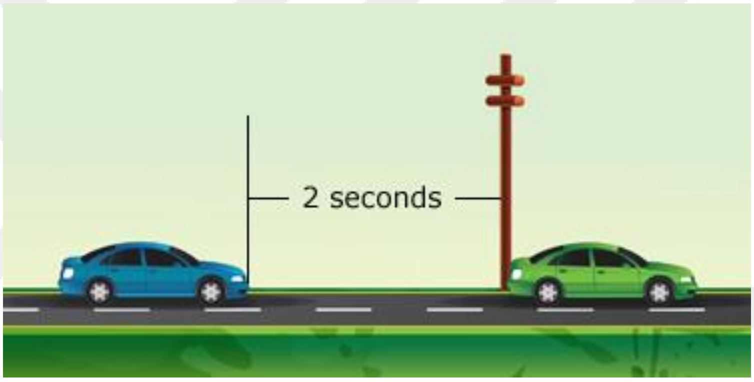


Figure 3. 1 Car Following

The car following models most general is:

- Pipe’s model: The basic assumption of this model is “A good rule for following another vehicle at a safe distance is to allow yourself at least the length of a car between your vehicle and the vehicle ahead for every ten miles per hour of speed at which you are traveling” According to Pipe’s car-following model, the minimum safe distance headway increases linearly with speed. A disadvantage of this model is that at low speeds, the minimum headways proposed by the theory are considerably less than the corresponding field measurements.
- Forbes’ model: In this model, the reaction time needed for the following vehicle to perceive the need to decelerate and apply the brakes is considered. That is, the time gap between the rear of the leader and the front of the follower should always be equal to or greater than the reaction time. Therefore, the minimum time headway is equal to the reaction time (minimum time gap) and the time

required for the lead vehicle to traverse a distance equivalent to its length. A disadvantage of this model is that, similar to Pipe's model, there is a wide difference in the minimum distance headway at low and high speeds

- General Motors' model: is the most popular of the car-following theories because of the following reasons:
  1- Agreement with field data; the simulation models developed based on General motors' car following models shows good correlation to the field data.
  2- Mathematical relation to macroscopic model; Greenberg's logarithmic model for speed-density relationship can be derived from General motors' car following model.

The basic philosophy of car following model is from Newtonian mechanics, where the acceleration may be regarded as the response of a matter to the stimulus it receives in the form of the force it receives from the interaction with other particles in the system. Hence, the basic philosophy of car-following theories can be summarized by the following equation

$$[\text{Response}]_n \alpha \quad [\text{Stimulus}]_n \tag{3.1}$$

for the nth vehicle (n=1, 2, ...). Each driver can respond to the surrounding traffic conditions only by accelerating or decelerating the vehicle. As mentioned earlier, different theories on car-following have arisen because of the difference in views regarding the nature of the stimulus.

- Follow-the-leader model: The car following model proposed by General motors' is based on follow-the leader concept. This is based on two assumptions; (a) higher the speed of the vehicle, higher will be the spacing between the vehicles and (b) to avoid collision, driver must maintain a safe distance with the vehicle ahead.

In computer, implementation of the simulation models, three things need to be remembered:

1. A driver will react to the change in speed of the front vehicle after a time gap called the reaction time during which the follower perceives the change in speed and react to it.

2. The vehicle position, speed and acceleration will be updated at certain time intervals depending on the accuracy required. Lower the time interval, higher the accuracy.
3. Vehicle position and speed is governed by Newton's laws of motion, and the acceleration is governed by the car following model.

Therefore, the governing equations of a traffic flow can be developed as below. Let ΔT is the reaction time, and Δt is the updating time, the governing equations can be written as,

$$v^t_n = v_n^{t-\delta t} + a_n^{t-\delta t} * \delta t \quad (3.2)$$

$$x^t_n = v_n^{t-\delta t} + a_n^{t-\delta t} * \delta t + \frac{1}{2} a_n^{t-\delta t} \delta t^2 \quad (3.3)$$

$$a_{n+1}^t = \left[\frac{\propto_{t.m} (v_{n+1}^{t-\delta t})^m}{(x_n^{t-\Delta\tau} - x_{n+1}^{t-\Delta\tau})^t}\right] (v_n^{t-\Delta\tau} - v_{n+1}^{t-\Delta\tau} \quad (3.4)$$

The equation (3.2) is a simulation version of the Newton's simple law of motion v = u + at and equation (3.3) is the simulation version of the Newton's another equation s = ut+ at2. The acceleration of the follower vehicle depends upon the relative velocity of the leader and the follower vehicle, sensitivity coefficient and the gap between the vehicles.

- Optimal velocity model: The concept of this model is that each driver tries to achieve an optimal velocity based on the distance to the preceding vehicle and the speed difference between the vehicles. This was an alternative possibility explored recently in car-following models. The formulation is based on the assumption that the desired speed undesired depends on the distance headway of the nth vehicle. i.e. undesired = vopt(Δxnt) where v opt is the optimal velocity function which is a function of the instantaneous distance headway Δxnt. Therefore, a nt is given by

$$a^t_{n+1} = 1/T\ [\ v^{opt}(\Delta x^t_n) - v^t_n] \quad (3.5)$$

where $\frac{1}{\tau}$ is called as sensitivity coefficient. In short, the driving strategy of nth vehicle is that, it tries to maintain a safe speed which in turn depends on the relative position, rather than relative speed [23, 24].

3.3 Lane Changing Models

The transfer of a vehicle from one lane to adjacent lane is defined as lane change. Lane changing has significant impact on traffic flow. Lane changing models are therefore an important component in microscopic traffic simulation modeling the behavior of a vehicle within its present lane is relatively straightforward, as the only considerations of any importance are the speed and location of the preceding vehicle. Lane changing, on the other hand, is more complex, because of the decision to change lanes depends on several objectives, and at times some of these may conflict [24].

3.3.1 Basic Lane Changing Model

The basic lane change model is described using the framework shown in Figure 3.2. The subject vehicle in the current lane tries to change direction either to its left or to its right. If the gap in the selected lane is acceptable the lane change occurs or else it will remain in the current lane [24].

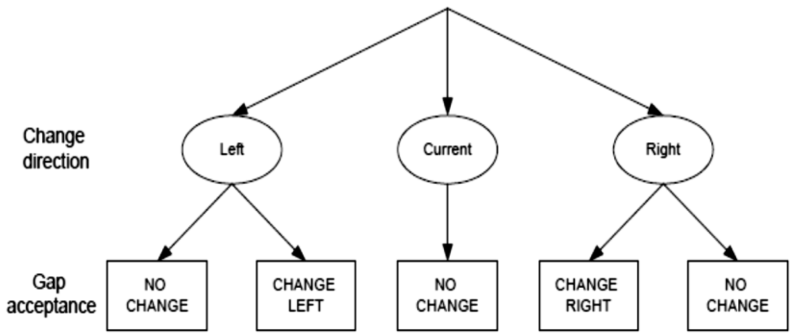


Figure 3. 2 Basic Lane Change Model

3.3.2 Explicit Target Lane Choice

The decision to seek a lane change and the direction of change in the models introduced so far have been based on an evaluation of the current lane and the adjacent lanes on the right and on the left. Therefore, in these models, the set of lanes the driver chooses from depends on the lane the vehicle is currently in. In multi-lane road facilities, only a subset of the available lanes is evaluated. This approach may result in unrealistic behavior in cases where drivers change lanes not because the lane they are changing to is preferable, but as a step on their way to another lane further away in the lane change direction. This type of situation may arise, for example, in multi-lane freeways with dedicated lanes (e.g. HOV lanes). Drivers may change lanes in the direction of the dedicated lane, even to lanes with undesirable characteristics (e.g. slower speeds) in order to eventually enter the dedicated lane, which may provide higher level of service. In order to tackle this problem, the model shown in Figure 3.3(for a driver currently in the second lane from the right, lane 2, of a four-lane freeway) has been tested. This model introduces an explicit target lane selection model. Rather than choosing a change direction, drivers choose a target lane among all the available lanes. The target lane is the lane that is perceived as the best lane to be in taking multiple factors and goals into account. The direction of a desired lane change, if any, is dictated by the direction of the target lane from the current lane the vehicle is in. As with previous models, the completion of the lane change depends on its feasibility, which is captured by gap acceptance models. Estimation of this model with trajectory data showed that important factors that affect the utilities of the various lanes are the microscopic and macroscopic traffic flow characteristics in the lane (e.g. Presence of heavy vehicles, average speed and density), the impact of the path-plan (e.g. whether it would be a correct one to follow the path), an inertia factor (e.g. whether it is the current lane and if not, the number of lane changes that would be needed to get to it) and characteristics of the driver (e.g. aggressiveness) [25].

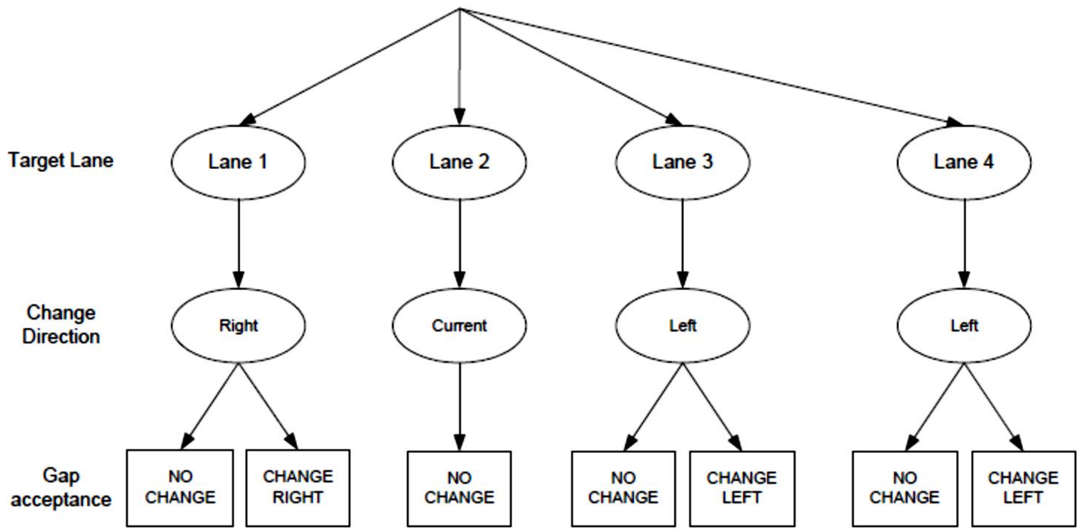


Figure 3. 3 Explicit Target Lane Change

3.3.3 Classification of Lane Change

The classification of lane changes is done based on the execution of the lane change and ac -condignly two types of lane changes exists.

Mandatory Lane Change (MLC): occurs when a driver must change lane to follow a specified path Suppose if a driver wants to make a right turn at the next intersection the he changes to the right most lane which is referred as Mandatory Lane change.

Discretionary Lane Change (DLC): occurs when a driver changes to a lane perceived to offer better traffic conditions, he attempts to achieve desired speed, avoid following trucks, avoid merging traffic, etc. Suppose if a driver perceives better driving conditions in the adjacent lane then he makes a Discretionary Lane change.

Forced merging model: happens if the gap on the target lanes not acceptable then the subject vehicle forces the lag vehicle on the target to decelerate until the gap is acceptable.

Cooperative Merging: The models discussed so far assume that lane changing is executed through gap acceptance. However, in congested traffic conditions acceptable gaps may not be available, and the resultant behavior would be different. For example, drivers may change lanes through courtesy and cooperation of the lag vehicles on the target lane that will slow down in order to accommodate the lane change.

Application of different lane: changing models most models classify lane changes as either mandatory or discretionary lane change. This separation implies that there are no trade-offs between mandatory and discretionary considerations [24].

3.4 Lane Changing Process

There are no analytic relationships that encompass the entire lane changing process. Instead, it is typically modeled as a sequence of several decision-making steps such as:

a. Desire to change the current lane
b. Selection of the target lane
c. Ensuring lane change is feasible
d. Decision to change lane based on gap acceptance

Lane changing process is explained using an example shown in Figure 3.4 where the subject vehicle traversing in the middle lane of a three-lane road undergoes lane changing. Each step of the lane changing process is explained in detail referring to the example. Note that a discretionary lane change is considered here [24].

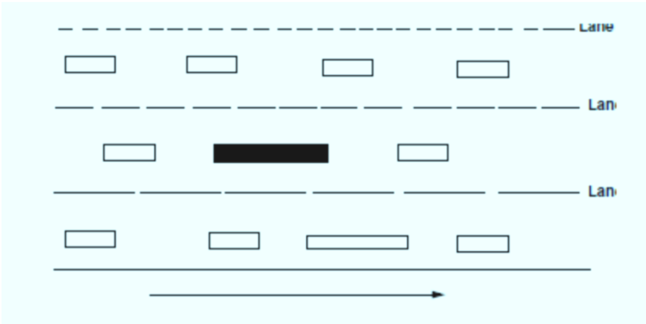


Figure 3. 4 Lane Change

1- Desire to change the lane: driver characteristics and behavior. Lane changes may be performed due to several factors such as reduced speed in the current lane, queuing, forced deceleration because of the lead vehicle, etc. The desire to change the lane becomes stronger when the driver also perceives a higher utility in the target lane in terms of higher speed or higher acceleration or a better position in the queue. Here we assume that the first step of deciding whether to change the lane arises basically from the current acceleration of the vehicle [24].

2- Selection of the target lane: when there is a desire to change the lane, the driver then targets a lane to shift. Modeling this decision is more for complex for discretionary lane changes, where the driver needs to select a lane based on several factors, such as queue length, operating speed, etc. (Discussion of this is outside the scope of this chapter) A simpler way of modeling target lane selection is based on the concept of utility maximization.

$$P(i) = \frac{e^{vi}}{\sum_{i=1}^{n} e^{ut}} \qquad (3.6)$$

Where n is the number of lanes. It is assumed that the driver will choose the lane that has the maximum probability as his target lane. Note that, in real traffic simulation, a random number is generated and used in the decision.

3- Ensuring lane change is Feasible: the lane change is said to be feasible if the subject vehicle will not collide with the rear vehicle in the target lane. For avoiding collision, the deceleration of the rear vehicle in the target lane needs to be less than the critical deceleration.

$$a_{n+1} = \frac{\propto v^{m} \Delta v}{\Delta x^{i}} \qquad (3.7)$$

If an+1 is less than the critical deceleration, it is feasible to change the lane to the selected target lane. Otherwise, the vehicle will continue in the current lane.

4- Decision to change lane based on gap acceptance: a gap is defined as the gap in between the lead and lag vehicles in the target lane The probability of gap acceptance as a function of time can be written as:

$$t = \frac{g}{v_{n+1}} \tag{3.8}$$

Where, g is the distance gap, and vn+1 is the velocity of the vehicle. Probability that the gap is accepted is the product of the probability that the lead gap is accepted and the probability that the lag gap is accepted [24].

3.5 Calibration of Micro simulation

Calibration is the act of regulate a model so that it represents reality as closely as possible Calibration refers to the process of adjusting model parameters and certain simulation inputs such that local traffic conditions, as measured by traffic volume, travel speed and/or time, or combinations of them can be matched closely by simulation outputs [26]. Microscopic simulation is a complex system that all parameters work together to influence its modeling results. In calibrating such a complex model, users could get trapped in the local optima of the objective function, due to the high dimension and numerous local optima [27]. Micro simulation usually involves many parameters that affect simulation results in complex ways parameters are divided into two distinctive groups: input parameters and model parameters, and calibrated one group at a time. The input parameters include those of geometry, controls, vehicle/driver population, and OD demands, while the model parameters are related to driver behavior and can further be categorized, in a microscopic simulation, as car-following, lane change/gap acceptance, and route choice parameters. It is suggested that the driving behavior parameters that apply to all road sections (called global parameters) be calibrated first, so that the flow capacity at typical road sections can be reproduced. Then driving behavior parameters that affect bottleneck capacities (called local parameters) are calibrated next OD demands and traffic control parameters are calibrated to make sure the level and locations of congestion are consistent with observations, as measured by traffic volume, and travel times or delays. Since route choice, OD demands and traffic control all affect traffic circulation patterns. Identified four global model parameters in Paramus: Mean Target Headway, Mean Reaction Time, Driver Aggressiveness, and Driver Awareness. These four parameters affect car-following behavior, hence the flow capacities, on all road sections. Local parameters are those driving behavior parameters that further affect road capacity besides global driving behavior parameters as it shows in Figure 3.5. [26].

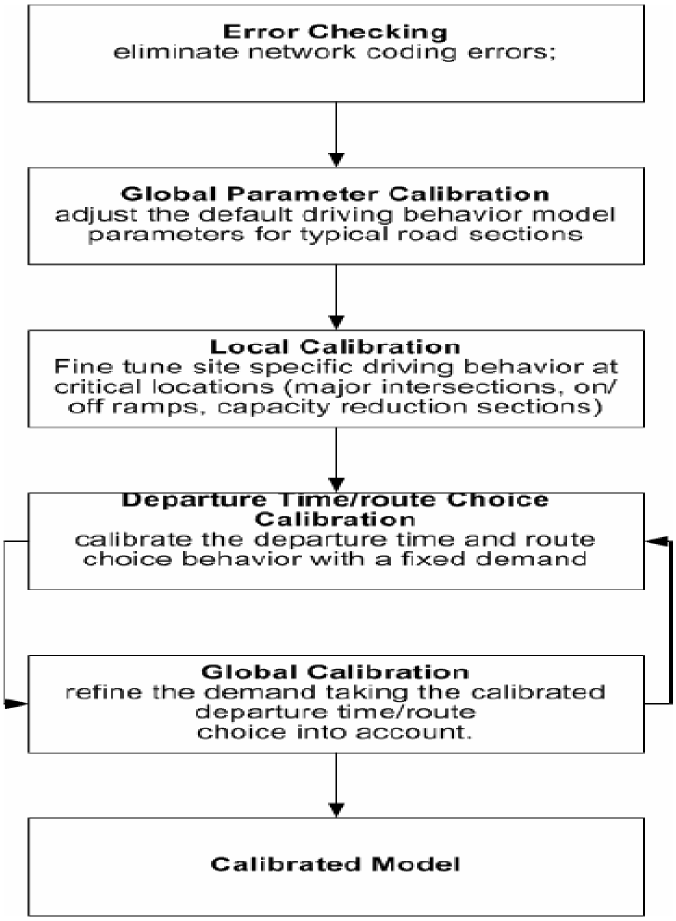


Figure 3. 5 Micro-Simulation Scama

The remaining four steps deal with the calibration of four different groups of parameters:

1. Global parameters that affect driving behavior throughout the network,
2. Local parameters that are peculiar to bottleneck locations, such as lane-drop locations or junctions where several roads meet,
3. Departure time and route choice parameters that affect the distribution of traffic in the network, and

4. Input parameters such as O-D demand that affect congestion levels throughout the network.

The idea of calibrating first the global model parameters then the local model parameters is akin to highway capacity analysis, where one first identifies a set of ideal conditions and the ideal capacity under such conditions, then adjusts the ideal capacity for non-ideal conditions through discount factors to obtain the prevailing capacity. Similarly, we want to identify typical road sections for the calibration of global parameters, and road sections with special features (such as sharp curvatures, lane drops, on-ramps and intersections) for the calibration of local model parameters. Through such calibration, we want to obtain a set of parameters that can reproduce the flow capacities of various types of road sections

Than the validation checks whether the model is valid if not valid, then any conclusions derived from it is of virtually no value. Validation and verification are two of the most important steps [26].

3.6 Evaluation Methodology (T-test Hypothesis)

T-test compares two averages (means) and show if they are different from each other. The T-test also shows how significant the differences are; In other words it shows if those differences could have happened by chance [28].

This approach consists of four steps: (1) state the hypotheses, (2) formulate an analysis plan, (3) analyze sample data, and (4) interpret results.

State of hypothesis: Every hypothesis test requires the analyst to state a null hypothesis and an alternative hypothesis. The hypotheses are stated in such a way that they are mutually exclusive. That is, if one is true, the other must be false; and vice versa [29].

Formulate an analysis: the formula for the t-test is a ratio.

$$\text{T-test} = \frac{\bar{X}_T - \bar{X}_C}{SE(\bar{X}_T - \bar{X}_C)} \tag{3.9}$$

$$SE(\bar{X}_T - \bar{X}_C) = \sqrt{\frac{S_T - S_C}{N_T - N_C}} \tag{3.10}$$

S = standard deviation

N= size of sample

T-test is difference between two group means divided by variability of group. The top part of the ratio is just the difference between the two means or averages. The bottom part is a measure of the variability or dispersion of the scores. The difference between the means is the signal that, the bottom part of the formula is a measure of variability that is essentially noise that may make it harder to see the group difference.

To test the significance, must have a set risk level (called the alpha level). In most social research, the "rule of thumb" is to set the alpha level at .05. This means that five times out of a hundred that would find a statistically significant difference between the means even if there was none (i.e., by "chance"). And also need to determine the degrees of freedom (df) for the test and the (df) was the exact shape of a t distribution is determined by its degrees of freedom. When the t distribution is used to compute a confidence interval for a mean score, one population parameter (the mean) is estimated from sample data as shown in Figure 3.6.

$DF = (s_1^2/n_1 + s_2^2/n_2)^2 / \{[(s_1^2 / n_1)^2 / (n_1 - 1)] + [(s_2^2 / n_2)^2 / (n_2 - 1)]\}$ (3.11)

In this study the Microsoft excel has been used for this purpose [28].

Analyze sample data and interpret results: as the conclusion, if t Stat < -t Critical two-tail or t Stat > t Critical two-tail, we reject the null hypothesis.

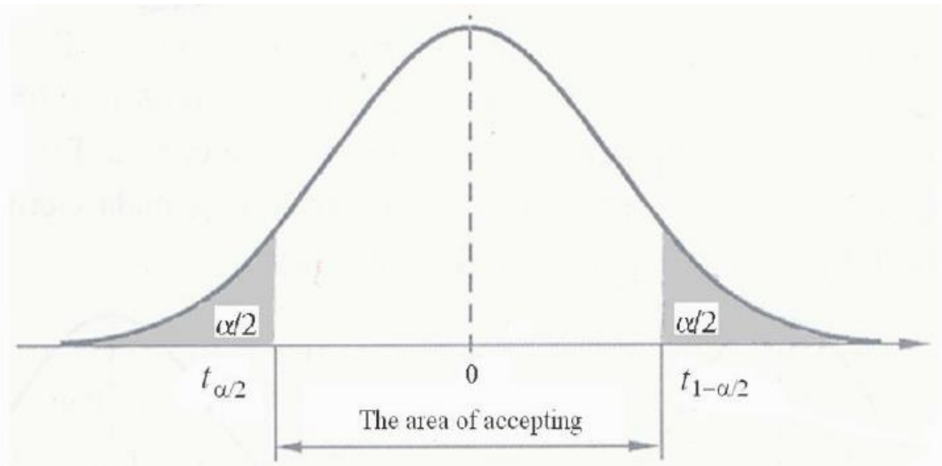


Figure 3. 6 T-Test Diagram

For tow sample in t-test need to:

- The samples are independent
- Each population at least 20 times larger than its respective sample

- The sampling distribution is approximately normal, which is generally the case if any of the following conditions apply.
- The population distribution is normal.
- The population data are slightly skewed, unimodal, without outliers, and the sample size is 16 to 40.
- The sample size is greater than 40, without outliers [29].

3.7 VISSIM Software Program

The simulation model used in this research was VISSIM (9), version 9.07.5256. VISSIM is a microscopic, time step, and behavior-based simulation model. The model was developed at the university of Karlsruhe, Germany, during the early 1970s and the commercial distribution of VISSIM was launched in 1993 by PTV Trans world AG. In the United States, ITC Inc. distributes and supports the program as show in Figure 3.7. Essential to the accuracy of a traffic simulation model is the quality of the actual modeling of vehicles or the methodology of moving vehicles through the network.

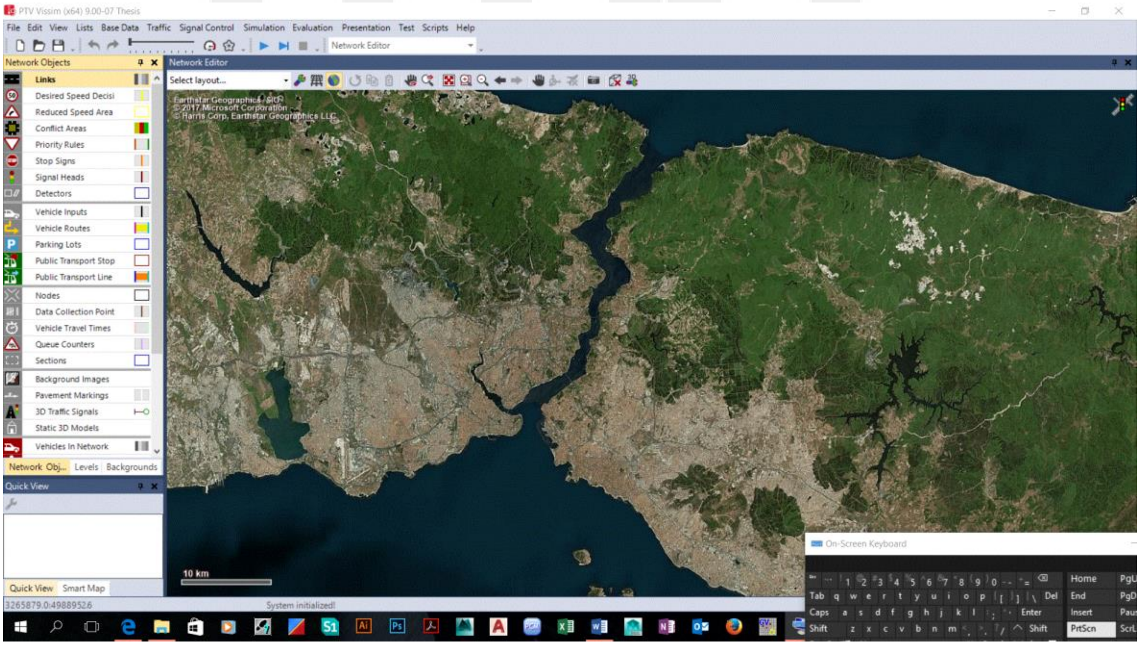


Figure 3. 7 PTV VISSIM Screem

VISSIM allows simulating traffic patterns exactly. Motorized private transport, goods transport, rail and road related public transport, pedestrians and cyclists – as the world's leading software for microscopic traffic simulation, PTV Vissim displays all road users and their interactions in one model. Scientifically sound motion models provide a realistic modeling of all road users as found the parameter in figure 3.8.

No.	Parameter	
1	*Simulation Resolution*	
2	Car Following Parameter	Maximum Look Ahead Distance
3		Average Standstill Distance
4		Additive Part of Safety Distance
5		Multiple Part of Safety Distance
6		CC0, Average Standstill Distance
7		CC1, Headway Time
8		CC2, "Following" Variation
9		CC3, Threshold for Entering "Following"
10		CC4, Negative "Following" Threshold
11		CC5, Positive "Following" Threshold
12		CC6, Speed Dependency of Oscillation
13		CC7, Oscillation Acceleration
14		CC8, Standstill Acceleration
15		CC9, Acceleration at 80km/h
16	Lane Change Parameter	Maximum Deceleration
17		Reduction Rate (-1m/s^2 per Distance)
18		Accepted Deceleration
19		Waiting Time Before Diffusion
20		Minimum Headway
21	Desired Speed Distribution	
22	Priority Rule	Minimum Gap Time
23		Minimum Headway

* Note: The range of each parameter was not presented because no specific ranges vary with different cases.

Figure 3. 8 PTV VISSIM Model Parameters

Car Following Parameter that is available:

- Maximum Look Ahead Distance value is maximum distance allowed for looking ahead, it needs to be extended only in rare occasions (e.g. for modeling railways if signals and stations are to be recognized in time)
- Car following model selects basic model for the vehicle behavior. Depended on the model parameters change, the models are Wiedemann74 and Wiedemann99.
- Wiedemann 74 parameters are:
  - Average Standstill Distance (ax) is average desired distance between stopped cars. It has a variation between -1.0m and +1.0m which normal distributed around 0.0m with a standard deviation of 0.3m.
  - Additive Part of Safety Distance (bx_add) and Multiply Part Safety Distance (bx_mult) affect the computation of the safety distance. The distance d between two vehicles is computed using this formula:

$$d= ax+bx \quad (3.12)$$

Where ax is the standstill distance

$$bx = (bx\_add + bx\_mult * z)\sqrt{v} \tag{3.13}$$

v is the vehicle speed (m/s)

z is a value of range (0.1) which is normal distributed around 0.5 with a standard deviation of 0.15

- Wiedemann99 model parameters are:

- CC0 (standstill distance) is the desired distance between stopped cars.
- CC1 (headway time) is the time in (s) that a driver wants to keep.
- CC2 (following variation) restricts the longitudinal oscillation or how much more distance than the desired safety distance a driver allows before he intentionally moves closer to the car in front.
- CC3 (threshold for entering following) controls the start of the deceleration process.
- CC4 and CC5 (following thresholds) control the speed differences during the following state.
- CC6 (speed dependency of oscillation) influence of distance on speed oscillation while in following process.
- CC7 (oscillation acceleration) actual acceleration during the oscillation process.
- CC8 (standstill acceleration) desired acceleration when starting from standstill (limited by maximum acceleration defined within the acceleration curves).
- CC9 (acceleration at 80 km/h) desired acceleration at 80 km/h (limited by maximum acceleration defined within the acceleration curves).

Lane Change in VISSIM basically has two components:

- Necessary Lane Change, the driving parameters contain the (maximum acceleration, acceptance deceleration and reduction rate (-1m/s2 per distance)) for the vehicle and the trailing vehicle on the new lane, depending on the distance to the emergency stop position of the next connector of the route.
- Free Lane Change, VISSIM checks for the desired safety distance of the trailing vehicle on the new lane and have the parameter:

- Waiting time before diffusion: the maximum amount of time a vehicle can wait at the emergency stop position waiting for a gap to change lanes in order to stay on its route.
- Minimum headway: the minimum distance to the vehicle in front that must be available for a lane change in standstill condition.

The Priority Rules consist of one stop line and one or more conflict markers that are associated with the stop line. The two main conditions to check at the conflict markers are:

- Minimum headway (distance): is determined by the distance between the conflict marker and the first vehicle approaching it.
- Minimum gap time: is determined every time step by the time an approaching vehicle will require reaching the conflict marker.

The software offers flexibility in several respects: the concept of links and connectors allows users to model geometries with any level of complexity. Attributes for driver and vehicle characteristics enable individual parameterization. Furthermore, a large number of interfaces provide seamless integration with other systems for signal controllers, traffic management or emissions models [30].

Functions and application of wide range for traffic engineering issues with VISSIM for an all modeling options like:

complex signalized and unsignalized intersections, traffic circles , U-turns, mixed-flow lanes, separate (2-lane) turn lanes , bicycle paths and lanes shared by different modes (bicycles and motorized vehicles), public transport stops and terminals , pedestrian-vehicle interaction, multi-lane freeways with user-defined curviness and roadway grades, freeway junctions, interchanges and merging and weaving areas, design, tests and evaluation of traffic-actuated signal control systems , design and dimensioning of intersections , modeling of complex public transport intersections , analysis of public transport acceleration measures , evaluation and optimization of intelligent transport systems , visualization of planning alternatives for supporting the political decision making process , creation of development plans, for instance for supermarkets, malls or entire city districts, set-up and coordination of roadwork sites , traffic calming measures ,environmental impact studies, including emission calculations , performance analyses of rail transport systems and planning of parking facilities and modeling of parking-related traffic[31].

Advantages:

- Leading tool for traffic simulation worldwide
- Modeling at the highest level of detail and complexity
- When, where and how data are reported is user defined
- Integrated Multimodal simulation
- Seamless and integrated with Vision Traffic software suite
- Industry leading technical support and service
- Flexible licensing solutions

For calibration in VISSIM model the first stage is volume-based calibration, the second stage is speed-based calibration, and the final (optional) stage is objective-based that depend on thesis research and point of calibrated [30].

CHAPTER 4

STUDY AREA / MALATYA

Malatya, the 27th most populated city in Turkey, as of 2016 had a population of 781,305. It is the largest city in the eastern Anatolia region and is located in the upper Euphrates division of the region as seen in Figure 4.1.

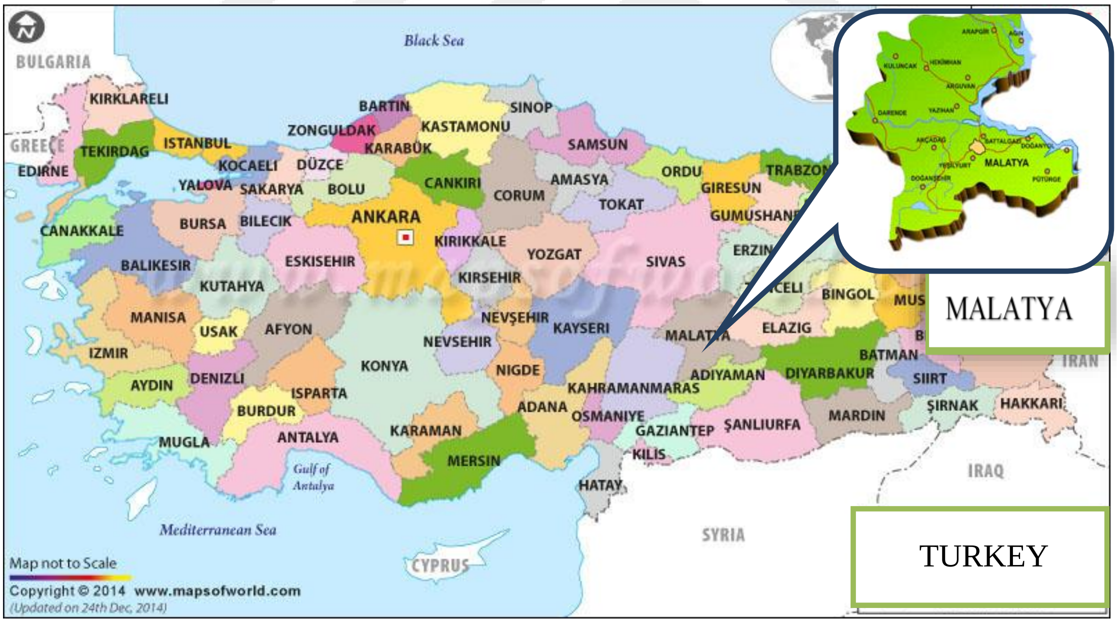


Figure 4. 1 Turkey/Malatya

Malatya highway transportation is provided from state roads between cities which are run by the TCK (General Directorate of Turkish Highways). Malatya was selected because it has one of the most important highways (D300) that connects the city to two other cities (Kayseri and Elazig).

Air transport in Malatya is provided by Erhaç Airport located in Malatya Akçadağ. Railway transportation in Malatya is provided by TCDD (Republic of Turkey State Railways). There are TCDD figures in four districts. These districts are:

- Battal Ghazi
- Akçadağ
- Hekimhan
- Doğanşehir

The Euphrates railway bridge on the Malatya-Elazıg railway line is the biggest and the longest bridge in Turkey [32].

4.1 Study Area

This study was conducted in Malatya, For the study, two intersections have been selected from the right side as shown in in Figure 4.2. The first interconnection known as MASTI has four street connections and the second intersection is known as BOSTANBASI.

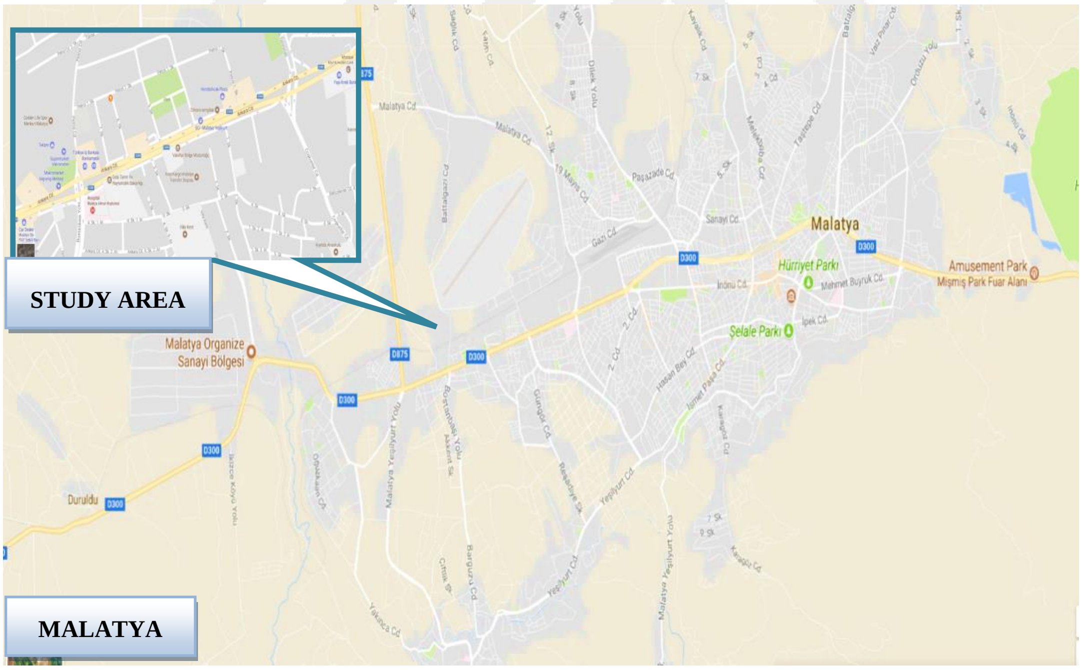


Figure 4. 2 Malatya /Study Area (MASTI and BOSTANBASI Intersections)

4.2 Data Collection

Roadway corridor between two intersections is presented in Figure 4.3.

Figure 4. 3 Study Area from Aerial View

Data of travel time and headway was collected between MASTI and BOSTANBASI intersections. Vehicle count data was collected in two hours from 7:00 am to 9:00 am continuously for the different vehicle types in corridors of the intersections. The municipality of Malatya provided the traffic count data that was used in the study. Travel time data was collected between the two intersections for three vehicle types namely car, van, and truck using video streaming in YTU ITS Lab. The average travel time and standard deviation of the data was calculated as shown in Table 4.1.

Table 4. 1 Average and standard deviation for travel time (s)

Type of Vehicle	Average	Standard Deviation
Car	40	5
Van	52	16
Truck	63	22

4.2.1 Traffic Count

Traffic volume was collected from the field for different types of vehicle from 7:00 am to 9:00 am with 15 minutes' intervals. The data was multiplied by four to convert to hourly volume and then input to the VISSIM software as it is appearing in Table 4.2. Then the vehicles that stopped at red light of each lane was counted

Table 4. 2 Data collection result

Time(s)	Field
0-900	367
900-1800	534
1800-2700	667
2700-3600	685
3600-4500	619
4500-5400	645
5400-6300	548
6300-7200	463

4.2.2 Travel Time Data Collection

Travel time data was collected for the three vehicle types (car, van, and truck) in corridor between MAST and BOSTANBASI intersections using video streaming in YTU ITS Lab. Calculations were made to determine average and standard deviation for travel time as shown in Table 4.3. The desired speed was then calculated by dividing the travel time by the distance of the corridor (738 m).

Table 4. 3 Average and standard deviation for headway(s)

Type of Vehicle	Average	Standard Deviation
Car	40	5
Van	52	16
Truck	63	22

Frequency shows how often something occurs. The frequency of an observation in statistics reveals the number of times the observation occurs in the data. In this study, the frequency was used to analyze the speed data and the time vehicles spend in corridor and to establish the gap between the default speed values provided by the VISSIM model and the field data. The frequency of the speed for the cars in the field occurs at the speed range values from 40 to 90 km/h as shown in the Table 4.4 whereas the default speed results from VISSIM model are in the range of 58-to 68 km/h as shown in Figure 4.4 shows that a gap exists between the model speeds and the field speeds.

Table 4. 4 Frequency speed for car

Speed (km/h)	Frequency	Cumulative fr.	% Frq.	Cumulative frq%
0	0	0	0	0
10	0	0	0	0
20	0	0	0	0
30	0	0	0	0
40	0	0	0	0
50	2	2	0.06	0.06
60	4	6	0.12	0.18
70	14	20	0.42	0.60
80	11	31	0.33	0.93
90	2	33	0.06	1
100	0	33	0	1
110	0	33	0	1
120	0	33	0	1
Sum	33	191	1	

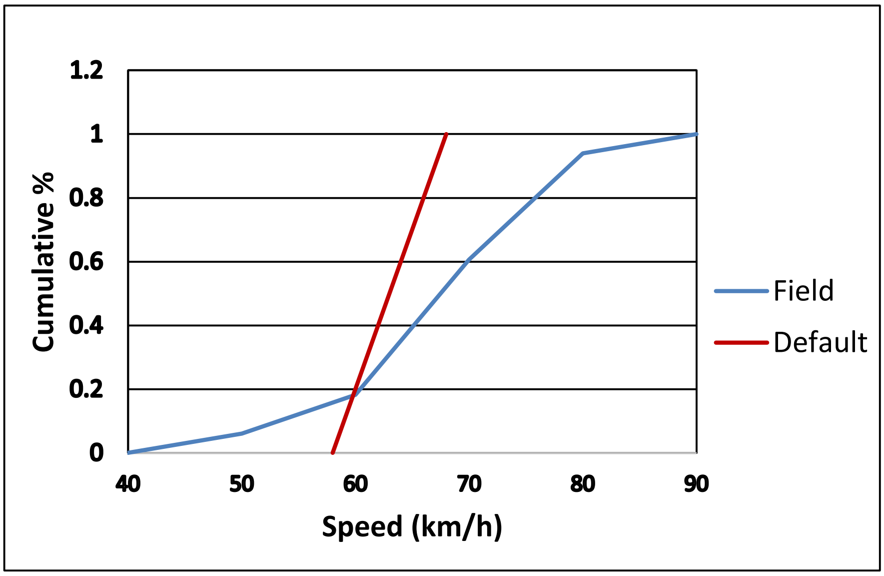


Figure 4. 4 Speed Car in Field and Default Value

Frequency analysis shows that van speeds range from 20 to 80 km/h as shown in Table 4.5 while the default value provided by VISSIM model shows that van speeds range from 40 to 45 km/h as shown in the Figure 4.5.

Table 4. 5 Frequency speed for van

Speed (km/h)	Frequency	Cumulative fr.	% Frq.	Cumulative frq%
0	0	0	0	0
10	0	0	0	0
20	0	0	0	0
30	1	1	0.03	0.03
40	3	4	0.09	0.12
50	7	11	0.21	0.33
60	7	18	0.21	0.54
70	13	31	0.39	0.93
80	2	33	0.06	1
90	0	33	0	1
100	0	33	0	1
110	0	33	0	1
120	0	33	0	1
Sum	33	230	1	

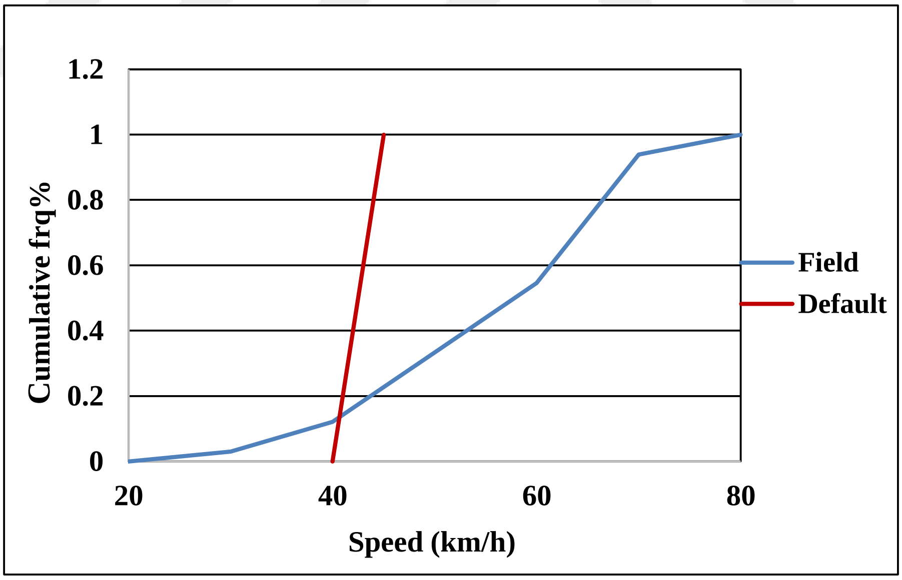


Figure 4. 5 Speed for Van in Field and Default Value

Frequency analysis shows that truck speeds range from 10 to 80 km/h as shown in Table 4.6 while the default value provided by VISSIM model shows that truck speeds range from 40 to 45 km/h as shown in the Figure 4.6.

Table 4. 6 Frequency speed for truck

Speed (km/h)	Frequency	Cumulative fr.	% Frq.	Cumulative frq%
0	0	0	0	0
10	0	0	0	0
20	1	1	0.03	0.03
30	2	3	0.06	0.09
40	7	10	0.21	0.30
50	12	22	0.36	0.66
60	8	30	0.24	0.90
70	1	31	0.03	0.93
80	2	33	0.06	1
90	0	33	0	1
100	0	33	0	1
110	0	33	0	1
120	0	33	0	1
Sum	33	262	1	

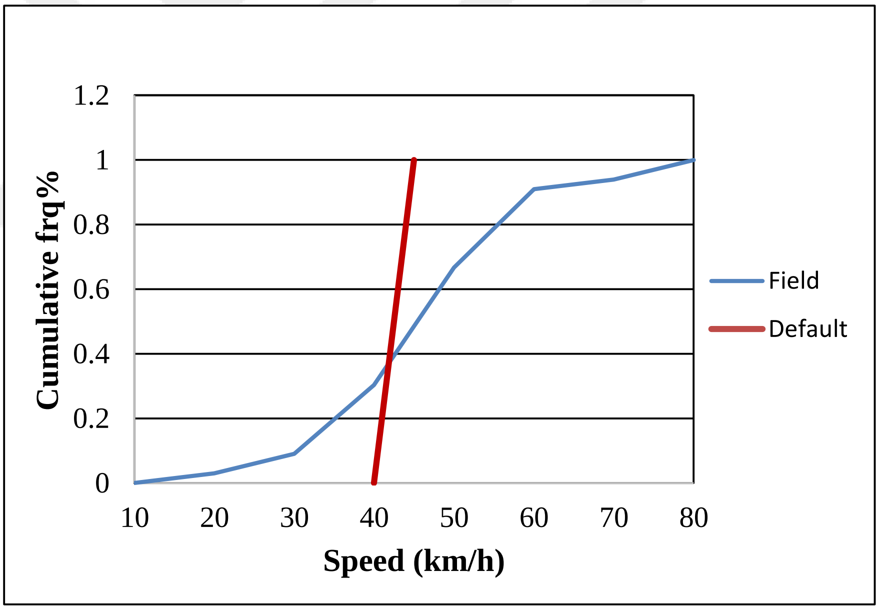


Figure 4. 6 Speed for Truck in Field and Default Value

4.2.3 Headway Data Collection

Headway is a measure of the temporal gap between two vehicles. Specifically, headway is the time that elapses between the arrival of the leading vehicle and the following vehicle at the designated test point. In this study, for each of three lanes, the starting and ending time of green light as well as the number of the cars are passing through the road during the period when the green light is on is recorded for a period of 2 hours.

Calculations were made to determine the average and standard deviation for head way as shown in Table 4.7.

Table 4. 7 Average and standard deviation for headway(s)

Lane	**Average**	**Standard dev.**
Left	2.2	0.59
Right	2.5	0.88
Middle	3.3	1.79

4.3 Evaluation of Data

This study analyzed data obtained using the T-test (two sample assuming unequal variance) to establish whether the vehicle speeds and the headway were acceptable or not followed by comparisons of the speeds in order to compares for each vehicle type and the headway of the each of the three lanes.

Essentially, if t Stat < -t Critical two-tail or Stat > t Critical two-tail, the null hypothesis is rejected. The results of the t-test hypothesis for speed are presented in Table 4.8.

Table 4. 8 Comparison of speed for three type of vehicle by t-test hypothesis

Value	**Comparison**		
	Car and Van	**Car and Truck**	**Van and Truck**
df	53	56	63
t Stat	4.58	8.99	3.2377328
P(T<=t)one-tail	1.45E-05	9.37E-13	0.01
t Criticalone-tail	1.67	1.67	1.67
P(T<=t) two-tail	2.89E-05	1.87E-12	0.01
t Critical two-tail	2.01	2.00	1.99

It can be seen in Table 4.8 that t stat (e.g. 4.58) is bigger than t critical two tails (e.g. 2.01), so the T- test is rejected.

This same process was repeated for headway. The comparison of headway between three lanes in MASTI intersection was applied in T-test and presented in the Table 4.9.

Table 4. 9 Comparison of headway in three lane for MASTI intersection by t-test hypothesis

Value	**Comparison**		
	Left and Right Lanes	**Left and Middle Lanes**	**Right and Middle Lanes**
df	69	53	45
t Stat	-1.90	-2.42	-3.60
P(T<=t) one-tail	0.03	0.01	0.01
t Critical one-tail	1.67	1.67	1.68
P(T<=t) two-tail	0.061	0.02	0.01
t Critical two-tail	1.99	2.01	2.01

In the Table 4.9, it can be seen that t stat (e.g. -1.90) is less than t critical two tails (e.g. 1.99); therefore, the t- test is acceptable.

4.4 Simulation of the Model and Calibration

The PTV VISSIM Model was used to build the framework of case study. The following steps were implemented:

1- Geometric information about study area, including corridors, connectors and intersections as shown in the Figure 4.7 was input in the program.

2- Information inserted about the vehicles such as vehicle name, vehicle type (car, van, midi-bus, big van, truck, bus, and articulated bus) together with the description of each type in 2D/3D Model.

3- Enter the information about interval time for the vehicle types that started from the intervals 0-900 s up to 3600-7200 s. Afterwards, the tool of composition vehicle used to add time interval, vehicle type and the desired speed for each vehicle type with flow interval of 98% for cars and 2% for heavy vehicles.

4- Enter hourly volume of vehicles for each time interval and for each vehicle type separately for each lane.

5- Build the routing road for the two intersections and the corridors to provide the model information about the starting point and the direction of the vehicles.
6- Input the information about the signal cycle for each lane and the direction in each intersection.
7- selected the travel time editor from the option of VISSIM and then choice the corridor between the two intersection for make simulation running to get the result.

Figure 4. 7 Study Area in VISSIM

In VISSIM model, the travel time detector was used to sample the time vehicles took travelling from the starting to the ending point of the corridor. The simulation was performed and travel time results obtained; the travel is then used to calculate speed. After detecting differences between the travel times in the field and in the model, the model is calibrated based on three parameters namely driving behavior parameters (CCO), headway (m) and headway(s). During the calibration, one parameter is varied independently and taken as variable whereas the other two parameters are considered to be stable until reasonable results are obtained by applying RMSE (Root mean square error) in Microsoft excel. The same process is repeated for other parameters as shown in Table 4.10, Table 4.11 and Table 4.12.

Traffic simulation consists of two main models; car following model and lane changing model. In VISSIM, the car following model uses the CCO and headway(s) parameters while the lane changing model uses the headway (m) parameter. In this study, CCO was calibrated using 10 different values including the default value as shown in Table 4.10.

Table 4. 10 RMSE for CCO parameter

Parameter CC0	Car	Van	Truck
0.6	58.46	50.11	277.65
1	29.48	20.44	85.11
1.1	28.32	18.56	105.65
1.2	28.41	20.61	83.60
1.26	27.66	18.88	84.79
1.5	27.94	19.34	86.06
1.7	29.00	20.63	96.38
2	31.82	22.28	124.46
5	28.88	19.43	101.10
6	35.19	28.08	140.87

Headway (s) was calibrated using 4 default different values and the least RMSE for travel time was (2, 90) as shown in Table 4.11.

Table 4. 11 RMSE for headway (s) parameter

Parameter headway s	Car	Van	Truck
1.50	31.1	20.78	96.86
2.90	29.68	20.46	98.9
20.20	95.37	46.99	208.94
30.30	187.48	179.15	325.24

Headway (m) was calibrated using 5 different values including the default headway and the least RMSE was (0, 1) as shown in Table 4.12.

Table 4. 12 RMSE for headway (m) parameter

Parameter headway m	Car	Van	Truck
0.1	28.09	17.84	94.40
0.5	28.91	19.90	97.58
1	29.40	19.65	100.13
1.5	31.10	20.67	97.51
7	28.43	19.88	88.24

Table 4.13 compares travel time defaults results and the travel times after calibration in different time intervals for the three vehicles types (car, van and truck). There is a difference between the results implying that calibration was done.

Table 4. 13 Comparison before and after calibration for travel time in VISSIM

Time Interval Sec.	Car		Van		Truck	
	Travel time default	**Travel time after change the value**	**Travel time default**	**Travel time after change value**	**Travel time default**	**Travel time after change the value**
0-900	63.39	64.59	63.09	64.01	198.19	164.54
900-1800	69.24	68.61	70.95	69.34	172.95	154.02
1800-2700	74.83	70.57	77.75	70.22	174.05	87.03
2700-3600	71.82	68.68	73.64	68.42	169.59	162.75
3600-4500	69.33	70.3	67.9	71.5	74.93	68.14
4500-5400	68.94	72.39	66.57	73.23	156.29	216.32
5400-6300	66.9	65.85	67.96	67.24	126.84	172.16
6300-7200	65.98	65.49	67.95	64.62	103.49	71.54

CHAPTER 5

CONCLUSION AND RECOMMENDATION

This research was conducted in Malatya. For this study attempts to analysis traffic data in two intersections for selected travel time, headway, and speed and to provide further understanding on the need for the calibration of traffic parameters in the traffic simulation model. Two consecutive intersections with 4-legs were selected in Malatya for calibration.

The traffic count data was collected during morning hours from 7:00 am to 9:00 am by analyzing the video streaming in the YTU ITS Lab. Data also collected include the signal timing, travel time between the two intersections and headway. Headway was statistically analyzed for vehicles in the intersections and results showed that there is no difference in headway(s). Additionally, travel time data was statistically analyzed to find out whether different vehicle types had similar travel times. It was established that travel time is dissimilar for the different types of the vehicle. Therefore, different vehicle types in the simulation had to have different speed parameters including desired speed, acceleration and deceleration.

In this study, traffic simulation consisted of two main models; the car following model and the lane changing model. In VISSIM, the car following model uses the CCO and headway(s) parameters while the lane changing model uses the headway (m) parameter. During the calibration, one parameter is varied independently and taken as variable whereas the other two parameters are considered to be stable until reasonable results are obtained by applying RMSE (Root mean square error) in Microsoft excel and the same process repeated for other parameters. The CCO was calibrated using 10 different values including default value, the headway (s) was calibrated using 4 default different values and the least RMSE for travel time was (2, 90). Finally, the headway (m) was calibrated using 5 different values including the default one and the least RMSE was (0, 1).

As a conclusion, the headway does not effect by the different lanes of the roadway and travel time of the roadway segment have differentiate by the type of the vehicle like car, van or truck.

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