REPUBLIC OF TURKEY
YILDIZ TECHNICAL UNIVERSITY
GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES

LIFE CYCLE COST ANALYSIS IN PAVEMENT DESIGN TO REACH THE BEST INVESTMENT DECISIONS FOR DEVELOPING COUNTRIES

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LIST OF SYMBOLS

i	Discount Rate
AM	Annual Maintenance Cost
Yrs	Years
kPa	kilopascal
psi	Pound squire inch
in	inch
ft	foot
m	meter
cm	centimeter
km	kilometer
ml	mile
W_{18}	Predicted Number of 18-kip ESAL
Z_R	Standard Normal Deviate
S_o	Combined Standard Error
p_o	Initial Design Serviceability Index
p_t	Terminal Design Serviceability Index
ΔPSI	Difference between p_o and pt
SN	Structural Number
R-Value	Resistance Value
U_f	Roadbed soil modulus
MR	Resilient modulus
D_d	directional distribution factor
D_L	lane distribution factor
$\hat{W}_{18}$	Cumulative Two-Directional 18-kip ESAL
R	Reliability
a_i	i^{th} layer coefficient
D_i	i^{th} layer thickness
m_i	i^{th} layer drainage coefficient
E	Elastic Modulus
°F	Fahrenheit
ºC	Celsius
gm	Gram
cm^3	Cubic centimeter
m^3	Cubic meter
cm^2	square centimeter
m^2	square meter

μm	Micrometer
hr	Hour
min	minute
G.F	Growth Factor
ESAL	Equivalent Single Axle Load
$	U.S. Dollar
θ	Sum of the principal stresses
k_1, k_2	material constants
E_{SB}	Elastic Modulus of Subbase
E_{BS}	Elastic Modulus of Base
D	Layer Thickness
PW	Present Worth
IC	Initial Cost
MC	Maintenance Cost
UC	User Cost
FRC	Future Rehabilitation Cost
S	Salvage Value
PWF	Present Worth Factor
AUC	Annual User Cost
crf	Capital Recovery Factor

LIST OF ABBREVIATIONS

LCCA	Life Cycle Cost Analysis
LCA	Life Cycle Analysis
AASHTO	American Association of State Highway and Transportation Officials
ESAL	Equivalent Single Axle Load
M-E	Mechanistic-Empirical
W.W. II	World War Two
USA	United States of America
PMB	Plent Mix Base
ICBP	Interlocking Concrete Block Pavement
HRA	Hot Rolled Asphalt
ISO	International Standard Organization
BSI	British Standards Institution
NCHRP	National Cooperative Highway Research Program
FPS	Flexible Pavement System
TTI	Texas Transportation Institute
CTR	Centre for Transportation Research
TxDOT	Texas Department Of Transportation
RPS	Rigid Pavement System
V/C	Volume to Highway Capacity ratio
FHWA	Federal Highway Administration
NPV	Net Present Value
PV	Present Value
THD	Traffic Hourly Distribution
EUAC	Equivalent Uniform Annual Costs
VOC	Vehicle Operating Costs
B/C	Benefit/Cost ratio
IRR	Internal Rate of Return
HMA	Hot Mix Asphalt
PCC	Portland Cement Concrete
CBR	California Bearing Ratio
ASTM	American Standards for Testing Materials
NSA	National Stone Association
SL	Shrinkage Limit
PI	Plasticity Index

AC	Asphalt Concrete
OTSS1	Automatic Vehicle Classification Counts 1
AADT	Annual Daily Traffic Data
D.R	Discount Rate
PP	Perpetual Pavement
CRCP	Continuously Reinforced Concrete Pavement
CPCD	Concrete Pavement Contraction Design
JRCP	Jointed Reinforced Concrete Pavement
PTCP	Post-Tensioned Concrete Pavement
PMS	Pavement Management System
M&R	maintenance and rehabilitation
TPMs	Transition Probability Matrixes
CEN	European Committee for Standardization (Comité Européen de Normalisation)
I	One
II	Two
III	Three
IV	Four
V	Five
VI	Six
RT	Road Tar
RC	Rapid Curing
MC	Medium Curing
SC	Slow Curing
AI	Anionic Emulsion
CIs	Cationic Emulsion
RS	Rapid sporadic
MS	Medium sporadic
SS	Slow sporadic
CRS	Cationic Rapid sporadic
CMS	Cationic Medium sporadic
CSS	Cationic Slow sporadic
KGM	Karayolları Genel Müdürlüğü

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ABSTRACT

LIFE CYCLE COST ANALYSIS IN PAVEMENT DESIGN TO REACH THE BEST INVESTMENT DECISIONS FOR DEVELOPING COUNTRIES

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MSc. Thesis

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Pavement design depends on different variables such as traffic, environmental condition, available materials, the budget of construction, government strategies and in sometimes the road users requirements, etc. Thickness of the pavements are so different based on the layer type and the materials that are used, therefore, it requires to evaluate the different alternatives. The common techniques that are used to evaluate the pavement alternatives are called Life Cycle Cost Analysis (LCCA). LCCA is denoted by analyzing all elements above for making an allowance for all significant costs and benefits for the project.

In this study, pavement design parameters are defined based on available materials in Iraqi City of Baghdad. Traffic data were obtained from the Republic of Turkey General Director of Highways for the road segment 400-23 in City of Gaziantep growth rate of the traffic was calculated based on that data too.

First of all, the literature review was conducted. In the second step, each of the pavement alternatives that were configured for the study was designed based on, American Association of State Highway and Transportation Officials (AASHTO), 1993 procedure; then LCCA was carried out to evaluate the pavements alternatives to decide the most appropriate pavement structure for the selected parameters. Design life for this project is 20 years. After analyzing the collected data for 12 years from the official data reference, the growth factors of these years were calculated, and they were used to reach the 20^{th} year (from 2017 through 2037).

In additional, the important stage in the pavement design is rendering all vehicle types homogenous by using the convert factors to get ESAL (Equivalent single Axle Load) and all axle groups will be as a single axle to be used in pavement design. As a result, outputs of the evaluation and the recommendations for the further researches are given.

Key words: Flexible pavement, rigid pavement, LCCA, AASHTO, pavement design, transportation.

ÖZET

GELİŞMEKTE OLAN ÜLKELERDE EN İYİ YATIRIM KARARLARINA ULAŞMAK İÇİN KALDIRIM TASARIMINDA ÖMÜR BOYU MALİYET ANALİZİ

Hasanain ASFOOR

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Üstyapı tasarımı, trafik, çevresel durum, mevcut materyaller, inşaat bütçesi, devlet stratejileri ve kimi zamansa yol kullanıcılarının gereksinimleri vs. gibi farklı değişkenlere dayanmaktadır. Üstyapıların kalınlığı, katman tipine ve kullanılan materyallere dayalı olarak farklılık göstermekte olup, bundan dolayı farklı alternatifleri değerlendirmeyi gerektirmektedir. Üstyapı türünü değerlendirmek için kullanılan genel tekniklere Ömür Döngü Maliyet Analizi (LCCA) adı verilmektedir. LCCA, proje için tüm kayda değer maliyet ve faydaları hesaba katarak yukarıda bahsi geçen tüm öğelerin hesabı yapılarak bulunur.

Bu çalışmada üstyapı tasarım parametreleri, Irak'ın Bağdat şehrinde mevcut materyallere dayanarak tanımlanmıştır. Trafik verileri, Türkiye Karayolları Genel Müdürlüğünün Gaziantep şehrinin 400-23 numaralı yol segmenti için alınmış olup, trafik gelişim hızı da bu verilere dayalı olarak hesaplanmıştır.

Öncelikle literatür taraması yürütülmüştür. İkinci adımda çalışma için oluşturulan üstyapı alternatiflerinin her biri Amerikan Eyalet Karayolları ve Ulaştırma Kurumunun (AASHTO) 1993, tasarım rehberi temel alınarak tasarlanmış ve ardından, seçilen parametreler için en uygun üstyapı yapısına karar vermek amacıyla kaldırımların değerlendirilmesi için LCCA tekniği uygulanmıştır. Bu projenin tasarım ömrü 20 yıldır. Resmi veri merkezinden toplanan 12 yıllık verilerin analizinin ardından, bu yıllara ait gelişim faktörleri hesaplanmış ve bunlar 20. yıla ulaşmak için kullanılmıştır (2017'den 2037'ye kadar).

Ayrıca, üstyapı tasarımında önem arz eden adım, ESAL'ı (Eşdeğer Tek Dingil Yükünü) elde etmek adına faktörlerin dönüştürülmesiyle tüm araç türlerinin homojenitesini sağlamak olup, tüm dingil grupları bir tek dingil olarak kabul edilip üstyapı tasarımında kullanılacaktır. Sonuç olarak, değerlendirmenin çıktıları ve daha detaylı araştırmalar için öneriler sunulmuş bulunmaktadır.

Anahtar Kelimeler: Esnek üstyapı, rijit üstyapı, LCCA, AASHTO, üstyapı tasarımı, ulaşım.

CHAPTER 1

INTRODUCTION

Transportation is the system, the planned networks of real and exact workings, which reacts and plays different roles in the process of physical movement of persons and freights from an origin or a source to a goal point through a motorized, non-motorized, or combined means [1].

In addition, transportation is essential for a nation’s growth and development. In fact, it has expended a considerable portion of human race’s time and funds for as long as it has occurred. There are many factors that should be taken into consideration in a pavement design; for example the traffic flow, the asphalt mixture materials and the environmental factors, which altogether defines the pavement performances. A satisfying pavement has to meet some conditions regarding the surface course of asphalt, which has to show sufficient strength and stiffness and suitable subbase layer strength to bearing capacity of the pavement structure. Furthermore, a stable subgrade and adequate drainage system has to be installed for preventing from the impact of moisture and for avoiding the base layer instability. Finally, a regular maintenance plan has to be fixed to avoid the pavement deterioration [2].

Life Cycle Cost Analysis (LCCA): The techniques used to make a comparison for the results of design alternatives during the design life of each phase, and making a budget for all important costs and profits.

LCCA provides a space for calculating the cost of an outcome or service during its design life. It is used to compare competing for design phases over the lives of each phase, taking all significance costs and returns into consideration, which are expressed in equivalent economic units. For construction - a project such as roads - a large amount of the total cost over the duration of such project is returned after the completion of construction; i.e.

during their design lives. It is probably to avoid most of the “unknown” costs by introducing long-term costs into the pavement evaluation processes as a substitute of comparing only initial material and substructure costs. The steps involved in the LCCA methodology are as follows [3]:

1. Make alternative design strategies,
2. Determine design life period,
3. Evaluate all agency costs,
4. Estimate user costs, and
5. Calculate life-cycle cost.

RealCost is the software was created with two distinct purposes. The first one is to provide an instructional tool for pavement design to help the decision makers to learn about LCCA. The software allows the user of LCCA to investigate the effects of cost, service life, and economic inputs on life-cycle cost. For this purpose, a graphical user interface (GUI) was designed to make the software easy to use. The second purpose is to provide an actual tool for pavement designers, which they can use to incorporate life-cycle costs into their pavement investment decisions.

RealCost automates FHWA’s LCCA methodology as it applies to pavements. The software calculates life-cycle values for both agency and user costs associated with construction and rehabilitation. The software can perform both deterministic and probabilistic modeling of pavement LCCA problems. Outputs are provided in tabular and graphic format. Additionally, RealCost supports deterministic sensitivity analyses and probabilistic risk analyses.

While RealCost compares two alternatives at a time, it has been designed to give the pavement engineer the ability to compare an unlimited number of alternatives. By saving the input files of all alternatives being considered, the analyst can compare any number of alternatives. Furthermore, the software has been designed so that a basic understanding of the LCCA process is sufficient to operate the software.

The software automates FHWA’s work zone user cost calculation method. This method for calculating user costs compares traffic demand to roadway capacity on an hour-by-hour basis, revealing the resulting traffic conditions. The method is computation intensive and ideally suited to a spreadsheet application.

The software does not calculate agency costs or service lives for individual construction or rehabilitation activities. These values must be input by the analyst and should reflect the construction and rehabilitation practices of the agency.

1.1 Literature Review

Transportation extremely developed through the 20th century. In spite of the fact that the first cars were invented in the end of the 19th century, they became cheaper and expanded after the end of the World War I. In 1940, the cars became cheap and broadly expanded to the point that just about one in 10 families in Britain had a car. They increased after the WWII; by 1959, 32% of the families owned a car. In addition, cars only became broadly used in the 1960s. By the 1970s, the most of the families owned a car or cars.

In 1903, the law of speed limits 20 MPH started to be applied in Britain in order to decrease the rate of deaths or fatal injuries. The law was not applicable in 1930. Then in 1934, a speed limit of 30 MPH in residence areas became effective. Meanwhile, Mary Anderson invented the windscreen scanner in 1903. In 1921, automatic wipers (scanners) were invented [4].

In the USA, in 1914, the first electric traffic lights were invented. In addition, the e-traffic was firstly implemented in London, Britain in 1925. The parking meter first appeared in the USA in 1935. A Swede named Nils Bohlin renewed the three-point seat belt in 1959. In 1983, using seat belt was obligatory in Britain. In 1983, the wheel clamps appeared in Britain and speed cameras appeared in 1992. The first appearing of busses, either motor busses or trolley busses, which ran by wires, was in 1950s. In the mid-20th century, there was a huge network of railways in Britain. On the other hand, flight was more expensive in the early 20th century; few people could afford the trip's cost [4].

The basic hypothesis of pavement design models and procedures applied in Mechanistic-Empirical (M-E) methodologies are commonly known. There is some number of assumptions that should be defined before the design implementation. One of the issues is that the design process should result from a regular pavement performance at a required level of reliability by considering many sources of reservations. Adequate reliability techniques should be involved in the M-E pavement design procedure in order to allow the designer to predict different reservations of pavement designs and produce a regular pavement performance level.

The main idea of the LCCA is that it is the economic tool that can determine which alternative has the best investment value and it is not an engineering tool that discovers how long phases will last or how the improvement will be achieved. That does not mean that engineering is not a significant factor in the LCCA; but suitable engineering knowledge must be owned to ensure each different phase achieves the requirements of design and gives the similar outcomes. If the phases do not achieve the same benefits, then an economic estimation used in LCCA to compare them is not realistic [5].

In addition, total revenues in pavement infrastructure investment, planning, design and durable service delivery in the economical road infrastructural preparation, budgetary and licenses in developing countries can be achieved from using LCCA in pavement design. The improving of confidence level with decision-makers require make the real reports and make some primary studies to persuade them [1].

Life cycle cost analysis can be divided into primary and secondary analysis. A Primary analysis is found if an exact set of phases should be done. For instance, a primary analysis finds if a roadway, a transport system, or neither of them should be constructed. Primary analysis defines if any improvements are required to be done in all phases. Therefore, in the primary analysis, a “does nothing” group is a possible phase that should be examined.

The secondary analysis compares to the phases that include the primary analysis. It is the assessment that shows which phase provides the best investment value. LCCA for the selection of pavement types (concrete or asphalt) is a secondary analysis. Because, the primary analysis will have already decided that a pavement project must be done. “Does nothing” is not a possible phase in secondary analysis, which means that an organization cannot advance the funds in stocks, qualifications, etc. The profits should only be based on investing [3].

In most developing countries, the road network constitutes one of the biggest attentions of estimation committees and takes the big budget in government financial plan. Road managers must rehabilitate, develop, operate and find another cost analysis of projects (or re-estimate); along with for project construction and careful management of the unique budget and human resources needed to building these objectives. All of these are done under the local investigation of the public who pay for regular road users, and who will increase the requirement of better levels of services along with the good levels of safety, reliability and less influence on the environment. The road administrations and Governments respond to improve the efficiency for road users, and their responsibility is limited to the management of the road networks. Really, in many countries, local highway authorities stand before official accountability and report requirements on how they manage their financial resources. Many road agencies have approved a financial management department, which are as applied to the roads sector signifies [6].

According to the number of the past studies on the LCCA, LCA and Sustainability, such studies provided benefits information and results to help the pavement designer in the selection of pavement types. The reports and studies of LCCA in initial planning help the decision makers to deal with costs estimation and sustainability conditions in a projected lifespan depending on project cost estimation methodologies and BS/ISO Standards [7].

The key aspect together with these works of researchers is their aim at possible the returns of the (LCCA) and LCA and Sustainability conditions for buildings using various economic estimation techniques with sets of best design alternatives, variables, and assumptions [8].

The active models that are available in the USA, Europe, and Canada are complex and unsuitable for application in developing countries because of weather, data, technical and local systems development sample; and a result the cultural differences from that of these countries. Road administrators in the developed countries have decided the models for LCCA with the purpose of decreasing the total costs for road construction, reducing the socio-economic profit with a lowered social economic cost, and reducing the influence on the environment. Mostly these models used to select of road infrastructure, alternatives or pavement types and other road construction, such as bridges and tunnels [9-11].

In developing countries, the models cannot be used as the standard models, since they are built-up according to the needs of minor road projects in some parts of the environment in those countries. Additionally, the limits of these existing models include the use of principles and roughly, predicted maintenance costs are in excess of the exact cost and the lack of regional unique through road design.

The NCHRP is the governmental research program - established by State Planning and Research funds and led by their needs. Each year the(AASHTO) is choosing Committee on Research selects 40 to 50 new projects that exercised the changing precedence and trials of states as they strategy, design, construct, achieve, or road maintenance the government highway structure. The program plans more than 60 projects annually that are planned to help states to expand their business [12-13].

The AASHO's "Red Book" of 1960, describes the term cost-benefit analysis (or life cycle cost analysis) and generally gives the huge highway creation. In that time, the only data given by AASHO for being used in life cycle costing belongs to the passenger cars in a rural region and truck costs. This procedure can help to explain the impression of the economic value of highway improvements in the planning level [14].

One of the major advancement in LCCA was the work embodied by Winfrey in 1963; the research combined and controlled the existing VOC data into a presentation that highway planners were able to use in the enhancement of LCCA above the next fifteen years. Also during the 1960s, they selected at least two roads to make analysis of them to find the good one, the theory of LCC values to the design and selection of pavement. The NCHRP made an investigation under project NCHRP 1-10 to rise the term of LCCA [15].

At the same time, the Flexible Pavement System (FPS) improved by the Texas Transportation Institute (TTI) and the Centre for Highway Research (which later became the Centre for Transportation Research or CTR) became the procedure and the computer program that is used to analyze alternative of asphalt concrete designs and ranks by the LCC. Then, TxDOT based a project to discover new Rigid Pavement System (RPS), in the similar basic with FPS, which achieves a life cycle cost analysis of rigid pavements and ranks alternative designs by overall life cycle cost [15-16].

Zhang (2009) developed a new life cycle prepared models for benefits of pavement management system. He assessed three possible overlay systems. One of these is a RPS. The application of active programming as a set in life cycle preparation of pavement overlay systems, which gets out significantly faster and exactly compared to standard methods, will be shown later. His results illustrate the significance of constant user costs and rough effects in PMS accounting [18].

William James and other; said; In developing countries, the background for a new life cycle cost is the significance thing all external of pavement efficiency, maintenance, social and economic impacts, and road users safety should be studied and involved. Many of the various features of a complete (LCC) model are included in the (LCC) methods. However, none of these methods or programs includes all the features, nor do they provide the means to add future works. Although, many of these components are not fully understood to easily determined or evaluated [15].

In 2013, Zhang ET presented the development way of a new PMS of networks, which gave the knowledge of analysis and realized the LCCA. The LCCA realization put into control for the perfect maintenance project for a pavement network and to decrease supportability metrics within a given analysis period of life. They talked about pavement damaging; it is the main aspect to recognize future pavement maintenance plans [18].

In 2006, Pradhan M. showed the choice of the appropriate economical and the beneficial pavement type was made by using LCCA, which takes into account the initial cost and the secondary (rehabilitation) cost. They also presented the cost of pavement for both types of pavement: Flexible and rigid. They also evaluated an economic cost analysis, which showed the life cycle cost of PCC is about 20-25% less than the bituminous pavement [18].

According to the Life Cycle Cost Analysis (LCCA) Primer, LCCA is one of the most useful engineering economic techniques that can find the lowest cost alternative among several phases to meet the aim of project. LCCA was considered by the FHWA recently as part of a widespread benefit management program that involves the good management, system protection, pavement operation, bridge management and checks up, and contraction and maintenance activities (FHWA, 2007). Two types of costs are considered in the LCCA method: Agency Costs, and User Costs.

Two economic elements are also used to compare the relative efficiency of phase pavement strategies, Net Present Value (NPV) and Equivalent Uniform Annual Costs (EUAC) [19].

1.2 Objective of the Thesis

The title of thesis mentions specific procedures for conducting LCCA in pavement design and discusses the relative significance of LCCA factors on analysis outcomes. In the interest of technical purity, the conversation includes all virtual LCCA factors; even though not all elements affect the final LCCA results to the similar degree. The analysis happens after the pavement design is complete; and at least has two alternatives to make the comparison between them by using the LCCA technics.

Three alternatives will be designed by using AASHTO guide (1993);to be a base for analyzing, after converting the vehicle's type to the ESAL to get the design traffic volume, and then to get the layers thickness. After getting the structural design of layers by using the LCCA, the results being influenced by the cost of material and the longer life and average cost will be discussed by using LCCA techniques and realize the results by using the RealCost software as the goal of the case study.

1.3 Hypothesis

The case study is to compare the three different alternatives of the roadway pavement design considering the layers and the cost. In each alternative layers different from the other alternatives, which mean different cost, the aim of this case study is to find the economical, sustainable and available design (can do it) alternative. The design procedure is according to AASHTO guide (1993), and the most parameters used in this case study took from tables and curves in this guide.

The design procedure is according to AASHTO guide (1993), and the most parameters used in this case study are taken from tables and curves in this guide.

The design assumptions will suppose that the layer in the alternatives will be different; the three alternatives will design after conversion of the vehicle's type to the ESAL to get the design traffic volume, and then to get the thickness of layers. That means to establish the area for analyzing the results and this analysis will be done by using the LCCA technique.

All three different alternatives in the case study will be a flexible pavement type; but the layers will be different as following, Figures (1.1 A, B and C).

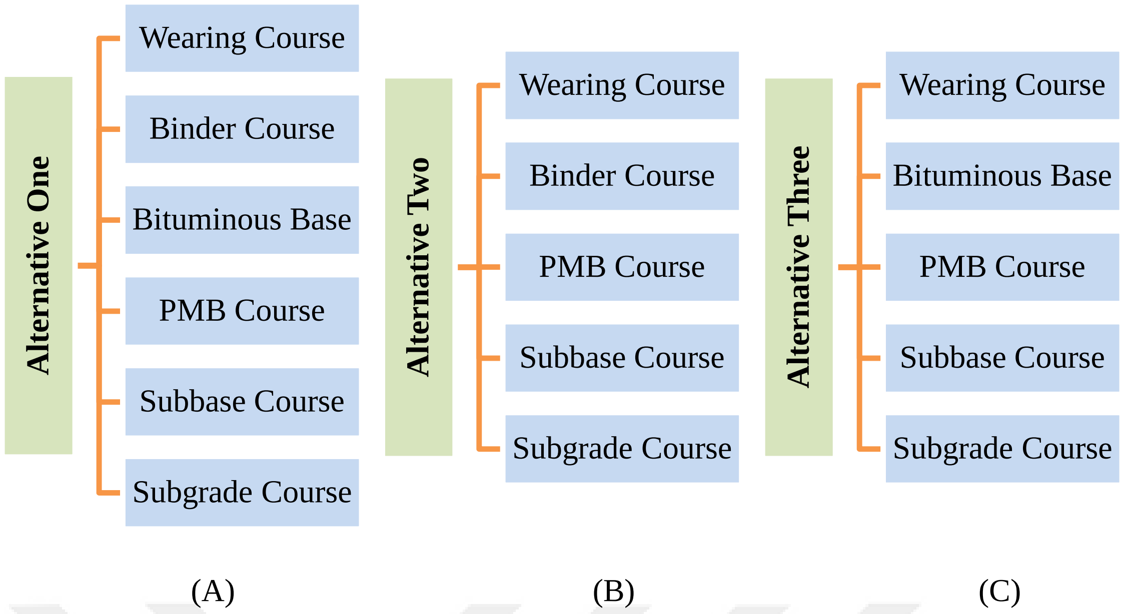

Figure 1.1: The Three Different Alternatives in the Case Study

After getting the design structures of layers, the results will be analyzed and discussed by using the RealCost software as LCCA techniques depending on the cost of material. Then the longer life and average cost will be chosen. This is the goal of the case study. The Figure (1.2) illustrates the flowchart of the study.

After doing the analyzing and checking the life cycle of roadway, the result appeared in the direction that the third alternative is the better one because it has the lower agency cost and lower user cost, the first one has also the same user cost of third alternative but its agency cost higher than third one.

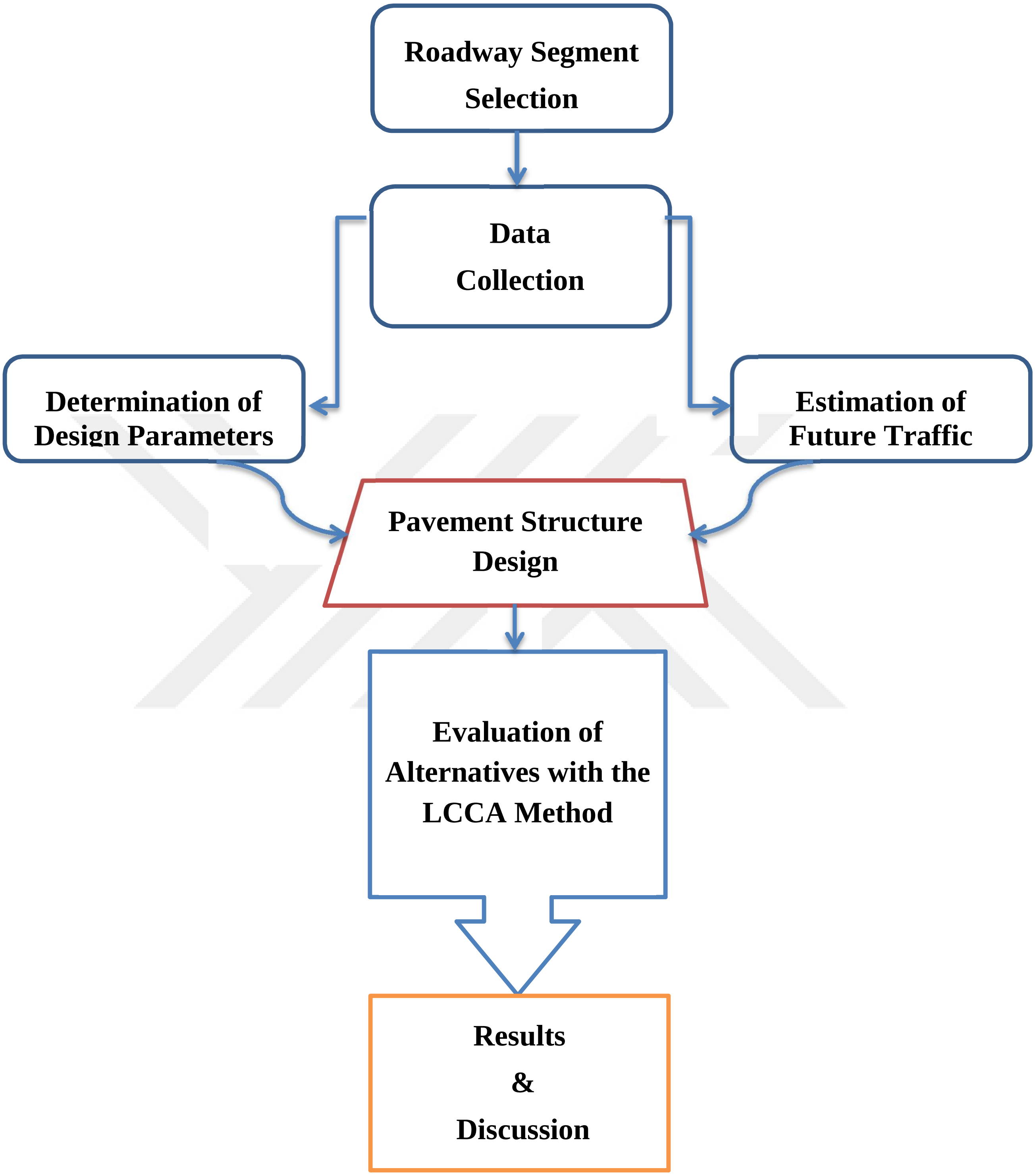

Figure 1.2 : Flowchart of the Study

CHAPTER 2

PAVEMENT TYPES AND PAVEMENT MANAGEMENT

2.1 Types of Pavement

According to AASHTO (1993), pavement type choosing is a three-part procedure that contains a pavement design analysis, life cycle cost analysis and evaluation of project exact details. Since the outcomes may exclude the need to continue with the rest of the pavement type choosing procedure, the pavement design should be done first [31].

Pavements are divided into two main types: Rigid and Flexible. The wearing surface of rigid pavements is usually constructed of Portland Cement Concrete (PCC), and the wearing surface of flexible pavements, on the other hand, is usually constructed of bituminous materials. Flexible pavements usually consist of a bituminous surface underlay with a layer of granular material and a layer of an appropriate mixture of coarse and fine materials such as subbase course. Traffic loads are transferred by the wearing surface to the underlying forced materials through the dovetailing of aggregates, the cohesion of the fine materials and the frictional effect of the granular materials [2].

2.1.1 A Rigid Pavement:

A rigid pavement structure is composed of a hydraulic cement concrete surface course and fundamental base and subbase courses (if used). The term that is commonly used is Portland Cement Concrete (PCC) pavement, with today's pozzolanic additives, cement might technically be categorized as "Portland". The surface course is the stiffest layer and provides the transfers loads to the lower layers and the majority of strength. The base or subbase layers are less stiff than the PCC layer, but they are important tools to pavement drainage, decrease the frost impact, and provide a working platform during the construction stages [21].

Rigid pavements are considerably 'stiffer' than flexible pavements because of the high modulus of elasticity of the PCC material, and the strength of deflections is very low under loading. Rigid pavements can have reinforcing steel, which is generally used to handle thermal stresses to decrease or eliminate joints and maintain tight crack widths [23].

2.1.2 A Flexible Pavement

Flexible pavements are divided into three subgroups: High Type, Intermediate Type and Low Type, which is illustrated in Table (2.1) [22]. High type pavements have wearing surfaces that sufficiently support the predictable traffic load without visible distress due to fatigue and are not susceptible to weather conditions. Intermediate type pavements have a wearing surface that vary from surfaces treated to those with qualities just below that of high type pavements. Low type pavements are used mainly for low-cost roads and have wearing surfaces that range from an untreated to loose natural materials to surface-treated earth [27].

In addition, Asphalt pavement (flexible pavement) can be called as untreated or treated base has a hot-mixed asphalt concrete surface that generally constitute a base layer, which may be either untreated or treated the granular material, and maybe a subbase layer (usually untreated) [29].

Table 2.1: Exact Range of 18-kip ESAL Applied at Each Traffic Level

Traffic Level	Range of 18-kip ESAL application
High	700,000 to 1,000,000
Medium	400,000 to 600,000
Low	50,000 to 300,000

All other groups of pavements - separately asphalt binder based - are named flexible. The sustainability of flexible pavements comes from being cohesive and transfers loads to the subbase and subgrade, which depends on aggregate and amalgamate particle friction and cohesion for stability [2].

2.1.3 Composite Pavement

This term has a tendency to imply new construction as an asphalt-surfaced concrete pavement, instead of a rehabilitated concrete pavement. Concrete pavements with current asphalt overlays are as a third main pavement type. Because of this pavement, type results from a substantial portion of the high-volume highway mileage of the United States. In some assessment and rehabilitation strategy choices for this type of pavement, it varies in some respects from assessment and rehabilitation strategy choosing for either asphalt pavements or concrete pavements [29].

There are other types of pavement classified depend on the dowels requirements, type of joints and structure for layers such as:

- Perpetual Pavement (PP),
- Continuously Reinforced Concrete Pavement (CRCP),
- Concrete Pavement Contraction Design (CPCD),
- Jointed Reinforced Concrete Pavement (JRCP), and
- Post-Tensioned Concrete Pavement (PTCP)

2.2 Pavement Management System (PMS)

Combined pavement management system can help highway agencies or pavement engineers to make strategic investment decisions in programming pavement maintenance and rehabilitation (M&R) projects for the protection of a road network and increased the life of these networks [30].

A necessity to have a system or programs for management and maintenance of road surfaces appeared in the 20^{th} century. Engineering companies all through the world invested money in developing a system that could find solutions to the problems by Pavement Management System (PMS), which was approved as an operative solution to pavement management in the 1950s [32].

Pavement design and management decisions are focused on economic, technical, and safety aspects [33].

The short span of additional service years - through the delay of maintenance - was developed at a very high price in terms of money increasing. It is for the reason that PMS was needed to prioritize the roads for earlier maintenance and cost-effective time [32].

2.2.1 Data for PMS

Primary types of data needed include pavement condition rankings, costs, roadway construction and maintenance periods according to the traffic loading [32].

A pavement management system (PMS) can be effectively applied by a highway agency depends on many factors such as the techniques used in these processes, with pavement network database and data for management, measurement and assessment of pavement condition, modeling of pavement spoilage, resolve of treatment strategies for pavement network protection, and the network (M&R) importance programming. In other words, the real application of PMS in real situations can be measured based on the quality of the techniques used in these efficient works [30].

A major emphasis of any PMS is to recognize and estimate pavement conditions and to find the reasons for deterioration. To achieve this, a pavement estimation system should be developed in a quick and economical way. The pavement condition data must be collected frequently in order to document the changes of pavement condition [32].

2.2.2 Benefits of PMS

For the understanding of the database, a check of the characteristic uses of a PMS can be assumed. The following material briefly describes the major areas where PMS is used and the benefits achieved from it [32].

a) Street Inventory

The direct use of the PMS is in having a whole and a readily nearby list of the country’s road system including up-to-date conditions. This data is often valued for day-to-day use in tracking maintenance work and for location in making reports or studies.

b) Developing Maintenance Budgets

With making the typical 1-year maintenance budget, the PMS lets a country to make a series of budgets. These budgets can be in the formula of a multi-year program, classifying not only short-term (1) year needs, but the exactness needs over the course of several years. In addition, phases can prepare and present to the budget stakeholders.

c) Prioritization

PMS enables the ranking of maintenance projects based on cost and condition ratings and other influences such as traffic. It can be used for selecting and ranking of projects for the future budget year, as well as for long-term financial planning. The system developed has three major ingredients [36]:

1) Using non-homogeneous (Le., time-related) Markovian estimate models to prediction pavement deterioration,
2) Using stochastic theory and Monte Carlo Simulation technique to establish the Markovian change possibility matrices (TPMs) for separate pavements,and
3) Utilizing cost-effectiveness based ranking program to select the optimal multi-year pavement M&R projects and action years.

CHAPTER 3

PAVEMENT DESIGN

Transportation is essential for a nation's growth and development. Actually, it has been using a significant portion of human race's time and funds for as long as it has existed. Many factors should be taken into account in a pavement design. For example, the traffic stream, the asphalt mixtures materials and the environmental factors, which of all are defined as the pavement performance [2].

Pavement performance is defined as the capability of a pavement to acceptably serving traffic-terminated time [22]. Serviceability is defined as the capability of a pavement to serve the traffic flow for which it was designed. By combining, both definitions will produce a new understanding of the presentation, which can be understood as the integration of the serviceability terminated time [34].

Serviceability is required for traffic estimation in terms of both design and rehabilitation. Since the pavement during construction the new road or that under rehabilitation is typically designed for ranging from 10 to 20 years or more, it is to evaluate or forecast the design loads for this period of time exactly.

Depending on AASHTO (1993); the choice of the pavement types has a three-part procedure that comprise an analysis of pavement design, life cycle cost analysis and estimation of the project parameters such as traffic data, environmental inputs and material properties using the observed data. In the first stage the pavement design should be made because of the impossibility to construct. It may be needed to continue with another kind of pavement selection to discover which one has the most suitable life cycle cost analysis and project exact data.

3.1 Design Consideration

Pavement design is based on the outcomes of experience or experiments. Normally, it requires a number of annotations to be made in order to determine the relationships between input variables and outcomes [39].

Especially, it is ignored to use experiments deriving relationships to define phenomena that happen out of the range of the original data used to find the new techniques to make this relationship stronger

The mathematical relations and empirical equations used in the AASHTO Guide (1993) are largely used in pavement design. Moreover, the pavement design considerations are [22]:

1) Reliability,
2) Time Constraints,
3) Traffic,
4) Environmental Effects,
5) Pavement Performance,
6) Materials of Construction,
7) Roadbed Soil,
8) Drainage,
9) Life Cycle Costs for Pavement Design, and
10) Shoulder Design.

3.1.1 Reliability (confidence level)

In fact, a reliability-based in the design procedure of pavement can be explained in the below advantages [37]:

1. Find the construction variableness, which differs between design and as-built elements, material unevenness, and variability related to traffic forecasting during pavement design life,

2. Model to compute the bias and refer to the assumption and explanation of the pavement analysis algorithm,

3. Choose the reliability level and display the outcome of failures. For instance, if it is felt that rutting is more dangerous than fatigue because of driver's safety, a higher reliability can be calculated to rut depth than to fatigue cracking,

4. Arrange for design pavement structures with the same performance level without which the contrast of lifecycle costs of different pavement types would be disingenuous and could result in the choosing of a less cost-effective pavement category, and

5. Update the standards in a rational method as more data becomes available to use in design.

3.1.2 Time Constraints

This refers to the period of time for which the analysis is to be shown, i.e. the length of time that any design plan must cover. The analysis period is similar to the term "design life" that is used by designers in the previous studies.

The maximum performance period plays the basic role in putting the plan for stage construction (i.e. an early pavement structure can be followed by one or more rehabilitation stage(s)) to reach the chosen analysis period.

Some time ago, pavements were normally designed and analyzed for a 20-year show period since the starting year. Now there is the option of giving more consideration compared to several years than the past where these may be better well matched for the estimation of alternative long-term plans based on life-cycle costs. The general guidelines illustrate in Table (3.1) [22].

Table 3.1: General Guidelines to Determine the Analysis Period for Pavement Life

Highway Conditions	Analysis Period (years)
High-volume urban	30 to 50
High-volume rural	20 to 50
Low-volume paved	15 to 25
Low-volume aggregate surface	10 to 20

3.1.3 Traffic

Traffic counting represents in two main types, manual counts, and automatic counts. There is no specific difference between the two methods. However, the economic use or choosing the suitable method of traffic counting depends on the level of traffic flow and the required level of data traffic [38].

One important function of a pavement design consideration is load distribution. Therefore, for appropriating and sufficient designing a pavement typical loading characteristics must be guessed about the predictable traffic it will pass on this road. Loads and the vehicle are the tools that apply forces on the pavement (e.g., by trucks, trailers, airplanes), and can be described by the following parameters:

1. Tire loads,
2. Axle and tire configurations,
3. Typical axle load limits,
4. Repetitions of axle loads,
5. Traffic distribution (by direction and lane), and
6. Traffic projections.

The damage in pavement is cumulative over the life of it and when it increases to the maximum value, the pavement is designed to reach the end of its service life. Pavements are structurally designed to the extent that they can carry all expected loads that a pavement would throughout its design life. These main parameters are usually done in one of two ways [40]:

1. Equivalent single axle loads (ESALs): This method converts axle configurations and axle loads of various sizes and repetitions such as mixed traffic to an equivalent number of (standard or equivalent) loads.

2. Load spectra: This method illustrates the loads directly by the number of axles, configuration, and load. It does not include conversion to the equivalent amount. Structural design calculations using load spectra are generally more difficult than using ESALs since the impact of each exact axle load is estimated. Both methods use the same type and quality of data, but the load spectra method has the possible to be more accurate in its load description.

The predicted traffic provided by the preparation group is generally the cumulative 18-kip ESAL axle applications predictable on the highway, whereas the designer needs the axle applications in the design lane. Therefore, if specifically furnished, the designer must feature the design traffic by direction and then by lanes (for more than two lanes). Equation (3.1) [22]; may be used to determine the traffic (w_{18}) in the design lane.

$$W_{18} = D_d \times D_L \times \hat{W}_{18} \qquad \text{Equation 3.1}$$

Where:

D_d = a directional distribution factor, represent as a ratio, that accounts for the distribution of ESAL units by direction.

D_L = a lane distribution factor expressed as a ratio, that accounts for spreading of traffic when two or more lanes are open in one direction, and

$W^{\wedge}_{18}$ = the cumulative two-directional 18-kip (8.2 t) ESAL units predicted for a specific section of highway during the analysis stage.

3.1.4 Environmental Effects

The environment conditions impact on the pavement in several ways. Temperature and variation in moisture content can have special effects on the stability, strength, load carrying, the capacity of the pavement and roadbed materials can influence on pavement performance. Another major environmental impact is the direct effect of roadbed disintegration, pavement blow-ups, frost heave and swelling [22].

3.1.5 Pavement Performance

The integration between elements of asphalt pavement is the basic for all pavement performance. Its importance is same to the significance of good base design for the road structures. The base of an asphalt pavement usually consists of two layers or lifts. The upper layer is called the sub-base and the lower one is called subgrade [9].

The term “lift” in highway terminology means a single layer or thickness of material. It is used slightly jointly with the term layer, and only the layer can include several lifts cannot be used in pavement construction. Mostly the sub-base is composed of a granular material like the crushed stone different in size; so it might also be a dense graded asphalt mixture as part of a Full-Depth asphalt pavement [23]. The subgrade enforces the pavement structure. The sub-base performs four main properties:

- Loads transferred by subbase to the subgrade.
- It provides a working platform for construction traffic and a paving platform onto which the asphalt materials can be placed and compacted in the site during the construction stage.
- It acts as an isolating layer or lift saving the subgrade from environmental impacts such as frost effects.
- It provides a drainage layer to observe some water from the pavement and reduce the disintegration.

The base course or binder course is the main structural component of an asphalt pavement. The base is lying under the surface course and usually, consists of granular sub-base.

The surface course sometimes includes the wearing and binder course. Surface course is the preferred terminology in the CEN standards. However, the surface course may consist of two layers – the lower layer being a thin smoothing binder or base course, followed by the upper layer being the material wearing course. The properties of wearing course are:

- It used to resist the deformation caused by traffic,
- Protection the lower layers,
- Used to resist the effects of climate change, traffic abrasion or wearing, and resists fatigue cracks because of its flexibility,
- Used to provide a skid-resistant surface and reduce the brake distance, and
- Providing a comfortable to the road users.

Normally, in asphalt pavements, the stiffness in each layer or lift is greater than that in the layer below and less than that of the layer above (Whiteoak 1991).

Mostly design procedures have three principles factors used to find the thickness of flexible pavement [47]:

- Traffic volume estimation and its' loading,
- Supporting of subgrade or increasing its strength layers and the material will be used for that purpose, and
- Characteristics of the asphalt materials selected for the pavement.

Shrinking and Swelling: Some soils change the volume considerably depending on their moisture content. This behavior is a special problem for pavement design; because this volume change can cause overlying pavements to drop down or heave up unequally resulting in a cracked or rutted pavement. Shrinking and swelling are generally attendant with fine-grained clay soils and sometimes the fine aggregates [28].

Soil shrinkage: It is confined to the upper layers of a soil. Shrinkage and shrinkage cracks are produced by a decrease in soil moisture content through:

- Evaporation from the soil in dry climates makes surface shrinking,
- Changing of the ground water table is important, and
- Desiccation of soil by trees during temporary dry spells in otherwise humid climates and dry air.

The following process is illustrated in Figure (3.1) [24]. The result of moisture content is decreased while the capillary stress in the void spaces increases because of the increased surface tension. This condition tends to pull adjacent soil particles to be closer resulting in an overall soil volume decreasing [24].

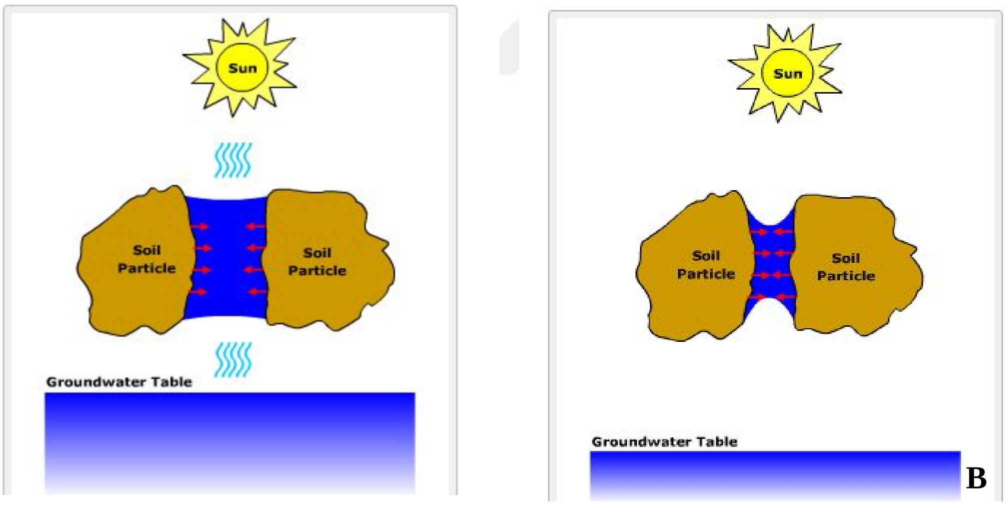

Figure 3.1: Shrinking Soil Mechanism

Swelling (Expansive): Swelling soils, also acknowledged as expansive soils are ones that swell in volume when subjected to wetness. These swelling soils typically contain clay minerals that appeal to and absorb the water. When water is introduced to expansive soils, the water molecules are pulled to be closer into gaps between the soil layers. For more absorbing of water, the plates are forced extra apart, leading to an increase in the soil hole pressure, Table (3.2) [10].

Table 3.2: Estimated the Possible Expansion by Using the Classification of Development of Soil

Degree of Expansion	Possible Expansion (as a percentage of the total volume change)	Granular Content (percentage less than 1μm)	Plasticity Index	Shrinkage Limit
Very High	More than 30	More than 28	More than 35	More than 11
High	20 to 30	20 to 31	25 to 41	7 to 12
Medium	10 to 20	13 to 23	15 to 28	10 to 16
Low	Less than 15	Less than 15	Less than 18	More than 15

Under a supplement of 6.9 kPa (1 psi).

3.1.6 Materials of Construction

The project here is the roadway and the type of this pavement is flexible pavement, which means the surface does not have any concrete layer. The structural layers are the surface course (wearing and binder), base course, subbase and subgrade following is the definitions for all these layers:

Prepared Roadbed: It is a layer of compacted roadbed soil or select borrows material; it has been compacted to a definite density and named Subgrade; the main role of it serves the foundation of the pavement structure depending on the type of pavement and transfer loads [2].

Subbase Course: The subbase course is the main part of the flexible pavement structure between the roadbed soil (subgrade) and the base course. It typically consists of a compacted layer of granular material, either treated or untreated or of a layer of soil cured with an exact mixture [22].

When the sub-base material does not match to the requirements, a process of treating soils to increase their engineering properties known as stabilization materials can be used. Actually, the available material should be cured with other materials to reach the essential properties [17].

Subgrade: The layer or layers of listed or selected material for the designed thickness constitute a subbase or a subgrade to support a surface course [22]. An asphalt binder selected for the surface course might be done in the design of asphalt binder for a base course [23].

Sometimes, the base course consists of granular materials such as sand, crushed stone, crushed or uncrushed gravel and crushed or uncrushed slag differently depending on the nearest of sources and the climate conditions. Generally, the base course materials include firmer necessities than those for sub-base course to afford the direct loads. In some cases, in order to increase the stiffness characteristics of heavy-duty pavements, the base course can be treated with asphalt or Portland cement [2].

Surface Course: One or more layers of a pavement structure designed to afford the traffic load; the role of top layer resists the skidding, traffic erosion, and the disintegrating effects of climate conditions. In addition, the top layer of flexible pavements is sometimes called "wearing course" [22].

3.1.7 Roadbed Soil

The subgrade must be able to support loads transferred from the pavement structure. This load bearing capacity is often influenced by the degree of compaction, moisture content, and soil material type. A subgrade that can afford a high quantity of loading without excessive deformation is considered a good soil [20].

The designer should avoid the poor subgrade if possible. However, when it is necessary to construct over weak soils should improve the mechanical properties of the subgrade. In short, a thick pavement structure over a poor subgrade might not make an acceptable pavement.

3.1.8 Drainage

Drainage of water from pavements is an important consideration in road design. However, the design has often resulted in base courses that do not drain well this excess water combined with increasing the traffic volumes and loads repetition leading to early pavement disintegration in the pavement structure [22].

Moisture in the subgrade and aggregate base layer can deteriorate these materials by increasing porous and reducing the resistances of the materials to shear and deformation. Additionally, moisture in the HMA layers can cause stripping; because it, rather than the asphalt binder, will follow to aggregate particles [23].

The designer should recognize what level (or quality) of drainage is reached under a specific set of drainage conditions. Table (3.3) illustrates the drainage quality depending on the time water is infiltrated from the surface of the pavement [22].

Table 3.3: Illustrate the Drainage Quality Depends on the Time Water is Removed from the Pavement Surface

Drainage quality term	Time water is removed
Excellent	2 hrs
Good	24 hrs
Fair	1 week
Poor	4 weeks
Very poor	No drainage

3.1.9 Life-cycle costs for pavement design

There are many steps, which should be applied to reach the best investment decision when designing more than one alternative. These steps will illustrate in chapter four.

3.1.10 Shoulder design

As defined by AASHTO, a highway shoulder is the “portion of roadways attached with road users used to stop vehicles for emergency use, and for lateral support of base and subbase courses”

Shoulders consider the safety side of the pavement and its associated road when the roadway is closed caused by the accidents or rehabilitation activities [22].

The Sweden agencies developed the standards of the shoulder depending on the roadway width and limit speed, the material for these shoulders selected according to the same standards.

3.2 American Association of State Highway and Transportation Officials (AASHTO)

The AASHTO definition of reliability is:

The reliability of the pavement design-performance procedure is the possibility that a pavement section designed using the process will perform acceptably over the traffic and environmental conditions for the design period [22].

Furthermore, design period can be defined as the time from primary construction or rehabilitation to the terminal serviceability index.

Empirical equations are used to calculate experiential or measurable phenomena (pavement properties) through products (pavement performance). Moreover, the form of the empirical equation is illustrated in Equation (3.2) [22]:

$$Log\, W_{18} = Z_r \times S_o + 9{,}36 \times Log(SN+1) - 0{,}20 + \frac{Log\left(\frac{P_0 - P_t}{4{,}2-1{,}5}\right)}{0{,}40 + \frac{1094}{(SN+1)^{5{,}19}}} + 2{,}32 \times Log(M_R) - 8{,}07 \qquad \text{Equation 3.2}$$

Where

W_{18} = Predicted number of 18-kip (8.2 ton) for the equivalent single axle load prediction,

Z_R = Standard normal deviate (desired probability of exceedance level),

S_o = Combined standard error of the traffic forecast and performance forecast,

ΔPSI= represent the difference between the initial design serviceability index, p_o, and the terminal design serviceability index, pt,

M_R = Resilient modulus (psi), and

SN =Equal to the structural number revealing of the total pavement thickness required.

3.2.1 Predicted Number of 18-kip Equivalent Single Axle Load Applications (W_{18})

Every segment of any road has the composite traffic volume, which means there are different types of vehicles and that results different axle loads. For the homogenization, the convert factors should be used (as shown in Table (3.4)) to find the Equivalent Single Axle Load (ESAL) [41]; and use it in Equation (3.2).

Table 3.4: Convert Factors to Find the Equivalent Single Axle Load (ESAL)

Used by (KGM)

Type of Vehicle	**Convert Factor**
Trailer	**4.10**
Truck	**2.90**
Bus	**3.20**
Medium Good Vehicle	**0.60**
Car	**0.0006**

The predicted traffic provided by the planning group is mostly the cumulative 18-kip ESAL axle applications predictable on the highway, while the designer requires the axle requests in the design lane. Therefore, unless specially furnished, the designer must make the design traffic by direction and then by lanes (if more than two) can use the Equation (3.1) to determine the traffic (w_{18}) in the design lane. The D_d factor is generally 0.5 (50 percent) for most roadways. For the DL factor, the Table (3.5), may be used as a guide to finding the DL [22].

Table 3.5: A Guide to finding the value of DL Factor

Number of Lane in Each Direction	Percentage of 18-Kip ESAL in Design Lane
1	100
2	80 to 100
3	60 to 80
4	50 to 75

3.2.2 Level of Reliability

It means that the integrating grade of certainty into the design process to ensure all the various design alternatives will continue the analysis period. The reliability design factor accounts for the eventuality of variations in both traffic prediction (W_{18}) and the presentation prediction (W_{I8}); and therefore provides a fixed level of assurance (R) that the pavement sections will protect the period for which they designed. The Table (3.6) illustrates the suggested levels of reliability for various purposeful classifications [22]:

Table 3.6: Illustrated the Classification for Various Purposeful as a Suggested Levels of Reliability

Functional Classification	Recommended Level of Reliability*	
	Urban	Rural
Interstate and Other Freeways	85 - 99.9	80 - 99.9
Principal Arterials	80 - 99	75 - 95
Collectors	80 - 95	75 - 95
Local	50 - 80	50 - 80

*Results base on a survey of the AASHTO Pavement Design Task Force

3.2.3 Standard normal deviate (ZR)

It means the standard normal table value corresponding to the desired probability of exceedance level. For example, a designer might specify that there should only be a 5% chance that the design does not sustain a specified number of years (e.g., 20 years). That means the design should achieve 95 % of the specified number of years. At that time, the reliability is 95% (100% – 5%) and the consistent ZR value is -1.645, Table (3.7) [22].

Table 3.7: The Guide Lines to Find the Standard Normal Deviate (ZR) Values According to Selected Levels of Reliability

Reliability (R) (percentage)	Standard normal deviate, ZR
50	-0.000
60	-0.253
70	-0.524
75	-0.674
80	-0.841
85	-1.037
90	-1.282
91	-1.340
92	-1.405
93	-1.476
94	-1.555
95	-1.645
96	-1.751
97	-1.881
98	-2.054
99	-2.327
99.9	-3090
99.99	-3.750

3.2.4 Standard Deviation (S_0)

The selection of S_0 according to AASHTO guide depends on the pavement type. AASHO Road Test does not include a traffic error. Therefore, the performance forecast error developed at the road test was 0.25 for rigid and 0.35 for flexible pavements. This leads to a total standard deviation for traffic is 0.35 and 0.45 for rigid and flexible pavements, respectively.

3.2.5 Present Serviceability Index (PSI)

The Present Serviceability Index (PSI) considers the primary measure of serviceability, which ranges from **0** (impossible road) to **5** (perfect road). The selection of the lowest acceptable PSI or Terminal Serviceability index (pt) is resulted from choosing the lowest index that will be accepted before rehabilitation, resurfacing or reconstruction is necessary. An index of 2.5 or higher is advised for the design of major highways and for highways; it is 2.0 with smaller traffic volumes. At the AASHO Road Test, the values are identified the p_o values experiential were 4.2 for flexible pavements and 4.5 for rigid pavements. The Table (3.8) illustrates the lowest allowable serviceability index (p_t) [22]. Furthermore, the Equation (3.3) [2]; can find the difference between the initial measure of serviceability and the terminal serviceability index (ΔPSI).

$$\Delta PSI = Po - Pt \qquad \text{Equation 3.3}$$

Table 3.8: Illustrated the Lowest Acceptable Serviceability Index (p_t)

Terminal Serviceability Level	Percent of People Felling Unacceptable
3.0	12
2.5	55
2.0	85

3.2.6 Resilient Modulus (MR)

The significantly different resilient moduli appear when the seasonal moisture conditions for which the roadbed soil samples need to be tested. For instance, in a climate, which is not put in danger to extended sub-freezing temperatures, it would be important to test for differences conditions between the dry seasons and wet (rainy) seasons. It would probably not be necessary. The first step should classify the region as climate change as US classification Table (3.9). As in the Tables (3.10) and (3.11), the M_R value can be calculated [22].

Table 3.9: The U.S Classification as Six Climatic Regions

(Traylor, M L, "Characterization of Flexible Pavements by Nondestructive Testing," Ph.D. thesis, University of Illinois,1979)

Region	Characteristics of Region
I	Wet, no freeze
II	Wet, freeze-thaw cycling
III	Wet, hard-freeze, spring thaw
IV	Dry, no freeze
V	Dry, freeze-thaw cycling
VI	Dry hard-freeze, spring thaw

Table 3.10: Proposed Seasons Length (Months) for the Six U.S. Climatic Regions classes

U.S Climatic Region	Season (Roadbed Soil Moisture State)			
	Winter (Roadbed Frozen)	Spring-Thaw (Roadbed Saturated)	Spring/Fall (Roadbed Wet)	Summer (Roadbed Dry)
I	0.0*	0.0	7.5	4.5
II	1.0	0.5	7.0	3.5
III	2.5	1.5	4.0	4.0
IV	0.0	0.0	4.0	8.0
V	1.0	0.5	3.0	7.5
VI	3.0	1.5	3.0	4.5

*Number of months for the season

Table 3.11: Proposed Seasonal Roadbed Soil Resilient Moduli, MR (psi), as a Role of the Qualified Quality of the Roadbed Material

Relative Quality of Roadbed Soil	Season (Roadbed Soil Moisture Condition)			
	Winter (Roadbed Frozen)	Spring-Thaw (Roadbed Saturated)	Spring/Fall (Roadbed Wet)	Summer (Roadbed Dry)
Very good	20,000*	2,500	8,000	20,000
Good	20,000	2,000	6,000	10,000
Fair	20,000	2,000	4,500	6,500
Poor	20,000	1,500	3,300	4,900
Very poor	20,000	1,500	2,500	4,000

*Values shown are Resilient Modulus in psi

3.2.7 Structural Number (SN)

It is equal to the structural number revealing of the total pavement thickness required Equation (3.4) [22]:

$$SN = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3 \qquad \text{Equation 3. 4}$$

Where

a_i = i^{th} layer's coefficient,

D_i= i^{th} layer's thickness (in), and

m_i = i^{th} layers drainage coefficient.

The procedure suggested is direct measurement using AASHTO Method, T274 (subbase and unbound granular materials) and ASTMD 4123 for asphalt concrete and other stabilized materials. Researchers and field studies specify many factors that influence on the layer coefficients. Therefore, the agency's experience must be included in applying the results from the procedures obtainable. For instance, the layer coefficient might differ with thickness, underlying support, location in the pavement structure, etc.

3.3 Pavement Type Selection

The principal factors of pavement selection lead to make an important decision. Such as other characteristics of pavement design, the AASHTO Guide says, “The choice of pavement alternative is not an exact science but one of the significant considerations, which the highway engineer must be careful while judging on different factors."

Primary factors for consideration can include [22]:

1. Traffic (volume, percentage of heavy trucks and degree of crowding resulting from following rehabilitation efforts)
2. Soils properties (shrink-swell potential, bearing capacity)
3. Climate/Weather change (amount of rainfall, freezing condition)
4. Construction deliberations (staged, determination of quick achievement, deviation requirements, expected future expansion)
5. Recycling for pavement (using material from obtainable structure or other sources)
6. Cost comparison (life-cycle cost analysis [LCCA] is the best, but initial costs might knowledge).

Secondary factors may include:

1. Presentation of similar pavements in the area (similar structures with similar traffic history),
2. Together current pavement sections (continuity of cross section),
3. Management of materials and energy,
4. Obtainability of local materials or contractor abilities,
5. Traffic safety (reflectivity amount under highway lighting, surface drainage, maintenance because of skid properties),
6. Traffic noise mitigation (added),
7. Combination of experimental structures (unique to one pavement type),
8. Stimulation of competition between major paving industries and local preference

3.4 Bitumen Classification

The term bituminous materials are generally used to refer to materials that compose the bitumen. Bitumen is defined as an amorphous, black or dark-colored, can divide to (solid, semi-solid, or viscous), cementitious substance, composed primarily of high molecular weight hydrocarbons, and solvable in carbon disulfide. The Figure (3.2) illustrates the classification of Bitumen [24].

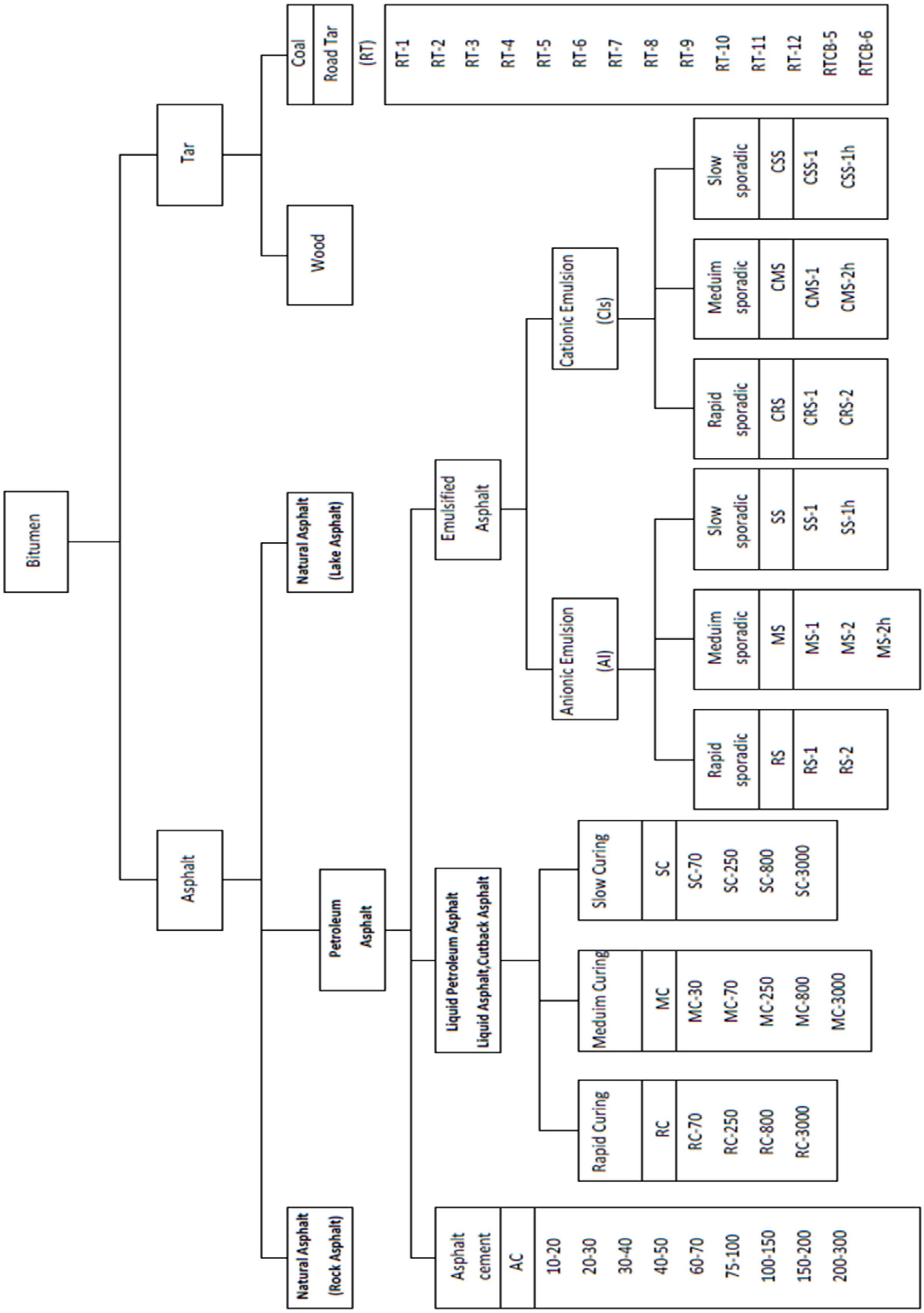

Figure 3. 2: Illustrates the Classification of Bitumen

CHAPTER 4

LIFE CYCLE COST ANALYSIS

The economic process used to compare the results of the design phases is called LCCA, in which throughout the lives of each phase efforts are spent for agreeing upon all significant costs and benefits.

If we need to understand, the LCCA is from the economical side, determining which alternative has the best investment value and not an engineering tool that discovers how long phases will last or how well it will be achieved. This does not mean that engineering is not an important factor of the LCCA, but suitable engineering must be applied to ensure that each differing phase achieves the design requirement and gives the similar products. If the phases do not provide similar efficiency, then an economic estimation using LCCA to compare them is not realistic [5].

There are several economic dials available in the economic estimation of projects. The common contain benefit/cost ratio (B/C), net present value (NPV), internal rate of return (IRR), and equivalent uniform annual cost (EUAC) [43]. The transportation organization's choice of the suitable indicator depends on a number of factors such as the level and situation of analysis or the economic environment in which the analysis is shown. For instance, the IRR is the preferred economic indicator when projects are assessed in developing countries where the discount rate is greatly indeterminate [42].

4.1 Types of Economic Analyses

There are two main parts of LCCA count on the analysis stage of projects [5]:

4.1.1 Primary analysis

It is found when an exact group of phases should be done that means we used it in exact projects or in Maintenance. For instance, a primary analysis uses if a roadway, a transportation system, or neither should be established. Primary analysis significant calculates if any improvements need to be done for all. Therefore, in the primary analysis, a “does nothing” category is a possible phase can be analyzed.

4.1.2 Secondary analysis

It compares the phases that satisfy the primary analysis. It is the evaluation that shows which phase provides the best value. For example, LCCA for pavement type selection (concrete or asphalt) is a secondary analysis. Because of the basic condition for applying the primary analysis that must be done for a pavement project, which means that an organization cannot invest the funds in stocks, bonds, etc. without constructing any activity of the project. The profit should only be based on investing in money.

There is no specific field for life cycle cost analysis process. Any alternative for any roads can be taken into achievement. Another thing that the life cycle cost analysis is a more economically efficient process for rigid pavements than flexible pavements is because of the long life for rigid pavement.

The simple definition of LCCA is an analysis process that discusses more report and - it is an emphasis - the Better Investment Decisions. It is represented a capital of some basic principles of economic analysis that have to evaluate highway and other public projects investments for years. However, LCCA has slightly stronger strategies management for a long time. In addition, the LCCA applies to broadly different elements of investment-related decision making to give improvement for the economic value of different designs, projects, phases, or strategies of investment to get the best profits [5].

4.2 Present Worth and Equivalent Uniform Annual Cost

The outcomes of an LCCA can be presented in many ways. The two most common are Present Worth (PW), and Equivalent Uniform Annual Cost (EUAC). PW is the summation of all costs (initial cost and benefits) through the project life in today's dollars, Figure (4.1) [5].

LCCA consists of initial costs with discounted value in future, maintenance costs or rehabilitation costs and salvage value. The prediction costs for future are reduced to define for the time value of cost using the discount (real interest) rate. Present value analysis is used to compare alternatives with the same analysis periods [35]. Equation (4.1) presents how the present worth can be calculated [5].

$$PW = IC + \Sigma[pwf(MC) + (UC) + (FRC)] - pwf(S) \qquad \text{Equation 4.1}$$

Where:

IC = Initial Cost

MC = Maintenance Cost

UC = User Cost

FRC = Future Rehabilitation Cost

S = Salvage Value

pwf = Present Worth Factor

$= 1 / (1+i)^{n}$

i = discount rate

n = Number of years to when cost (benefit) will occur

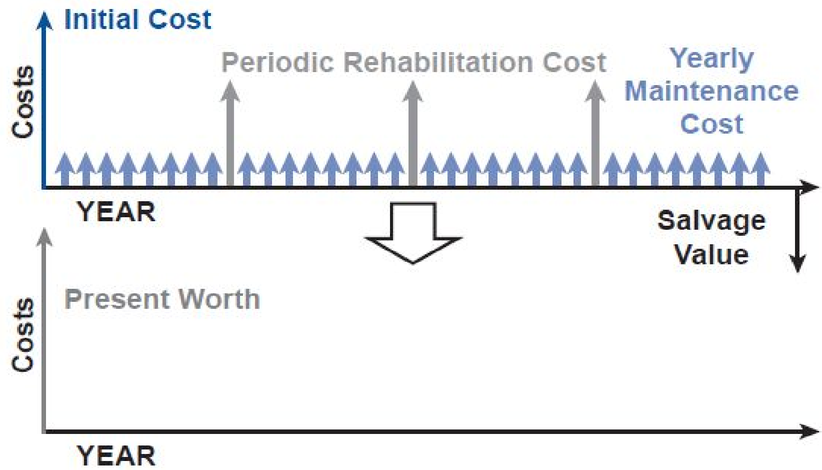

Figure 4.1: Diagram presenting all costs converted to the Present Worth

EUAC is used to convert the cost of all parameters (initial, user, maintenance, and predictable rehabilitation costs) to an annual cost through all the analysis period, Figure (4.2) [5]. An analysis using EUAC technique lets the identical varieties analysis periods for alternatives. Therefore, when doing the analysis for comparison between different phases, the customer should understand the main thing - that the analysis is supposing for the same group of actions will be frequent for an unlimited period. Equation (4.2) is used to compute the EUAC [5]:

$$EUAC = crf\ (IC) + AM + AUC + S[crf\ (pwf\ \{FRC\}) - crf\ [pwf\ (S)\ ] \qquad \text{Equation 4. 2}$$

Where:

AM = Annual Maintenance Cost

AUC = Annual User Cost

crf = Capital Recovery Factor

$= [i\ (1 + i)\ n] / [(1 + i)\ n - 1]$

i = discount rate

n = Number of years to when cost (benefit) will occur

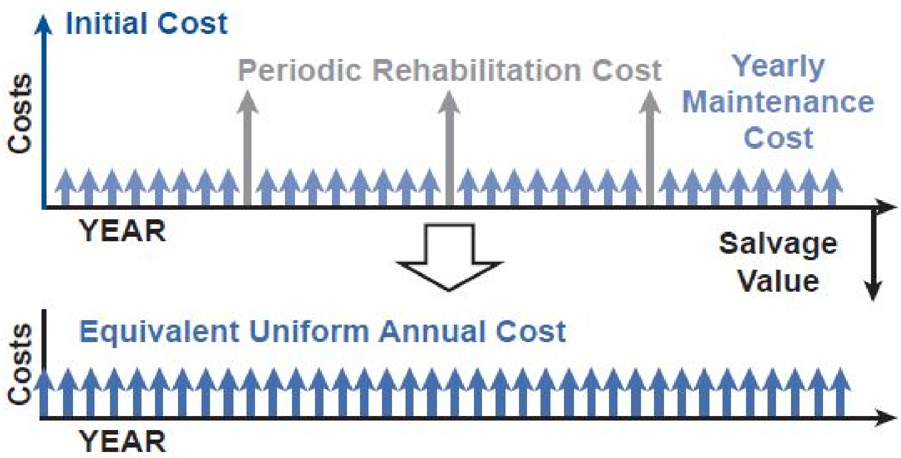

Figure 4.2: Diagram presenting all costs converted to the EUAC

4.3 Performance of LCCA

The implementing of the life cycle cost analysis is not complex. It is just a mathematical determination of the early outflow running during the life of the project. Although a computer program or worksheet is useful in the presentation of these calculations, a predictor does not principally need these presentation tools [3].

There are six basic steps to present LCCA [6]:

1. Make a design similar pavement parts.

2. Developing the money flowing schedule over the period of the project by:

a. Put the strategy for the rehabilitation and maintenance (description and schedule time) to use it during the design life.

b. Estimate agency costs for all activities separately.

3. Predicate the user costs.

4. Calculate PW or EUAC.

5. Analyze the results by using the computer programs (models) or worksheets.

6. Re-work on plans and arrangement strategies.

The explanation of the six stages of LCCA are:

STEP ONE: Making a design similar pavement parts

The LCCA process is made after estimation of all activities and puts all these activities in specific ranks to improve project activities. The minimum number of comparison alternatives is two, and the economic difference between cases is supposed to be accessible to compute the total cost of each phase.

In this step, main actions for each phase are explained and the analysis period is determined. The difference in alternatives of the project requires different maintenance and rehabilitation stages.

The importance of this stage is how we can find the period life of the project and evaluate at least only one main rehabilitation stage for every phase defined for the project with put all possible data of this stage.

STEP TWO: Developing money flowing timetable over period of the project

After the approximation, the plans for maintenance and rehabilitation for each activity separately must put a plan to develop the alternative’s maintenance and rehabilitation plan. Figure (4.3) illustrates the cycle of the example of construction, weakening, and rehabilitation that normal transportation approximation afford [5].

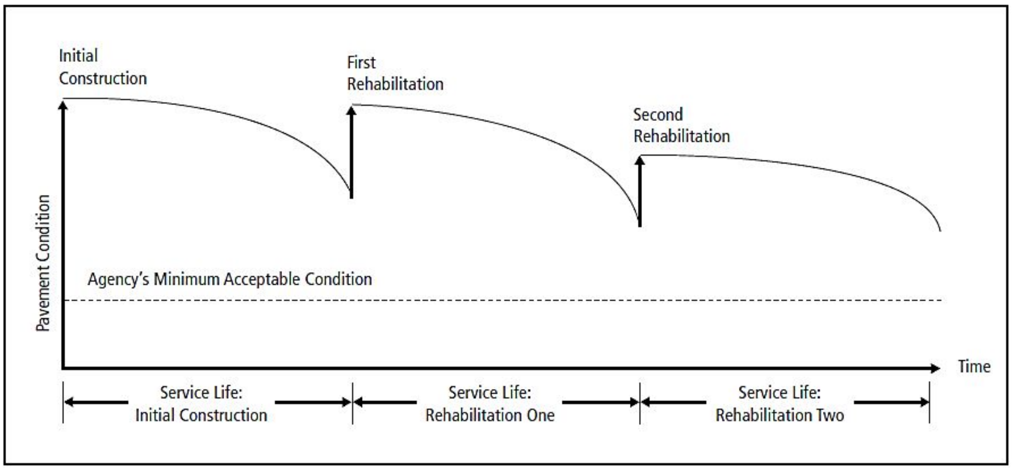

Figure 4.3: Example of Lifetime for One Design Alternative

STEP THREE: Predicating the user costs

Costs considered as the important part in LCCA for highway agencies and to users of the highway system because of construction and maintenance activities. LCCA does not require all costs to be related to each alternative to be calculated. Only costs that demonstrate the differences between alternatives need be discovered.

STEP FOUR: Computing LCCA (or Calculating PW, EUAC)

In the previous steps, the alternatives were defined with respect to agency costs, user costs, and the time when these actions will happen. At this step, the objective is to calculate the total LCCs for each phase so that they may be directly compared with other alternatives. On the other hand, because the amounts paid in dollars paid at different times have different present values, the projected activity costs for an alternative cannot be added together to calculate total LCC for that phase.

STEP FIVE: Analyzing the results by using the computer programs or worksheets This step includes analyzing and understanding the LCCA results. With the certainty or probabilistic LCCs computed, the PVs of the different costs may be associated across competing alternatives.

STEP SIX: Re-evaluating the alternatives or re-working on plans and planning strategies. The LCCA arranges with a review of the results to determine adjustments or modifications to any of the proposed alternatives might be indicated prior to completing the alternative selection. Revisions might contain design changes, newly definite work zone criteria for the contractors or different traffic plans to reduce high user costs.

4.4 Basic Factors for LCCA

The factors that influence on the results of LCCA for pavement type selection are:

- Agency costs (including engineering costs),
- User costs,
- Discount rate,
- Selection of rehabilitation activities,
- Use of comparable sections, and
- Length of the analysis period.

Other considerations, such as softness over time, safety, and environmental friendliness, may enter the pavement selection decision but can be hard to relate to cost or performance. If possible, it is better to use engineering adjustments on the alternatives to the explanation for these concerns before performing the LCCA. After the adjustments are made, LCCA will allow a rational comparison between the alternatives [3].

4.4.1 Agency Costs

Agency costs are all direct costs incurred by the agency over the lifetime of the project and it consists of [26]:

• Initial costs,

• Operation and maintenance costs (including staffing),

• Rehabilitation costs (including engineering and traffic control for each rehabilitation), and

• Salvage value.

When looking at agency costs, it is only necessary to contain costs that are specific to the separate alternatives and set them separately. The costs such as public hearing and informational meetings, permits, real estate and land development, legal fees, etc. are called common costs and they are sustained no matter which pavement alternative is selected. Consequently, these costs do not affect the comparison results.

4.4.2 User Costs

By calculating users' costs, we can see the influence of road works on road users. User Costs change during maintenance and rehabilitation stages. During rehabilitation and maintenance, user costs can raise dramatically. It is recognizable that road works cause delay and increase the vehicle operating costs (fuel and maintenance of vehicles) as well as the number of traffic accidents are increased. User costs can be classified into following categories: [44]:

1. Delay-of-use costs,

2. Roadway deterioration costs, and

3. Accident or crash cost.

4.4.3 Discount Rate

The discount rate is a highly important factor in LCCA and can have a major influence on the outcome. When analyzing long-term public investments, discounting is an important element in comparing costs happening at different points in time [42].

Wisconsin Department of Transportation's procedure for defining the value of discount rate was a 4% discount rate.

The discount rate has two advantages associated with its use. Firstly, not only the total values of the interest and increase rates that matter, but also their difference is important [45].

Secondly, using of the discount rate force the analysts to realize, the results are fake values on the total costs of ownership. Namely, the results are not the actual amounts in dollars that will be wanted in the future to complete each of the processes. LCCA can only be used to compare alternatives, and not determine exactly how much a pavement will cost over its total life [3].

4.4.4 Selection of Rehabilitation Activities

The rehabilitation activity selection should be based on the type of riskiness and the extent of suffering in the pavement. Because of a pavement deteriorates the type of rehabilitation will be changed.

In addition, considering the important changes in the materials and pavement design methodologies that have occurred over last few decades, the values generated from any historic database may not exactly reproduce the life of a future rehabilitation.

The suggested values are shown in Table (4.1) can be used as standard values for initial LCCA evaluations [42].

Table 4.1: Illustrate Minimum, Most Likely and Maximum Service Lives Standards for Rehabilitation and Preliminary Service Lives

Items	Flexible Pavement years			Rigid Pavement years		
	Min	Most Likely	Max	Min	Most Likely	Max
Preliminary Service Life	8*	11*	14*	18	24	30
	12ǂ	15ǂ	18ǂ			
Rehabilitation Service	8	11	14	8	11	14

* Conventional HMA; ǂ Polymer-Modified HMA

4.4.5 Equivalent Sections

In order to achieve a realistic and reliable life cycle cost analysis, the two alternatives must have equivalent and similar designs and should make available similar results over the analysis period. Therefore, the design of all alternatives should be the same in terms of inputs whereas the outcomes may not [46]:

- Structural (traffic-carrying) capacity,
- Reliability,
- Subgrade properties, and
- Terminal condition.

Additionally, they need to provide the same or rationally similar levels of service over the analysis period. If the two designs being compared do not have these same characteristics over the analysis period, the resulting LCCA is mistaken. Unfortunately, this is difficult because of the difficulty in:

1. Exactly calculating performance over time, and

2. The difference in performance between the alternatives is counted.

At this time, the only national design procedure that straightly compares flexible and rigid pavement designs is the AASHTO Pavement Design Procedure.

4.4.6 Length of Analysis Period

It means the period over which all costs are compared. It does not have to be the same to the design or service life; and in most cases, it is not. The main standard is that it is long enough to reflect the cost differences between the two pavement kinds. Therefore, it should be long enough to contain at least one rehabilitation cycle for all alternatives.

However, the analysis period can be founded on module life or a common multiple of module life, facility life or investment life. Most governmental agencies base their analysis period on a random period that is equal for each alternative. Typically, the values are 30-40 years for highways, 20-30 years for streets, and 30 years for airports [3].

In general, a cost that is experienced in 25-30 years or more plays a minimum role in the life cycle cost analysis. If an agency is performing LCCA on the alternatives where the analysis periods are not equal, then the life cycle cost analysis must be performed with equal uniform annual cost.

However, in doing this, the analyst must understand that this procedure assumes that each alternatives' activities will be repeated indeterminately (i.e. the activity streams will be frequent forever). Therefore, the analysis period shall be long enough to incorporate at least one rehabilitation activity for each alternative, Figure (4.4) [3].

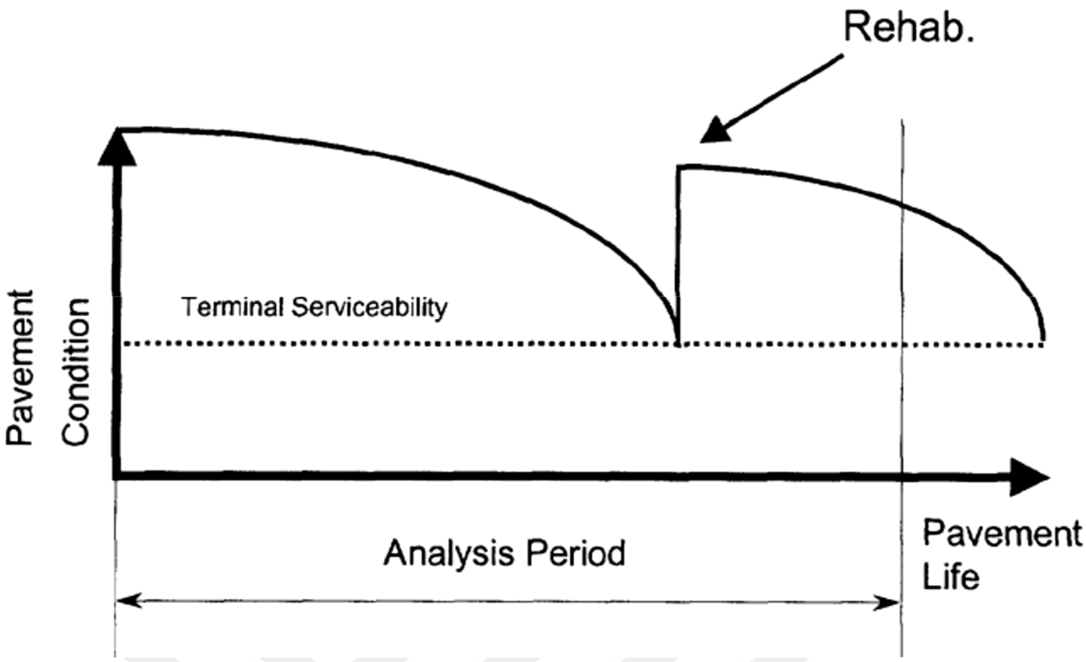

Figure 4.4: Analysis Period for a Pavement Design Alternative (general case)

The 1993 AASHTO Guide for Design of Pavement Structures also makes available some guidelines on the selection of an analysis period. The optional analysis periods, depending on the highway conditions can be seen in Table (4.2) [22].

Table 4.2: Suggested Analysis Period

Highway Type	Analysis Period (yrs)
High Volume Urban	30 - 50
High Volume Rural	20 - 50
Low Volume Urban	15 - 25
Low Volume Aggregate Surface	10 - 20

4.5 Net Present Value (NPV):

The NPV is the present discounted monetary value of predictable net benefits to computing NPV. The values need to be assigned to profits and costs. These values need to be discounted to present day costs using a suitable discount rate. As a final point, the sum of total present discounted costs needs to be subtracted from the total present discounted profits. Meanwhile, the benefits of keeping the pavement above a certain terminal serviceability level are the same for all phases; the benefit component decreases. The resulting Equation (4.3) for NPV is [3]:

$$NPV = InitialCost + \sum_{k=1}^{n} \mathrm{Re}\,hab \cdot Cost_k \left[\frac{1}{(1+i)^{n_k}} \right] \qquad \text{Equation 4.3}$$

Where:

i = Discount Rate

n= Year of Expenditure

The $\left[\frac{1}{(1+i)^{n}\mathrm{k}} \right]$ component of the above formula is referred to the Present Value (PV) As an example when the D.R = 4% the value factor for a single future amount is illustrated in Table (4.3) [3].

Agency, user cost and social costs are calculated separately before the net present value of the total project is computed in order to improve the understanding components of the total cost. In the procedure defined in the FHWA Technical Bulletin for calculating serviceable life, the value of the pavement is determined by multiplying the cost of the newest rehabilitation activity by the percent of design life lasting at the end of the analysis period. Studying the procedure by joining the cost of initial construction instead of the latest rehabilitation activity is currently being measured by FHWA [3].

Table 4.3: Present Value for the Discount factors, for single future payment

Discount Rate (i)			
years	i=0.04	years	i=0.04
1	0.9615	11	0.6496
2	0.9246	12	0.6246
3	0.8890	13	0.6006
4	0.8548	14	0.5775
5	0.8219	15	0.5553
6	0.7903	16	0.5339
7	0.7599	17	0.5134
8	0.7307	18	0.4936
9	0.7026	19	0.4746
10	0.6756	20	0.4564

CHAPTER 5

CASE STUDY

The case study is to compare the three different alternatives of the roadway pavement considering variables. In each alternative, different variables such as the thickness of layers and different type of the layers are used. The aim of this case study is to find the economical, sustainable and available design of the pavement structure. The design procedure of the pavement followed the AASHTO 1993 Guide, and all parameters were used in the case study was taken from tables and curves of AASHTO 1993 Manual. The roadway that is designed has the geometric details in Table (5.1), the geometric design of it is illustrated in Figure (5.1), and the cross section in the roadway is illustrated in Figure (5.2). The case study will design and analyze just one kilometer from this roadway.

Table 5.1: Illustrates the Lane Width and the Details of the Roadway

Detail	Unit	Dimension	Notes
Length of Whole Roadway	km	34	The case study took just 1 km from the whole length
Lane Width	m	3.5	
Median Width	m	3.0	
Shoulder Width	m	2.5	Both Side

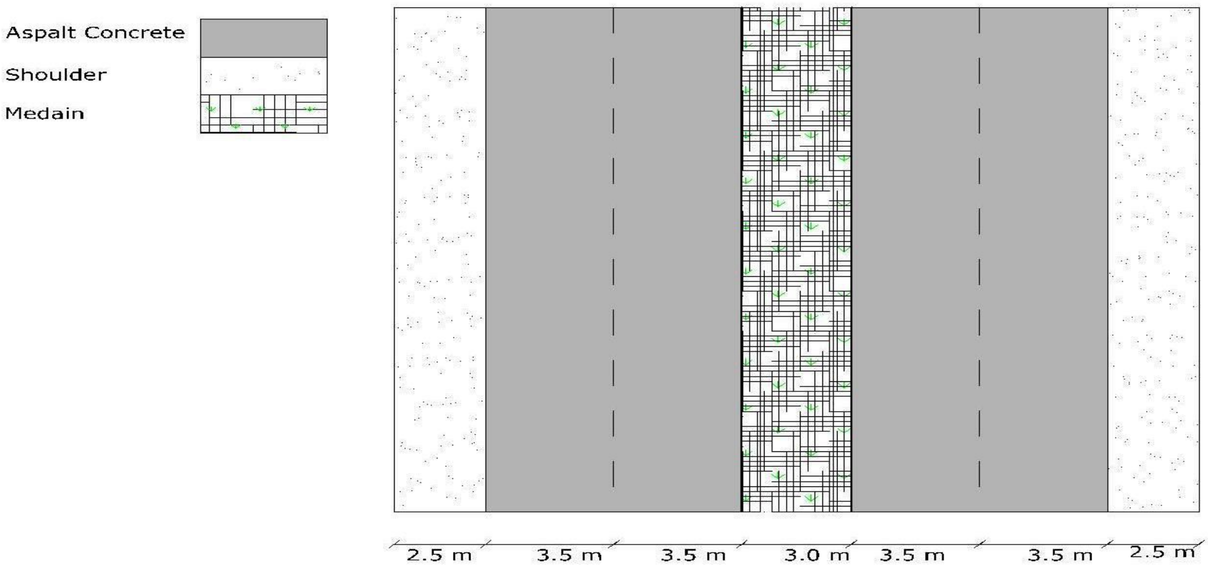

Figure 5.1: Illustrates the Lane Width and the Geometric Details of the Roadway

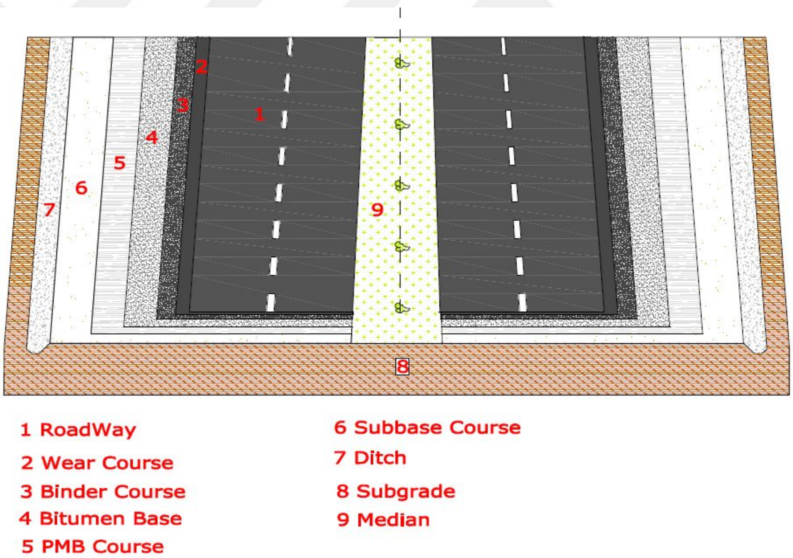

Figure 5.2 Illustrates the Cross Section in the Roadway

5.1 Data Collection

Every roadway design needs the data for starting the design procedure and the success of these projects influenced by the reliability of these data. The data collection should contain details for the project activities. As an example, the data should contain the length of road segment, width, shoulder details, etc.

5.1.1 Traffic Data

The traffic volume data were obtained from the General Administration of Highway in Turkey for the segment 400-23 in GAZİANTEP the southern part of Turkey where the environmental characteristics is the similar with the southern part of Iraq. Study with the traffic data consist of three main parts:

a) Obtaining Traffic Data

General Directorate of Highways permanently and portably performs traffic counts and special counts. Permanent counts are realized continuously within the year. Portable counts are short-term counts and are performed in every season, seven days and 24 hours according to annual systems [49].

The vehicles are classified as light (passenger cars), medium goods and heavy (busses, trucks, and articulated trucks) vehicles.

In this study, traffic count data between 2004 and 2015 were used to find out the input traffic volumes as an ESAL for pavement design. Turkish agency provided the data for twelve years from 2004 up to now, Table (5.2) illustrates the traffic data from Turkish agency for 12 years; the Figure (5.3) illustrate the ESALs for all vehicle types [49].

The case study involves twenty years where the base year is 2017, which means that the design life of this structure will be finished in 2037. The calculation of base year data was made by using the growth factor, Table (5.3).

In addition, the speed data are evaluated in three different groups [49]:

According to Single or Dual Carriageway; the percent of cars, which exceed 90 km/h or 110 km/h speed, the percent of medium goods vehicles, and buses, which exceed 80 km/h or 90 km/h speed, and the percent of trucks, and articulated trucks, which exceed 80 km/h or 85 km/h speeds, are given. The average speeds are 85 km/h percentile speeds for all vehicle classes.

Table 5.2: Illustrates the Official Traffic Data from General Administration of Highway in Turkey GAZIANTEP in the Southern Part for Years (2004-2015)

Traffic Data (GAZIANTEP OTSS1)													
Year	Length of segment	Summation Of AADT	Car		Medium Goods Vehicle		Bus		Truck		Articulated Truck	
			AADT	ESAL	AADT	ESAL	AADT	ESAL	AADT	ESAL	AADT	ESAL
2004 OTSS1	34	10615	5633	493	669	58604	299	139693	3305	1399337	709	424407
2005 OTSS1	34	11513	6860	601	599	52472	303	141562	3061	1296027	690	413034
2006 OTSS1	34	12083	6890	604	705	61758	368	171930	3310	1401454	810	484866
2007 OTSS1	34	9919	5630	493	586	51334	331	154643	2700	1143180	672	402259
2008 OTSS1	34	9585	5915	518	591	51772	351	163987	2022	856115	706	422612
2009 OTSS1	34	8617	5101	447	496	43450	300	140160	2052	868817	668	399865
2010 OTSS1	34	8818	5565	487	488	42749	225	105120	1957	828594	583	348984
2011 OTSS1	34	9301	6272	549	489	42836	225	105120	1798	761273	517	309476
2012 OTSS1	34	10555	7624	668	528	46253	212	99046	1755	743067	436	260990
2013 OTSS1	34	11451	8315	728	581	50896	212	99046	1826	773128	517	309476
2014 OTSS1	34	11393	8259	723	581	50896	169	78957	1851	783713	533	319054
2015 OTSS1	34	13018	9684	745	638	55889	170	79424	1912	809541	614	367540

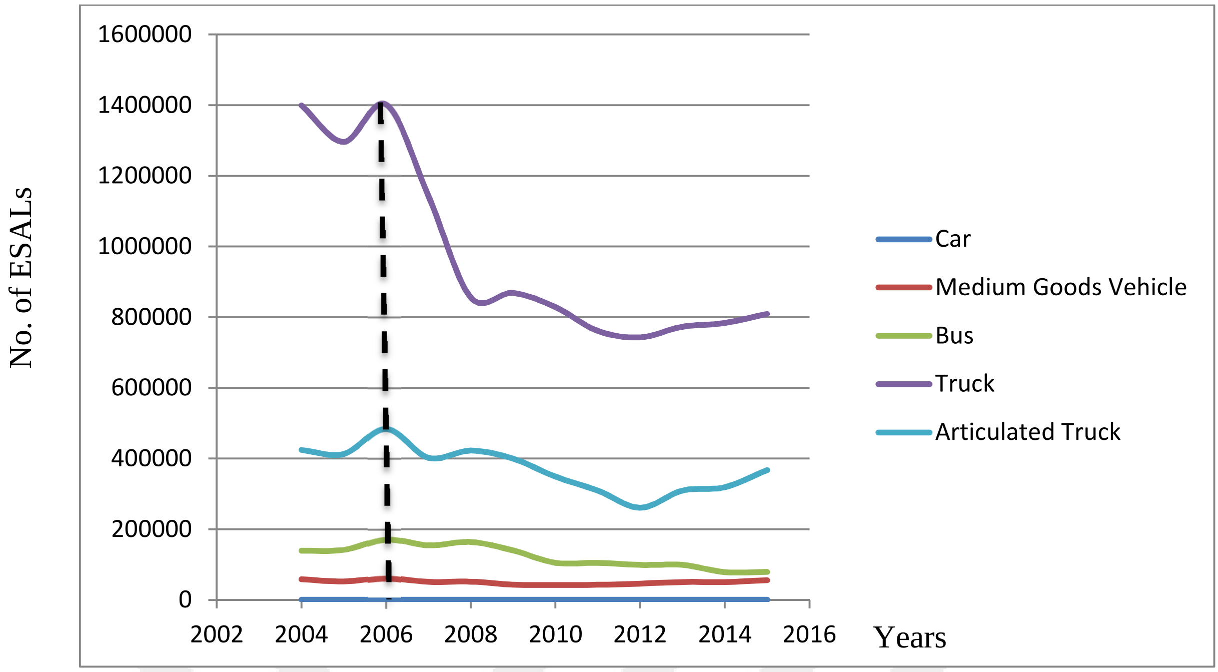

Figure 5. 3: Number of ESALs for All Vehicle Types

b) Estimation Growth Factor

Design life for this project is 20 years. The data are collected from the analysis' data of traffic volume for 12 years from Turkish data reference. The growth factor calculated depends on the Equation (5.1) [22].

$$W_{18}^{n} = W_{18}^{base} \frac{(1+r)^{n} - 1}{r} \qquad \text{Equation 5.1}$$

Firstly, the years 2004-2006 were used to calculate the G.F. for all vehicle types separately. In Table (5.3) illustrates the value of G.F. Because of the decrease in some years and return to increase in the traffic data, and the recording data of the last year in 2015 OTSS1 is less than 2004 OTSS1; when the G.F. is calculated, some irregular results were obtained. So, only the first three years were used. After calculating the growth factors from years 2004-2006, we used it to calculate the Equivalent single Axle Load for 20 years (starting from 2017 to 2037) by means of Equation (5.1).

The coverage count data were found by expanding this "old" estimate to a modern estimate using a growth factor resulting from data on segments predictable to show similar growth over the period of design life [48].

c) Calculation of Equivalent Single Axle Load (ESAL)

There are two values of ESALs in this study, the first one is the ESAL $_{predicted}$ and the second one is the ESAL $_{calculated}$. The procedure of finding these ESALs are:

1- By using, the convert factors in Table (3.4) to get ESAL (Equivalent single Axle Load) by multiplying the number of vehicles for each type in its factor to make all types of axle groups homogenous. Table (5.3) illustrates the ESAL after using the convert factors and growth factor [49].

2- By finding, the growth factor from part (growth factor) above for all vehicle types calculates the future traffic volume, Table (5.3).

Table 5.3: Illustrated the Design ESAL and Growth Factor

NO.	Year	Length	ESAL Car	ESAL Medium Goods Vehicle	ESAL Bus	ESAL Truck	ESAL Articulated Truck
2004	2004 OTSS1	34	493	58,604	139,693	1,399,337	424,407
2005	2005 OTSS1	34	601	52,472	141,562	1,296,027	413,034
2006	2006 OTSS1	34	604	61,758	171,930	1,401,454	484,866
Summation			1698	172835	453184	4096818	1322307
Growth Factors			0.03	0.02	0.05	0.02	0.05
2014	2014 OTSS1	34	723	50,896	78,957	783,713	319,054
2015	2015 OTSS1	34	745	55889	79424	809541	367540

Table 5.3 (continued)

2016	predication	34	768	57007	83395	825732	385917
2017	predication	34	791	58147	87565	842246	405213
2037	predication	34	21243	1412812	2895419	20464369	13398764
Summation Of ESALs (Design ESALs)			**38,193,000**				

3- The calculation of prediction equivalent single axle load (ESAL) based on which year will be the base year in calculation and by using the (Equation (5.1)) and the growth factor will calculate the Design ESALs. In this case study, the five growth factors (for all vehicle types) were used, which means that we have five ESAL $_{predicted}$ and they are added together to get the total design ESAL $_{predicted}$.

4- Now we use the value of ESAL $_{predicted}$ to calculate the ESAL $_{calculated}$ using the Equation (3.2) and assumption values of Structural Number (SN), Table (5.9) and Table (5.14) illustrates all parameters in Equation (3.2).

5.1.2 Materials Characteristics

A flexible pavement consists of a subgrade (prepared roadbed), subbase, base course and surface course. The performance of the pavement depends on the satisfactory performance of each layers material characteristics bring information for each layers.

- Subgrade**:** The subgrade is the natural granular material located along the horizontal alignment of the pavement (It works as the foundation of the pavement structure). According to the type of pavement constructed, it is essential to treat the subgrade material to achieve the required strength properties.

- Subbase Course: Located just above the subgrade, the subbase component consists of a material of a higher quality, which is generally used for subgrade construction. When the quality of the subgrade material meets the requirements of the subbase material, the subbase component may be deleted. When the subbase material does not correspond to the requirements, a process of treating soils to improve their engineering properties known as stabilization can be used. In fact, the available material should be treated with other materials to achieve the required properties.

- Base Course: The base course is placed above the subbase (above the subgrade if the subbase course is not used). It consists of granular materials such as sand, crushed stone, crushed or uncrushed gravel and crushed or uncrushed slag.
  Usually, the base course materials include firmer requirements than those for subbase course. In some cases, to increase the stiffness characteristics of heavy-duty pavements, the base course can add the asphalt to achieve the required properties.

- Surface Course: The surface course is the upper layer of the pavement section located directly above the base course. The surface course in flexible pavements generally consists of a mixture of mineral aggregates and asphaltic materials. It must be able to resist a wide variety of factors that can accelerate the deterioration process of the pavement. The surface course can be divided into layers the upper is wearing and the lower is the binder.

In Table (5.4) illustrated the pavement layer characteristics and the coefficients influenced on layers properties [50]. The coefficients layer (a_i) assumed consideration on the previous studies and used a high quality for surface course [50]. There is no value calculation on subgrade layer because of unlimited thickness for it; also, there are no drainage coefficients for surface course and subgrade.

Table 5.4: Illustrate the Pavement Layer and Them Properties

NO.	Layer	Coefficient Layer	Elastic Modulus E (psi)	Drainage Coefficient	Layer Thickness cm
1	Wearing Course	0.44	250000		D_1
2	Binder Course	0.4	200000		D_1
3	Bituminous Base	0.32	175000	1	D_2
4	PMB Course	0.15	35000	1.1	D_2
5	Subbase	0.11	15000	1.1	D_3
6	Subgrade		6000		

5.2 AASHTO Design Pavement Parameters

The American Association of State Highway and Transportation Officials is one of the common manuals for pavement design and the empirical equation used in it the Equation. (3.2).

- Reliability (R)

  From Table (3.6) AASHTO suggested levels of reliability for various functional classifications, so R=0.95 for Interstate and Other Freeways (Rural).

- Standard Normal Deviate (ZR)

  The design life is 20 years; the reliability is 95 % (100 % – 5 %) and the corresponding ZR value is ZR = -1.645 Table (3.7).

- standard Deviation (S_0)

  The value of S_O = 0.45 for flexible pavements from (AASHTO, PART II).

- Serviceability Index (PSI)

There are two types of serviceability indexes: The initial serviceability (p_o) and the terminal (p_t). The p_o values observed at AASHO Road Test were (4.4) for flexible pavements, and p_t an index of (2.5) or higher is suggested for the design of major highways according to percent of people stating unacceptable from AASHO Road Test, Table (3.8). In this case study $p_o = 4.5$ and $p_t = 2.5$.

- Resilient Modulus (MR)

The calculation of MR depends on assumptions:

a) Climate Region: Here, the fifth region (V) is chosen according to the Six Climatic Regions in the United States (this region is near to the Iraqi climate conditions), Table (3.9).

b) By using Table (3.10) Suggested Seasons Length (Months) for the Six U.S. Climatic Regions and Table (3.11) Suggested Seasonal Roadbed Soil Resilient Moduli, as a Function of the Relative Quality of the Roadbed Material MR (psi) will be in Table (5.5).

Table 5.5: Illustrate the Calculation of Resilient Modulus (MR) [(1)]

Calculation of MR according to the fifth climate condition (V[(2)])				
Titles	Winter (Roadbed Frozen)	Spring-Thaw (Roadbed Saturated)	Spring/Fall (Roadbed Wet)	Summer (Roadbed Dry)
Number of months	1	0.5	3	7.5
Roadbed soil modulus [psi] for GOOD conditions	20000	2000	6000	10000
Relative damage Uf = 1.18 X 108 X MR -2.32	0.01	2.6	0.2	0.062
Uf × number of months	0.01	1.3	0.6	0.465
The Mean of Uf=∑ Uf /12	0.2			
MR	≈ 6000 psi			

[(1)]Some of these parameters assumed according to Iraqi climate conditions.

[(2)]Sixth climatic region; as the U.S climatic classification, AASHTO – 1993.

- Structural Number (SN)

The structural number can be determined from the Equation (3.5).

There are three main layers (Surface (wearing and binder), Base and Subbase course). Only the Base and Subbase layer have drainage coefficient (mi).

Layer Coefficient (a_i): It can be calculated from charts and equations depending on the elastic modulus (E).

For the surface a_1 can calculate from the (from AASHTO Guide1993, Figure 2.5., PART II)

For the base: $a_2 = 0.249 \log E_{BS} - 0.977$

For the subbase: $a_3 = 0.227 \log E_{SB} – 0.839$

Both the resilient modulus of the base, E_{BS}, and the subbase, E_{SB}, are stresses depend on following:

$E = K_1\ \theta^{K2}$

Where:

E = Elastic modulus [psi],

θ = sum of the principal stresses [psi], Table (5.6).

k_1, k_2 = material constants, Table (5.7).

The sum of the principal stresses in the base and subbase indeed depends on the thickness and stiffness of the layers placed on top of them in addition to the magnitude of the load. Optional values for θ are presented in Table (5.6) [22].

As it can be noticed from Table (5.7), the material constants k_1 and k_2 depend on the moisture content of the material (dry, saturated, wet) like the quality of the material (shown by the range of values) [22].

Table 5.6: Approximation Value of θ for Base and Subbase*

	Roadbed Resilient modulus [psi]		
AC Thickness	3000	7500	15000
< 2	20	25	30
2 to 4	10	15	20
4 to 6	5	10	15
> 6	5	5	5

* From AASHTO Guide – 1993, PART II.

Table 5.7: Standards of k_1 and k_2 for Base and Subbase Materials*

Base		
Moisture Condition	K_1**	K_2**
Dry	6000 to 10000	0.5 to 0.7
Damp	4000 to 6000	0.5 to 0.7
Wet	2000 to 4000	0.5 to 0.7
Subbase		
Dry	6000 to 8000	0.4 to 0.6
Damp	4000 to 6000	0.4 to 0.6
Wet	1500 to 4000	0.4 to 0.6

* From AASHTO Guide – 1993, PART II.

**Range in k_1 and k_2 is a function of the material quality

The drainage coefficient (m_i) is a very important feature of pavement structures. Insufficient drainage might result in moisture conditions close to saturation. As seen in Table (5.7), such conditions result in significantly lower values for k1, implying that the modulus of the unbound base and subbase can be three times lower in wet conditions than when they are dry. Recommended (m_i) values are illustrated in Table (5.8) [22].

Table 5.8: Illustrates the Drainage Coefficient depends on the drainage coefficient (m_i)

Quality of Drainage	Percent of Time Pavement Structure is shown to Moisture Levels Future Saturation			
	< 1%	1 to 5 %	5 to 25 %	> 25 %
Excellent	1.40 to 1.35	1.35 to 1.30	1.30 to 1.20	1.20
Good	1.35 to 1.25	1.25 to 1.15	1.15 to 1.00	1.00
Fair	1.25 to 1.15	1.15 to 1.05	1.00 to 1.80	0.80
Poor	1.15 to 1.05	1.05 to 0.80	0.80 to 0.60	0.60
Very Poor	1.05 to 0.95	0.95 to 0.75	0.75 to 0.40	0.40

After calculated the parameter of pavement design the result are illustrating in the Table (5.9).

Table 5.9: Illustrate the Parameter of Pavement Design*

Parameter	Units	Definition	Value
R	-	Reliability	0.95
Z_R	-	standard normal deviate	-1.645
S_o	-	Standard deviation	0.45 flexible pavements
PSI_i (p_o)	-	Initial Serviceability Index	4.50 for flexible pavements
PSI_t (p_t)	-	terminal serviceability index	2.50
M_R	psi	Resilient Modulus	6000

*The value of SN will calculate in Design of Alternatives

5.3 Design of Alternatives

The case study has three different layer systems as alternatives. However, there are some layers did not change such as the wearing course: PMB course, subbase and subgrade. This means the modification in two layers are shown in Table (5.10).

In the first alternative, two layers of the surface course are used with also bituminous under it, then the PMB, subbase and subgrade, respectively.

The second alternative layers are as the same with the alternative one; but the bituminous base is eliminated. The third alternative does not have the binder course.

In following parts, the difference between the three alternatives and their costs will be shown Table (5.10).

Table 5.10: Illustrate the Layer That Used in Alternatives of Pavement Design

Layer	Alternative 1	Alternative 2	Alternative 3
Wearing Course	√	√	√
Binder Course	√	√	
Bituminous Base	√		√
PMB Course	√	√	√
Subbase	√	√	√
Subgrade	√	√	√

There are some parameters influencing on the thicknesses of layers; the values of layers differ from layer to layer as shown in Tables (5.11, 12, and 13).

For alternative one Table (5.11), when used for the five layers and the design to some extent regular, the design thickness of the layers shown in the last column start from the small thickness for the surface course going up to rather big thicknesses for the rest of the layers.

Table 5.11: Layers Design (Alternative 1)

Layer's name	Elastic Modulus Mr (psi)	Layer coff. (a)	Drainage coff. (m)	$W_{18calculated}$	$SN_{req.}$ (cm)	Thickness (cm)
Wearing Course	250000	0.44	1	-	-	4
Binder Course	200000	0.4	1	-	-	5
Bituminous Base	175000	0.32	1	38195217	9.2	20
PMB Course	35000	0.15	1.1	38196971	12.2	25
Subbase	15000	0.11	1.1	38196430	16.2	40
Subgrade	6000	0	0			94

The modification in design in alternative two eliminates the bituminous base and that influences on the thicknesses of the rest of the layers; increases in surface course and also the PMB and subbase layers are huge to achieve the requires properties, Table (5.12).

Table 5.12 : Layers Design (Alternative 2)

layer's name	Elastic Modulus Mr (psi)	Layer coff. (a)	Drainage coff. (m)	$W_{18calculated}$	$SN_{req.}$ (cm)	Thickness (cm)
Wearing Course	250000	0.44	1			10
Binder Course	200000	0.4	1			12
Bituminous Base	175000			0	0.0	0
PMB Course	35000	0.15	1.1	38196971	12.2	40
Subbase	15000	0.11	1.1	38196430	16.2	50
Subgrade	6000	0	0			112

In alternative three, the modification in design eliminates the binder course while the bituminous base remains, but that does not influence on the thicknesses of the rest of the layers, just increasing the wearing course 3cm because of the bituminous base is found, Table (5.13).

Table 5.13: Layers Design (Alternative 3)

Layer's name	Elastic Modulus Mr (psi)	Layer coff. (a)	Drainage coff. (m)	$W_{18calculated}$	SN req. (cm)	Thickness (cm)
Wearing Course	250000	0.44	1			7
Binder Course	200000	0.4	1			0
Bituminous Base	175000	0.35	1	38195217	9.176	20
PMB Course	35000	0.15	1.1	38196971	12.227	25
Subbase	15000	0.11	1.1	38196430	16.215	40
Subgrade	6000					92

The rest of the parameters that influence on the design - such as the coefficient layer and drainage coefficient - for all of the layers are illustrated in Table (5.14).

Table 5.14: Illustrate All Values of Require Parameter for Layer Pavement

Parameter	Units	Definition	Value		
			1st Alt.	2nd Alt.	3rd Alt.
$a_{1(wearing)}$	-	Structural layer Coefficient for surface course(wearing)	0.44	0.44	0.44
$a_{1(binder)}$	-	Structural layer Coefficient for surface course(binder)	0.40	0.40	0.40
$D_{1(wearing)}$	cm	Thickness surface layer(wearing)	4	10	7
$D_{1(binder)}$	cm	Thickness surface layer(binder)	5	12	0
$a_{2(bitu)}$	-	Structural layer Coefficient for base course(bitumen)	0.32	0.32	0.32
$a_{2(PMB)}$	-	Structural layer Coefficient for base course(PMB)	0.15	0.15	0.15
$D_{2(bitu)}$	cm	Thickness base layer(bitumen)	20	0	20
$D_{2(PMB)}$	cm	Thickness base layer(PMB)	25	40	25
$m_{2(bitu)}$	-	Drainage coefficient	1	1	1
$m_{2(PMB)}$	-	Drainage coefficient	1.1	1.1	1.1
$E_{BS(bitu)}$	psi	resilient modulus of base course(bitumen)	175,000	175,000	175,000
$E_{BS(PMB)}$	psi	resilient modulus of base course(PMB)	35,000	35,000	35,000
a_3	-	Structural layer Coefficient for subbase course	0.11	0.11	0.11
D_3	cm	Thickness subbase layer	40	50	40
m_3	-	Drainage coefficient	1.1	1.1	1.1
E_{SB}	psi	resilient modulus of subbase course	15,000	15,000	15,000

5.4 Traffic Hourly Distribution (THD)

In RealCost software, the one of the important inputs is Traffic Hourly Distribution in this software, there is a default values of it for rural and urban traffic, in both condition inbound and outbound traffic; these values influence on the user cost directly, where the increasing in the number of this traffic will increase the user cost.

For illustrating the impacts of THD on NPV and user costs, some modification will do on the default values.

Firstly used these values to draw the graph of AADT rural as a percentage, Inbound - Rural and Outbound - Rural percentage with Traffic Hourly Distribution (THD) and then divided these curves to four zones depends on the THD percentages, Table (5.15) and Figure (5.4).

Table 5. 15 The Default Values of Traffic Hourly Distribution and Zones

Zone NO.	Hour	AADT Rural %	Inbound - Rural %	Outbound - Rural %
1	0 - 1	1.80	48.0	52.0
	1 - 2	1.50	48.0	52.0
	2 - 3	1.30	45.0	55.0
	3 - 4	1.30	53.0	47.0
	4 - 5	1.50	53.0	47.0
2	5 - 6	1.80	53.0	47.0
	6 - 7	2.50	57.0	43.0
	7 - 8	3.50	56.0	44.0
	8 - 9	4.20	56.0	44.0
	9 - 10	5.00	54.0	46.0
	10 - 11	5.40	51.0	49.0
	11 - 12	5.60	51.0	49.0

Table 5.15 (continued)

	12 - 13	5.70	50.0	50.0
3	13 - 14	6.40	52.0	48.0
	14 - 15	6.80	51.0	49.0
	15 - 16	7.30	53.0	47.0
	16 - 17	9.30	49.0	51.0
4	17 - 18	7.00	43.0	57.0
	18 - 19	5.50	47.0	53.0
	19 - 20	4.00	47.0	53.0
	20 - 21	3.80	46.0	54.0
	21 - 22	3.50	48.0	52.0
	22 - 23	2.60	48.0	52.0
	23 - 24	2.70	47.0	53.0
Summation		100.00		

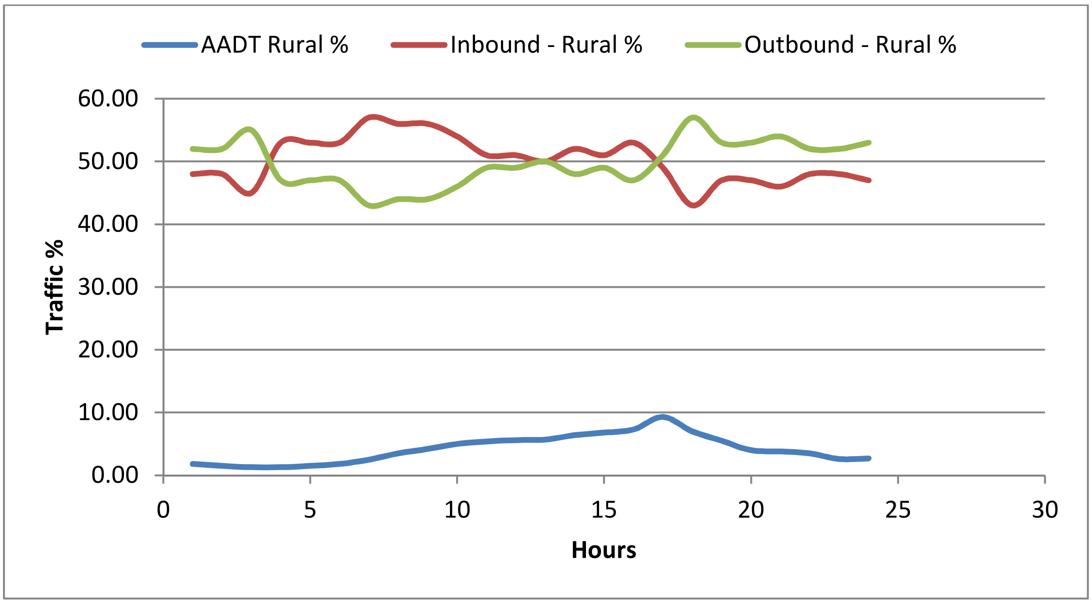

Figure 5.4 The Default Values of Traffic Hourly Distribution

Secondly, to make the modifications will be done with change the percentage of AADT (rural) by:

- Decrease the values of zones (1 and 4), 10%, and increase the values of zones (2 and 3), 10%, also; with draw the graphs Figure (5.5).
- Increase the values of zones (1 and 4), 10%, and decrease the values of zones (2 and 3), 10%, also; with draw the graphs Figure (5.6).

The outcomes of these modifications shown, there is no difference in NPV and user costs.

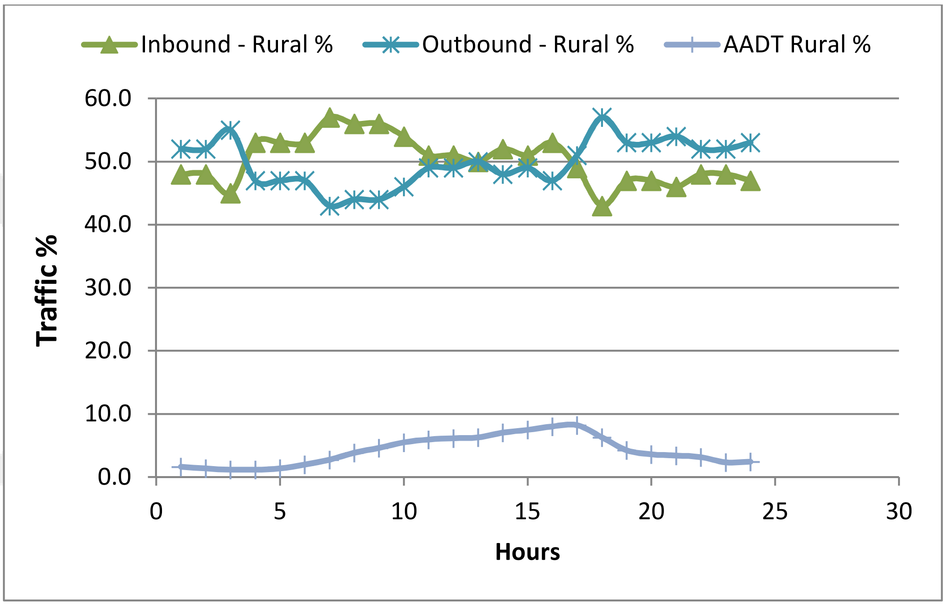

Figure 5.5 The First Modification of AADT Rural (%) with Traffic Hourly Distribution

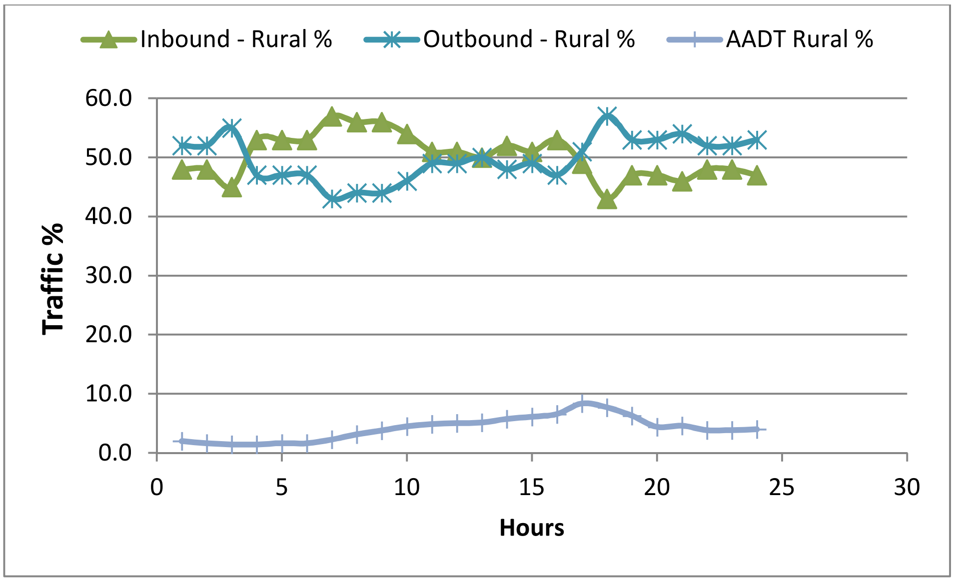

Figure 5.6 The Second Modification of AADT Rural (%) with Traffic Hourly Distribution

1. To increase the impacts of THD on NPV and user costs will be modified the, Inbound - Rural and Outbound - Rural percentage values with the same procedure that mentioned in point (1 and 2) above. The first modification outputted huge impact in zones (2nd and 3rd) more than the impact in zones (1st and 4th). The second modification did not output huge impact in zones (2nd and 3rd) and the impacts in zones (1st and 4th) were so less, Figures (5.7 and 5.8).

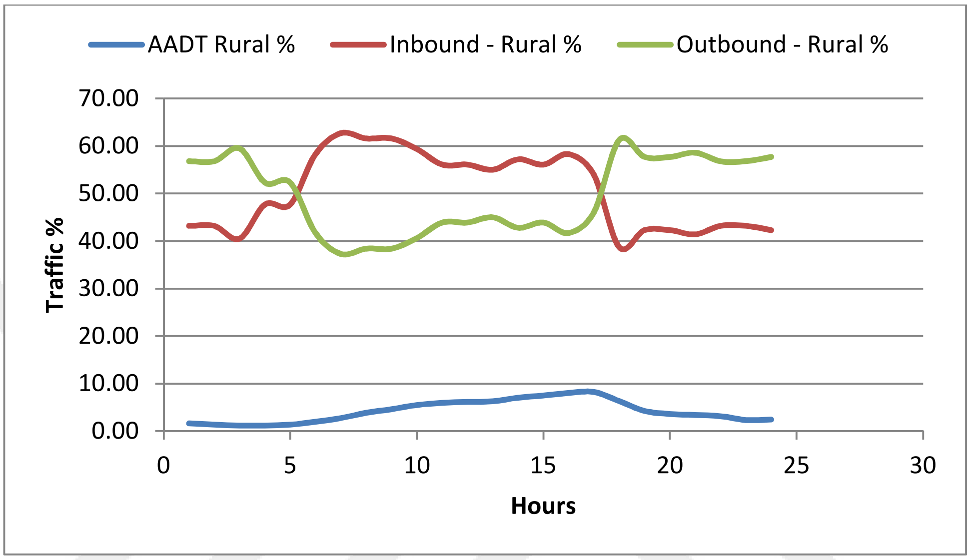

Figure 5. 7 The First Modification of Inbound - Rural and Outbound - Rural percentage

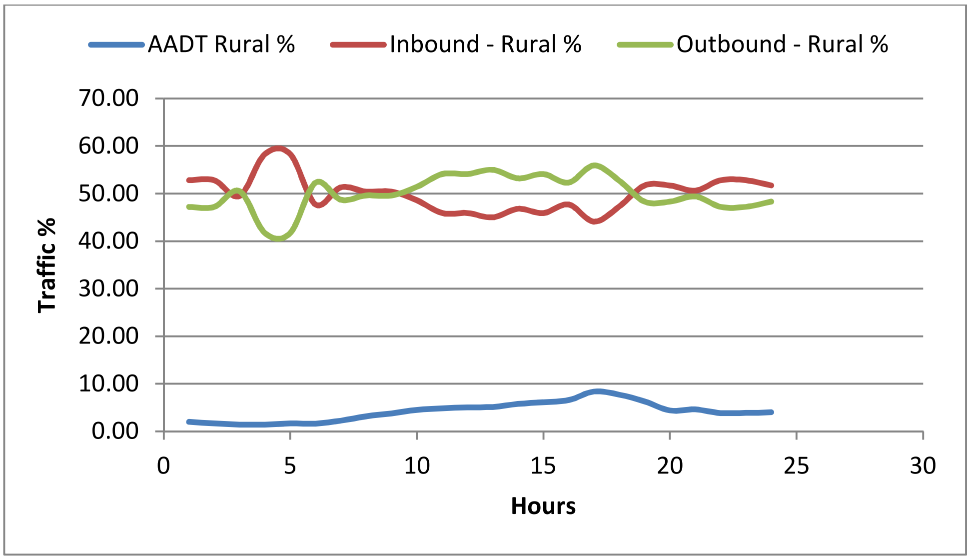

Figure 5. 8 The second Modification of Inbound - Rural and Outbound - Rural percentage

5.5 Evaluation of Alternatives with LCCA Method

FHWA Technical Bulletin lists the steps involved in conducting a life cycle cost analysis as follows [3]:

- Establish Alternative Pavement Design Strategies for the Analysis Period:

In section (5.3) Alternative Design, the alternatives and their design layers have been identified.

- Determine Performance Periods and Activity Timing:

The analysis period for highway (rural) is 20 years - from section (4.4.4) Selection of Rehabilitation Activities) - the initial service life is (10 years) and rehabilitation service life is (10 years also), Table (4.1).

- Estimate Agency Costs:

Agency costs are all direct costs incurred by the agency over the lifetime of the project. Section 4.4.1 Agency Cost illustrates all elements that might be determining. In this study, these costs will not be mentioned about; because they are fixed for all alternatives and the discussion about the variety cost.

- Estimate user costs:

From the RealCost software the user cost is found depended on the Value of User Time for all vehicle types (Cars, Trucks and Combination Tracks) and the unit of this value is ($/hr). In Iraqi and Turkish case, there is not specific value of user cost, it depends on the surveying condition and prices in time of preparing the study; here the assumptions of the Value of User Time will be used to calculate different value of user cost, Table (5.16).

Table 5.16: The Assumptions of the Value of User Time

Vehicle Type	Value of User Time ($/hr)	
	Min.	Max.
Cars	10	20
Trucks	15	30
Combination Tracks	20	40

To present the influencing of user cost on Total NPV will increase the user costs two times by 10% increasing every step and that influenced on the Total Net Present Value where it also increased dramatically by different percentages as illustrate in Table (5.17),and Figure (5.9) as an example of these increases.

Table 5.17 : The Vehicle Type with Different Cases of Total NPV (By Using RealCost)

Vehicle Type			Total NPV ($)		
Cars	Trucks	Combination Tracks	First Alternative	Second Alternative	Third Alternative
10	15	20	1,253,120	1,752,710	1,197,880
11	16.5	22	1,259,110	1,759,360	1,203,870
12	18	24	1,265,110	1,766,000	1,209,870
13	19.5	26	1,271,110	1,772,640	1,215,860
14	21	28	1,277,100	1,779,290	1,221,860
15	22.5	30	1,283,100	1,785,930	1,227,860
16	24	32	1,289,100	1,792,570	1,233,850
17	25.5	34	1,295,090	1,799,210	1,239,850
18	27	36	1,301,090	1,805,860	1,245,850
19	28.5	38	1,307,080	1,812,500	1,251,840
20	30	40	1,313,080	1,819,140	1,257,840

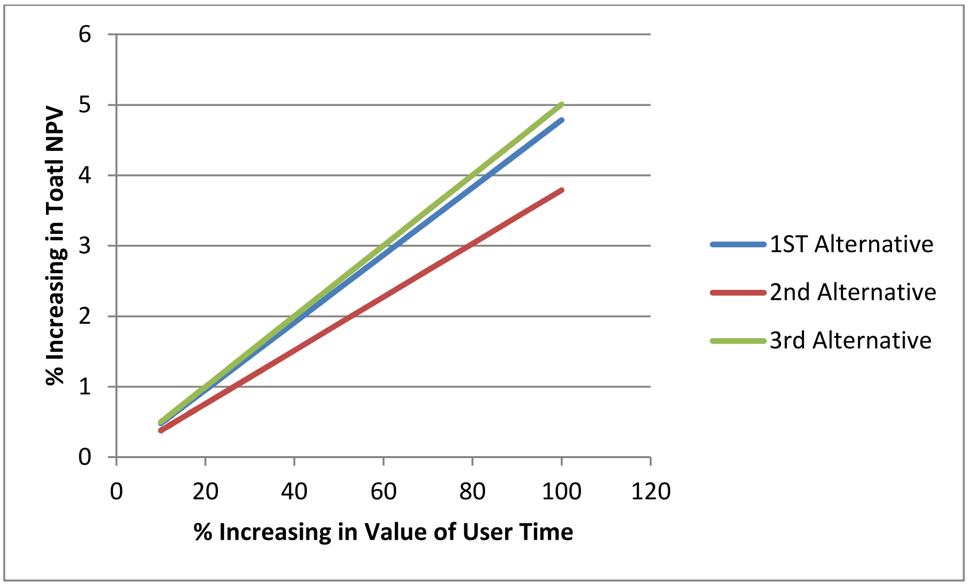

Figure 5. 9 : The Percentage Increasing in Value of User Time with Percentage Increasing in Total NPV

- Develop Expenditure Stream Diagrams:

The expenditure will be different between the alternatives and the section 5.5.1 and section 5.5.3 illustrate these different costs.

- Compute Net Present Value:

The NPV illustrated in section 4.5 and the discount rate in section 4.4.3 and the value of discount rate in our case is (4%). After the 10% as value of D.R, the influencing on the NPV will be equal for the alternatives which have the closed initial cost (1st alternative and 3rd alternative), Figure (5.10). The increasing or decreasing in D.R effected on the NPV or User cost, therefore; the different values of D.R used to show the influencing of this factor on NPV and user cost, Figure (5.11) and Table (5.20) illustrate these values.

- Analyze results.

The analysis results will explain in chapter six.

- Re-evaluate design strategy.

Re-evaluate design strategy depends on opinion of decision-making after present the outcomes.

5.5.1 Cost of Alternatives

The cost of construction (materials, equipment and teamwork) according to the Iraqi governmental contracts with the private and governmental companies, and the technique used in the analysis was the Life Cycle Cost Analysis (LCCA).

In Table (5.18) illustrates that the Iraqi governmental cost for the elements required in comparison in this case study does not indicate the fixed values for this roadway.

The investigation just for 1 KM length from the whole roadway segment and the detailed width are illustrated in Figures (5.1) and (5.2) for both two directions.

In first alternative, there are five layers. Two of them are the surface layers, two base layers, subbase and subgrade. In addition, the other alternatives are the second and third ones that eliminate Bituminous and binder layers, respectively. The costs for these layers are illustrated in Table (5.18).

Table 5.18: Illustrates the Cost of Materials Using in Layers of Pavement

The part of roadway has length 1000 m							
Detail	Unit	Layer's width m	Quantity m^2	Price $ for m^2	Total Price $	Price for 1 cm thickness $	Notes
		(a)	(b) (a x 1000)	(c)	(d) (b x c)	(e) (d/specific thickness)	
Supply and lie the subbase layer at least 20 cm	M^2	21.00	21000	5.00	52,500	2,625	The bed of roadway and shoulder for two directions

Table 5.18 (continued)

Supply and lie the base layer PMB at least 20 cm	M^2	14.00	14000	12.00	168,000	8,400	
Supply and lie the bituminous layer at least 20 cm	M^2	14.00	14000	15.00	210,000	10,500	
Supply and lie the binder layer at least 5 cm	M^2	14.00	14000	10.00	140,000	28,000	
Supply and lie the wearing layer at least 5 cm	M^2	14.00	14000	12.00	168,000	33,600	

5.5.2 Evaluation of Alternatives

This case study does not indicate the fixed costs for each alternative. This means that all comparison costs for layers depend on the layers thickness. Only the cost of the subgrade is neglected; because all alternatives have the same subgrade. Table (5.19) illustrates the thickness and the cost of these layers for each one of alternatives.

Table 5.19 : Illustrates the Thickness of Layer and the Cost of Layers*

Alternative	Activity	Length m	Width m	Thickness cm	Volume m3	Cost of m3 ($)	Total Cost of Layer ($)
1	Wearing course	1000	14	4	560	240	134400
	Binder course	1000	14	5	700	200	140000
	Bituminous base	1000	14	20	2800	75	210000
	PMB course	1000	14	25	3500	60	210000
	Subbase	1000	14	40	5600	25	140000
	Summation						834400

Table 5.19 (continued)

	Activity	Length m	Width m	Thickness cm	Volume m3	Cost of m3 ($)	Total Cost of Layer ($)
2	Wearing course	1000	14	10	1400	240	336000
	Binder course	1000	14	12	1680	200	336000
	Bituminous base	1000	14	0	0	75	0
	PMB course	1000	14	40	5600	60	336000
	Subbase	1000	14	50	7000	25	175000
	Summation						1183000
3	Activity	Length m	Width m	Thickness cm	Volume m3	Cost of m3 ($)	Total Cost of Layer ($)
	Wearing course	1000	14	7	980	240	235200
	Binder course	1000	14	0	0	200	0
	Bituminous base	1000	14	20	2800	75	210000
	PMB course	1000	14	25	3500	60	210000
	Subbase	1000	14	40	5600	25	140000
	Summation						795200

5.5.3 Assumption of LCCA

The initial cost of pavement design is illustrated in Table (5.20), and it has maintenance cost of 15 % of the alternative cost that will be incurred at (5^{th} and 15^{th}) Year. Rehabilitation cost of 30% of the alternative cost that will be incurred at (10^{th}) Year. In addition, the user cost will be calculated at (0 and 10^{th}) Year.

Therefore, the salvage value at 20^{th} year, based on a prorated cost of the year-20 rehabilitation design and remaining life, will be (zero). Figure (5.10) illustrate the Total Net Present Value for alternatives are shown.

Table 5. 20: Illustrate the Initial Cost, Rehabilitation and Maintenance Cost for All Alternatives

No. of Alt.	Cost Component Activity	Years	Costs $	Discount Factor for (Discount Rate=2%)	Discounted Cost $ i=2%	Discount Factor for (Discount Rate=4%)	Discounted Cost $ i=4%	Discount Factor for (Discount Rate=6%)	Discounted Cost $ i=6%	Discount Factor for (Discount Rate=8%)	Discounted Cost $ i=8%	Discount Factor for (Discount Rate=10%)	Discounted Cost $ i=10%	Discount Factor for (Discount Rate=12%)	Discounted Cost $ i=12%	Discount Factor for (Discount Rate=14%)	Discounted Cost $ i=14%	Discount Factor for (Discount Rate=16%)	Discounted Cost $ i=16%
1	Initial Construction	0	834400	1	834400	1	834400	1	834400	1	834400	1	834400	1	834400	1	834400	1	834400
	User Cost	0	66650	1	66650	1	66650	1	66650	1	66650	1	66650	1	66650	1	66650	1	66650
	Maintenance	5	125160	0.9057	113357	0.8219	102869	0.7473	93532	0.6806	85184	0.6209	52891	0.5674	30010	0.5194	15587	0.4761	7421
	Rehabilitation	10	250,320	0.8203	205337	0.6756	169116	0.5584	139779	0.4632	115948	0.3855	96498	0.322	66119	0.2697	45611	0.2267	31688
	User Cost	10	15,680	0.8203	12862	0.6756	10593	0.5584	8756	0.4632	7263	0.3855	6045	0.322	4142	0.2697	2857	0.2267	1985
	Maintenance	15	125160	0.743	92994	0.5553	69501	0.4173	52229	0.3152	39450	0.2394	29963	0.1827	16990	0.1401	9737	0.1079	5636
	Salvage Value	20	0	0.673	0	0.4564	0	0.3118	0	0.2145	0	0.1486	0	0.1037	0	0.0728	0	0.0514	0
Total NPV					**1325601**		**1253130**		**1195346**		**1148896**		**1086447**		**1018310**		**974842**		**947779**
2	Initial Construction	0	1183000	1	1183000	1	1183000	1	1183000	1	1183000	1	1183000	1	1183000	1	1183000	1	1183000
	User Cost	0	66,650	1	66,650	1	66,650	1	66,650	1	66,650	1	66,650	1	66,650	1	66,650	1	66,650
	Maintenance	5	177450	0.9057	160716	0.8219	145846	0.7473	132608	0.6806	120772	0.6209	110179	0.5674	91191	0.5194	75752	0.4761	78,980
	Rehabilitation	10	354,900	0.8203	291124	0.6756	239770	0.5584	198176	0.4632	164390	0.3855	136814	0.322	93742	0.2697	64666	0.2267	44927
	User Cost	10	15680	0.8203	12862	0.6756	10593	0.5584	8756	0.4632	7263	0.3855	6045	0.322	4142	0.2697	2857	0.2267	1985
	Maintenance	15	177450	0.743	131845	0.5553	98538	0.4173	74050	0.3152	55932	0.2394	42482	0.1827	24088	0.1401	13805	0.1079	7990
	Salvage Value	20	0	0.673	0	0.4564	0	0.3118	0	0.2145	0	0.1486	0	0.1037	0	0.0728	0	0.0514	0
Total NPV					**1846199**		**1744398**		**1663240**		**1598007**		**1545169**		**1462812**		**1406731**		**1383531**
3	Initial Construction	0	795200	1	795200	1	795200	1	795200	1	795200	1	795200	1	795200	1	795200	1	795200
	User Cost	0	66650	1	66650	1	66650	1	66650	1	66650	1	66650	1	66650	1	66650	1	66650
	Maintenance	5	119280	0.9057	108032	0.8219	98036	0.7473	89138	0.6806	81182	0.6209	74061	0.5674	61297	0.5194	50920	0.4761	42439
	Rehabilitation	10	238,560	0.8203	195691	0.6756	161171	0.5584	133212	0.4632	110501	0.3855	91965	0.322	63012	0.2697	43468	0.2267	30199
	User Cost	10	15680	0.8203	12862	0.6756	10593	0.5584	8756	0.4632	7263	0.3855	6045	0.322	4142	0.2697	2857	0.2267	1985
	Maintenance	15	119280	0.743	88625	0.5553	66236	0.4173	49776	0.3152	37597	0.2394	28556	0.1827	16192	0.1401	9280	0.1079	5371
	Salvage Value	20	0	0.673	0	0.4564	0	0.3118	0	0.2145	0	0.1486	0	0.1037	0	0.0728	0	0.0514	0
Total NPV					**1267060**		**1197887**		**1142731**		**1098393**		**1062476**		**1006493**		**968375**		**941843**

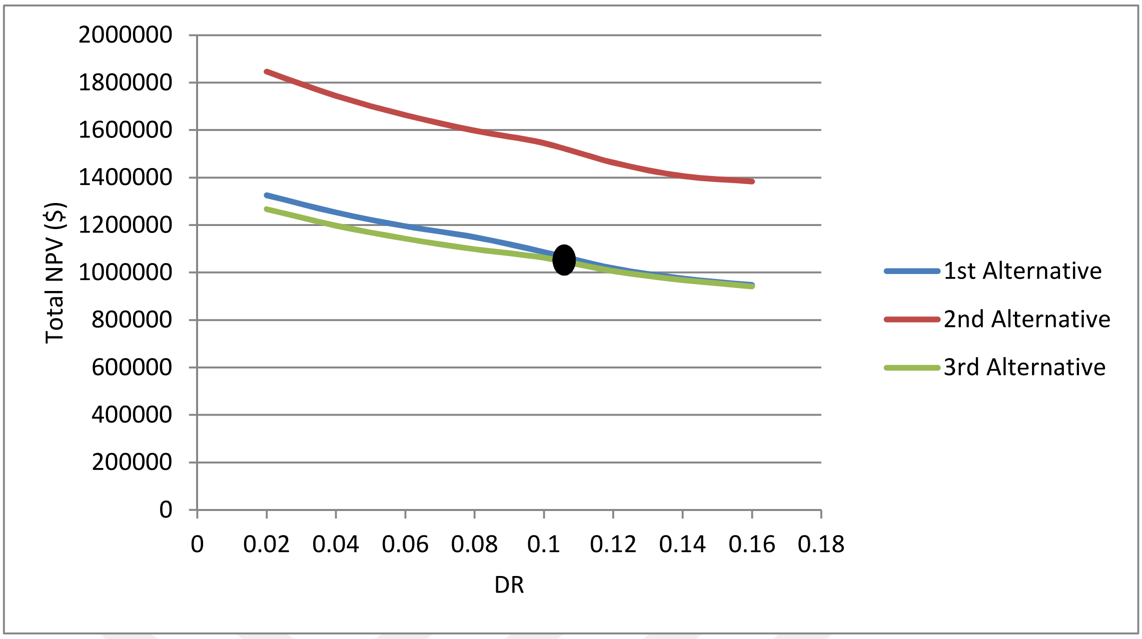

Figure 5. 10 The Relationship Between Different Values of D.R and Total NPV

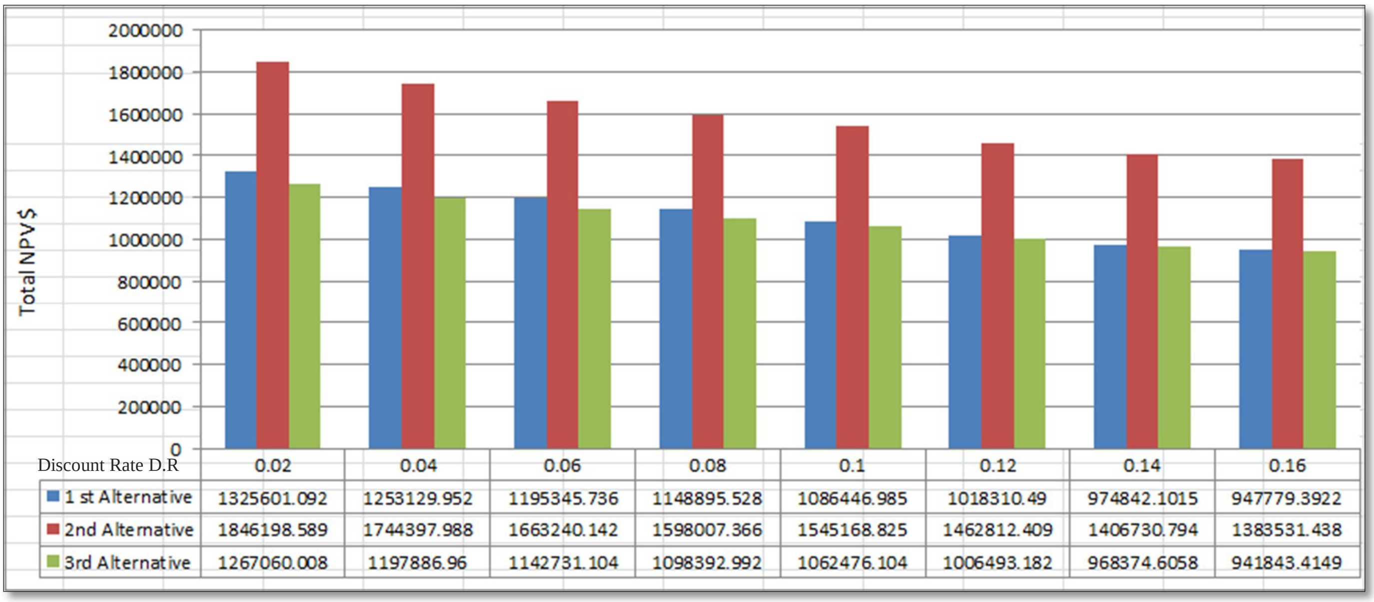

Figure 5. 11 The Chart of Total Net Present Value for Alternatives with Different Values of Discount Rate

CHAPTER 6

RESULTS AND DISCUSSION

Once accomplished, LCCA should at least be subjected to a compassion analysis. Compassion analysis is a process that is used to define the effects of major input assumptions, forecasts, and approximations on LCCA results. In a compassion analysis, the main input values are same. Whereas all other input values stay constant and the amount of change in outcomes is well known. The input variables might be ranked according to their effects on outcomes. Compassion analysis allows the analyst individually become accustomed to the impact of the variability of individual inputs on general LCCA results.

Many times a compassion analysis emphasizes on best case/worst case category in an attempt to support results. Most LCC compassion analyses in minimum assess the effect of the discount rate used on LCCA outcomes.

The previous chapters illustrate the principles and procedures of pavement design, and by using the traffic volume data that are collected from the past years that is for 12 years, Table (5.2). Traffic volume for future years through 2017-2037 (20 years as a design life) are calculated based on growth factors then the alternatives of pavement are designed according to the AASHTO Guide (1993).

The three different alternatives are evaluated in this thesis. Each layer of the pavement is differentiated between them based on the materials. In addition, the thickness also differs between the layers, but the Equivalent Single Axle Load (ESALs) is the same for all alternatives. Some layers are deleted from the alternatives. Such as the second alternative, that does not have the Bituminous Base and the third alternative does not include the

Binder Course. The comparison will be between the different layers, the wearing course, subbase course and subgrade course remaining in all alternatives Table (5.10).

The costs of these alternatives are as well different. This means that there is more than one value to construct this project. A good designer should use his knowledge to find the economical, sustainable and shorter construction period with protecting the engineering side for a capability of establishing this roadway.

In the comparison, the material and the costs are different. Neither the alternative one having a lower cost makes it is the best, nor the expensive one good. However, it should be realized which alternative will have the best cost and the best engineering properties, and that it will be chosen.

The Life Cycle Cost Analysis (LCCA) invented to solve this problem by using some techniques that some of them depends on the past experiments and the engineer's knowledge. Some costs for roadway construction are fixed (i.e. location preparing, road furnishing, etc.).

The alternative (# 3) has the lower agency cost because it has lower initial and user costs while the highest agency cost is the alternative (# 2), and the middle one is the alternative (# 1). The number of rehabilitation is the same (just one); also, the maintenance is two for all alternatives. The design life is 20 years. The alternatives have the same type of pavement (flexible pavement), which leads to keeping the arrangement of the alternatives according to the cost being the same, and the economical one is the third alternative.

The third alternative is the most economical one, although the second alternative having the bituminous base eliminated. This layer is not less than 20 cm of the expensive material (bitumen), which means high construction cost, but the high thickness of other layers influenced on the cost of this alternative. In addition, the engineering should be careful during the implementation of the project. Because, the layers have the same structure need more than one layer for the same course. For example, the PMB and subbase course in (# 2) alternative is (40, 50) cm, respectively and should implement in two layers, which means more precision work.

When the Binder course eliminated in the third alternative, the cost decreased less than the second and first alternative because of the material of Binder course is high-quality bitumen, while the thickness is slightly changed if comparing with the first alternative.

When calculated the total net present value (NPV) by using different discount rates (D.R), produces different values of NPV, not only results differ between each other but also influence on the choosing of alternatives, as mentioned in following.

- When the value of D.R =10% the difference between alternatives (#1 and #3) is so less, that resulted from closing in them initial cost and the equivalence of user cost of these alternatives. Because they have the same work zone duration that means after this value of D.R can choose any one of them, but the alternative (#2) is different (Figure 5.10).
- The user cost also differed when used the different values of D.R, because the user cost is calculated two times (0 year and 10^{th} year) in initial and rehabilitation stages.

The user cost is changed according to the value of user time as shown in Table (5.16), which means every value inputs in the RealCost software will be influence on the outputs with different value. From Figure (5.9), can recognize the increasing in value of user time will be influenced on the (#1 and #3) alternative after two times increasing in these values, but before (50%) increasing it will be so small. In addition, the changed in work zone duration will be influenced on the user cost too.

The alternative (#2) is the highest user cost, but the others have the same user cost resulted from the equivalence in work zone duration.

The user cost did not influence by the initial cost just during rehabilitation and maintenance, user costs can raise dramatically. It is recognizable that road works cause delay and increase the vehicle operating costs (fuel and maintenance of vehicles) as well as the number of traffic accidents are increased, therefore rehabilitation and maintenance should choose carefully to reduce the delay time and in results get the lower user cost.

In our case, the user cost after 10^{th} year will be the same because of the value of D.R is small and the rehabilitation time will be the same, that led to, after 10^{th} year the influencing of D.R and user cost will not be affected on the closed alternatives (#1 and #3).

The first and second modifications in Traffic Hourly Distribution (THD) did not effect on the values of NPV and user costs, because the decreasing and increasing of AADT is regular, that means any decreasing face by increasing and the total AADT still the same, therefore; there is no any important modification, Figures (5.5 and 5.6).

The modification of Inbound - Rural and Outbound - Rural percentage values did not influence on the NPV but influenced on the User Cost; especially when decreased the values of zones (1 and 4), 10%, and increased the values of zones (2 and 3), 10%, also. Because the third zone has a huge influencing on the TDH, that resulted from the big percentage of AADT (rural) any increased in this zone will be influenced on the results, Figures (5.7 and 5.8).

By using the RealCost software, with inputted all the value of user time, work zone duration and the Net Present Value (NPV) calculated from Equation (4.3) with the discount rate (4%), the outcomes will be the suitable alternative is the (# 3) alternative considering the agency and user cost.

The advantage from these alternatives appeared when a comparison was made between the cost of them, the cost advantage when compare the (#2) and (#3) alternative is (546,511 $), (491,268 $) between the (#1) and (#2) alternative and the value becomes minus when compared to the rest of the cases.

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