EFFECTIVESS OF USING GEOTEXTILES IN FLEXIBLE PAVEMENTS: LIFE-CYCLE COST ANALYSIS

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(ABSTRACT)

Using geotextiles in secondary roads to stabilize weak subgrades has been a well accepted practice over the past thirty years. However, from an economical point of view, a complete life cycle cost analysis (LCCA), which includes not only costs to agencies but also costs to users, is urgently needed to assess the benefits of using geotextile in secondary road flexible pavement.

Two design methods were used to quantify the improvements of using geotextiles in pavements. One was developed at Virginia Tech by Al-Qadi in 1997, and the other was developed at Montana State University by Perkins in 2001. In this study, a comprehensive life cycle cost analysis framework was developed and used to quantify the initial and the future cost of 25 representative low volume road design alternatives. A 50 year analysis cycle was used to compute the cost-effectiveness ratio when geotextiled is used for the design methods. The effects of three flexible pavement design parameters were evaluated; and their impact on the results was investigated.

The study concludes that the cost effectiveness ratio from the two design methods shows that the lowest cost-effectiveness ratio using Al-Qadi's design method is 1.7 and the highest is 3.2. The average is 2.6. For Perkins' design method, the lowest value is 1.01 and the highest value is 5.7. The average is 2.1. The study also shows when user costs are considered, the greater Traffic Benefit Ratio (TBR) value may not result in the most effective life-cycle cost. Hence, for an optimum secondary road flexible pavement design with geotextile incorporated in the system, a life cycle cost analysis that includes user cost must be performed.

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CHAPTER 1 INTRODUCTION

The road system of the United States does not exist only for the fast and comfortable movement of interstate or inter-regional traffic. The American road system consists, in large part, of connector and access roads whose principal function is to open the countryside. The traffic volume and traffic loads on these low volume roads are generally limited. Therefore, these roads usually have less strict structural requirements than interstate highways or other principal roads which are designed to guarantee the fast flow of traffic. In fact, heavily built road structures are not required for relatively low traffic volume and loads. It is common practice to lay an unbounded aggregate layer directly on the original subgrade or in a shallow cutting. However, construction such road on a soft subgrade, which can only support relatively small loads, can be a problem. For a soft subgrade, it is often impossible to build a stable base course without losing a large amount of base material into the subgrade. Due to the escalating cost of materials and limited resources, new methods which can stabilize the pavement over the soft subgrade have emerged. Using fabric to enhance pavement stabilization provides such a solution. If it can either reduce the use of the materials or extend the service life by using the same material, this technique would give a feasible solution.

The first use of fabricated material to enhance pavement performance in the US was in the 1920s. The state of South Carolina used a cotton textile to reinforce the underlying materials in a road that had poor quality soils (Beckham et al., 1935). However, because of their susceptibility to degradation, cotton fibers have since been replaced with synthetic polymers, which can better resist harsh conditions. The use of geotextiles for the construction of roads on soft subgrade soil has become popular in the past thirty years, and it has been successfully applied in several cases (Sprague and Cioff, 1993; Tsai et al., 1993). Currently, one of the important roles of geotextiles is as a separator between the granular base layer and the natural subgrade (Al-Qadi et al., 1998).

Life Cycle Cost Analysis (LCCA) is a useful economic tool for the consideration of certain transportation investment decisions. In the 1986 AASHTO *Guide for the Design of Pavement Structures*, the use of LCCA was encouraged and a process lay out to evaluate the cost-effectiveness of alternative designs (AASHTO, 1986). Until the National Highway System (NHS) Designation Act of 1995, which specifically required agencies to conduct LCCA on NHS projects costing $25 million or more, the process was only used routinely by a few agencies (Walls and Smith, 1998). The Federal Highway Administration (FHWA) position on LCCA is defined in its Final Policy Statement published in the September 18, 1996, *Federal Register*. Federal Highway Administration policy indicates that LCCA is a decision support tool. As a result, FHWA encourages the use of LCCA in analyzing all investment decisions. Although the Transportation Equity Act for the 21st Century (TEA-21) has removed the requirement for agencies to conduct LCCA on high cost projects, it is still the intent of FHWA to encourage the use of LCCA for NHS projects. As a result, FHWA has developed a training course titled "Life Cycle Cost Analysis in Pavement Design" (Demo Project 115) to train agencies on the importance of and the use of sound procedures to aid in the selection of alternate designs and rehabilitation strategies (FHWA, 1998).

Since the usefulness of geosynthetic materials in pavements has been recognized (Holtz et al., 1998; AASHTO, 1997), the next question to answer is whether or not this material is cost effective. Therefore, a comprehensive Life Cycle Cost Analysis (LCCA) is needed to quantify the cost-effectiveness of geotextile applications in pavement. Such analysis should include initial construction, rehabilitation, and user costs.

1.1 PROBLEM STATEMENT

Geotextiles have been used in pavements to either extend the service life of the pavement or to reduce the total thickness of the pavement system. The economic benefits of using this material are not well documented. Only initial cost is usually reported. A study considering the LCCA of geosynthetically stabilized pavements, including initial construction, future maintenance, rehabilitation, and user costs, is needed.

1.2 OBJECTIVES

The main objective of this study is to evaluate the cost-effectiveness of using geotextiles (the most used geosynthetic in pavements) at the subgrade-granular material interface. To achieve this objective, the improvement of the pavement system due to the use of geotextiles will be investigated. A simplified geosynthetically-stabilized pavement performance prediction model, a maintenance and rehabilitation schedule, and a proper user cost model will be incorporated into this study. Finally, a sensitive analysis will be conducted to examine the influence of several cost parameters on this study.

1.3 HYPOTHESIS

The roadway considered in this study is a secondary road system. It is hypothesized that geotextiles work as a cost effective separator between the granular base layer and the natural subgrade of the pavement. It is reported that geotextiles improve pavement performance by preventing the intermixture of subgrade fines and base layer. If, in the absence of a geotextile at the subgrade/base course interface, aggregate contamination by the subgrade fines occurs, the overall strength of the pavement system will be weakened. As for cost considerations, vehicles keep a uniform speed through the workzone. The vehicle arrival and discharge rate from queue remains constant. The user delay costs must be represented by a constant average per vehicle hour. In addition, the traffic volume for both directions is available; the maintenance or rehabilitation cost is a linear function of the workzone length. The time required to maintain or rehabilitate a work zone is also a linear function of the workzone length. The combined traffic volume from both lanes should be smaller than the capacity of one lane.

1.4 RESEARCH APPROACH

In order to quantify the cost-effectiveness of using geotextiles in pavement, 25 different low volume road pavement design alternatives were taken into consideration. Four different hot-mix asphalt (HMA) thicknesses (50, 75, 100 and 125 mm), four different base thicknesses (100, 150, 200, and 250mm), and four different subgrade strengths in terms of California Bearing Ratio (CBR) (0.5%, 2%, 4% and 6%) were considered, composing several pavement structures.

Two models have been adopted to quantify the improvement of using geotextiles in the pavements. One was developed at Virginia Tech by Al-Qadi in 1997 and another developed at Montana State University by Perkins in 2001. Based on the work of these two researchers, the traffic benefit ratio (TBR) can be calculated. The TBR is defined as the ratio of the number of load cycles needed to reach the same failure state for a pavement section with geotextiles to the number of cycles to reach the same failure state for a section without geotextiles.

The predicted pavement service life of different pavement structures without geotextiles were determined by using American Association of State Highway and Transportation Officials (AASHTO) pavement performance model and different levels of applied traffic. The service life of the pavement with geotextiles can then be estimated as the cumulative equivalent single axial load (cEASL) value of the pavement without geotextiles at the year in which the rehabilitation work will be applied. This value is then multiplied by the TBR obtained from the previous two models.

A FHWA recommended LCCA process was used to quantify the initial and the future cost of different design alternatives. The costs which were considered in the LCCA process include agency costs and user costs. The agency costs include all costs incurred directly by the agency over the life of the project. These costs include expenditures for initial construction, all future maintenance and rehabilitation activities. The user costs are those incurred by the highway user over the life of the project. They include vehicle operating costs (VOC), user delay costs, and accident costs.

The effects of three flexible pavement design parameters were evaluated; and their impact on the results was investigated.

1.5 THESIS SCOPE

This thesis includes six chapters. Chapter Two gives the present state of knowledge, applications of geosynthetics, and a detailed review of the literature on the improvement of using geotextiles in pavement. In addition, the background of LCCA is discussed and the models currently being used are described. Chapter Three gives a description of how the two selected design methods work and how much improvement could be obtained if the representative design alternatives were selected. Chapter Four discusses the detailed steps of the LCCA including agency and user costs. Chapter Five analyzes the cost results and cost-effectiveness ratios for the representative design alternatives, as well as the sensitivity analyses of the LCCA model. Chapter Six presents the summary of the research findings and conclusions.

CHAPTER 2 PRESENT STATE OF KNOWLEDGE

2.1 GEOSYNTHETICS

The definition of Geosynthetics is a planar product manufactured from polymeric material used with soil, rock, earth or other geotechnical engineering related material as an integral part of a man-made project, structure, or system (ASTM Committee D35 on Geosynthetics).

The main functions of geosynthetics in the pavement industry are separation, reinforcement, filtration, and drainage. The major product uses in this area are geotextiles, geogrids, geosynthetic clay liners, geocomposites, and geonets. The main purpose of using geosynthetic materials is to have better performance and to save money. However, this review, will concentrate on the separation function of geotextile products.

2.1.1 Geotextiles

Textiles were first applied to roadways in the days of the Pharaohs. Even they struggled with unstable soils which rutted or washed away. They found that natural fibers, fabrics, or vegetation improved road quality when mixed with soils, particularly unstable soils.

The first use of textiles in American roadways was in the 1920s. The state of South Carolina used a cotton textile to reinforce the underlying materials in a road with poor quality soils. Evaluation several years later found that the textile was still in good workable condition. When synthetic fibers become more available in the 1960s, textiles were considered more seriously for roadway construction and maintenance.

During the past thirty years, geotextiles have been known to be good for improving the performance of paved or unpaved roads. Both woven and nonwoven geotextiles can be effectively used in the separation/stabilization of primary highway, secondary or low volume roads, unpaved and paved (access roads, forest roads, haul) roads, parking lots, and industrial yards.

Modern geotextiles are usually made form synthetic polymers- polypropylenes (85%), polyysters (12%), polyethylenes (2%), and polymides (1%) which do not decay under biological and chemical processes (Koerner, 1999). This makes them useful in road construction and maintenance.

Basically, the process of making geotextiles can be summarized into three steps. The first is production of polymers from polymeric materials. Then, combinations of the polymers are melted into fibers (or yarns, where a yarn can consist of one or more fibers). The resulting fiber filaments are then hardened or solidified by one of three methods: wet, dry, or melting, to form the different types of fibers. The principle fibers used in the construction of geotextiles are monofilament, multifilament, staple yarn, slit-film monofilament, and slit-film multifilament.

Finally, the fibers or yarns are formed into geotextiles using woven, non-woven or knit methods (although knit fabrics are seldom used as geotextiles) (Koerner, 1999). Woven geotextiles are manufactured using traditional weaving methods and a variety of weave types. Non-woven geotextiles are manufactured by placing and orienting the fabrics on a conveyor belt and subsequently bonding them by needle punching or melt bonding. The needle punching process consists of pushing numerous barbed needles through the fiber web. The fibers are thus mechanically interlocked into a stable configuration. The heat (or melt) bonding process consists of melting and pressurizing the fibers together. Table 2-1 shows the commonly used interlayer geotextile products.

2.1.2 Function of Geotextiles

Geotextiles have four main functions in pavements: separation, drainage, filtration, and reinforcement. However, geotextiles have had over three decades of successful use as stabilizers for very soft and wet subgrade. Based on experience, people found out that when the subgrade condition consists of poor soil, low undrained shear strength, a high water table and high sensitivity, the primary function of geotextiles in stabilizing the subgrade is separation. A brief introduction of each function follows (Koerner, 1994):

Separation: Inserting a flexible porous geosynthetic will keep layers of different sized particles separated from one another.

Drainage: Geosynthetics allow the passage of water either downward through the geosynthetic into the subsoil or laterally within the synthetic material.

Reinforcement: The geosynthetic can actually strengthen the earth or it can increase apparent soil support. For example, when placed on sand, it distributes the load evenly to reduce rutting.

Filtration: The fabric allows water to move through the soil while restricting the movement of soil particles.

Table 2-1 Subgrade-Gradular Interface Products (Elseifi, 2003)

Interface Type	Picture	Applications
Woven Geotextile		**Separation Filtration**
Non-Woven Geotextile		**Moisture Barrier Filtration Separation Stress Relief**

2.1.3 Separation Function

The separation function is now recognized as the main function of geotextiles in pavements, especially when they are used to enhance the road with low bearing capacity subgrade (Al-Qadi, 2002 and Al-Qadi et al., 1994). The main idea is to place a thin layer between two dissimilar materials to prevent the intermixing of the two materials. In this way, each material can fully perform their original function. In this circumstance, there are basically two mechanisms happening with the geotextile as a separator in the wet, soft, weak subgrade road. One is preventing the intrusion of subgrade soil up into the base course aggregate. The other is that the base course aggregate tends to penetrate into the subgrade soil, which affects the strength of the base layer (Al-Qadi et al., 1994; Austin and Coleman, 1993; Barksdale et al., 1989; Bell et al., 1982; Christopher and Holtz, 1991;

FHWA, 1989; Nishida and Nishigata, 1994; Yoder and Witczak, 1975; Koerner and Koerner, 1994; Van Santvoort, 1994).

The action of subgrade soil intruding up to the base course can also be called pumping. Two situations can cause pumping. One is that when dynamic wheel loading is applied, the base course will tend to spread the load. However, if there is loading over the limited capacity which the base course can absorb, the space between grains will increase and the soft subgrade material can have an opportunity to intrude into the base course material. Another recognized conditions which, pumping enhance are the following (Alobaidi and Hoare, 1996): Subgrade soil with a high percentage of fines, subbase layers which lack fine particles (medium to fine sand), free water at the subbase/ subgrade interface, and cyclic loading. When dynamic wheel loading is applied, high pore water pressure is induced. This will cause hydraulic gradient between the subgrade soil and base layer. Therefore, placing a proper geosynthetic layer at the subgrade base interface can reduce the upward plastic flow of subgrade soil because the geosynthetic layer can help to dissipate the excess pore water pressure (Christopher and Holtz, 1991; Alobaidi and Hoare, 1996).

However, in 1996, research by Alobaidi and Hoare also indicated that although the geosynthetic layer can help dissipate the excess pore water pressure, there are some limitations for geosynthetic selection. They found that although higher permeability geosyntextiles can dissipate pore pressure faster, this will also cause the erosion of the subgrade surface and upward movement of the eroded material. Thicker geosynthetic material can reduce the critical hydraulic gradient at the boundary of the contact area between a subbase particle and subgrade soil and therefore can reduce pumping. Also, compressible geotextiles cause higher cyclic pore pressure and therefore cause higher pumping.

In addition, the base course particles penetrate into the subgrade soil. This results in the reduction of base course thickness. When aggregate mixes with subgrade soil; the base aggregate loses its original strength. Figure 2-1 illistrates stone base intrusion and its prevention.

When using geotextile as a separator, there are some requirements which must be followed: (Koerner, 1999, Van Santvoort, 1994).

Burst resistance: The geotextile has to resist the underneath soil entering the upper base layer. Traffic loads cause force which encourages this movement. The traffic loads are transmitted to the stone, through the geotextile, and into the underlying soil. The stressed soil then tries to push the geotextile up.

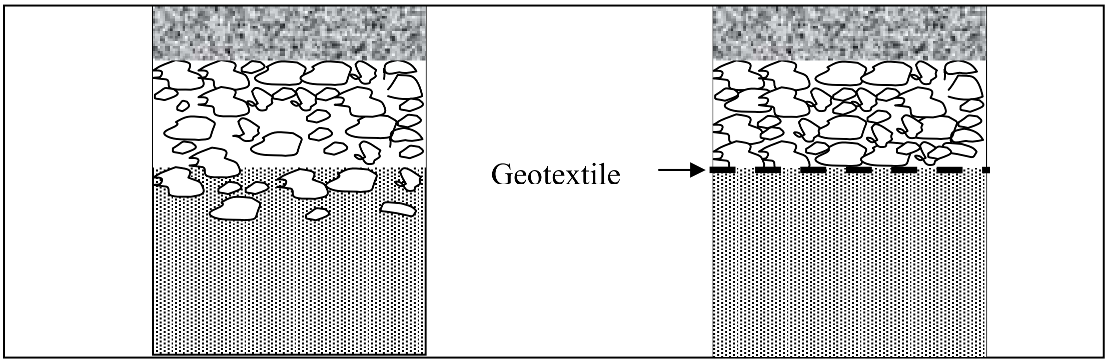

Figure 2-1 Stone Base Intrusion and Prevention in Roadway (after Rankilor, 1981)

Tensile strength: The geotextile must resist lateral or in-plane tensile stress which is mobilized when an upper piece of aggregate is forced between two lower pieces that lies against the fabric.

Puncture resistance: The geotextile must be strong enough to resist puncture. During use, sharp stones, tree stumps, roots, and base course load could puncture through the geotextile.

Impact resistance: The geotextile must resist many impacts of various objects. Impacts usually come from free falling objects such as falling rock, construction equipment, or materials.

Water permeability: If the water level rises, it should not be possible for water pressure to build up under a separation layer to such an extent that the structure stability is endangered. Water in the base layer should drain out through the geotextile. If not, the base may become unstable. For that reason, the water permeability of the geotextile should be greater, or at least equal to, that of the subgrade.

Nishida and Nishigata (1994) found the boundary between the separation and the reinforcement functions. There is a distinct relationship between the opening size of woven geotextiles and total weight of soil mass through a geotextile. However, for nonwoven geotextiles, the relationship was unclear. Excess pore water pressure in each case increased as the number of loadings increased. Separation was found to be the primary function when $/c_u < 8$ and reinforcement was the primary function when $/c_u > 8$, where is the vertical stress on the top of the subgrade and c_u is the undrain shear strength of the subgrade.

Christopher and Holtz (1985) suggest the functions of geotextiles for the corresponding subgrade strengths as presented in Table 2-2:

Table 2-2 Functions of Geotextiles (after Christopher and Holtz, 1985)

Un-drain Shear Strength (KPa)	**Subgrade CBR (%)**	**Functions**
60-90	2-3	Filtration and possible separation
30-60	1-2	Filtration, separation, and possible reinforcement
<30	<1	All functions, including reinforcement

2.2 IMPROVEMENTS DUE TO GEOTEXTILES

In the past few decades, many researchers have attemptted to quantify the improvements of using geotextiles in pavements. Two tests were most commonly used to quantify the improvements in geosynthetically-stabilized pavement: (i) Falling Weight Deflometer (FWD), and (ii) surface rutting of the pavement. The FWD is a nondestructive testing device used to measure the structural capacity of the pavement. Through surface-deflection measurements at different distances from the loading point of the pavement, one can backcalculate the resilient moduli and the thicknesses of the different layers of the tested pavement surface. If the presence of geosynthetics affects (directly or indirectly) the pavement layer moduli, that can be detected. The failure criterion of rutting according to recommendations by the Asphalt Institute pavement design method is 12.7 mm (0.5 in); however, for low volume roads, 0.8 in (20mm) rutting may be considered the failure criterion (Asphalt Institute, 1991). After testing, two major indexes that express the benefits of geotextiles can be defined: (i) traffic benefit ratio (TBR), and (ii) base course reduction ratio (BCR). The TBR is the ratio of the number of load cycles needed to reach the same failure state for a pavement section with geosynthetics to the number of cycles to reach the same failure state for a section without geosynthetics. The BCR ratio is the ratio of the percent reduction in base thickness for a pavement section with geosynthetics to a pavement section without geosynthetics, but with same base pavement properties (Geosynthetic Manufacturer Association, 2000).

Significant research has been conducted on the effectiveness of using geotextiles to improve the performance of flexible pavements. Some of the studies were performed in laboratories, while other studies were performed in the field.

2.2.1 Laboratory and Field Studies

Le (1982), in his Ph.D. thesis, describes the use of various geotextile fabrics in the construction of streets in New Orleans. The fabrics were placed between the subgrade and the base, between the base and the hot-mix asphalt (HMA) layer or between the HMA layers and overlays. All the applications pertained to soft clay subgrades. Strain gages within the subgrade, moisture sensors and falling weight deflectometer (FWD) measurements were used to monitor the performance of constructed roadways. FWD results from his studies showed an increase in pavement stiffness in sections where geotextiles were placed as a separator. His conclusion was that the inclusion of fabric provided an increase in pavement strength by improving load distribution and acting as a separating membrane. No analytical modeling was performed to predict the observed behavior.

Tsai et al. (1993) staged a full scale field test to compare the ability of different types and weights of geotextiles to stabilize a soft subgrade during construction and to investigate their respective influences on the long-term performance of the pavement system. About a 32 km test section located on Washington State Highway in Bucoda was constructed. The subgrade of the test section consisted primarily of clayed soils, with some organic materials in the northbound lane. The soil sample collected from subgrade had more than 80% passing a No. 200 sieve. Test sections with five different separator geotextiles (nonwoven heat-bonded polypropylene, nonwoven needle-punched with different mass/area, and woven slit-film polypropylene), plus a control section were installed in each lane of the roadway. The length of the each test section was 7.62m. Instrumentation included soil strain gages for measuring vertical strain, a grid of rivet point on the geotextile surface to measure geotextile deformation in the wheel path, and moisture/ temperature sensors.

The pavement performance was evaluated by FWD. The short term result showed that the use of a geotextile was in all cases found to eliminate base/subgrade intermixing, though soil migration was found in the control section up to 130mm. The geotextile section presented more uniform rut depth. However, the rut depth could not be reduced by geotextiles if the subgrade had modest shear strength.

Black and Holtz (1999) reported a long-term evaluation result five years after a geotextile installation. The subgrade soils at the test site had consolidated since the geotextile was installed. Density tests suggested that the subgrade in the section containing the geotextile consolidated more than the subgrade in the section without the geotextile. They also stated that the long term performance of geotextiles may not be critical because of the increase in subgrade strength and reduced compressibility due to consolidation.

Sprague et al. (1989) conducted a short-term and long-term field evaluation on using separation geotextiles in a permanent road. A LCCA which included agency costs only was included in the study to examine the cost-effectiveness of using geotextiles in the pavement. A 2.5 km (8100 ft) trial section was built in Greenville County, Virginia. Three pavement cross-sections: 64mm full depth HMA, 38mm HMA over 76mm stone base, and a triple treatment surface course over 75mm stone base were evaluated. Approximately 150m each of three different types of geotextiles, 135 and 203 g/m^2 needle-punched nonwoven geotextile and a 135 g/m^2 silt film woven geotextile, were installed between the subgrade and each pavement section. The remaining length of the road was to act as a control section for the long-term evaluation of each pavement section. Periodically, an independent pavement visual surface inspection program was applied through the sections. A pavement management computer program, Micro Paver, was utilized in this study. The trigger point of the pavement deterioration was set to a PCI of 50 before resurfacing or other rehabilitation work would be required. The result shows that geotextiles provide subgrade/stone base interface stability which increases the life and reduces the maintenance cost of a pavement section. The short-term evaluation result shows that utilizing fabric in low volume pavement design is promising but inconclusive. The long-term evaluation indicates that, in most cases, life cycle costs for pavement incorporating separation geotextiles were lower than the costs associated with the control section which did not utilize geotextiles. An annual cost savings ranging from 5 to 15% can be expected when using an appropriate separation geotextile with low volume paved roads.

Austin and Coleman (1993) conducted a full-scale field study to evaluate the use of polypropylene geogrids and woven/nonwoven geotextiles as reinforcement in aggregate placed over soft subgrades. The initial soil strength of the test area was too high for the research objectives, so the test site area was flooded for nearly eight months. The flooded area was drained, resulting in a CBR value of approximately 1% for the test site subgrade. The subgrade was covered with either a geogrid or a geotextile, and then overlaid with an aggregate layer of varying thickness. Four geogrids and two geotextiles were used as reinforcement in the study. The granular subbase of the test sections was constructed with crushed limestone at depths between 150mm and 260mm. Rod and level measurements were used to monitor the deflection at the surface of the aggregate. The test vehicle used in this study had a front axle loaded to 25.1kN and rear dual axle loaded to approximately 81.4kN. The tire pressure for both axles was 550kPa. A rut depth of 75mm was chosen as the failure criterion.

It was found that geogrids and geotextiles improved the performance of the unsurfaced pavements. For an equal amount of permanent deflection, it was found that 2 to 3 times the numbers of equivalent single axle load (ESAL) repetitions were applied to the reinforced sections compared to the control sections. There was no significant correlation between the geosynthetic tensile strength and the measured performance.

In the unreinforced sections, there was considerable mixing of the aggregate and subgrade layers. Contamination of the aggregate layer by the soft subgrade was evident in the sections not containing a geotextile separator. The base layer had been contaminated by thicknesses ranging from 35mm to 190mm above the original subgrade.

Approximately 4.5 times the number of ESAL repetitions was applied to the composite reinforced section compared to the control section. Approximately three times as many ESAL repetitions were applied to the geotextile sections compared to the control section. The improvement gained from the geogrid reinforcement varied from about 1.5 to 3 times the number of ESAL applications compared to the control section. The separation function of the geotextile is the main reason for the improved performance of the unsurfaced roads.

Perkins (1999) performed an indoor laboratory test to evaluate the performance geosynthetics in flexible pavement. A test box was constructed with inside dimensions of 2x2x1.5m. The bottom of the floor was left open and the front wall was removable in order to facilitate excavation of the test sections. Loading was provided through the application of a cyclic 40kN load applied to a stationary plate resting on the pavement surface. The 20-constructed test sections were instrumented for stress, strain, temperature, moisture, displacement, and load measurements. Two polypropylene biaxial geogrids (one stiffer than the other) and a polypropylene woven geotextile were included in the test. The HMA layer for all the test sections were the same: 75mm. A crushed stone base layer (Montana DOT classification Type 3) with a thickness varying from 200mm to 375mm was also constructed. Two different types of subgrade strengths: one constructed from highly plastic clay with a CBR of 1.5% and another one of mainly silty sand with a CBR of 15%. The test sections contained as many as 90 instruments measuring the various pavement loads, surface deflections, stresses, strains, temperatures, and moisture contents in the various pavement layers.

Significant improvements in pavement performance, as defined by surface rutting, resulted from the inclusion of geosynthetic reinforcement. Substantial improvement was seen when a soft clay subgrade having a CBR of 1.5% was used. For the stronger subgrade (CBR=20%), it appears that little to no improvement occurred from the geosynthetics. For all test sections, mixing of the subgrade and base course aggregate was not observed, indicating that any improvement due to the geosynthetics was due to reinforcement functions. Test sections with the two geogrids performed better than the sections with geotextiles.

Perkins et al. (2000) developed a 3D finite element model of unreinforced and geosynthetic reinforced pavements. The material model for the HMA layer consisted of an elastic-perfectly plastic model. This model allowed for the HMA layer to deform with the underlying base aggregate and subgrade layers as repeated pavement loads were applied. A bounding surface plasticity model was used to predict the accumulated permanent strain of the base and subgrade under repeated loading. A material model including elasticity, plasticity, and creep and direction dependency was used for the geosynthetic. The shear interaction between the base and geosynthetic was molded by the Coulomb friction model, which is an elastic-perfectly plastic model. The finite element models were then modified through comparing them to the laboratory test results obtained by Perkins (1999). The result shows that reinforcement reduced the lateral permanent strain at the bottom of the base, as well as the vertical stress on top of the subgrade. This model is claimed to have the ability to simulate an accumulation of permanent strain and deformation, and can qualitatively show the mechanisms of reinforcement observed from the laboratory test section. However, some deficiencies were found in the base aggregate material model. It was not sensitive to the effects of restraint on the lateral motion of the material. However, the response of the vertical strain on top of the subgrade and the mean stress in the base course layer shows a reasonable result. The amount of vertical deformation after 10 cycles was significantly reduced in the reinforced model compared to the unreinforced case. Perkins (2001) combined the result of the finite element modeling and previous laboratory testing to develop a systematic design model of using geosynthetics in flexible pavement.

In 1992, Al-Qadi et al., (1994) performed experimental and analytical investigations to evaluate the performance of pavement with and without geotextile or geogrid materials. Eighteen pavement sections were tested in three groups: one without geosynthetics which served as control section, one geotextile stabilized section and one geogrid reinforced section. Two types of woven polypropylene geotextiles were installed between the base and subgrade. The ratio of HMA layer/base layer thickness was 70mm/150mm and 70mm/200mm, and there was no subbase layer installed. A 550kPa dynamitic loading by a 300mm rigid plate at a requency of 0.5Hz was applied to the pavement surface. The applied loading was to simulate the dual tire load from an 80kN axle with a tire pressure of 550kPa. A 44.5kN load cell was placed directly on top of the loading plate to monitor the applied loading. The resulting displacement was continuously monitored and recorded by an array if linear variable displacement transducers (LVDT). The subgrade used in the study was a weak silt sand which has the resilient modulus of CBR=2 to 5%. The base layer was dense granite aggregate (VDOT classification 21-A). The test sections were evaluated by studying the effect of loading cycles on deformation. The AASHTO pavement design procedure was used in the study by Al-Qadi et al. (1994) as the basis for converting loads to equivalent single axle loads as part of their service life prediction. The KENLAYYER computer developed by Huang (1993) was used to determine the frequency of the applied load. The predicted equivalent single axle loads (ESALs) to failure were calculated and presented for a speed of 64 and 96 km/hr.

The experimental results showed that the geotextile stabilized sections sustained 1.7 to over 3 times the number of load repetitions of the control sections for 25mm of permanent deformation. The study shows that the geotextile stabilized sections didn't have the intermixed phenomena between the granite aggregate layer and silty sand subgrade layer. This phenomenon proves that the geotextile material was effective in preventing fines migration between the base and subgrade layers. In the section with a CBR value of 4% or less, the geotextile section showed more significant improvement than the control and geogrid-stabilized sections. This result was further validated by the later field experiment. The benefit of geotextile-stabilization is quantified in the increased number of ESAL repetitions to failure and the increase in the service life of the pavement sections.

Al-Qadi et al. (1994) conducted a field evaluation using geosynthetics in a 150m-long low volume traffic secondary road in Bedford County, VA. Nine test sections were constructed in three groups. An average 90-mm-thick HMA layer was constructed in each section, and no subbase layer was installed. Sections one to three had a 100-mm-thick limestone base course, sections four to six had a 150-mm-thick base course, and sections seven to nine had a 200-mm-thick base course. Three geotextile stabilizers and three geogrids were installed at the interface of the base and subgrade layers. The other three sections were kept as control sections. One section from each stabilization category was included in each base course thickness group. All nine sections were instrumented. The subgrade and base layers were instrumented with Kulite earth pressure cells, Carlson earth pressure cells, soil strain gages, thermocouples and moisture sensors. The HMA layer was instrumented with strain gages, thermocouples, and four piezoelectric sensors. Pavement performance was based on the instrumentation response to normal and vehicular loading. Performance was also verified through some other measurements such as rut depth, ground penetration radar (GPR), and FWD.

The results show that the measured stress at the base course subgrade interface for the geosynthetic-stabilized sections was lower than the control sections. The final rut depth of the test was about 17mm. The rut depth measurement in the control sections was more severe than in the geosynthetic-stabilized sections. The FWD result shows that weaker subgrade strength was measured in geogrid-stabilized and control sections. An intermixed layer was found in the control and geogrid-stabilized sections when an excavation was applied in the 100mm thick base section. The final result based on rut depth measurement shows that the geotextile-stabilized section can carry 134% more ESALs before failure than the control section, and that the geogrid-stabilized section can carry 82% more ESALs before failure than the control section.

2.3 LIFE CYCLE COST ANALYSIS

In the National Council of Highway Research Programs (NCHRP) Synthesis of Highway Practice, Peterson (1985) defined LCCA as follows: To evaluate the economics of a paving project, an analysis should be made of potential design alternatives, each capable of providing the required performance. If all other things are equal, the alternative that is the least expensive over time should be selected. According to FHWA recommendations, an analysis period of at least 35 years should be used. The different economic indicators commonly used in the LCCA procedure are present worth (PW), method (of benefits, costs, benefits and costs-NPV), equivalent uniform annual cost (EUAC), internal rate of return (IRR), and the benefit cost ratio (BCR).

Therefore, LCCA can be perceived as an analysis technique used to evaluate the overall long-term economic efficiency of different alternative investment options. This is a decision support tool that helps to choose a cost effective alternative from several competitive alternatives. It includes all current and future costs associated with investment alternatives (NCHRP, 1985). The different steps involved in a LCCA are described as follows (FHWA, 2002; Hass et al., 1993; Hicks and Epps, 2002):

Develop alternative pavement design strategies and relevant activities: This step is to assume the type of pavement and its related rehabilitation and maintenance strategies for the analysis period. Each type of pavement has its own specific service life (Figure 2-2).

Establish the timing of various activities: For different types of pavement, the rehabilitation, maintenance strategies, and service life will be different. Therefore the timing of these activities can be determined. Pavements which receive different rehabilitation (or maintenance) strategies will have different performance curves as shown in Figure 2-3.

Estimate the appropriate costs for agency costs and user costs: For agency costs, the cost of initial construction, the cost of future maintenance and rehabilitation, and the value of salvage return or residual value at the end of the project is needed. Non-agency costs can be described in two parts: One is user costs, which include vehicle operation costs, travel time costs, and accident costs. Another is nonuser costs, which include environmental pollution and neighborhood disruption.

Develop an expenditure stream diagram: The expenditure stream diagram (Figure 2-4) helps to visualize the quantity and timing of expenditures over the life of the analysis period. Three kinds of elements would be presented in the expenditure stream diagram: initial and future activities, agency and user costs related to these activities, and the timing and costs of these activities. The upward arrows on the diagram are expenditures. The horizontal arrow and segments show the timing of work zone activities and the period of time between them. The remaining service life (RSL) value (salvage value) is presented as a downward arrow and reflects a negative cost at the end of the analysis period.

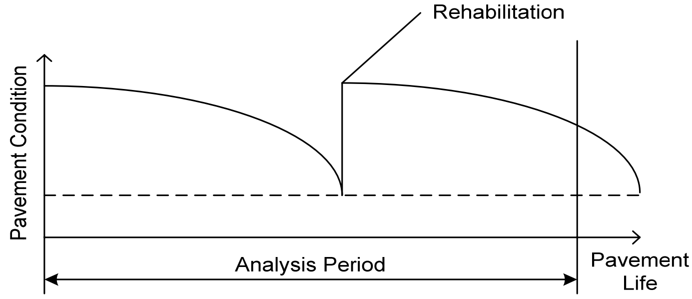

Figure 2-2 Analysis Period for a Pavement Design Alternative

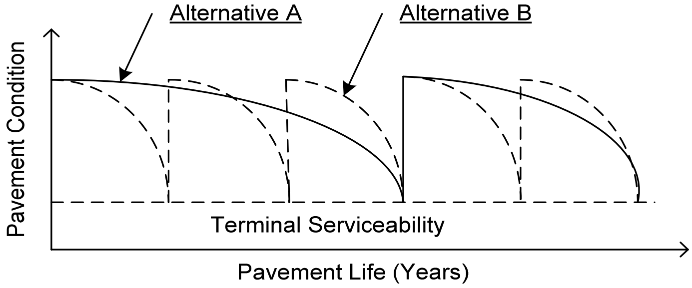

Figure 2-3 Performance Curve for Different Rehabilitation or Maintenance Strategies

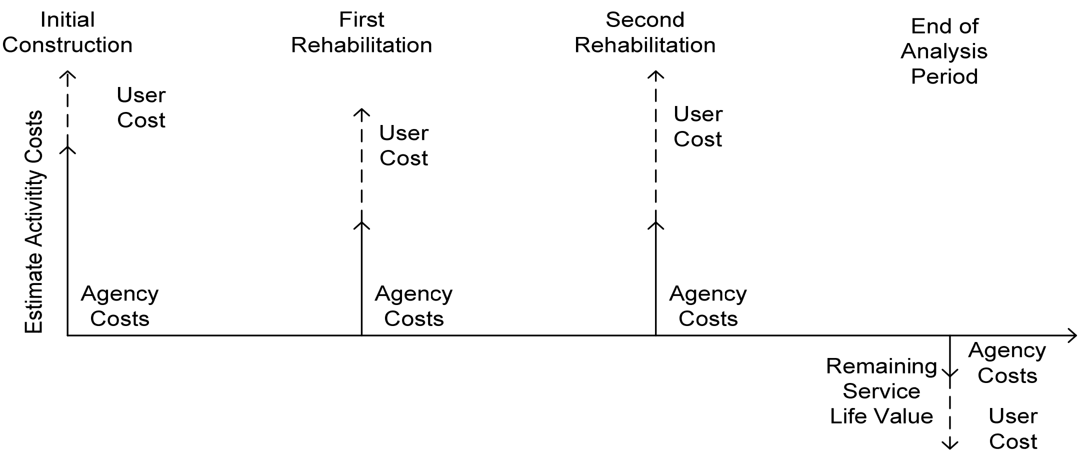

Figure 2-4 Expenditure Diagram

Select an economic analysis method: The value of the money spent at different times over the LCCA period must be converted to a value at a common point in time. The reason is that the value of the amount of money today is higher than the same amount money at a later date. The techniques usually used to implement this concept are: 1) the present worth method; 2) the equivalent uniform annual cost method.

Analyze the result using either a deterministic or probabilistic approach. The differences between these two are that the deterministic approach gives each LCCA input variable a fixed discrete value. On the contrary, the probabilistic approach takes the input parameter as a sampling distribution of all the possible values.

Analyze the result: The final step of the analysis is to compare the sum of the agency and user costs among the alternatives. The output obtained with the deterministic approach, does not show the full range of possible outcomes. To improve this, the most commonly used technique is to apply a sensitivity analysis. This analysis can help show where the result may be depending on the given range. The probabilistic approach method attempts to model and report on the full range of possible outcomes and also show the predicted outcomes.

2.3.1 Pavement Performance Prediction Model

According to past research (Mahoney, 1990; Lytton, 1987; Alsherri and George, 1988; Butt et al., 1994; Davis and Van Dine, 1988; Hutchinson et al., 1994; Lee, 1982, Zhang et al., 1993), pavement performance prediction models can be generally categorized into two groups: deterministic (mechanistic) models and probabilistic models.

The deterministic or so called mechanistic type of prediction model is based on some primary responses of the pavement such stress, strain, and deflection. Alternatively, it can be based on some structural or functional deterioration indexes such as distress or roughness of the pavement, or by regression techniques to find the relationship between the dependent variables such as measured structural or functional deterioration and independent variables such as subgrade strength, axle load applications, pavement layer thickness and properties, and environmental factors (Haas, 1993).

A well known example of the mechanistic type of performance prediction model was developed by Queiroz, 1983. Sixty three flexible pavement test sections were evaluated to develop a model to consider the compressive strain at the top of the subgrade related to roughness prediction (Eq. 2-1) or vertical compressive strain at the bottom of the HMA surface related to crack development prediction (Eq. 2-2).

$$Log(QI) = 1.426 + 0.01117AGE - 0.1501RH + 0.001671VS_3N * Log(N) \quad (2\text{-}1)$$

where,

QI = roughness (quarter-car index);

AGE = number of years since construction or overlay;

RH = state of rehabilitation indicator (0 for as constructed and 1 for overlay);

VS_3 = vertical compressive strain at top of the subgrade; and

N = cumulative equivalent single axle loads (ESAL).

$$CR = -8.7 + 0.258HST1 * Log(N) + 1.006 * 10^{-7} * HST1 * N \quad (2\text{-}2)$$

where,

CR = percent of pavement area crack (%);

HST1 = horizontal tensile stress at the surface bottom (kgf/cm^2); and

N = cumulative equivalent single axle loads (ESAL).

Another way to build an empirical prediction model is based on a large long-term monitoring database to form a regression model (Eq 2-3). A good example of this is the equation developed by the AASHTO road test for flexible pavement. The data on traffic, subgrade strength, and present service index (PSI) data was correlated to form a deterioration regression model as follows:

$$\Delta PSI = (p_t - p_i)10^{\left\{(0.4+\frac{1094}{(SN+1)^{5.19}})\left[\log W_{18} - Z_R S_0 - 9.361\log(SN+1) + 0.20 - 2.321\log M_R + 8.07\right]\right\}} \quad (2\text{-}3)$$

where,

PSI = changes of present serviceability index of the pavement;

P_t = terminal percent serviceability index;

P_i = initial present serviceability index;

W_{18} = allowable ESAL;

Z_R = normal standard deviation;

S_0 = standard deviation;

SN = structural number of the pavement; and

M_R = resilient modulus of the subgrade soil.

However, the deterministic prediction model is not suitable for all kinds of pavement management (Salem et al., 2003) because of the following: Pavement behavior is uncertain during environmental and traffic changes, difficulties with the factors or parameters which affect pavement deterioration, and error and bias when doing the measurements or surveys of pavement conditions.

Another type of the performance prediction model is the probabilistic method. One of the most important challenges of this model is to build transition probability matrixes (TPMs). Building TPMs can be based on the Markov process. In the Markov process, the future state of the model element is estimated only from the current state of the model element. Therefore, defining those elements in the matrixes is important work. The common way to do this is to use an experienced engineer as a base to define each element. A study by Karan (1976) used the Markov process model for the maintenance of a pavement covering at Waterloo, Ontario. The pavement performance deterioration versus age was modeled as time independent with a constant TPM. The element of TPM is based on the average of the results of a survey from expert engineers. The disadvantage of using this approach is that TPMs need to be developed for each combination of factors that affect pavement performance. Another disadvantage is that pavement historical data is difficult to include in the Markov model because the future state of pavement is only based on its current state (Hass et al., 1993).

2.3.2 **Agency Cost**

Agency cost typically consists of fees for initial preliminary engineering, contract administration, construction supervision, construction, maintenance and rehabilitation, and costs associated with administrative needs (FHWA, 1998). When considering agency costs, routine annual maintenance costs and sink value are usually neglected. Because sink value does not affect decision making and routine maintenance costs when discounted to the present value, the cost differences will have a negligible effect on the net present value (NPV).

The American Association of State Highway and Transportation Officials (AASHTO) define maintenance as, "A program to preserve and repair a system of roadways with its elements to its designed or accepted configuration" (AASHTO 1987). The purpose of maintenance is described as, "…to offset the effects of weather, vegetation growth, deterioration, traffic wear, damage, and vandalism. Deterioration would include effects of aging, material failures, and design and construction faults" (AASHTO 1987).

Maintenance work can be categorized in to two major types: routine maintenance and preventive maintenance (Haas, 1994). Routine maintenance is the day-to-day maintenance activities that are scheduled by the highway agency in charge and do not improve the pavement structural capacity. Examples of routine maintenance activities include patching potholes, roadside maintenance, drainage maintenance, localized spry patching, localized distortion repair, and minor crack sealing. According to the definition noted by the Minnesota DOT, "Preventive maintenance is performed to improve or extend the functional life of a pavement. It is a strategy of surface treatments and operations intended to retard progressive failures and reduce need for routine maintenance and service activities." For example, activities considered to be preventive maintenance in Minnesota include crack sealing, chip sealing, fog sealing, rut filling, thin overlay, and some emergency techniques such as micro-surfacing and ultra-thin overlay (Johnson, 2000).

According to the NCHRP, C1-38, pavement rehabilitation is defined as, "A structural or functional enhancement of a pavement which produces a substantial extension in service life, by substantially improving pavement condition and ride quality." The concept of rehabilitation can be described by four different tasks commonly referred to as "The Four Rs" – restoration, resurfacing, recycling, and reconstruction. Each of the four types of rehabilitation treatments are defined below (Hall, 2001).

Restoration is a set of one or more activities that repair existing distress and significantly increase the serviceability (and therefore, the remaining service life) of the pavement, without substantially increasing the structural capacity of the pavement.

Resurfacing may be a structural or functional overlay. A structural overlay significantly extends the remaining service life by increasing the structural capacity and serviceability of the pavement. A functional overlay significantly extends the service life by correcting functional deficiencies that may not increase the structural capacity of the pavement.

Recycling is the process of removing pavement materials for reuse in resurfacing or reconstructing a pavement (or constructing some other pavement).

Reconstruction is the removal and replacement of all HMA layers, and often the base and subbase layers, in combination with remediation of the subgrade and drainage. Due to its high cost, reconstruction is rarely done solely on the basis of pavement condition.

2.3.3 User Cost

According to FHWA Life Cycle Cost Analysis in Pavement Design (1998), user cost is usually classified into two categories: (i) normal operation user cost, and (ii) work zone user cost. The normal operation user cost reflects highway user costs related to using a facility during free construction, maintenance, and/or rehabilitation activities that restrict the capacity of the facility. This type of user cost is a function of different pavement performance (roughness) of the facility. Another type of user cost is referred to as work zone user cost, which is associated with using a facility during construction, maintenance, and/or rehabilitation activities that restrict the capacity of the facility and disrupt the normal traffic flow.

In the normal operation user cost, the crash cost and delay cost should have minimal differences between the alternatives. The major difference would be the vehicle operation cost. The theory behind having different vehicle operation costs consists of two major parts: (i) an accurate estimate of pavement performance curves, and (ii) the ability to quantify the differences in the vehicle operation cost (VOC) rate for small differences in pavement performance at relatively high performance levels.

In the United States, the National Highway System (NHS) and State Highway Agencies typically consider pavements that have a present service index (PSI) of 2.0 to have reached their terminal serviceability index, and subsequently plan some form of rehabilitation. Therefore, the VOC in normal operational conditions will not be insensitive in such condition. In addition, a low volume road was consid in this study. Because of above two reasons, the normal operation user cost will not be included in the study. The work zone user cost is usually composed of three items: (i) travel time, (ii) vehicle operation, and (iii) accident costs. Road-user costs related to work zones may be calculated by methods presented by the Transportation Research Board (TRB) and AASHTO (TRB, 2000; AASHTO, 1987).

2.3.3.1 User Delay Cost

Generally, travel time costs vary by vehicle class, trip type (urban or interurban) and trip purpose (business or personal). The work zone user delay costs may be significantly different for different rehabilitation alternatives, depending on traffic control plans associated with the alternatives. Therefore, the work zone delay costs should take into consideration not only average daily traffic volumes but also daily and hourly variations in traffic volume. The NCHRP Project 7-12, "Microcomputer Evaluation of Highway User Benefits," recommends a typical traffic volume distribution value for urban or rural roadways. Table 2-3 shows the user cost consideration in different research studies for economic evaluations of pavement alternatives.

2.3.3.2 Vehicle Operation Cost (VOC)

The first VOC model in the United States was developed by the AASHTO Red Book in 1978. It was then replaced by the Texas Research and Development Foundation (TRDF) VOC model. The TRDF data was incorporated into the MicroBENCOST model by the Texas A&M Research Foundation. Another famous system is the work done by the World Bank with the Highway Design and Maintenance Standard Model (HDM-4). The main difference between the HDM-4 and MicroBENCOST is that MicroBENCOST does not have performance prediction models. It simply uses the Pavement Serviceability Rating (PSR) as direct input for each year of the program. The HDM-4 has performance prediction models and predicts IRI based on pavement condition. In the FHWA "Life Cycle Cost in Pavement Design" technical bulletin, vehicle operation costs were considered only associated with vehicle speed changes because of work zones. In this study, only work zone VOC was considered.

Table 2-3 User Cost Components Considered in Several Studies

Study	User Cost Item Considered
HDM-4	VOC, Time delay, Accident cost
Road and transportation in Canada	VOC, Time delay cost,
Schonfeld and Chien	Time delay cost, Accident cost
AASHTO (1977)	Travel time, VOC

2.3.3.3 Accident/Crash Costs

While accidents are rare events in terms of total vehicular utilization of highway infrastructure in the United States, they nonetheless create unfortunate consequences. These consequences may be grouped into two categories: First, those relating to the accident site and the incident (covering personal, vehicular and property damage); and second, the external costs driven by the impacts that the incident has on the rest of the traffic flow. An important type of accident study examines accidents associated with elements of infrastructure. The consequences of these events are described in terms of personal injury and property damage.

Accident rates have traditionally been determined by studying records, generally those created by the police and by the insurance industry. The problems associated with collecting relevant factors about an accident on a form designed by these sources have, however, frustrated efforts to understand and model accident causes. There is little empirical evidence to support consistent relationships between accident rates or severity and infrastructure characteristics that are relevant to work zone configurations.

Much of the literature associated with accidents has been related to political needs as well as to federal and state highway planning of a very general nature. For example, the decision to reduce highway speeds in the 1970s was promoted both on fuel efficiency grounds and on accident reduction grounds. Also, it can be seen that improvements in highway design, such as those related to guardrails and collapsible devices, have reduced the severity of many accidents. Finally, accident rates are used to justify enforcement, particularly the expenditures related to policing the highways both to enforce speed standards and to control driver behavior impaired by drugs and intoxication.

Modeling accidents, in general, can be difficult because of the complexity of factors affecting an incident and the difficulties in obtaining data to develop statistical relationships. Moreover, the problems extend into the valuation of accidents, particularly those related to injury and fatalities. The substantial debate over the figures used by planning authorities to justify accident reductions suggests that any modeling of accidents that extend into the area of cost-benefit analysis must be accompanied by sensitivity testing to permit variations in the valuations to be tested. The review of accident cost in this study focuses on accident costs due to work zones.

2.3.3.3.1 Accidents in Work Zones

Researchers have shown that drivers have different perceptions of, and behaviors towards, work zone safety, in spite of such zones being inherently more dangerous than open lanes. An extensive literature search by Ha and Nemeth (Ha et. al, 1995) presented data on the increase in accidents in several project sites in ten states where work zones were present. Table 2-4 shows their findings, including the site where the project was performed and the change in the accident rate when a work zone was present.

The results shown in the table vary widely, from a 7% to a 119% increase. Part of the variability in the results of these studies is due to the rarity of accidents, and especially to the rarity of accidents in work zones. As part of the above research, a more detailed project was conducted in Ohio. Of accidents occurring in Ohio from 1982 to 1986, 1.7 percent was attributed to work zones.

Table 2-4 Accidents Experienced during Construction Periods

Project	Project Site	% Change in Accident Rate
California	California	+21.4 to +7.0
Virginia	Virginia	+119
Georgia	Georgia	+61.3
Midwest Research Institute	Colorado Minnesota Ohio New York Washington	+6.8
Ohio	Ohio	+7
New Mexico	New Mexico	+33.0 (Rural Interstate) +17.0 (Federal-Aid Primary) +23.0 (Federal-Aid Secondary)

In Table 2-5 (Nemeth and Rathi, 1983), the number of accidents and their respective percentages for the Ohio Turnpike, a 240-mile freeway, are reported. The Turnpike in general seems to be less dangerous than the interstate system as a whole, judging by fatality rates. In 1978 there were 0.6 fatalities per 100 million vehicle miles traveled. On the entire U.S Interstate system, the rate for the same year was 1.9 per 100 million vehicle miles traveled.

Table 2-6 (Garber and Woo, 1990) gives the percent increase in accident rates depending on the length of the work zone and the duration of the construction project that caused the work zone to be needed.

Accident rates decrease according to project distance until about 1km. At that point, accident rates slowly rise again and, at a distance of 3.2km, reach a level equal to what was seen at a distance of about 0.3km. Within each set of construction duration times, the accident rate increases with the project duration until about 350 days, and then decreases. In long work zones, the percent increase in the accident rate during a 500-day construction project is less than that of a 100-day project.

Table 2-5 Accident Types: Construction and All Turnpike Accidents (after Nemeth and Rathi, 1983)

Type of Accident	Construction Accidents		All Accidents	
	Number	Percent	Number	Percent
Rear End	42	22.7	696	20.29
Hitting Objects	97	52.43	1,297	37.82
Side Swipe	18	9.73	451	13.16
Non Collision				
and other	28	15.14	985	28.73
Total	185	100	3,249	100

Table 2-6 Percentage Difference in Accident Rates

Work Zone Lengths (miles)	Duration (Days)								
	100	150	200	350	300	350	400	450	500
0.3	50.96	71.89	79.26	80.7	79.11	75.82	71.53	66.64	61.39
0.6	32.54	52.28	58.81	59.6	54.47	53.73	49.05	43.82	38.25
1.0	30.52	49.56	55.59	56	53.56	49.56	44.65	39.21	33.47
1.3	32.99	51.54	57.23	57.36	54.71	50.51	45.44	39.86	33.99
1.6	37.15	55.32	60.73	60.66	57.83	53.49	48.29	42.6	36.63
1.9	42.01	59.86	65.05	64.81	61.84	57.38	52.08	46.3	44.31
2.2	47.13	64.71	69.72	69.33	66.24	61.68	56.29	50.44	44.31
2.6	52.31	69.68	74.52	74	70.81	66.17	60.7	54.78	48.6
2.9	57.48	74.63	79.33	78.71	75.42	70.7	65.18	59.19	52.95
3.2	62.56	79.53	84.1	83.38	80.01	75.22	69.64	63.6	57.31

(Source: Garber and Woo, 1990)

2.3.3.3.2 Accident Cost Model

Several different methods of estimating accident cost were reported in the literature. In a FHWA technical report titled "Life Cycle Cost Analysis in Pavement Design," the accident/crash cost related to work zones was generally higher than that on the roadway without work zones. Crash rates are based on the number of crashes as a function of exposure, typically vehicle miles of travel (VMT). Crash rates are commonly specified as crashes per 100 million vehicle miles of travel (100 M VMT). Accident/crash costs can be calculated by multiplying the unit cost per crash, by the differential crash rates between work zones and nonwork zones, by the vehicle miles of travel during the duration of the work zone. Schonfeld and Chien (1999) presented an accident cost model related to the moving delays and queue delays of work zone activity.

2.3.4 Economic Analysis Method

The purpose of doing economic analysis is based on the concept that the value of money changes over time. Expenditures at different times over the life of a pavement will not be equal. Therefore, to convert the present and future costs to common method of cost assessment, people usually use some economic techniques such as the present worth (PW) method, the equivalent uniform annual cost (EUAC) method, the rate of return method, and the benefit-cost ratio (BCR) method.

2.3.4.1 Present Worth (PW) Method

The present worth method converts all present and future costs to a common baseline. This baseline is typically set at the time of the first expenditure or in the year of construction. To determine the present worth, it simply adds the present worth of each component related to the selected alternative over the analysis period. The PW of each component is calculated by multiplying the cost of the task by the present worth factor. People can separately calculate the present worth of costs and benefits or the net present value (difference between the present values ("worths") of benefits and costs). The present worth factor for discounting either benefits or costs to their present value is expressed as follows (Zimmerman, 2000; Haas, 1993):

$$pwf_{i,n} = \frac{1}{(1+i)^n} \tag{2-4}$$

where,

$pwf_{i,n}$ = present worth factor for particular i and n;

i= discount rate; and

n= number of years to when the sum will be expended, or saved.

2.3.4.2 Net Present Value

Net present value is slightly different from the present worth method. For a project to be justifiable on economic grounds, the benefits should be greater than the costs. The net present value can be expressed as:

$$NPV_1 = TPWB_1, n - TPWC_1, n \tag{2-5}$$

where,

NPV_1 = net present value of alternative x1;

$TPWB_{1,n}$ =total present worth of benefits of alternative x1; and

$TPWC_{1,n}$ = total present worth of costs of alternative x1.

2.3.4.3 Equivalent Uniform Annual Cost (EUAC) Method

The equivalent uniform annual cost (EUAC) method converts all initial investments and future costs into equal annual payments over the analysis period. The result will present the amount that would have to be invested each year over the analysis period. The equation form of the method can be represented as:

$$AC_{X_1,n} = crf_{i,n}(ICC)_{X_1} + (AAMO)_{X_1} + (AAUC)_{X_1} - sff_{i,n}(SV)_{X_{1,n}} \tag{2-6}$$

where,

$AC_{x1,n}$= equivalent uniform annual cost for alternative x1, for a service life or analysis period of n years;

$Sff_{i,n}$ =sinking fund factor for interest rate i and n years;

$(ICC)_{x1}$= initial capital cost of construction (including actual construction costs, maintenance costs, engineering costs, etc.);

$(AAMO)_{x1}$=average annual maintenance plus operation costs for alternative x1;

$(AAUC)_{x1}$=average annual user cost for alternative x1 (including vehicle operation, travel time, accidents and discomfort if designated); and

$(SV)_{x1,\,n}$=salvage value if any, for alternative x1 at the end of n years.

2.3.4.4 Benefit Cost Ratio (BCR) Method

This method is usually used to select among projects when funding is restricted. The mathematical expression of the BCR can be written as follows:

$$BCRxi, n= (TPWBxi)/ (TPWCxj) \tag{2-7}$$

where,

$BCR_{xj,\,n}$ =benefit-cost ratio of alternative x_j over an analysis period of n years;

$TPWB_{xj}$ =total present worth of benefits respectively for alternative x_j; and

$TPWC_{xj}$ = total present worth of costs respectively for alternative $x_{j.}$

It shows that the higher the BCR, the greater the ratio cost benefit ratio of the project. According to FHWA recommendations, the denominator considers the initial construction cost. All other parameters will be considered as a positive or negative benefit in the numerator of the equation.

2.3.4.5 Cost-Effectiveness Method

Cost-effectiveness is a systematic quantitative method for comparing the costs of alternative means of achieving the same stream of benefits or a given objective. An alternative is cost-effective if, on the basis of a life cycle cost analysis of competing alternatives, it is determined to have the lowest costs expressed in present value terms for a given amount of benefits. Cost-effectiveness analysis is appropriate whenever it is unnecessary or impractical to consider the dollar value of the benefits provided by the alternatives under consideration. This is the case whenever each alternative has the same annual benefits expressed in monetary terms, or each alternative has the same annual effects, but dollar values cannot be assigned to their benefits. Table 2-7 shows the advantages and disadvantages of the above economic analysis methods.

Table 2-7 Advantages and Disadvantages of Economic Analysis Methods

Analysis Method	Advantages	Disadvantages
Present worth	Simple and easy to understand	Benefits and costs might not be distinguished clearly enough
Net present value	The benefits and costs for a project are related and expressed by a single value. Projects having different service lives can be directly compared using this method. The cost and benefits are expressed in present day terms. The method is computationally simple.	When there is only a single alternative the benefits cannot be calculated and NPV cannot be applied in this case. The results in terms of a lump sum may not be understandable to some people as a rate of return or an annual cost method.
Equivalent uniform annual cost	Simple and easy to understand	If vehicle operating costs are among the alternatives then this assumption becomes questionable.
Benefit cost ratio	High public appeal due to the emphasis on benefits	Benefit-cost ratio is abstract and can be difficult to comprehend
Cost-effectiveness	Good to use when there is no monetary benefit from the output	Comparisons need to have the same effectiveness criteria

CHAPTER 3 BENEFIT QUANTIFICATION OF USING GEOTEXTILES IN PAVEMENT

3.1 BENEFIT QUANTIFICATION MODEL

Significant research has been conducted regarding the effectiveness of geosynthetics in improving flexible pavements. Some of the studies were performed in laboratories, while others were conducted in the field. However, little work has been done to quantify the cost-effectiveness of using geosynthetics in pavements. Two of the most widely used methods for designing secondary road pavements with geosynthetics are described below.

3.1.1 Al-Qadi's Method for Geosynthetically Stabilized Road Design

In 1992, Al-Qadi et al. began a series of laboratory and field experiments to evaluate the benefits of using geotextiles and geogrids to stabilize secondary roads. In the lab, secondary road pavements were simulated through 18 test sections with different subgrade strengths and granular base layer thicknesses. These sections were tested using pneumatic loading. The study concluded that significant improvement in pavement service life was achieved when geosynthetics were used. The effectiveness was quantified using the number of loading cycles needed for pavement with and without geotextile incorporation to reach a 19mm rutting failure after converting the applied loading to the equivalent single-axle load (ESAL).

This study shows that geotextiles are most effective when used on top of a weak subgrade. A design method was proposed to predict improvement in pavement service life resulting from the use of geotextiles as shown in Figure 3-1 (Al-Qadi and Bhutta, 1999). Using this model, one may determine the design number of ESALs in accordance with the AASHTO pavement design guide. The design number of ESALs is then used as the without-geotextile y-axis value. Corresponding higher with-geotextile ESAL values can be obtained from the x-axis. This means that if a geotextile is incorporated into the pavement design, the pavement would reach the same failure at the newly obtained ESAL (x-axis). Conversely, if the AASHTO design number of ESALs is used as the with-geotextile value, the corresponding without-geotextile value represents the lower number of ESALs (y-axis). This lower number of ESALs would allow a reduction in required pavement structure capacity to achieve the same service life (Al-Qadi et al., 1997). This model was validated in a field study using geosynthetics in an instrumented section with different pavement designs (Al-Qadi et al., 1998).

3.1.2 Perkins' Method for Geosynthetically-Reinforced Flexible Pavements

An analytically based method for determining reinforcement benefits was proposed by Perkins (2001). A design method used to calculate reinforcement benefits in terms of pavement structure thickness, subgrade strength, and properties related to the geosynthetic was also proposed.

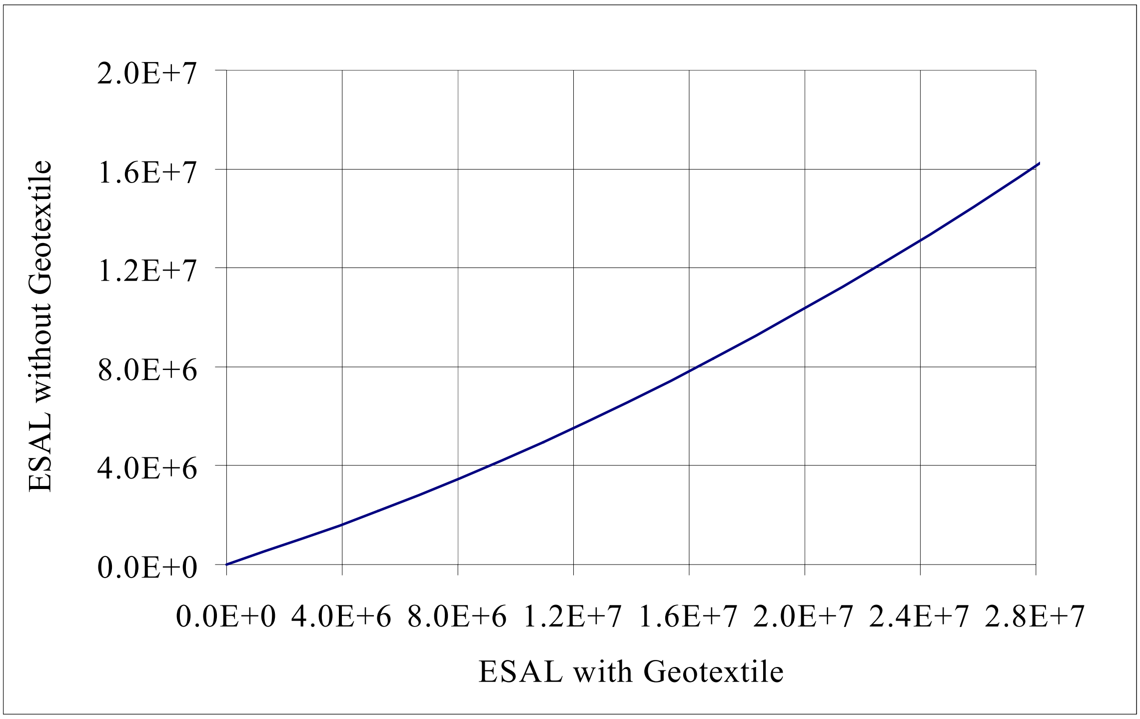

Figure 3-1 Effect of Using Geotextile on Cummulative ESAL (after Al-Qadi et al., 1998)

The design equation was based on 465 cases analyzed using both a finite element response model and empirical damage models. The finite element model involved elastic-plastic material models for the HMA, base aggregate, and subgrade layers, and an anisotropic linear elastic model for the geosynthetics. The empirical distress models were developed to relate the stress and strain responses obtained from the finite element model to pavement performance and, ultimately, to the reinforcement benefit. The distress model was calibrated from previous work by Perkins (Perkins, 1999a), which observed permanent surface deformation (rutting) caused by permanent vertical strain in the base and subgrade layers. The design model in Perkins' study specifically addressed the pavement distress feature of rutting. Reinforcement benefits were obtained by comparing the ESAL needed to reach the 12.5mm rutting depth between the reinforced and un-reinforced cases. Design equations were developed for reinforcement benefits in terms of traffic benefit ratio (TBR), base course reduction ratio (BCRR), or a combination of the two. The woven geotextile's strength at 2% strain on cross machine direction is 12.7kN/m. The secant tensile modulus at 2% axial strain is 637kN/m. And the ratio of minimum to maximum 2% secant tensile modulus is 0.641.

It has to be noted that the objective is not to compare the two aforementioned methods. The development of the first method was based on stabilization, while the second was based on reinforcement. Although both of the aforementioned methods clearly showed that the incorporation of geosynthetics in pavements, especially with weak subgrade strength, is beneficial and would increase the pavement service life, detailed cost-effectiveness and life cycle cost analyses are needed.

3.2 TRAFFIC BENEFIT RATIO (TBR)

Therefore, according to the aforementioned performance prediction model, the benefits in terms of traffic benefit ratio (TBR) obtained from the two design models (Al-Qadi's and Perkins') are presented. Figure 3-2 shows the benefits in terms of TBR versus allowable ESAL value when Al-Qadi's model is used. The TBR increases rapidly when the pavements have lower allowable ESALs, and the increase rate decreases as the allowable ESAL increases beyond 200,000.

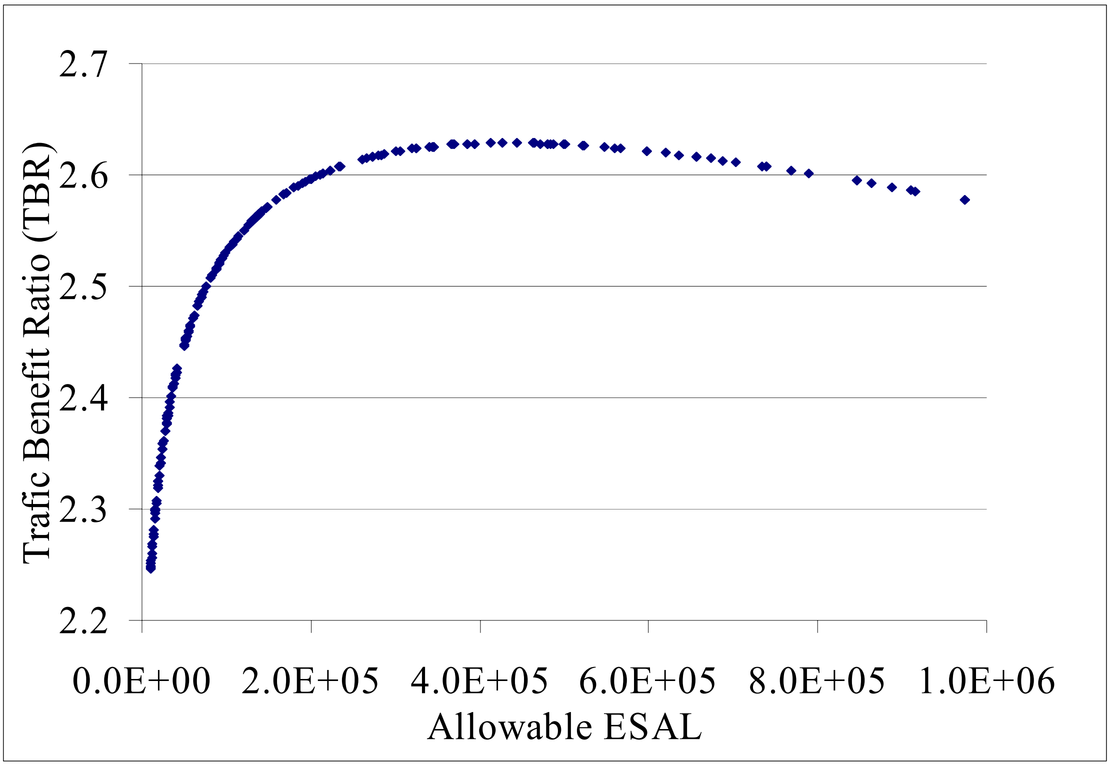

Figure 3-2 Traffic Benefit Ratio Versus Allowable ESALs Using Al-Qadi's Method

In the model presented by Perkins (Figure 3-3), the benefits from the base aggregate layer and the subgrade layer are considered individually. The model shows that the benefits decrease as the pavement becomes stronger. The model also shows significant improvement at low CBRs and in weak pavement. However, the TBR value approaches unity when the CBR is greater than two and when the pavement structural capacity increases. This result fits the assumption of that study.

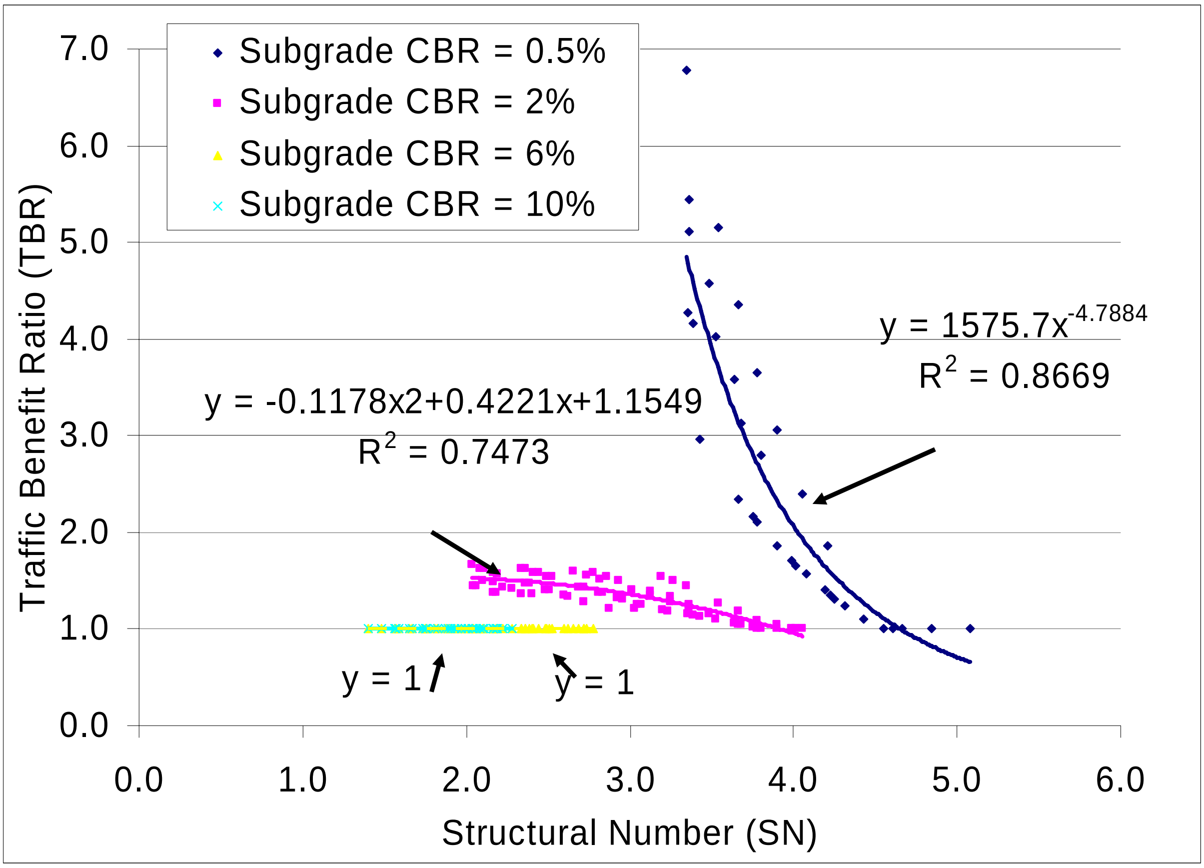

Figure 3-3 Traffic Benefit Ratio Versus Structural Number (SN) for Different CBR Values Using Perkins' Method

CHAPTER 4 LIFE CYCLE COST ANALYSIS

4.1 PAVEMENT STRUCTURE CONSIDERATIONS

Life Cycle Cost Analysis can be used to determine the relationship between performance and cost when geotextiles are incorporated in pavements. The AASHTO 1993 Pavement Design Guidelines were used in this study. Pavement reliability is considered as 70%, and the standard deviation is considered as 0.49 (secondary road). Table 4-1 shows the matrix of possible secondary road pavement design combinations based on four different HMA thickness (50, 75, 100, and 125mm), four different granular base thicknesses (100, 150, 200, and 250 mm), and four different subgrade strengths (CBR=0.5, 2, 6 and 8%). The design layer coefficient was considered as 0.44 for the HMA layer and the drainage coefficient as 1.0. Using a combination of the aforementioned pavement composition and characteristics, there are 64 design combinations; however, only a fraction of these combinations are considered to be realistic and somewhat representative of secondary road traffic conditions. According to the Virginia DOT traffic count data in 2004, the annual average daily traffic (AADT) of a secondary road varies from several hundred to several thousand. Therefore, 25 design combinations based on the traffic features of the secondary roads in the state of Virginia were selected on which to conduct the cost-effectiveness analysis comparison in this study. The 25 representative designs are designated 1 through 25 in Table 4-1.

Table 4-1 Pavement Structures Considered

HMA Thickness (mm)	Base Thickness (mm)	Subgrade Strength (*CBR %)			
		0.5	2	4	6
50	100				1
	150			2	3
	200		4	5	6
	250		7	8	9
75	100			10	11
	150		12	13[!]	14
	200		15	16	
	250		17		
100	100		18	19	
	150		20		
	200		21		
	250		22		
125	100				
	150		23		
	200	24			
	250	25			

*California Bearing Ratio ;!(ref) represents the reference design

4.2 PAVEMENT PERFORMANCE PREDICTION

The evaluation of pavement performance is a crucial step in the life cycle cost framework. The ability to predict the remaining life or the distress levels of a pavement section allows engineers, planners, and highway agencies to plan ahead for maintenance and rehabilitation activities, budget for future expenditures, and makes decisions about the timing of those rehabilitation activities. With ample time to plan, state transportation agencies can minimize their costs as well as minimize the impact of their construction activities on the traveling public and others affected by such construction.

Therefore, the first step in the life cycle cost framework is to evaluate a pavement design and the conditions under which it is expected to operate throughout its design life or its analysis period. The framework presented in Figure 4-1 shows the steps required to prepare an analysis for the life cycle cost procedure. The general inputs relating to the project as a whole, independent of pavement type must be defined prior to idetifying pavement design alternatives. These inputs include such conditions as predicted traffic patterns, pavement loading, and economic variables. Once the general and specific conditions are defined, the life cycle cost framework simulates the predicted traffic loading and environmental conditions for each year of the analysis period. At the end of each year, the performance models predict the level of distress or damage to the pavement based on that year's current traffic conditions.

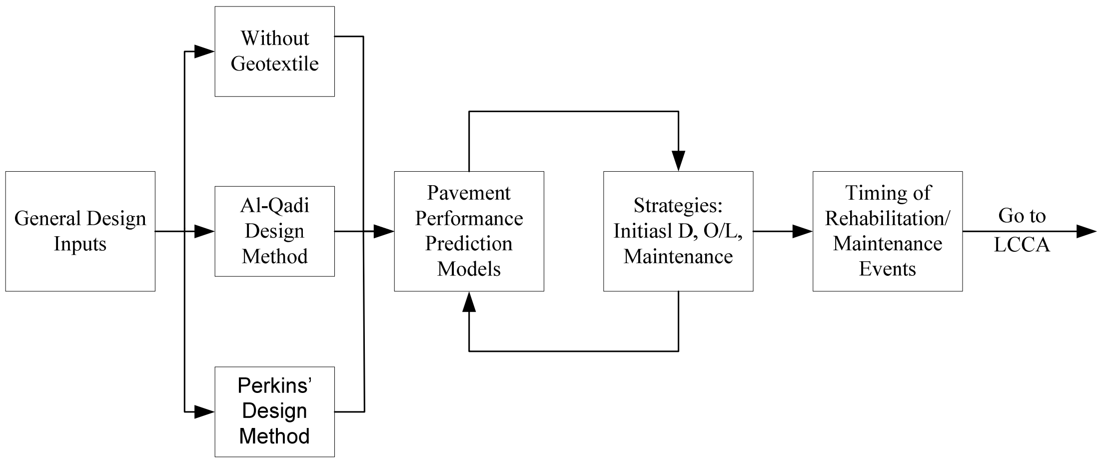

Figure 4-1 Life Cycle Cost Framework- Pavement Performance

4.3 PAVEMENT LOADING

Often, vehicular loading of the pavement is the parameter that has the greatest effect on the performance of pavements. Although other factors, such as environmental conditions, affect the performance of pavements, they only help to modify and calibrate performance models to local conditions. The effects of vehicular loading, however, are universal and affect all pavements in any locale. This section will address the method by which the amount of vehicular loading is determined and predicted for the entire analysis period.

In the method that will be discussed, the engineer obtains, or predicts, the equivalent single axle loads (ESALs) for the first year and an estimated annual growth rate. Another method uses the average daily traffic (ADT) for the first year, predicts the

ADT for the final year of the analysis period, the percentage of trucks throughout the analysis period, and then designs an ESAL value for the entire analysis period.

In order to determine the appropriate ESAL value for each year, the traffic evaluation module begins with the initial year ESAL and increases this value annually by the growth rate. This is represented by equation 4-1 below, which shows the calculation for the current year's ESAL value:

$$ESAL_{current} = ESAL_{initial} \bullet (1 + g)^{i} \tag{4-1}$$

where:

g = annual ESAL growth rate, and

i = current year, between 0 and analysis period

In the study, the user cost model not only requires the current annual ESAL value, it also requires the cumulative value to predict the level of serviceability. The algorithm used to determine the annual cumulative values is as described below. Given the first and last year ADT values, an annual growth rate can be derived by the following formula:

$$ADT_{final} = ADT_{initial} \bullet (1 + g)^{n} \tag{4-2}$$

where:

g = annual growth rate, and

n = analysis period.

Then, solving for g,

$$g = \left(\frac{ADT_{final}}{ADT_{initial}} \right)^{1/n} - 1 \tag{4-3}$$

The annual cumulative ESAL value, then, is calculated by deriving the first yearESAL value from the growth rate and the total ESALs:

$$ESAL_{cumulative} = ESAL_{initial} \bullet \frac{(1+g)^n - 1}{g} \quad (4\text{-}4)$$

$$ESAL_{cumulative} = ESAL_{initial} \bullet \frac{g}{(1+g)^n - 1} \quad (4\text{-}5)$$

From this point, the cumulative ESAL values for each year are determined by:

$$ESAL_{annual,\ cumulative} = ESAL_{initial} \bullet \frac{(1+g)^i - 1}{g} \quad (4\text{-}6)$$

where,

i = current year.

4.4 AASHTO PAVEMENT SERVICEABILITY PREDICTION MODEL

The AASHTO design guide equation for flexible pavements, which is the major model in use today for predicting the pavement serviceability ratings, is used in predicting the remaining serviceable life of a pavement in the study.

The equation is used to determine the design thickness of a flexible pavement, or the allowable loads for a specific thickness. This equation can also be used to determine the decrease in PSI for given inputs and traffic loading. The AASHTO design equation is shown below:

$$\log W_{80} = Z_R S_0 + 9.36 \log(SN+1) - 0.2 + \frac{[\log(\Delta PSI)/(4.2-2.5)]}{0.4 + 1094/(SN+1)^{5.19}} + 2.32 \log M_R - 8.07 \quad (4\text{-}7)$$

where,

W_{80}= number of 80kN equivalent single axle load applications estimated for a selected design period and design lane;

R = reliability;

Z_R= the normal deviate for a given reliability R;

S_0= standard deviation;

ΔPSI= Present Serviceability Index difference between initial value (P_i) and the terminal value (P_t);

SN= design structure number indicative of total required pavement layer thickness and their corresponding moduli; and

M_R= subgrade resilient modulus.

Each year the current level of traffic is updated in the AASHTO equation and the equation is solved for the PSI value, which provides an estimate of the structural condition of the pavement. This equation can be used with a known or predicted value of ESALs to predict the PSI of a pavement, given other design parameters that will be readily available to the pavement design engineer. Using the PSI prediction, rehabilitation requirements will be evaluated. The AASHTO model is used for consistency, since it is the same model that will be used for pavement thickness design. NCHRP Report 277 (Darter et. al, 1985) suggested that it was an effective approach as pavement thickness affects the rate of loss of pavement serviceability. Figure 4-2 shows a typical PSI curve with respect to time or traffic. This example shows a major rehabilitation toward the end of the predicted service life, and no action was taken after that.

Figure 4-2 Typical Time or Traffic Versus PSI Curve with One Rehabilitation

4.5 PAVEMENT SERVICE LIFE PREDICTION OF GEOTEXTILE INCORPORATED PAVEMENT

The pavement service life for the design alternative which incorporates geotextiles needs to be quantified. The method to account for the service life benefit due to the utilization of geotextiles in pavement is presented as follows. The AASHTO pavement design equation 4-7 can be rewritten into the following form:

$$\Delta PSI = 2.7 \times 10\wedge\left[(0.4+\frac{1094}{(SN+1)^{5.19}})\left[\log W_{80} - Z_R S_0 - 9.36\log(SN+1) + 0.20 - 2.32\log M_R + 8.07\right]\right] \quad (4\text{-}8)$$

Changing Present Serviceability Index (ΔPSI) is altered with increases in the applied ESAL: as the applied ESAL increases, ΔPSI increases. The terminal PSI value (P_t) is equal to initial PSI (P_i) value minus ΔPSI. Therefore, P_t decreases as applied ESAL increases. When the P_t reaches 2.0, a major rehabilitation may need to be applied. Hence, the service life of the pavement can be determined. The applied cumulative ESAL up to the year of rehabilitation is cESAL. Using the calculated cumulative ESAL (cESAL) and the TBR, corresponding to the model used, the allowable ESAL for the pavement incorporating geotextiles can be determined:

$$ESAL_G = cESAL \times TBR \tag{4-9}$$

Utilizing $ESAL_G$ and ΔPSI at rehabilitation in equation 4-1 will give SN_G (structure number for pavement with geotextiles). The latter can be used to calculate the ΔPSI_G. ΔPSI_G represents the change in PSI when geotextiles are used between P_i and the corresponding P_t to the ESAL equivalent to that of P_t without geotextiles.

An example of predicted service life of a pavement among these three design alternatives is shown in Figure 4-3. In this example, the pavement has an HMA layer of 100mm, a base layer of 375mm, and a subgrade of CBR 0.5%. For this low CBR, the TBR value is 2.45 (Al-Qadi's method) and 1.86 (Perkins' method), respectively.

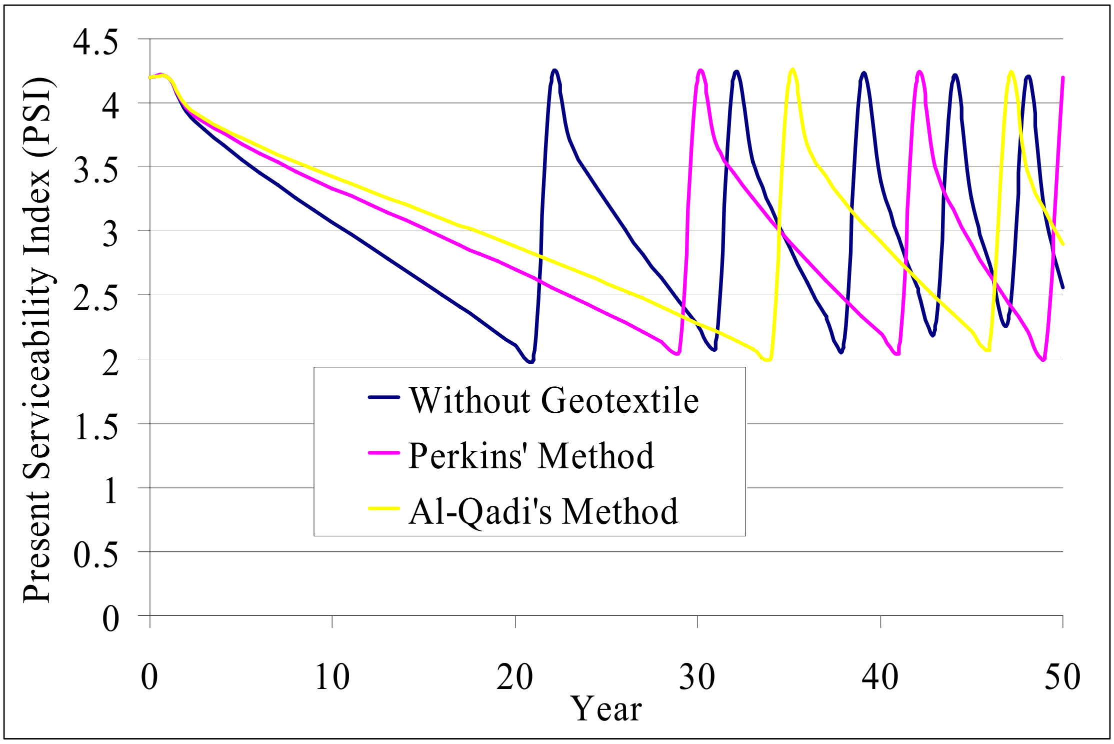

Figure 4-3 Service Life Comparison between Three Alternative Flexible Pavement Design Approaches

4.6 OVERLAY THICKNESS

Rehabilitation work (usually an overlay) is assumed to bring the pavement back to better condition when the P_t value of the pavement reaches 2.5. The rehabilitation work considered in this study is to apply an overlay. The thickness of the overlay is based on the recommendation of overlay design in the AASHTO 1993 Pavement Design Guide (AASHTO, 1993). To determine the effective structure number (SN_{eff}), the condition of the pavement is needed. A remaining service life (R_L) at the time of rehabilitation can be obtained by one minus the ratio of the cumulative applied ESAL (N_i) to allowable ESAL (N_T):

$$R_L = 1 - \left(\frac{N_i}{N_T} \right) \qquad (4\text{-}10)$$

Substituting the remaining service life into Eq (6), the condition factor (C_F) can be obtained.

$$C_F = 1 - 0.7 \times e^{-(R_L + 0.85)^2} \qquad (4\text{-}11)$$

The SN_{eff} can then be determined.

$$\mathrm{SN}_{\mathrm{eff}} = \mathrm{C}_{\mathrm{F}} \times \mathrm{SN}_{0} \qquad (4\text{-}12)$$

where,

R_L = remaining service life;

N_p = cumulative applied ESAL at the year rehabilitation work is applied;

N_T = allowable ESAL for a specific pavement cases;

C_F = condition factor;

SN_{eff} =effective structure number; and

SN_0 = original design structure number.

This assumes that after the rehabilitation work has been done, the PSI and SN will return to their original values. Prior to the overlay application, a 50mm cold milling is assumed. The layer coefficient of the existing HMA layer is considered to be 0.33. Therefore, an overlay thickness can be calculated as follows:

$$d = \frac{SN_0 - (SN_{eff} - 0.33 * 2)}{0.44} \tag{4-13}$$

where,

d = overlay thickness at year of rehabilitation work is applied;

SN_{eff} = effective structure number at rehabilitation year;

SN_0 = structure number at initial year;

0.33 = layer coefficient of existing HMA layer; and

0.44 = layer coefficient of new HMA.

4.7 MAINTENANCE AND REHABILITATION STRATEGIES

The pavement is evaluated at the end of each year by the performance models and the predicted distress levels are evaluated by the strategies module. This is shown by the flowchart in Figure 4-4. This module takes the distress levels evaluated in the pavement performance module as inputs and determines appropriate rehabilitation timing.

4.8 AGENCY COSTS

The cost of highway construction is very well understood and tracked, both by construction companies and by departments of transportation. Contractors compile cost data in order to bid on new highway construction projects, and highway departments compile the same types of data in order to make proper project estimates and to control those costs once a project has begun.

In this study, the life cycle costs for a design alternative are calculated based on the cost of the materials for the entire pavement structure. This would be similar to the

Engineer's Estimate of construction costs. The costs were considered for mainline paving only; each rehabilitation section is assumed to have a dimension of 7.3m (24ft) wide and 1.6km (1mi) long. The input value used in the study for the initial construction and rehabilitation costs is listed in Table 4-2.

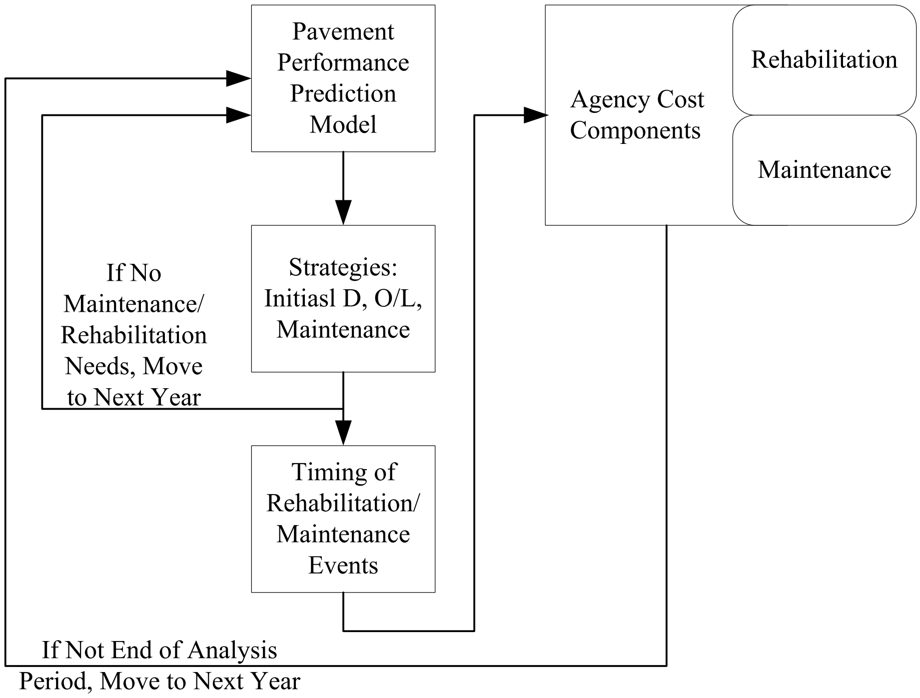

Figure 4-4 Performance Prediction and Maintenance/ Rehabilitation Framework

Table 4-2 Material Unit Price per Agency Cost

Item Name	Cost ($)
HMA Surface Course	36/ ton
HMA Intermediate Course	30/ ton
HMA Milling Cost	1.5/ m^2 (for 50-mm-thick)
Base Aggregate Cost	6.15/ ton
Geotextile Cost	0.40/ m^2

4.8.1 Pavement Construction Cost Items Considered

This section contains a list of the general categories that should be considered for initial construction, rehabilitation, and annual maintenance costs. All of the items in this list are viable options for construction and rehabilitation activities and should be considered as agency costs in the analysis of life cycle costs for a highway pavement project.

- *Initial Construction*

Subgrade

Clear and Grub
Scarify and Recompact
Subgrade Compaction
Borrow Material

Base

Aggregate
Aggregate Treatments
Base Compaction

Drainage Layer

Coarse Graded Aggregate
Filter Material

Hot-Mix Asphalt Layer

Surface HMA layer
Intermediate HMA layer

- *Rehabilitation*

HMA Overlays

- *Routine Maintenance*

Pothole Repair
Localized Spray Patching
Minor Crack Sealing

All the items in the table can be tabulated and included in the total cost of a pavement structure. The initial construction items can be assigned quantities, while unit costs can be provided for the other aspects of maintenance and rehabilitation activities. For each alternative design condition, the quantities of the maintenance and rehabilitation items based on the results of the performance models will be estimated. All these costs are then converted into a single cost per kilometer of pavement structure for easy comparison.

4.9 USER COSTS AND WORK ZONE EFFECTS

When construction maintenance activities are undertaken on highway sections that continue to allow traffic on the facility, a system of traffic controls and protective barriers are instituted to ensure worker and traffic safety. Traffic management in work zones is influenced by the type of infrastructure, environment, traffic characteristics, duration, type of work, and available sight distance. Work zone configurations are trade-offs, balancing contractor efficiency against traffic speeds and safety. When vehicle flows are light, impacts on speed and safety may be slight. As demand increases, however, such impacts rise substantially and rapidly. Modeling these impacts must therefore not only incorporate work zones' impact on speed, but must also account for how those changes in speed translate into estimates of user costs. In addition to the impact of speed on traditional user costs, increased traffic through a work zone impacts the amount of accidents and fuel consumption in work zones.

4.9.1 User Costs

Speed changes are manifested as additional costs that are measured in a variety of ways. These costs, categorized under the general label of user costs, comprise four elements for the purposes of work zone evaluation. The first group is related to delay, or travel time costs. Reduced speeds and speed cycle changes lengthen trip time, which means that time is lost in making a journey (compared with the time expended on the same route without the work zone). Such time elements are typically aggregated and then converted it monetary values by dollar rates for work and social values.

The second group of user costs relate to vehicle operating costs. These costs concern elements of vehicle operation that result in costs incurred by the vehicle owner.

These costs comprise fuel consumption, oil consumption, tire wear, vehicle maintenance, vehicle depreciation, and spare parts. Again, speed changes and queuing alter the consumption of these items, particularly those related to fuel.

The third group of user costs is those associated with accidents, which are generally higher in work zones for the reasons given in the previous section. Again, these are costs that would not ordinarily be generated by a regular trip, but are a result of imposing a work zone on traffic and should be included in the total user cost evaluated in a full-cost approach to work zone impacts.

In light of these factors, four user cost components are included in this study: queue delay costs, moving delay costs, work zone accident costs, and fuel consumption costs. Fuel costs, as mentioned above, are one of the components of vehicle operation cost. They usually represent about 20% of vehicle operation costs. The reason for using fuel consumption costs instead of whole vehicle operation cost is because the other two components of vehicle operation cost, tire costs and maintenance costs, are not directly related to work zone operation. Hence they are not considered in this study. However, fuel consumption costs are related to the delay associated with work zone operation.

A work zone user cost model developed by Schonfeld and Chien was adopted here. The work zone user cost includes the user delay and accident costs. The user delay costs consist of the queue delay costs upstream of the work zone and the moving delay costs through the work zone. The queue delay cost (4-5) is a function of traffic volume in both directions, work zone hours, average headway through the work zone, average work zone speed, and the cost of time. The moving delay cost (4-6) is a function of traffic volume in both directions, the length of the work zone, average work zone speed, average normal operation speed, and the cost of time. The only traffic accidents considered are those occurring in the work zone and queue areas. Accident costs (4-7) are incurred by the traffic flow passing through the work zone and queue area. Schonfeld and Chien do not consider vehicle operation costs in their model. Therefore, the vehicle operation cost component (fuel consumption) which is most related to work zone operations is considered in this study. The method of calculating fuel consumption cost reported by the Federal Highway Administration (FHWA, 1998) was adopted in this study as shown in

(4-8). Fuel consumption cost is a function of traffic volume, work zone length, fuel consumption rate, and fuel price. The fuel consumption of passenger cars and light trucks is given in equation (4-9).

$$C_q\ (\$) = \text{queue delay cost} = \frac{D\left[Q_1\left(\frac{3600}{H} - Q_1\right) + Q_2\left(\frac{3600}{H} - Q_2\right)\right]v}{V\left(\frac{3600}{H} - Q_1 - Q_2\right)} \tag{4-14}$$

$$C_m\ (\$) = \text{moving delay cost} = \left((Q_1 + Q_2)D\left(\frac{L}{V} - \frac{L}{V_0}\right)v\right) \tag{4-15}$$

$$C_a\ (\$) = \text{accident cost} = \left(\frac{C_q + C_m}{v} + \frac{n_a v_a}{10^8}\right) \tag{4-16}$$

$$C_f\ (\$) = \text{fuel consumption cost} = (Q_1 + Q_2)Lv_f\,. \tag{4-17}$$

$$v_f\ (\$/\ \text{veh-mil}) = \text{average fuel consumption cost} = 0.0362 + \left(\frac{0.746v}{C_q + C_m}\right) \tag{4-18}$$

where,

D = total work zone hours (h) = z_3L;

H = average headway (s);

L = work zone length (km);

n_a = number of accidents per 100 million vehicle hours;

Q = combined flow rate (e.g., $Q = Q_1 + Q_2$) (vph);

Q_i = hourly flow rate in direction i (vph);

V = average work zone speed (km/h);

V_0 = average normal speed (km/h);

v = average user delay cost (\$/veh-h);

v_a = average accident cost (\$/acc); and

z_3 = average M&R time per kilometer (h/km).

The average daily traffic volume can be obtained by converting the annual ESAL value into average daily traffic (ADT):

$$ESAL = (ADT) \times T \times T_f \times D \times L \times (365) \qquad (4\text{-}19)$$

where,

ADT= Average daily traffic;

T= percentage of truck;

T_f= truck factor;

D= directional distribution factor; and

L= lane distribution factor.

The traffic flow (Q) for each hour is obtained by average daily traffic by multiplying the hourly distribution factor (Table 4-4 Default Hourly Distribution of All Function Classes (after TTI, 1993)

Therefore, for the 25 selected cases, the ESAL values can be converted from the ADT value. Five percent of the traffic is assumed to be trucks, thus the truck factor of 5%. The directional distributional factor of 0.5 and lane distribution factor of 1 were used to calculate the equivalent single axle load (ESAL) as shown in the following table.

Table 4-3 ESAL Values for the Selected 25 Pavement Designs

D_1	D_2	SN	MR	ESAL
50	100	1.34	9000	12,416
50	150	1.57	6000	11,820
50	150	1.57	9000	30,280
50	200	1.81	3000	5,314
50	200	1.81	6000	26,535
50	200	1.81	9000	67,975
50	250	2.05	3000	11,116
50	250	2.05	6000	55,504
50	250	2.05	9000	142,186
75	100	1.77	6000	23,310
75	100	1.77	9000	59,713
75	150	2.01	3000	9,873
75	150	2.01	6000	49,300
75	150	2.01	9000	126,294
75	200	2.24	3000	19,616
75	200	2.24	6000	97,951
75	250	2.48	3000	36,912
100	100	2.20	3000	17,565
100	100	2.20	6000	87,709
100	150	2.44	3000	33,336
100	200	2.68	3000	60,273
100	250	2.91	3000	104,487
125	150	2.87	3000	95,587
125	200	3.11	750	6,446
125	250	3.35	750	10,497

*a_1: 0.44, a_1: 0.12, and m_2: 1

Table 4-4 Default Hourly Distribution of All Function Classes (after TTI, 1993)

Hour (24-hr)	Rural			Hour (24-hr)	Urban		
	ADT (%)	Direction (%)			ADT (%)	Direction (%)	
		In	Out			In	Out
0 - 1	1.8%	48	52	**0 - 1**	1.2%	47	53
1 - 2	1.5%	48	52	**1 - 2**	0.8%	43	57
2 - 3	1.3%	45	55	**2 - 3**	0.7%	46	54
3 - 4	1.3%	53	47	**3 - 4**	0.5%	48	52
4 - 5	1.5%	53	47	**4 - 5**	0.7%	57	43
5 - 6	1.8%	53	47	**5 - 6**	1.7%	58	42
6 - 7	2.5%	57	43	**6 - 7**	5.1%	63	37
7 - 8	3.5%	56	44	**7 - 8**	7.8%	60	40
8 - 9	4.2%	56	44	**8 - 9**	6.3%	59	41
9 - 10	5.0%	54	46	**9 - 10**	5.2%	55	45
10 - 11	5.4%	51	49	**10 - 11**	4.7%	46	54
11- 12	5.6%	51	49	**11- 12**	5.3%	49	51
12 - 2	5.7%	50	50	**12 - 2**	5.6%	50	50
13 - 14	6.4%	52	48	**13 - 14**	5.7%	50	50
14 - 15	6.8%	51	49	**14 - 15**	5.9%	49	51
15 - 16	7.3%	53	47	**15 - 16**	6.5%	46	54
16 - 17	9.3%	49	51	**16 - 17**	7.9%	45	55
17 - 18	7.0%	43	57	**17 - 18**	8.5%	40	60
18 - 19	5.5%	47	53	**18 - 19**	5.9%	46	54
19 - 20	4.7%	47	53	**19 - 20**	3.9%	48	52
20 - 21	3.8%	46	54	**20 - 21**	3.3%	47	53
21 - 22	3.2%	48	52	**21 - 22**	2.8%	47	53
22 - 23	2.6%	48	52	**22 - 23**	2.3%	48	52
23 - 24	2.3%	47	53	**23 - 24**	1.7%	45	55

The baseline numerical input associated with user costs is presented in the following tables (Tables 4-5 a, b and c). The average headway (H) through the work zone is 3sec. The work zone length is considered to be 1.6km (1mile). The number of accidents per 100 million vehicle hours is 40. The normal operation speed (V_0) is 80km/hr (50mph), and the work zone speed (V) is 40km/hr (15mph). User cost rates used in this study refer to the dollar values assigned to each user cost component. The unit price of user time (v) and accident cost (v_a) after escalation to the dollar value of 2004 are listed in Tables 4-5 b and c, respectively. The fuel cost (v_f) per gallon is assumed to be $1.8.

Table 4-5a Notation and Baseline Numerical Inputs

Variables	Definition	Baseline Values
H	Average Headway through work zone area	3sec/veh
L	Work Zone Length	1.6 km
n_a	Number of accidents per 100 million vehicle hours	40 acc/100mvh
V_o	Normal operation speed	80km/hr
V	Average work zone speed	40km/hr

Table 4-5b User Delay Costs for Cars in 1996

Agency	Delay Cost Rate Value ($/ veh-hr)	Escalation Factor	Delay Cost Rate Value Adjusted for 2004 ($/ veh-hr)
HERS	14.3	1.195	17.09
NCHRP	11.78	1.195	14.08
MicroBENCOST	11.37	1.195	13.59
Mean			14.92

Table 4-5c Accident/ Crash Costs

Agency	Accident/ Crash Cost ($1000/ fatality)	Reference Year	Escalation Factor	Accident/ Crash Adjusted Cost for 2004 ($1000/fatality)
MicroBENCOST	1,182	1996	1.198	1,416
US DOT	2,700	1995	1.234	3,330
FHWA	2,723	1991	1.380	3,758
Mean				2,835

Again the user cost components were added into the life cycle cost framework of the study as shown in Figure 4-5. The components that make up the user cost calculations are evaluated each time a work zone is placed in the roadway.

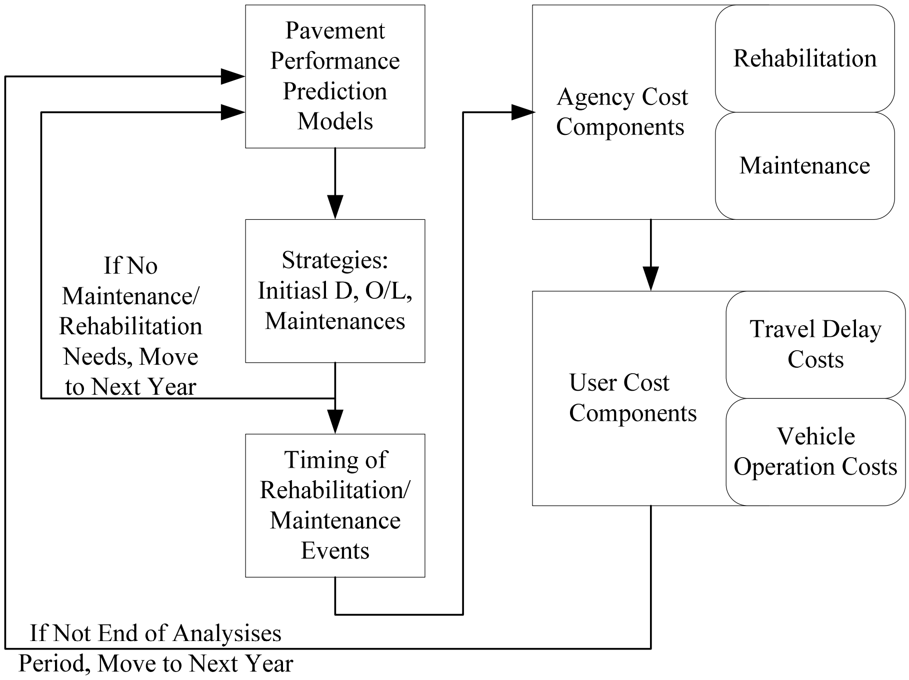

Figure 4-5 User Cost Components Added to Framework

4.10 PAVEMENT LIFE CYCLE COST CALCULATION

The life cycle cost calculation component takes the events and their timing, as predicted by the performance and rehabilitation strategy models, and assigns a cost for each applicable component of each event. Thus a modified total cost model that includes agency costs and user costs, which was developed by Schonfeld and Chien (1999), was adopted in this study to calculate the total cost of the pavement over the analysis period.

The total cost (C_T) of maintaining or rehabilitating a work zone of length L can be considered as a linear function:

$$C_T = C_{M\&R} + C_U \qquad (4\text{-}20)$$

where,

$C_{M\&R}$ = agency cost; and

C_U= the user cost.

The user cost includes queue delay cost (C_q), moving delay cost (C_m), fuel consumption cost (C_f), and work zone accident cost (C_a). This model assumes that the vehicle keeps a uniform speed through the work zone, that the vehicle arrival and discharge rate from queue remains constant, and that the user travel and delay costs must be represented by a constant average per vehicle hour. In addition, the traffic volume for both directions is given. The $C_{M\&R}$ of maintaining or rehabilitating a zone of length L is a linear function of the following form:

$$C_{M\&R} = z_1L + z_2L; \tag{4-21}$$

where,

z_1= maintenance or rehabilitation cost per work zone km; and

z_2 = average installation geosynthetics cost per km.

The time required to maintain a work zone of length L is a linear function of the form

$$D = z_3L + z_4L. \tag{4-22}$$

where,

z_3 = working time per work zone km; and

z_4 = time required to install geosynthetics per work zone km.

It is also assumed that the combined traffic volume in two directions does not exceed the capacity of one lane for an extended period (Schonfeld and Chien, 1999). To calculate the agency and user costs of each maintenance/ rehabilitation event, this study proposes a total cost modification to the previous model presented by Schonfeld and Chien. The modification consists of using different parameters, such as adding geosynthetic materials installation time, considering vehicle operation costs, and eliminating work zone set-up costs and time. Based on the aforementioned assumptions, the total cost model for a two-lane highway, including the agency and user costs of a work zone, can be expressed as follows:

$$C_T = C_{M\&R} + C_U = C_{M\&R} + C_d + C_m + C_f + C_a \quad (4\text{-}23)$$

where,

C_T = average total cost per kilometer per lane (\$/km-lane);

$C_{M\&R}$ = total agency cost (\$/L) = $Z_1L + Z_2L$; and

C_U = total user cost.

The estimated agency costs for the maintenance/rehabilitation will be calculated and entered for the appropriate year. Depending on the maintenance/ rehabilitation strategy selected, the traffic impacts will be considered and calculated. User costs will then be considered and calculated.

A conceptual graph of expenditures over a pavement analysis period is shown in Figure 4-6. In the graph, the black arrows represent the agency costs associated with each construction activity and the gray arrows represent the user costs, which are associated with construction activities every time a construction work zone is in place over the life of the project. User costs vary greatly, depending on the number of vehicles passing through the work zone, but can easily be much greater than the total cost of the actual construction activities.

As the result, the combined agency and user costs for each event will be entered in the life cycle cost analysis at the predicted age of the pavement. The total cost calculated for each year is then discounted to the present time to obtain its present value, for comparison.

In conclusion, an expanded view of the cost components of the framework is needed. For each year that a design alternative is evaluated, the maintenance and rehabilitation routine determines if repair work is necessary. If such work is needed, the appropriate cost components of the framework are invoked to estimate the total cost, given present day unit costs and production rates. These are costs to the agency in terms of construction costs, and to the users in terms of time delay, vehicle operating costs, and all other costs that can be measured and valuated.

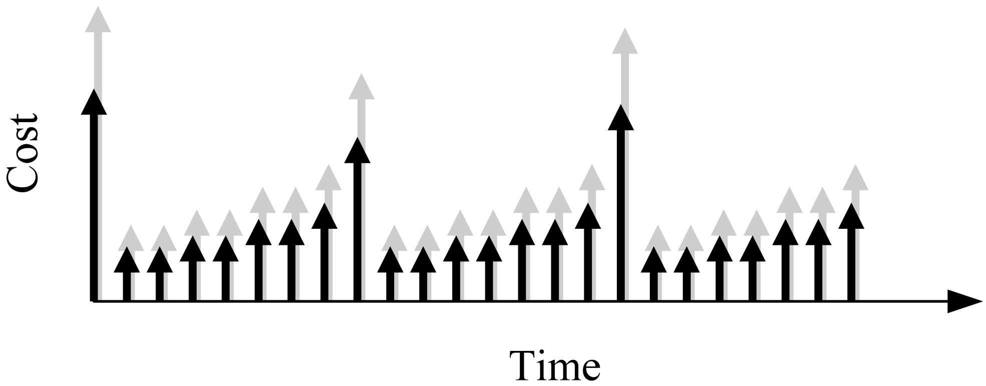

Figure 4-6 Expenditure Diagram Including Agency and User Costs Associated with Construction Activities

The final cost components of the LCC framework are schematized in Figure 4-7. After computing the total cost for a particular year, including agency and user costs, this total is discounted to the present time, or the time at which the analysis is being performed. This means that the calculated cost at a future time is adjusted for inflation and prevailing interest rates. After summing costs for the current analysis year and discounting those costs to the present, the framework records the level of all pavement distresses, notes any maintenance or rehabilitation work that is performed during the year, and returns to the pavement performance models to begin a new analysis year.

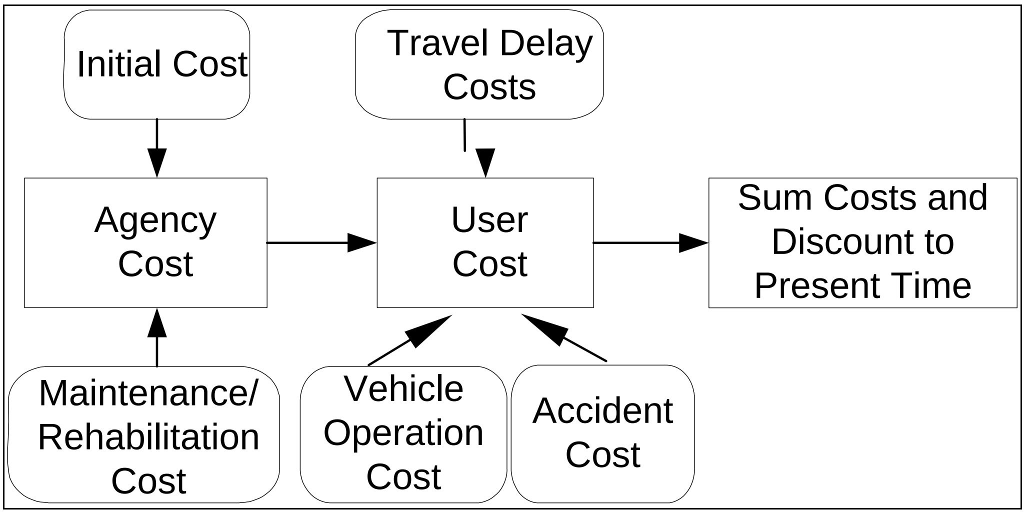

Figure 4-7 Cost Component of the Framework

4.11 THE DISCOUNT RATE

All the pavement cases considered in this study have different rehabilitation timings. Therefore, comparisons were made using inflation-adjusted dollars. A discount rate is then used to account for the time value of money. This provides an agency, planner, or decision maker with a way to compare future expenditures with those occurring in the present. In a proper economic analysis, all future costs and benefits are *discounted* to the present, accounting for the prevailing and expected interest and inflation rates. Since the future value of a sum of money in the present is greater owing to compounding interest, the reverse must also be true. The present value of a future sum is worth less than it would be at the present. Because an economic analysis can be highly sensitive to the discount rate, it must be selected with care. Here, this section will discuss how the interest rate and inflation rate can be used to determine a proper value for the discount rate. Also discussed are some common values that are suggested by various agencies in the United States.

The discount rate can be calculated from the interest rate and the inflation rate that may be expected over the life of the highway project. The equation to calculate the discount rate is as follows:

$$Discount = \left(\frac{\text{interest} - \text{inflation}}{1 + \text{inflation}} \right) \tag{4-24}$$

where:

Discount = calculated discount rate;

interest = expected interest rate; and

inflation = expected inflation rate.

The difficulty in predicting interest and inflation rates over the life of the project is somewhat attenuated by using the discount rate. The Federal Highway Administration has recommended standard discount rates for use in various highway pavement projects. The level of the discount rate depends largely on the type of agency that is financing the work. The rate also depends on the agency's cost of funds, tax-exempt status (for bond investors), creditworthiness (based on ratings by investment banks), and many other factors. In general, the following rates have been recommended for the indicated entities. In this study, a 2.5% discount rate was used to similar state/ municipal condition.

Table 4-6 Recommended Discount Rates

Agency Type	Appropriate Discount Rate
State/ Municipal	2.50%
Federal/ Long Term Project	3.50%
Privately Funded Projects	4.50%

4.12 ANALYSIS PERIOD

The period of time over which the pavement and the total life cycle cost will be analyzed is called the *analysis period*. This term has been used throughout this study when discussing other aspects of the life cycle cost analysis framework. The analysis period is used to define the time during which the pavement performance is analyzed, and during which all the costs and other aspects of the framework are calculated. During the analysis period, the pavement must be kept above minimum standards for structural and functional capacity.

The Federal Highway Administration provides some guidelines regarding appropriate analysis periods for highway pavement projects. The analysis period should be sufficiently long to reflect the long-term differences between various design alternatives (Walls et. al, 1998). The FHWA Final Policy of September 1996 recommends a minimum analysis period of at least 35 years for all pavement projects, both flexible and rigid. This includes all types of construction as well, such as new construction, rehabilitation, restoration, and resurfacing projects (*Federal Register*, 1996). This recommendation also suggests, however, that shorter analysis periods may be appropriate in some cases (e.g., low volume roads) or when a short rehabilitation project will extend the life of a pavement a few years until a total reconstruction is planned.

The FHWA's life cycle cost *Interim Technical Bulletin* (Walls et al, 1998) also indicates that the analysis period should be greater than the pavement design period, which allows enough time to incorporate at least one rehabilitation activity. This concept was also promoted in a lecture by Witczak to the National Asphalt Paving Association in 1997 (Witczak, 1997).

It is important to understand that future pavement performance becomes more difficult to predict as the analysis period grows longer. This is offset, however, by the more important component of life cycle cost analysis, namely, user costs associated with frequent maintenance and rehabilitation. Using reliability concepts, pavements can be designed with a high degree of confidence. The result can be pavement structures that are very likely to provide performance throughout their entire design life, with minimal disruptions to traffic flow.

4.13 ECONOMIC ANALYSIS STRATEGY

Here, the cost-effectiveness method was used to quantify the engineering benefit into the economic benefit. Calculations of cost effectiveness are based on the areas under the performance curve.

Effectiveness is the net area under the pavement performance curve multiplied by length of section and volume of traffic, such that the effectiveness can be expressed as follows:

$$Effectiveness = \left[\sum_{\text{Reha Year}}^{PSI_R \geq PSI_M} (PSI_R - PSI_M) - (\sum_{PSI_N \geq PSI_M}^{\text{Reha Year}} (PSI_M - PSI_R) \right] \times (ADT) \times (\text{Length of section}) \qquad (4\text{-}25)$$

where:

PSI_R = Pavement Serviceability Index (PSI) after rehabilitation and for each year until PSIM is reached;

PSI_M = minimum acceptable level of PSI; and

PSI_N = yearly PSI from the needs year to the rehabilitation year.

Therefore the calculation of cost-effectiveness, CE, would be a simple ratio of effectiveness divided by cost. This value has no economic or physical meaning but is valuable in the relative comparison of alternatives or design methods.

4.14 SENSITIVITY ANALYSES

Major assumptions should be varied and net present value and other outcomes recomputed to determine how sensitive outcomes are to changes in assumptions. The assumptions that deserve the most attention will depend on the dominant benefit and cost elements and the areas of greatest uncertainty in the project being analyzed. In this study, several features of flexible pavement such as layer thickness, structure number, subgrade strength, ESAL value, and traffic benefit ratio were selected to investigate the cost-effectiveness of the selected design method.

CHAPTER 5 RESULTS AND DISCUSSION

5.1 SERVICE LIFE COMPARISON

The service lives of the representative pavements were compared using the AASHTO pavement design equation. Table 5-1 lists the service life predictions of the pavements without geotextiles and the pavements with geotextiles for all 25 pavement cases considered in the study.

Table 5-1 Service Life Estimates for the 25 Representative Pavement Designs

Representative Design	Pavement without Geotextile (year)	Pavement with Geotextile (year)	
		Al-Qadi Design Method	Perkins' Design Method
1	20, 30, 37 42, 46, 50	32, 44, 50	22, 32, 39, 44, 48, 50
2	20, 30, 37 42, 46, 50	32, 44, 50	27, 38, 46, 50
3	20, 30, 37 42, 46, 50	32, 44, 50	23, 34, 41, 46, 50
4	20, 30, 37 42, 46, 50	32, 44, 50	39, 50
5	20, 30, 37 42, 46, 50	33, 45, 50	27, 38, 46, 50
6	20, 30, 37 42, 46, 50	34, 46, 50	23, 34, 41, 46, 50
7	20, 30, 37 42, 46, 50	32, 44, 50	39, 50
8	20, 30, 37 42, 46, 50	34, 46, 50	27, 38, 46, 50
9	20, 30, 37 42, 46, 50	34, 47, 50	23, 34, 41, 46, 50
10	20, 30, 37 42, 46, 50	33, 45, 50	26, 37, 44, 50
11	20, 30, 37 42, 46, 50	34, 46, 50	22, 32, 39, 44, 48, 50
12	20, 30, 37 42, 46, 50	32, 44, 50	37, 50,
13	20, 30, 37 42, 46, 50	34, 46, 50	26, 37, 44, 50
14	20, 30, 37 42, 46, 50	34, 47, 50	22, 32, 39, 44, 48, 50
15	20, 30, 37 42, 46, 50	33, 45, 50,	37, 50
16	20, 30, 37 42, 46, 50	34, 46, 50	26, 37, 44, 50
17	20, 30, 37 42, 46, 50	33, 46, 50	37, 50
18	20, 30, 37 42, 46, 50	33, 45, 50	34, 46, 50
19	20, 30, 37 42, 46, 50	34, 46, 50	24, 35, 42, 48, 50
20	20, 30, 37 42, 46, 50	33, 45, 50	34 ,46, 50
21	20, 30, 37 42, 46, 50	34, 46, 50	34, 46, 50
22	20, 30, 37 42, 46, 50	34, 47, 50	34, 46, 50
23	20, 30, 37 42, 46, 50	34, 46, 50	31, 43, 50
24	20, 30, 37 42, 46, 50	32, 44, 50	50
25	20, 30, 37 42, 46, 50	32, 44, 50	50

The purpose of this study is to compare the service life among the pavements with or without geotextiles instead of comparing different pavement design methods. The service life predictions of the pavements without geotextiles are all the same because the input traffic volume used corresponds to its design traffic Therefore, the service life of the pavements with geotextiles are presented. Table 5-1 using Al-Qadi's design method and Perkins' design method. As expected, the service life extension based in Al-Qadi's design model gives more uniform results compared to Perkins' design method. This is because the TBR values obtained from Al-Qadi's design method are between 2.1 and 2.6. However, the TBR obtained from Perkins' prediction model are varied from 1.1 up to 8.34. Besides, although the improvement values from the two design models are greater than one, this does not mean the pavement can actually extend its service life by simply multiplying the obtained TBR value by the original service life. The improvement obtained here is improvement in the pavement's allowable ESALs rather than pavement service life. For example, the 13^{th} representative design, which considers a pavement with a 75mm thick HMA layer, a 150mm thick granular base layer and a subgrade at a CBR of 4%, has a TBR of 2.44 from Al-Qadi's design model and a TBR of 1.45 from Perkins' design model, respectively.

The effect of subgrade CBR on pavement service life is also compared. As shown in the Figure 5-1, pavement service life decreases with increasing subgrade strength when Perkins' design model is used and increases slightly when Al-Qadi's design method is used. Perkins' design method gives very high a TBR value when the subgrade is weak. In this instance, with subgrade CBR at 0.5%, the model gives TBR values of 19. This means that when geotextiles are used in the pavement, then the allowable ESALs can increase to 19 times the original design ESALs which is unrealistic.

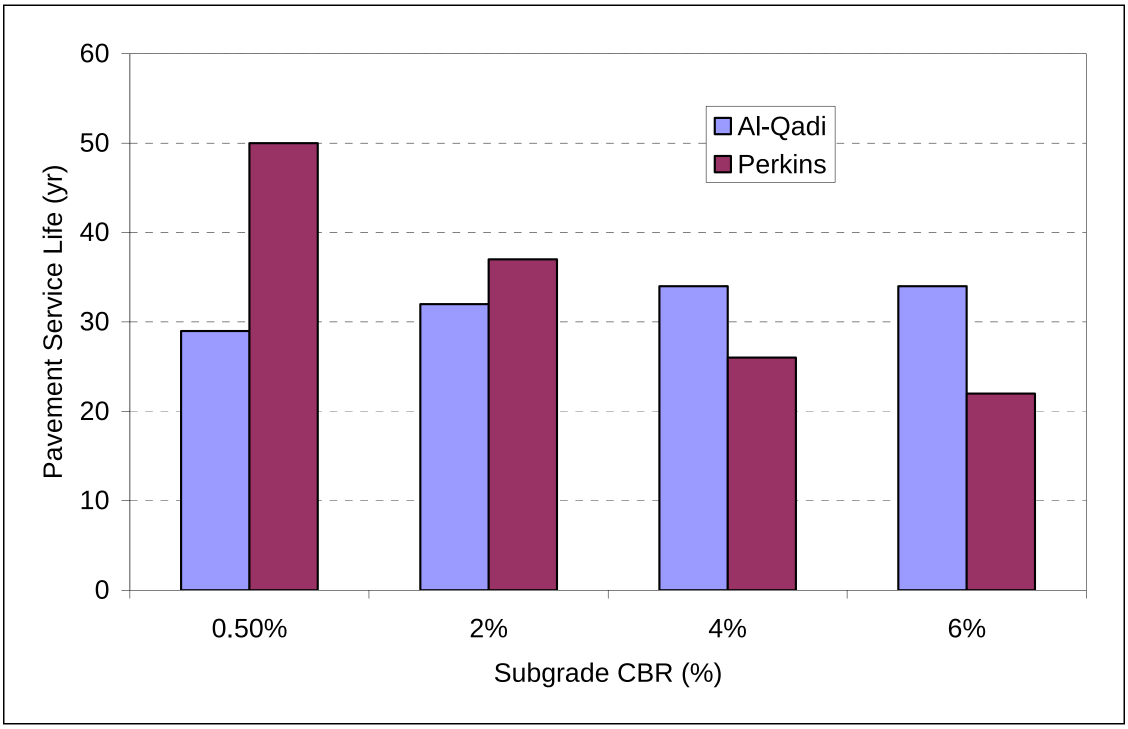

Figure 5-1 Service Life Comparison among Different Subgrade Stengths

5.2 Agency and User Cost Comparison

Most of the studies related to the incorporation of geosynthetic materials in pavement systems report that by using geotextiles in pavement, the service life of the pavement can be extended. Hence, the question here is how much money can be saved relatively if the pavements incorporate geotextiles. Therefore, the focus here was aimed at transforming the engineering benefit into economic profit. The following sections present the agency and user costs of the 25 representative pavement designs. All the values presented are transferred back to the net present value of the initial construction year.

5.2.1 Agency Costs

The agency costs of the 25 pavement design alternatives were compared. Initial construction costs, maintenance costs, and total agency cost are presented in Figure 5-2. Three design methods show similar initial construction costs. Al-Qadi's and Perkins' designs show a little higher value in initial construction cost since these design methods incorporate geotextiles in the pavement, which slightly increases initial construction costs. However, if this can provide the engineering benefits stated in the design methods, the future maintenance costs will show significant savings in agency cost considerations. The maintenance costs are calculated and shown in Figure 5-3. As can be seen from the figure, enormous differences are evident in the maintenance costs among the three design methods.

To have an idea of how much savings there were between the different design alternatives due to the service life extension, the total agency costs of the pavement using Al-Qadi's and Perkins' design methods were each separately divided by the total agency cost using AASHTO pavement design method. The result is shown in Figure 5-4. In general, Al-Qadi's design method gives a 20% reduction in the agency cost for the 25 preventative pavement design alternatives, and Perkins' design method gives from almost no cost reduction to a 40% cost reduction in the total agency cost. Figure 5-5 shows the total agency cost for the 25 representative design pavements.

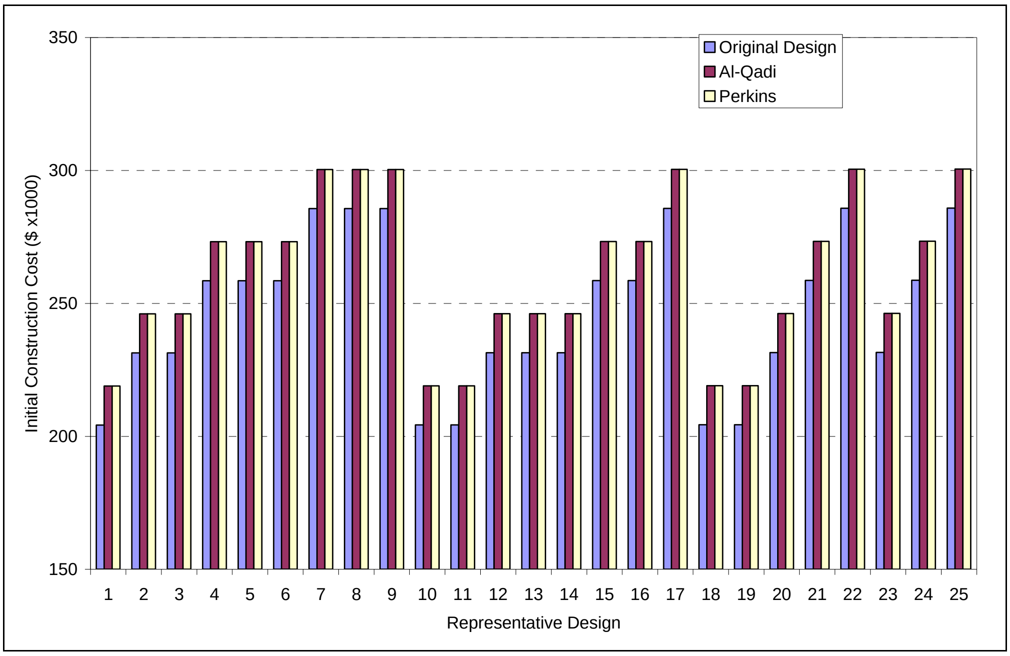

Figure 5-2 Initial Construction Cost Comparison for the 25 Representative Design Alternatives

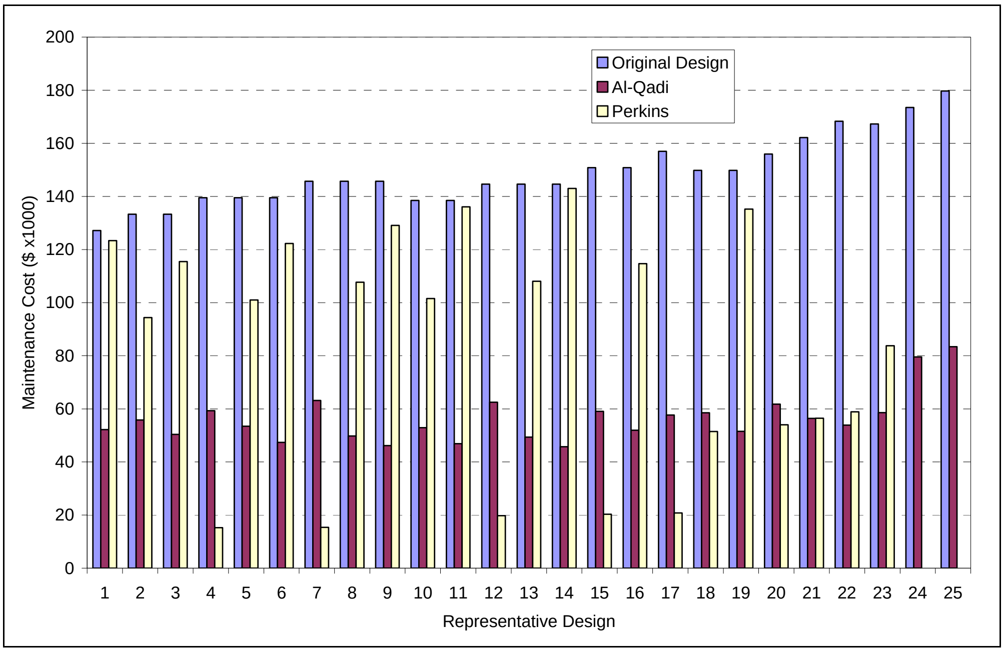

Figure 5-3 Maintenance Cost Comparison for the 25 Representative Design Alternatives

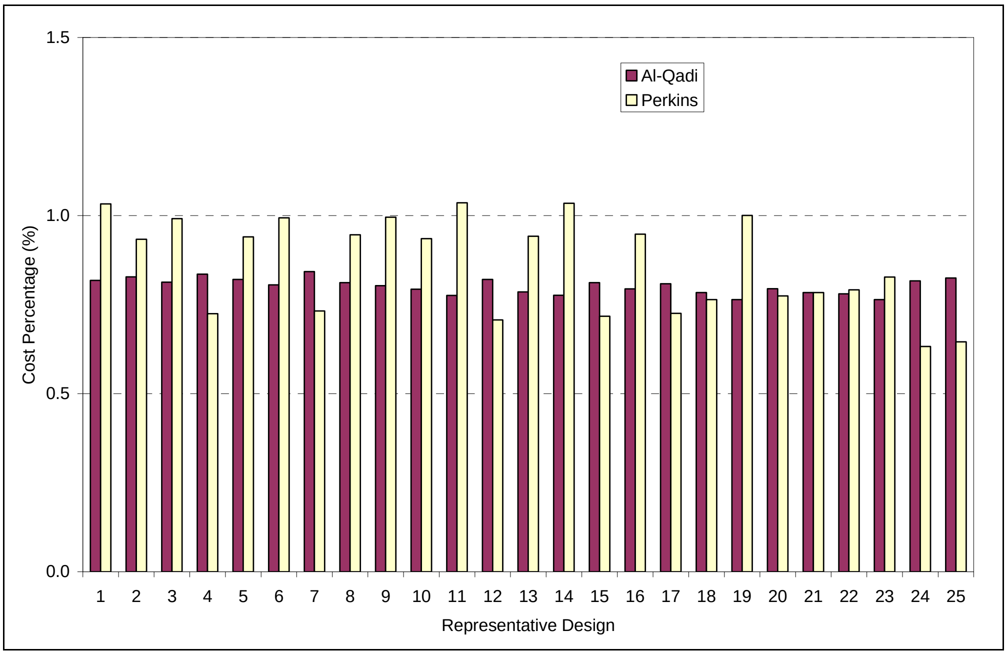

Figure 5-4 Relative Agency Cost Compared to the AASHTO Design Method

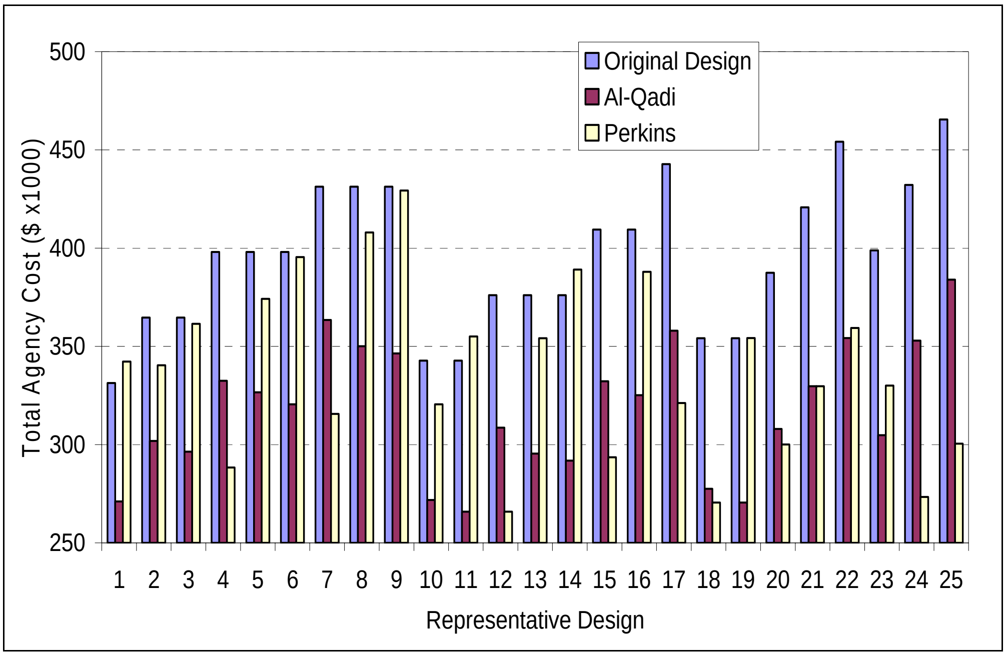

Figure 5-5 Total Agency Cost Comparison of the 25 Representatives Design Alternatives

5.2.2 <u>User Costs</u>

In many cases, while state highway engineers conduct a LCCA, the user cost components are believed to be difficult to quantify, thus they are usually excluded from LCCA. This exclusion has resulted in total cost underestimation. In this study, three work zone induced user cost components are considered: delay, fuel consumption and accident cost.

In terms of the work zone induced user costs, user delay costs usually occupy the highest proportion among the three user cost components. User delay costs are mainly affected by the amount of traffic on the roadway. As a result, roadways with higher traffic volumes will have higher user delay costs. User delay costs were compared in Figure 5-6. In general, the AASHTO design method still gives higher user delay costs compared to the other two design methods. However, in design 14 (D1=75mm, D2=150mm, and subgrade CBR=6%), following Perkins' design method, the design alternative gives greater user delay costs compared to the AASHTO and to Al-Qadi's design method. In representative design alternatives 24 (D1=125mm, D2=200mm, and subgrade CBR=0.5%) and 25 (D1=125mm, D2=250mm, and subgrade CBR=0.5%), Perkins' design method shows that there is no rehabilitation needed if the geotextile is incorporated into the pavement.

For fuel consumption and accident costs, as can be seen in Figure 5-8 and Figure 5-7, the values are relatively small if compared to user delay costs. However, fuel consumption costs are just one of the components of vehicle operation costs.

Similarly, the total user cost of pavement using the AASHTO design method was divided by the total user cost obtained from Al-Qadi's and Perkins' design methods. The results are shown in Figure 5-9. In general, Al-Qadi's design method gives about a 70% reduction in user costs for the 25 representative pavement design alternatives, and Perkins's design method gives from almost no cost reduction to a 100% cost reduction in total user costs. Figure 5-10 shows the total user cost for the 25 representative pavement designs.

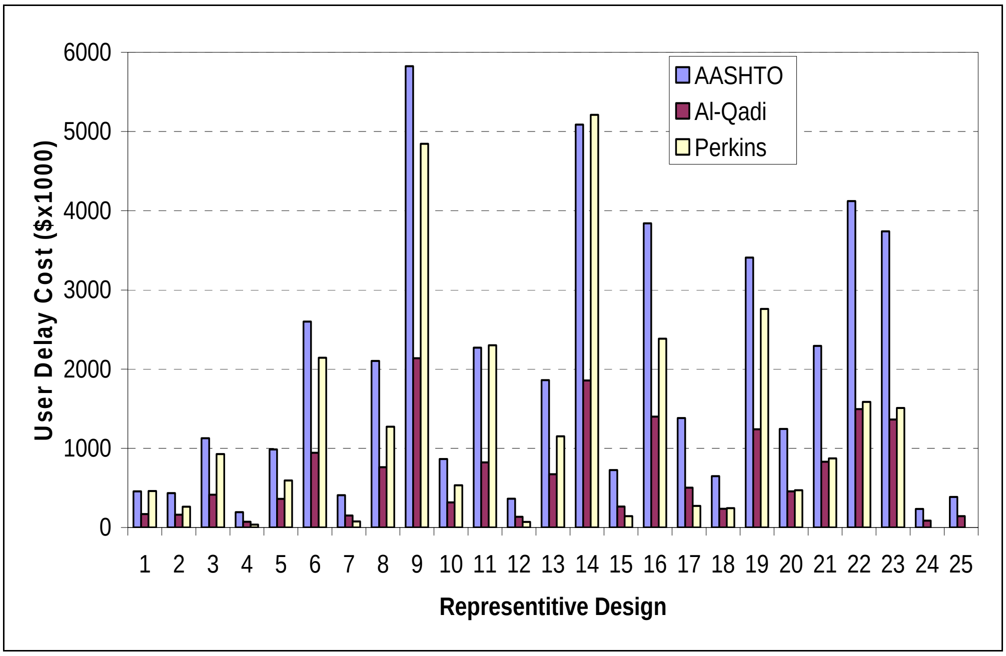

Figure 5-6 User Delay Cost Comparison for the 25 Design Alternatives

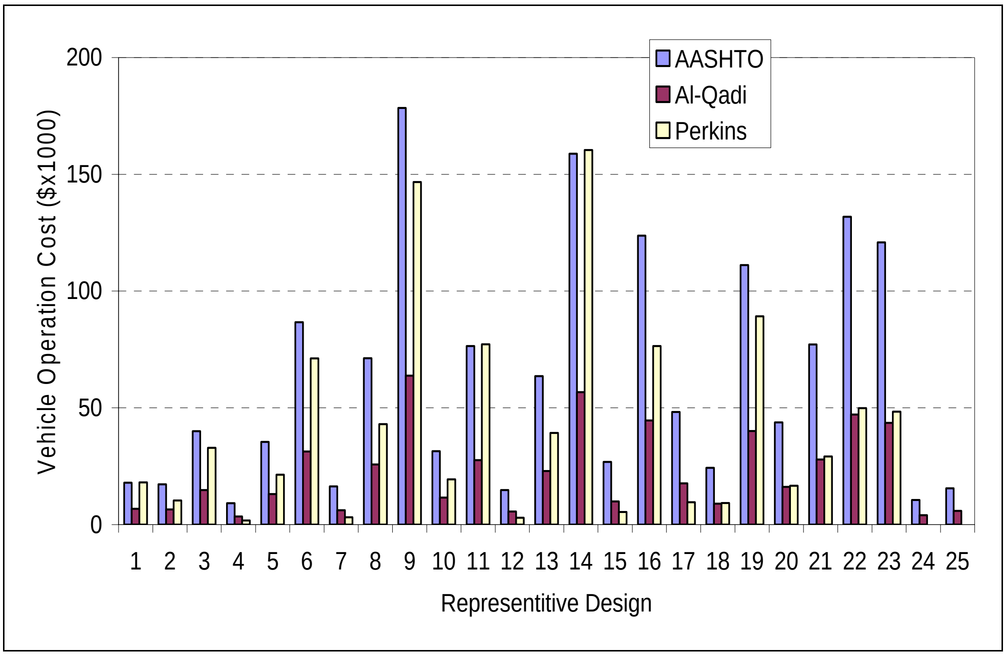

Figure 5-7 Fuel Consumption Cost Comparison for the 25 Design Alternatives

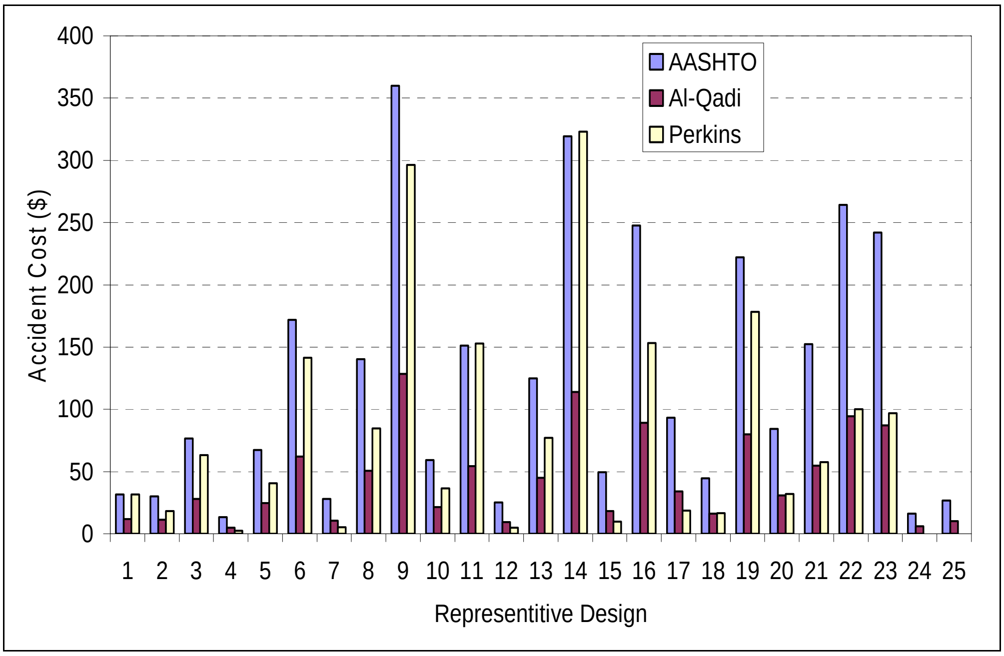

Figure 5-8 Work Zone Accident Cost Comparison for the 25 Design Alternatives

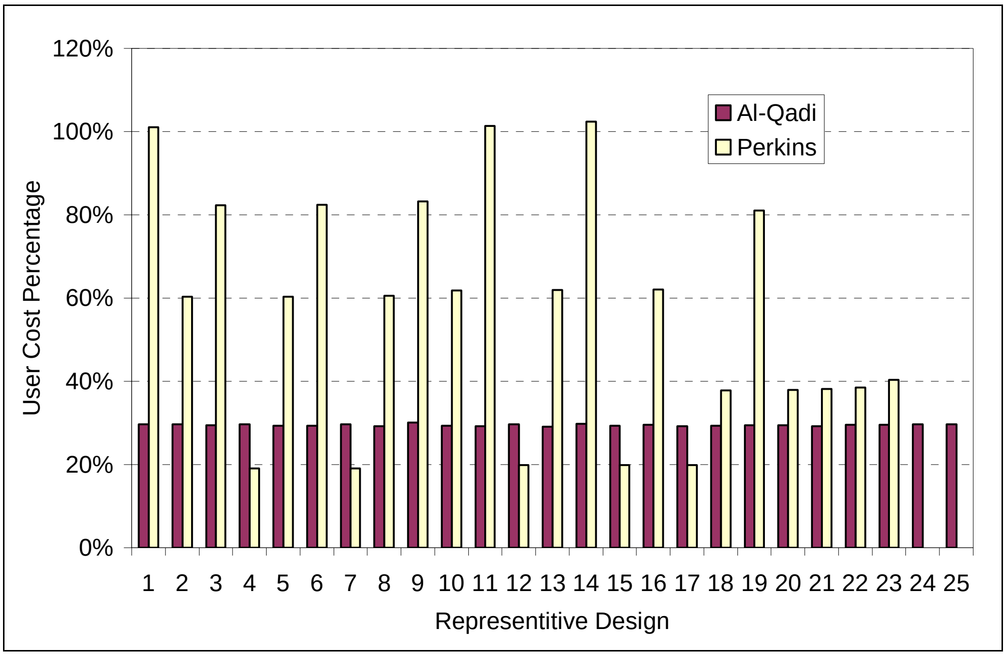

Figure 5-9 Relative User Cost Compared to the AASHTO Design Method

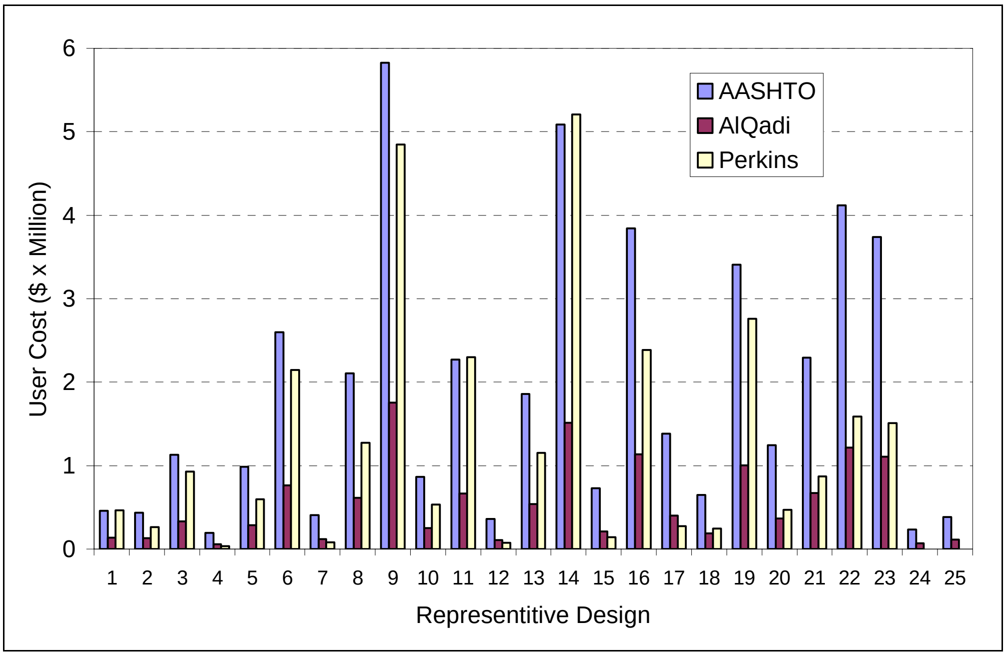

Figure 5-10 Total User Cost Comparison for the 25 Representatives Design Alternatives

5.2.3 Total Cost

Using the aforementioned information, the total life cycle cost for the 25 representative design alternatives is calculated and the results are shown in Figure 5-11. It is worthwhile to notice that when only agency costs are included in the LCC analysis, the difference among the three design methods is not obvious. However, when the user costs are taken into consideration, the three design methods are clearly distinguished from one another. Al-Qadi's design method suggests that when a geotextile is placed in a pavement as a separator, the range of the total pavement life cycle cost savings can be as high as 70% to as low as 40%. This depends on the selected design alternative. When Perkins' design method is used, the suggested total pavement life cycle cost savings varies from no savings to 70% savings compared to the AASHTO's design method.

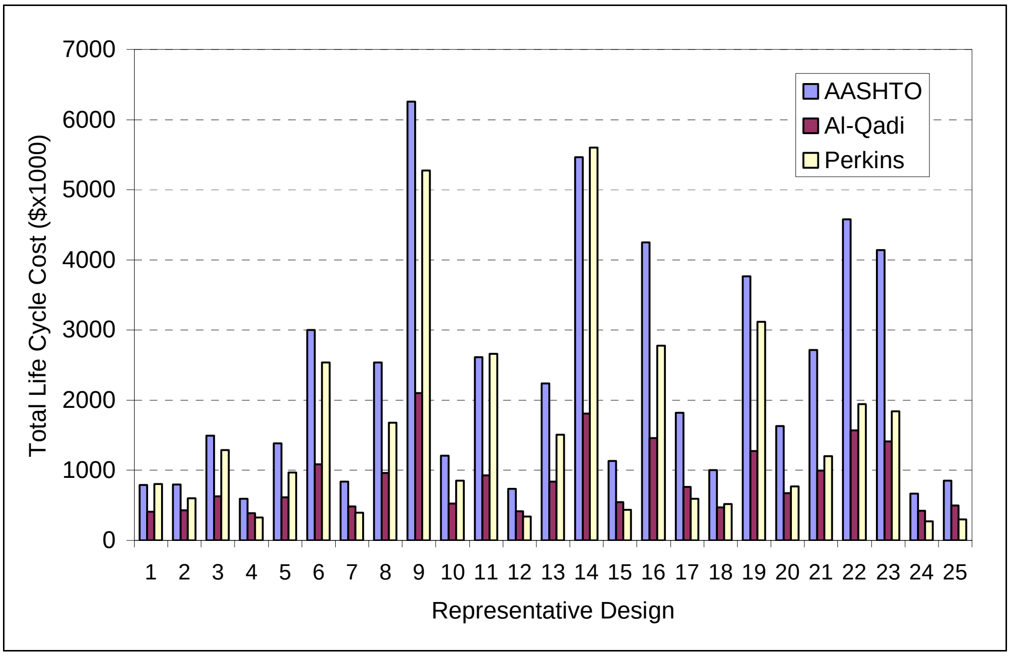

Figure 5-11 Total Life Cycle Cost for the Three Design Methods

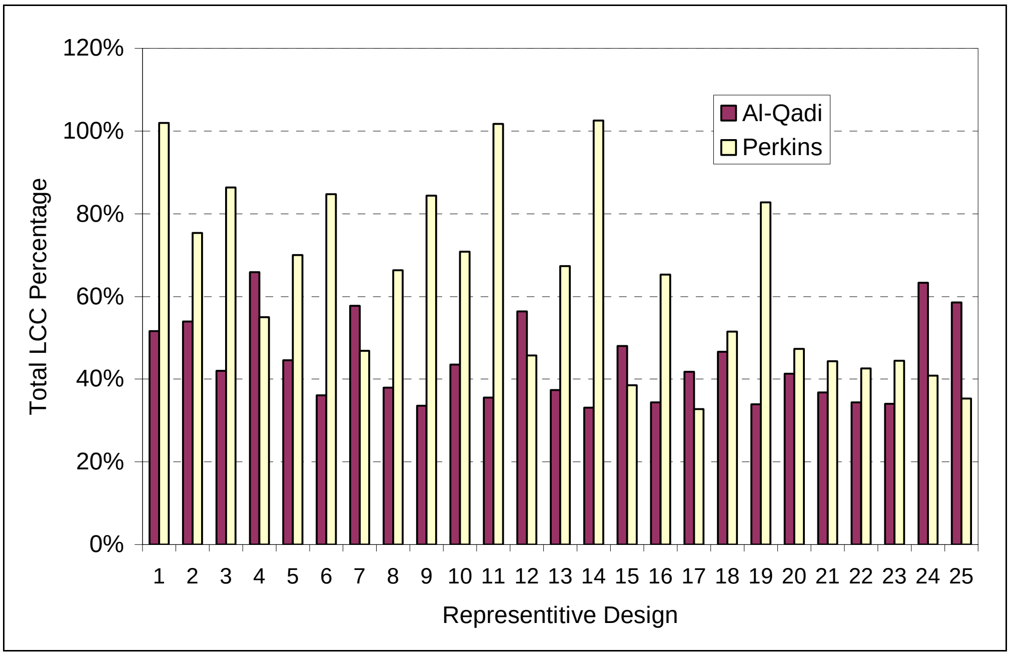

Figure 5-12 Relative Total Cost Compared to the AASHTO Design Method

When the representative design alternatives are arranged in order of traffic volume and their agency and user costs are compared, the following results. For the AASHTO method, when the traffic volume is greater than 11,800 ESALs over the 20 year design period, the user costs will exceed the agency costs (Figure 5-13). Figure 5-14 illustrates that when Al-Qadi's design method was used, the threshold number of ESALs for this tipping point is 30,000. However, as Figure 5-15 indicates, Perkins' design method does not indicate such a specific trend when agency and user costs are compared.

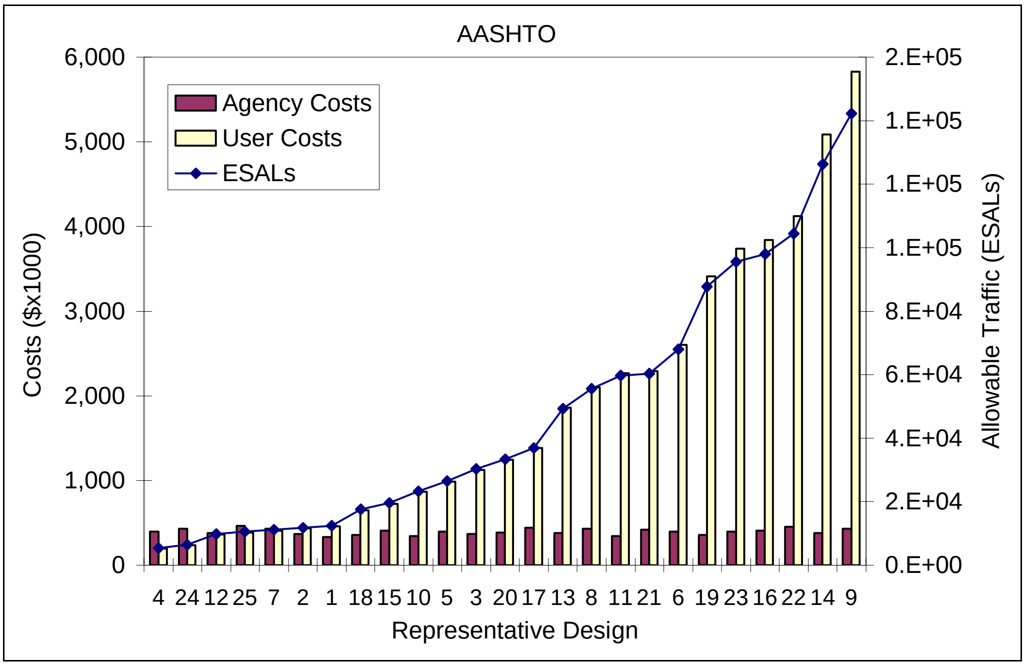

Figure 5-13 Agency and User Costs Comparison as ESALs Rise using the AASHTO Design Method

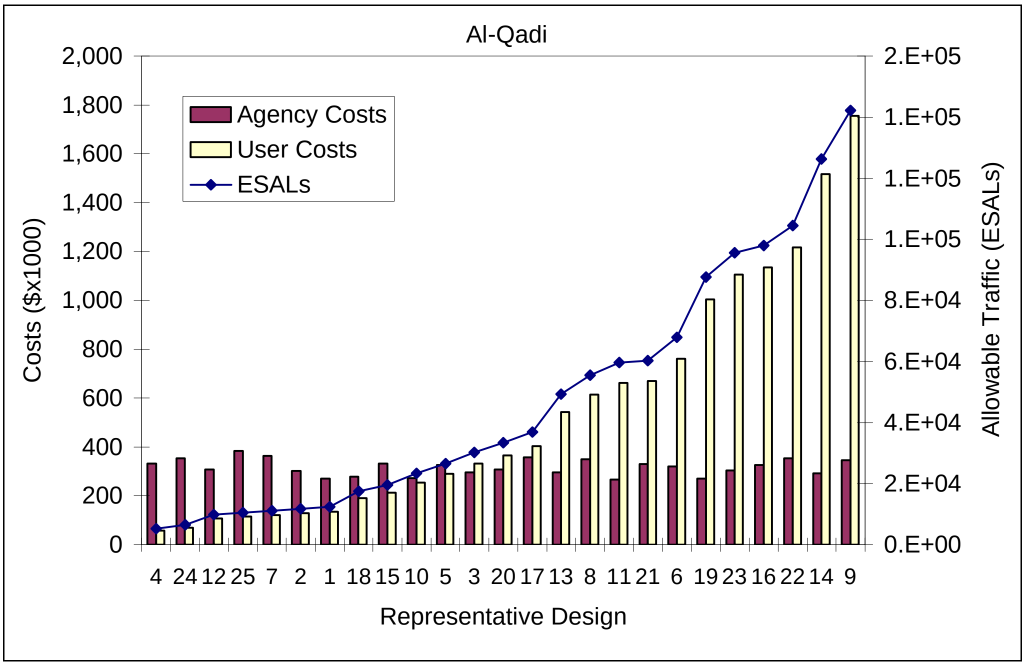

Figure 5-14 Agency and User Costs Comparison as ESALs Rise using the Al-Qadi Design Method

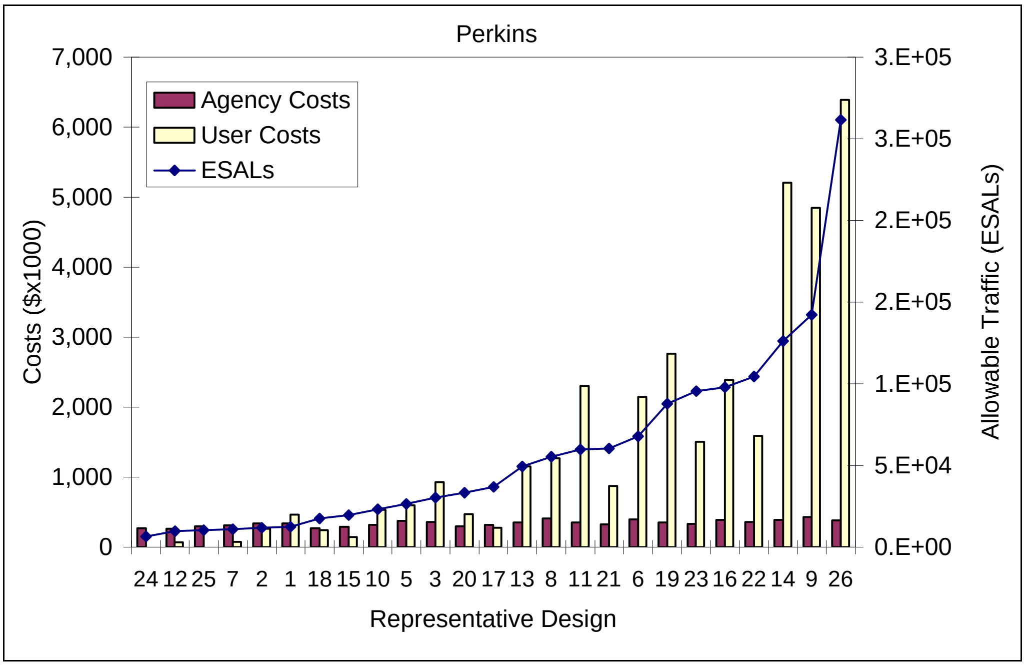

Figure 5-15 Agency and User Costs Comparison as ESALs Rise using Perkins' Design Method

5.3 Cost-Effectiveness Ratio

Another parameter usually used in LCCA is comparing the cost-effectiveness of selected design alternatives. First, the effectiveness of the representative pavement alternatives of different design methods is evaluated by calculating the area under the present service index (PSI) versus the time (t) curve. The cost-effectiveness is then obtained by taking the ratio of the effectiveness and total pavement life cycle costs for the selected pavement alternative. Once the cost-effectiveness is obtained, the cost-effectiveness ratio can be calculated by comparing the cost-effectiveness value of one design method to another design method. In this study, the design methods of Al-Qadi and of Perkins are compared to the AASHTO design method to acquire the cost-effectiveness ratio of the first two design methods.

The result of the cost-effectiveness ratio of the two design methods is shown in Figure 5-16. Therefore, if a design method shows a higher cost-effectiveness ratio, this means that the design method suggests that such a pavement design alternative is better. If the result is less than unity, this means that the design method suggests that incorporating geotextiles into the pavement is worse than not incorporating geotextiles into the pavement. If the value is equal to one, it means the pavement does not get any benefit from having geotextiles incorporated. If the value is greater than one, it means the pavement benefits from having geotextiles added. Therefore, the higher the value, the greater the benefit. From the figure, the lowest cost-effectiveness ratio using Al-Qadi's design method is 1.7 and the highest is 3.2. The average is 2.6. For Perkins' design method, the lowest value is 1.01 and the highest value is 5.7. The average is 2.1.

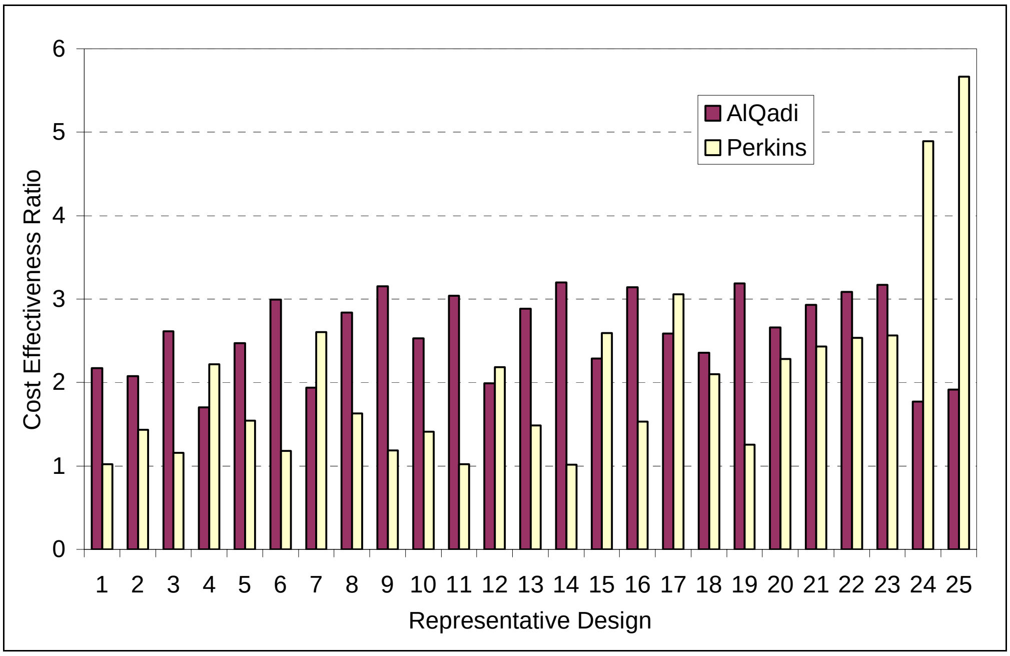

Figure 5-16 Cost-Effectiveness Ratio of Adopting Al-Qadi's and Perkins' Design Methods

5.4 SENSITIVITY ANALYSIS

The ideal situation in the sensitivity analysis would be to alter a particular design feature and to experience as large a percentage increase as possible in service for the smallest possible percentage increase in cost. The following section will investigate which design feature of a flexible pavement, such as thickness of HMA, thickness of the base, subgrade strength, structure number, and TBR value, improves cost-effectiveness.

5.4.1 Thickness of HMA

The most common parameter for designing a pavement is the thickness of the HMA layer. The thickness of the HMA is investigated here to see the how changing the thickness of this layer influences the cost-effectiveness ratio. Figure 5-17 indicates that with subgrade CBR=2% with a base layer thickness of 150mm, both designs suggest that the thicker HMA layer will give the higher cost-effectiveness ratio. When the subgrade CBR increases to 4%, a similar result can be observed. However, further increasing subgrade strength using Perkins' design method suggests the opposite trend in the cost-effectiveness ratio. This trend, in which thicker HMA layers give lower cost-effectiveness ratios, can be seen in Figure 5-19. In addition, when the thickness of the base layer is also evaluated, at a base thickness of 100mm, Al-Qadi's design method still suggests that with an increase in the thickness of the HMA, the cost-effectiveness ratio of the design alternative will increase. The opposite trend is observed when Perkins' method is used, as shown in Figure 5-20 and Figure 5-21.

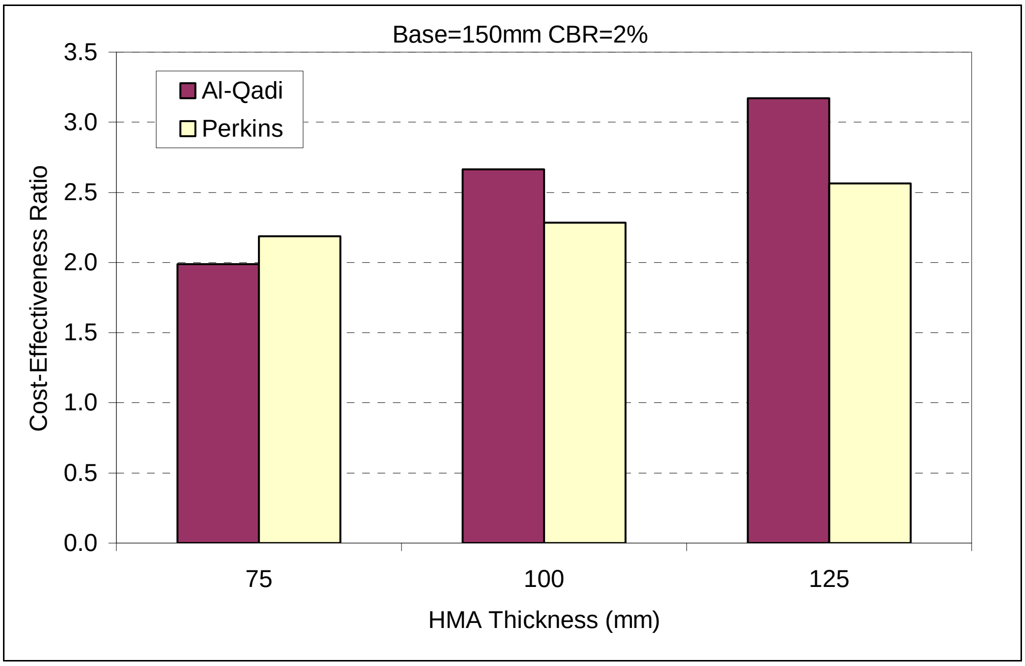

Figure 5-17 Influence of Cost-Effectiveness Ratio by HMA Thickness Variation using 150mm Base Layer and Subgrade Strength of 2%

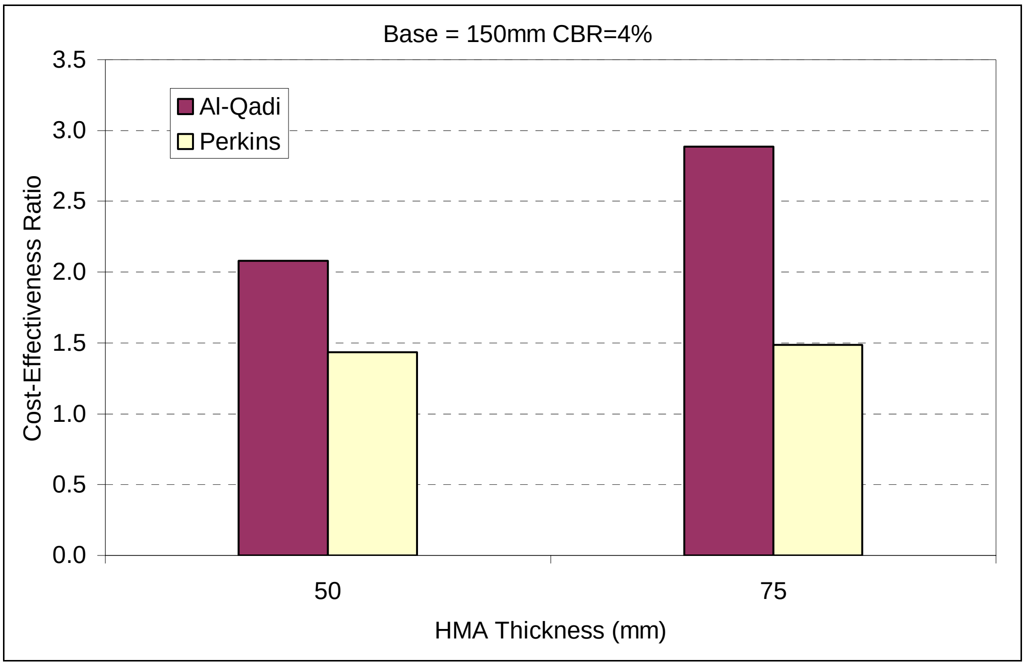

Figure 5-18 Influence of Cost-Effectiveness Ratio by HMA Thickness Variation using 150mm Base Layer and Subgrade Strength of 4%

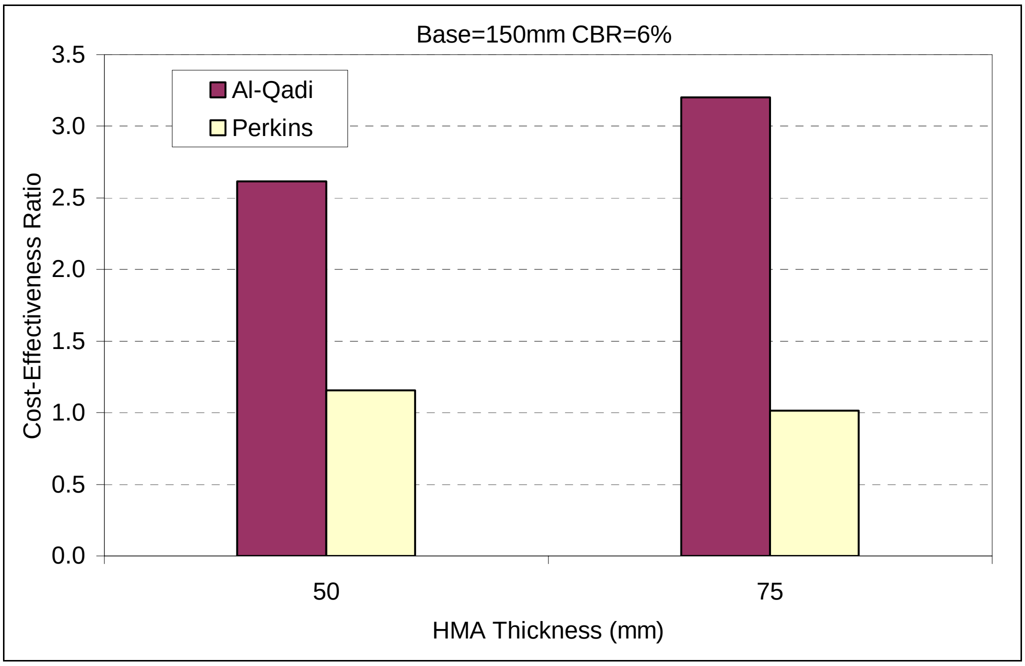

Figure 5-19 Influence of Cost-effectiveness Ratio by HMA Thickness Variation using 150mm Base Layer and Subgrade Strength of 6%

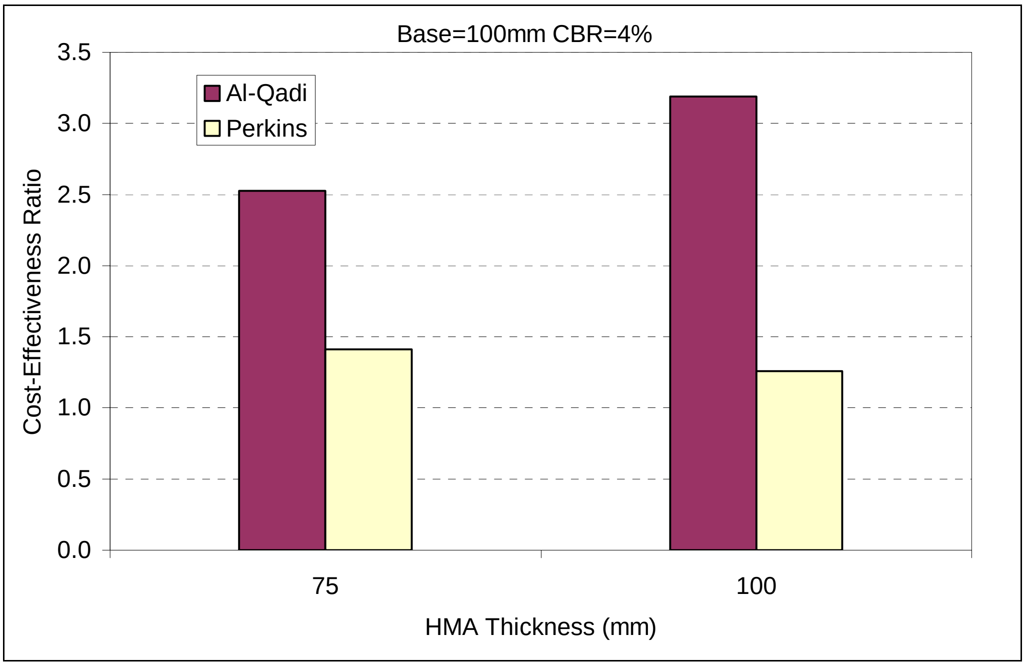

Figure 5-20 Influence of Cost-effectiveness Ratio by HMA Thickness Variation using 100mm Base Layer and Subgrade Strength of 4%

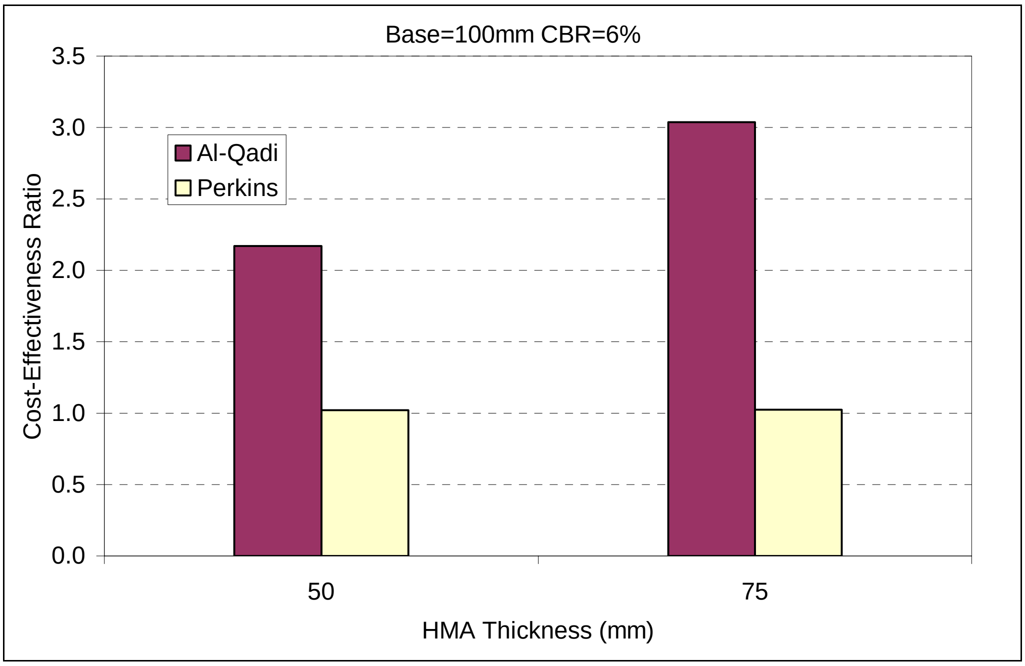

Figure 5-21 Influence of Cost-effectiveness Ratio by HMA Thickness Variation using 100mm Base Layer and Subgrade Strength of 6%

5.4.2 Thickness of Granular Base

The second layer in flexible pavement design is the granular base layer. The following comparison shows the effect of changing the base thickness on the cost-effectiveness ratio. Figure 5-22 to Figure 5-24 illustrate that changing the base layer thickness results in an increase of the cost-effectiveness ratio for both Al-Qadi's and Perkins' design methods. Under the condition of subgrade CBR=0.5% with an HMA thickness of 125mm, there is a slight 8% increment in the cost-effectiveness ratio when Al-Qadi's method is used. Under the same condition, Perkins' design method suggests a 16% increment.

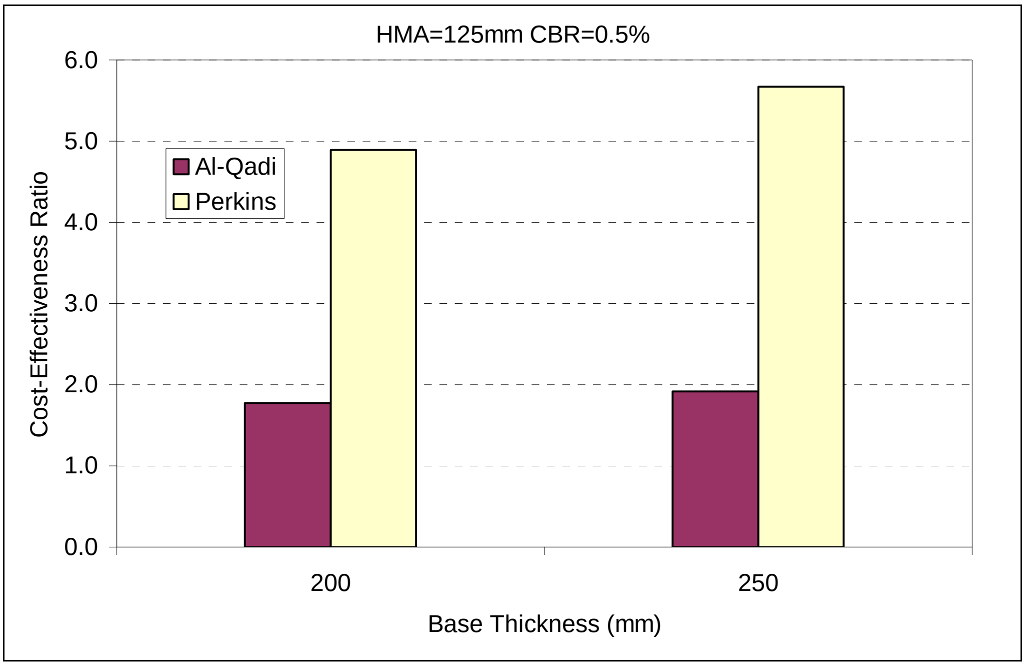

Figure 5-22 Effect of Cost-Effectiveness Ratio by Base Thickness Variation using 125mm HMA Layer and Subgrade Strength of 0.5%

Furthermore, in the Figure 5-23 (a) and (b), the CBR=2% and 4% with HMA of 75mm, the same trend of increasing cost-effectiveness ratio is present for both design methods used. A similar observation can be found in Figure 5-24 where the CBR=4% and 6% with an HMA thickness of 50mm. However, one can notice that when the subgrade is weak, the cost-effectiveness ratio increment obtained from Perkins' design method is greater compared to Al-Qadi's design method.

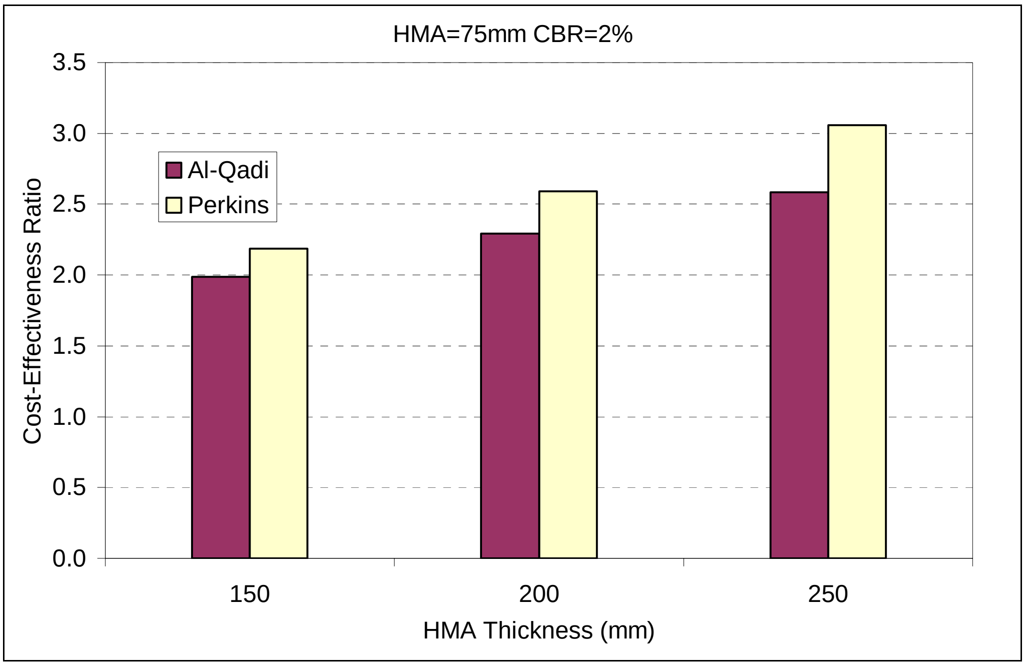

Figure 5-23 (a) Effect of Cost-Effectiveness Ratio by Base Thickness Variation using 125mm HMA Layer and Subgrade Strength of 2%

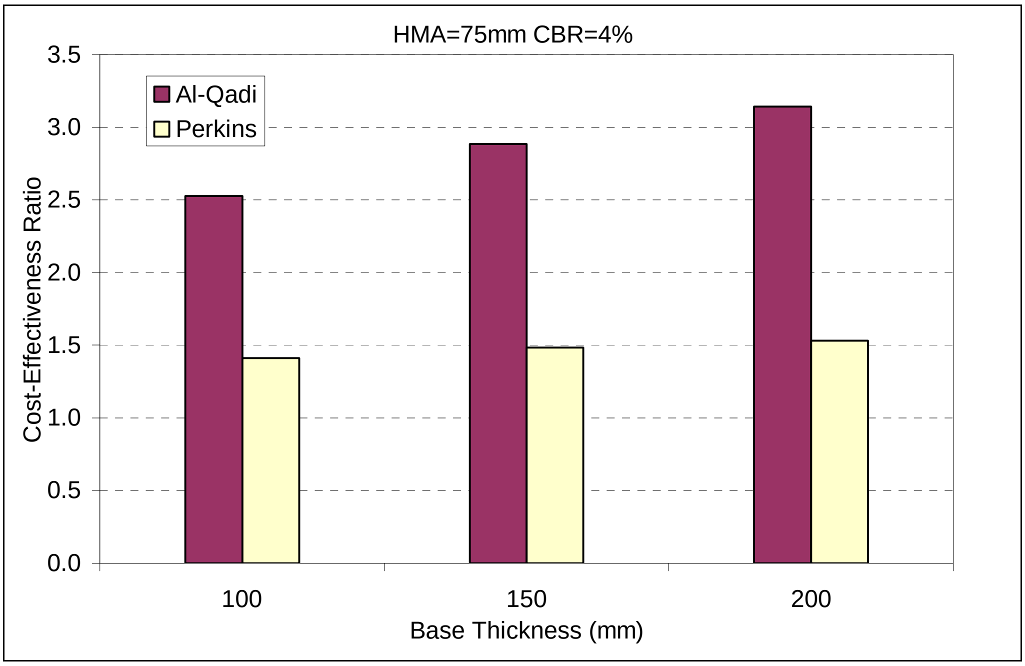

Figure 5-23 (b) Effect of Cost-Effectiveness Ratio by Base Thickness Variation using 125mm HMA Layer and Subgrade Strength of 4%

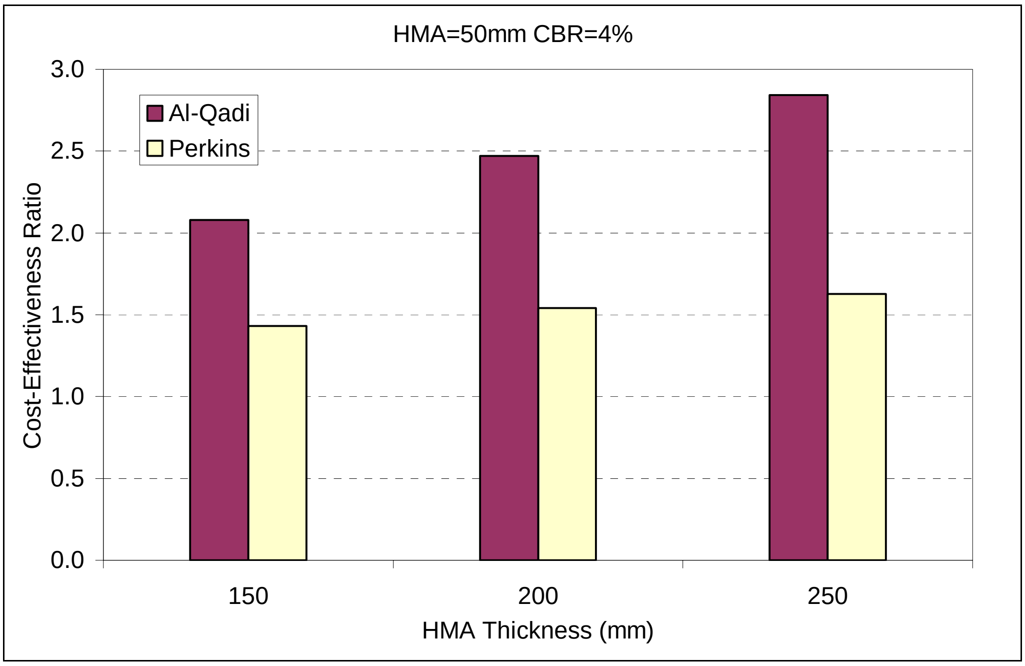

Figure 5-24 (a) Effect of Cost-Effectiveness Ratio by Base Thickness Variation using 125mm HMA Layer and Subgrade Strength of 4%

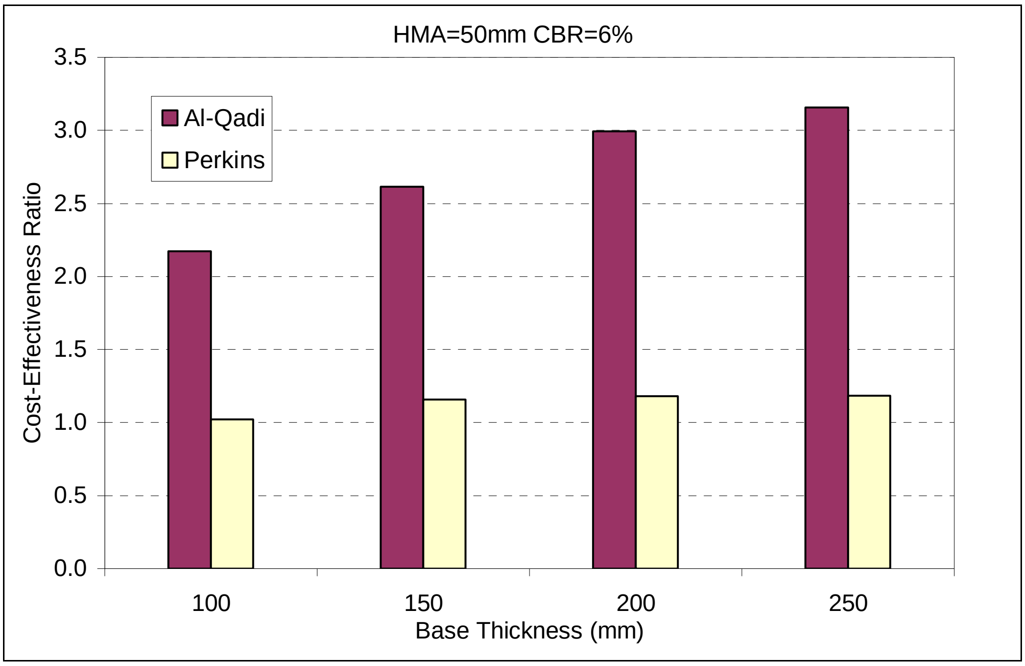

Figure 5-24 (b) Effect of Cost-Effectiveness Ratio by Base Thickness Variation under 125mm HMA Layer and Subgrade Strength of 6%

5.4.3 Structure Number

If the influence of the HMA and the base layer are combined and converted into a single parameter, the structure number (SN), the effect of the variation of the SN on the cost-effectiveness ratio can be seen. In Figure 5-25 to Figure 5-27, the subgrade CBR is varied from 2% to 6%. The plot shows the changing of the SN versus the cost-effectiveness ratio. It is clearly shown that there is no direct proportion between increasing the SN and an increase in the cost-effectiveness ratio.

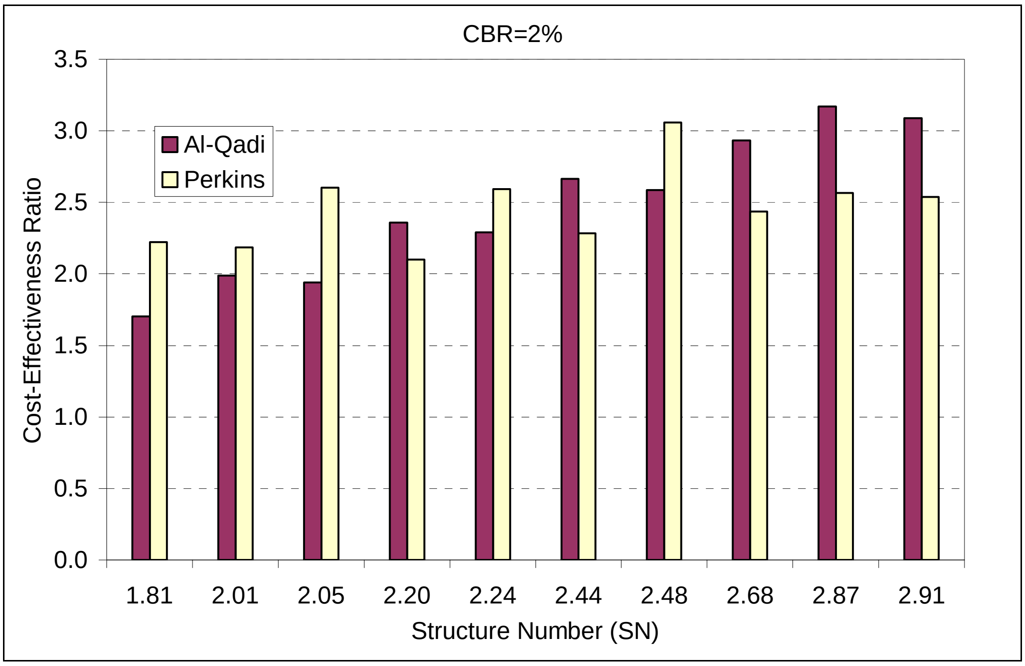

Figure 5-25 Cost-Effectiveness Comparison among Pavement Alternatives with Different Structure Numbers at CBR=2%

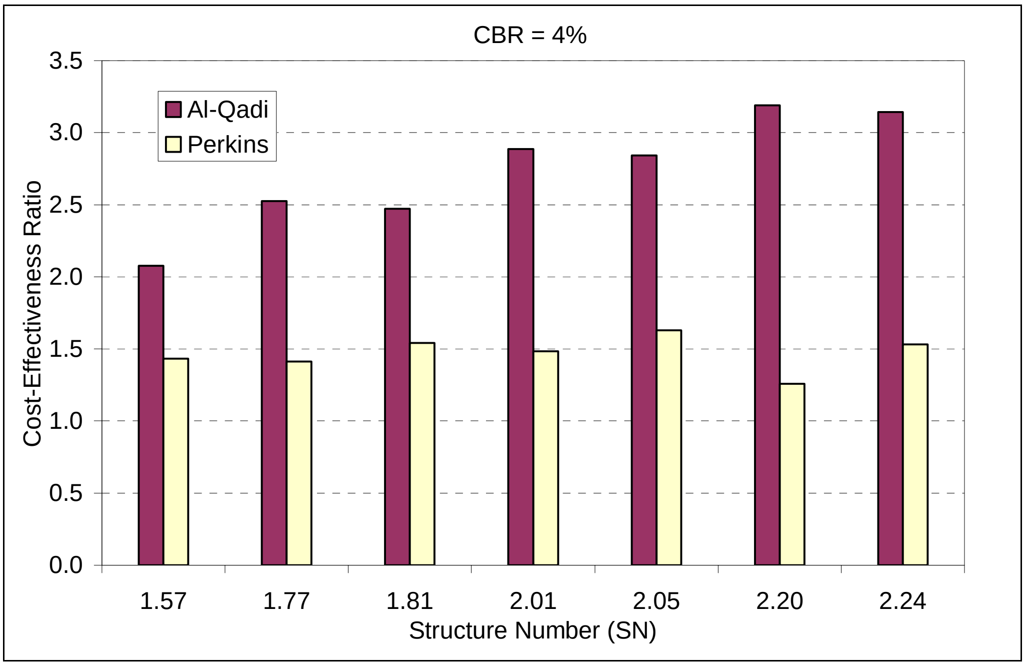

Figure 5-26 Cost-Effectiveness Comparison among Pavement Alternatives with Different Structure Numbers at CBR=4%

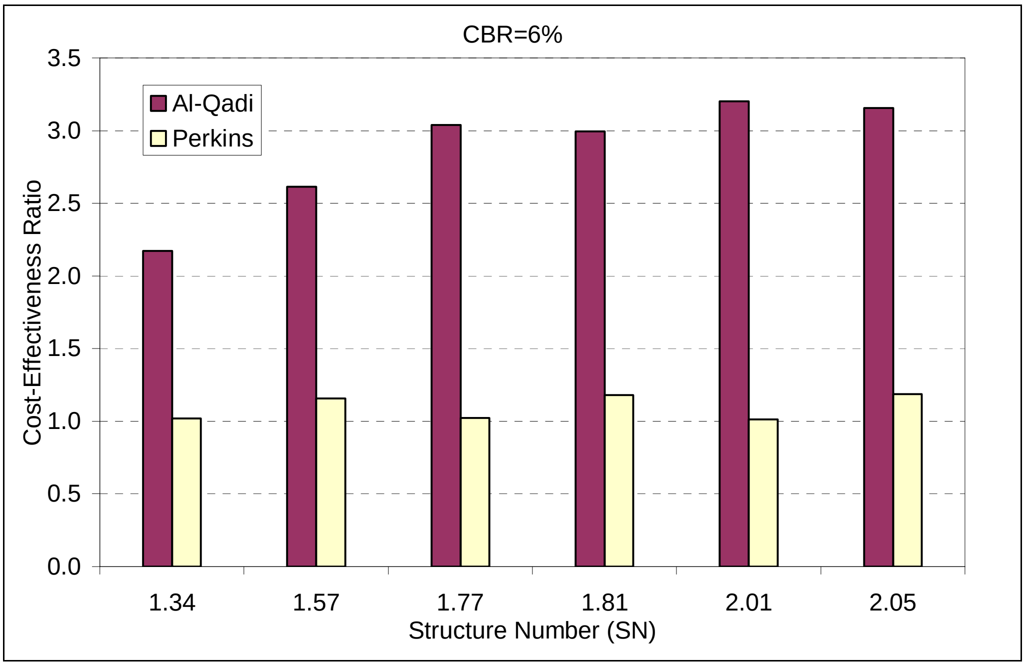

Figure 5-27 Cost-Effectiveness Comparison among Pavement Alternatives with Different Structure Numbers at CBR=6%

5.4.4 Strength of Subgrade

The same pavement design alternative is chosen to see the effect of subgrade strength on the cost effectiveness ratio. For example, when SN=2.05 and 1.81 were reviewed, Al-Qadi's design method shows that by increasing the strength of the subgrade, the cost-effectiveness ratio of the pavement increases. However, Perkins' design method gives the totally opposite result from Al-Qadi's design method. Perkins' method shows that by increasing the strength of the subgrade, the cost-effectiveness ratio of such a pavement alternative will decrease.

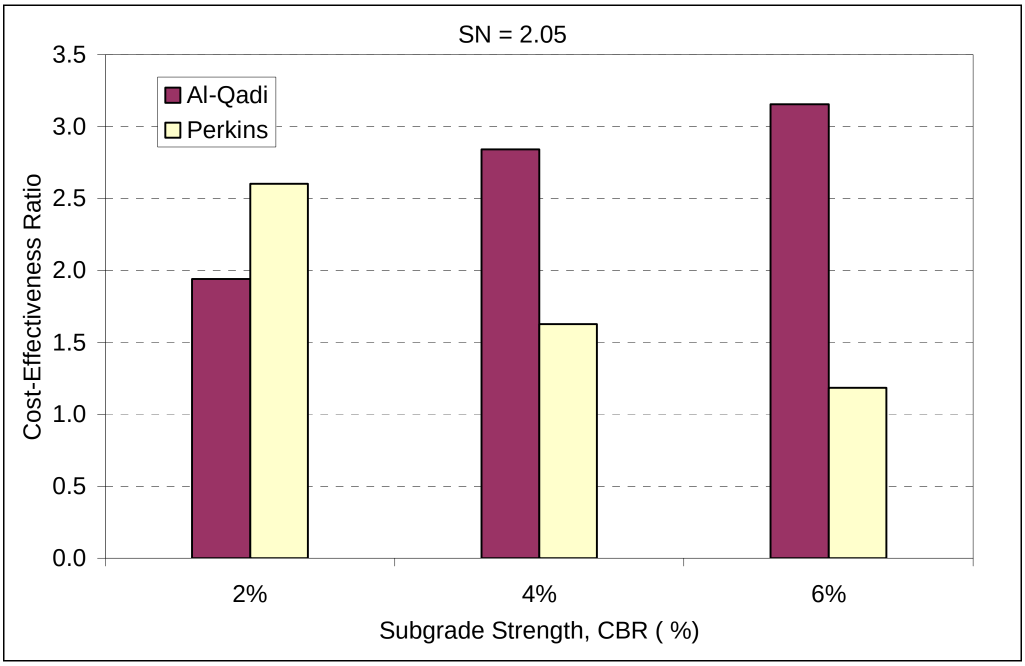

Figure 5-28 Effect of Subgrade Strength on the Ratio of Cost-Effectiveness of the Design Alternative at SN=2.05

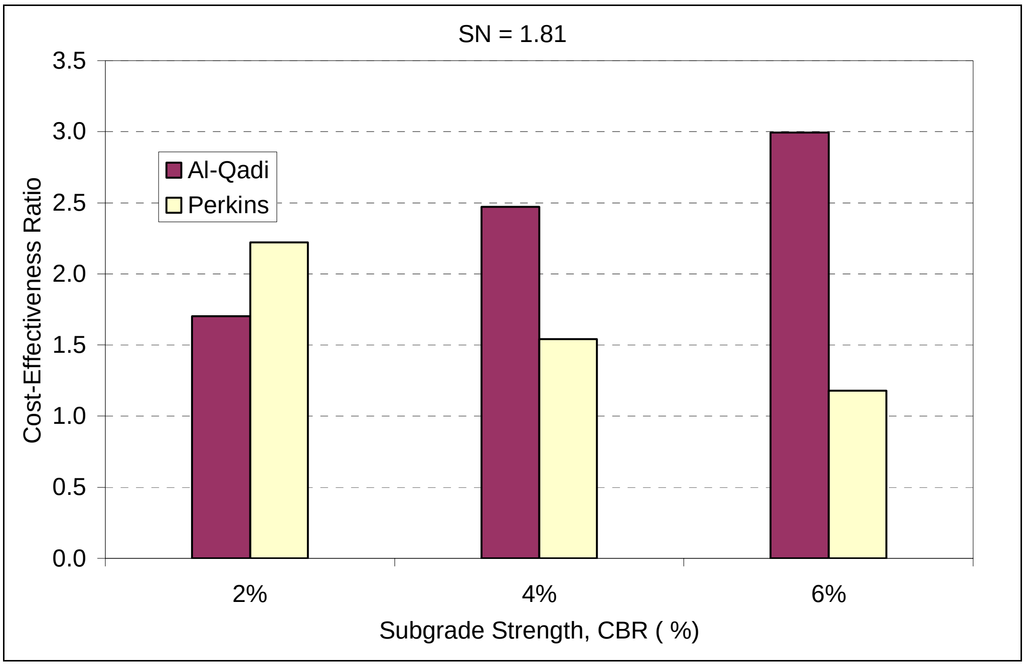

Figure 5-29 Effect of Subgrade Strength on the Ratio of Cost-Effectiveness of the Design Alternative at SN=1.81

5.5 COST-EFFECTIVENESS RATIO PREDICTION

One thing worth investigating is if the traffic benefit ratio obtained from these two design methods can be directly related to the cost-effectiveness ratio for a particular design alternative. In this way, one could use the TBR value to estimate the cost effectiveness ratio of a selected design alternative instead of going through a lengthy LCCA process. To this end, the figures below show the TBR and cost-effectiveness ratio for the representative design alternatives. They show that for both Al-Qadi's and Perkins' design methods; there are no particular trends between the traffic benefit ratio and the cost-effectiveness ratios of a selected design alternative.

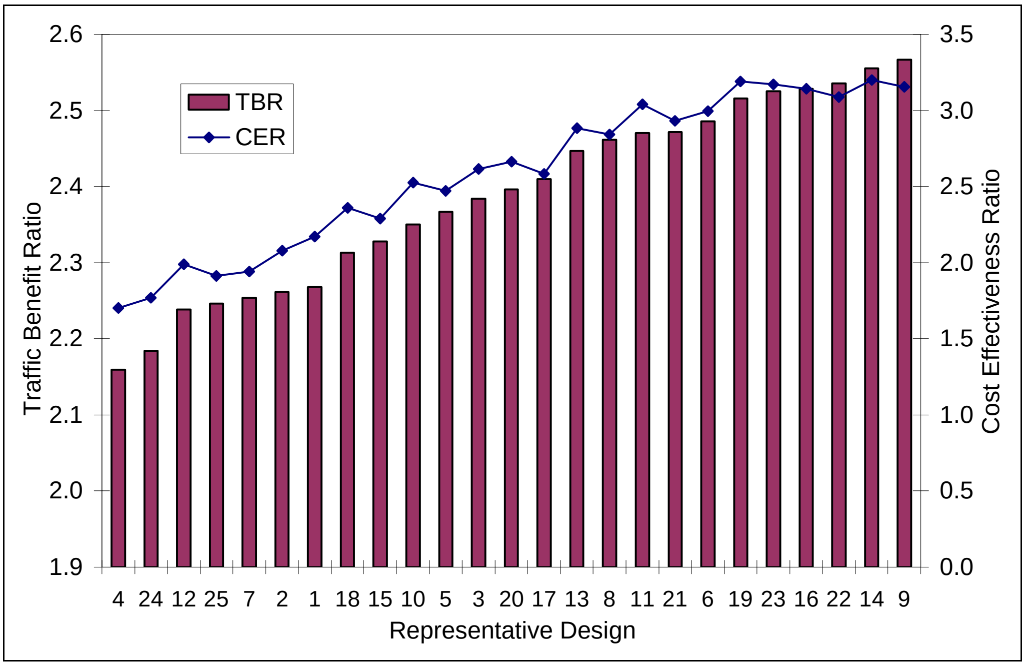

Figure 5-30 Relationship between Traffic Benefit Ratio and Cost-Effectiveness Ratio Using Al-Qadi's Design Method

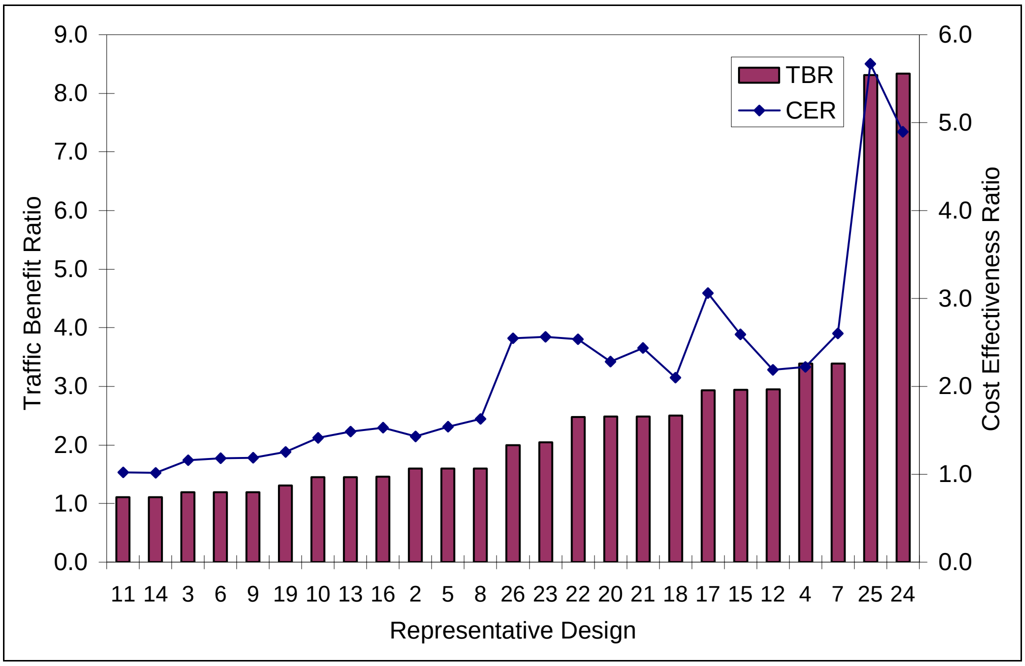

Figure 5-31 Relationship between Traffic Benefit Ratio and Cost-Effectiveness Ratio Using Perkins' Design Method

Another potential design feature of flexible pavement, equivalent single axle load (ESAL), was also considered. The designed ESAL for a particular pavement was used to predict the corresponding cost-effectiveness ratio for the pavement. Figure 5-32 shows the plot of the designed ESAL value versus the cost-effectiveness value using Al-Qadi's design method. The result shows that a simple power law equation can be found to predict the cost-effectiveness ratio of a pavement when the 20 year traffic design alternative is less than 160,000 ESALs. However, if Perkins' design method is used, no feasible model can be found between the designed ESALs and the cost-effectiveness ratio for the pavement, as can be seen in Figure 5-33.

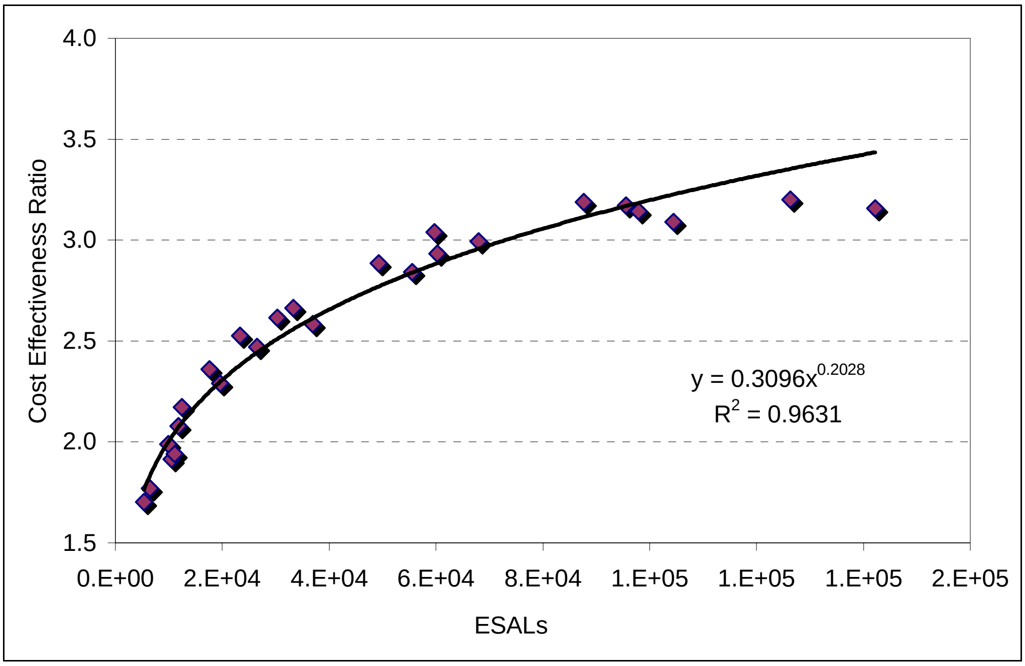

Figure 5-32 Prediction Model for Al-Qadi's Design from the Designed ESAL Value

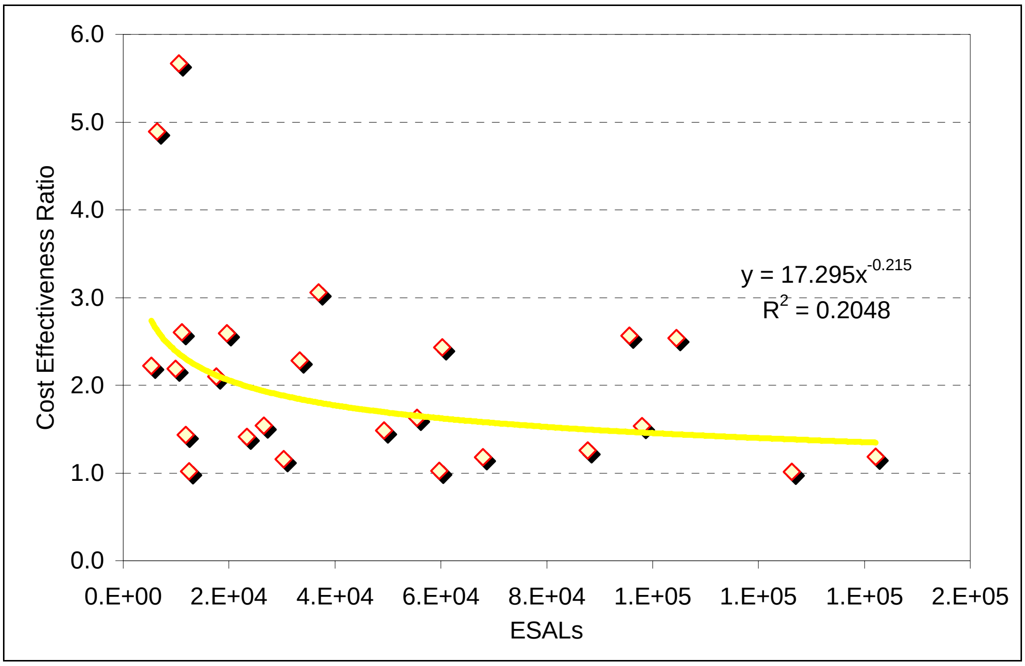

Figure 5-33 Prediction Model for Perkins' Design from the Designed ESAL Value

CHAPTER 6 SUMMARY CONCLUSION AND RECOMMENDATIONS

6.1 SUMMARY

Geotextiles have been used in pavements to either extend the service life of the pavement or to reduce the total thickness of the pavement system. However, the economic benefits of using this material are still not clear. In general, most of the geotextile related life cycle cost analysis studies only account for agency costs. In this study, a comprehensive life-cycle cost analysis of geotextile stabilized pavements, including initial construction, future maintenance, rehabilitation, and user costs is considered.

Two design methods were used to quantify the improvements of using geotextiles in pavements. One was developed at Virginia Tech by Al-Qadi in 1997, and the other was developed at Montana State University by Perkins in 2001. Based on these two methods, the Traffic Benefit Ratio (TBR), defined as the ratio of the number of load cycles needed to reach the same failure state for a pavement section with geotextiles to the number of cycles needed to reach the same failure state for a section without geotextiles, is taken and combined with an AASHTO pavement design method to develop a pavement performance model which can account for the benefits of incorporating geotextiles.

In this study, a comprehensive life cycle cost analysis framework was developed and used to quantify the initial and the future cost of 25 representative design alternatives. A 50 year analysis cycle was used to compute the cost-effectiveness ratio for the design methods. The costs which were considered in the LCCA process include agency costs and user costs.

Four flexible pavement design features were selected to test the degree of influence of the frame's variables. The analysis evaluated these variables and examined their impact on the results. An attempt to predict the cost-effectiveness ratio for a particular pavement from the base input of the design was also made.

6.2 FINDINGS AND CONCLUSIONS

A comprehensive life cycle cost analysis framework has been developed in this study in order to evaluate the economic benefit of using geotextiles in pavement. The resulting benefit has been quantified in terms of a cost-effectiveness ratio. The findings in this study are limited to the design features, unit costs and performance models assumed in this analysis.

During the study several findings were uncovered. Using Al-Qadi's design method, it was found that when the designed ESAL value is greater than 30,000, the user costs will be greater than the agency costs over the 50 year analysis period. However, no such pattern was found when Perkins' design method was adopted. In addition, the results showed that a simple power law equation could be found to predict the cost-effectiveness ratio of a pavement when the 20 year traffic design alternative is less than 160,000 ESALs. However, if Perkins' design method was used, no feasible model could be found between the designed ESALs and the cost-effectiveness ratio.

Three flexible pavement design parameters were also evaluated. The following findings are made with respect to these parameters:

HMA thickness variation was investigated. It was found that at the same subgrade strength and same base thickness, the cost-effectiveness ratio will increase when Al-Qadi's design method is used. However, the percentage increase in cost-effectiveness decreases with an increase in the thickness of the HMA. When Perkins' design method is utilized, an increase in HMA layer thickness would increase the cost-effectiveness of the pavement only if the subgrade CBR is less than 2% with a base layer thinner than 250mm and an HMA thickness less than 100mm.

A granular base thickness increase resulted in an increase in cost-effectiveness in both design methods. However, a greater increase in base thickness obtained a smaller percentage increase in cost-effectiveness.

Structure number was thought to be a common parameter to characterize the strength of pavement. However, during the sensitivity analysis, this feature does not show a regular pattern in the cost-effectiveness ratio among the different structure numbers for both design methods.

Subgrade strength is an important feature in the performance model. When Perkins' design method was used, the stronger the subgrade, the smaller the cost-effectiveness ratio was gained. A similar pattern was found when Al-Qadi's design method was used. However, there was a difference between these two design methods. Although the benefit decreased when the strength of the subgrade increased, overall, Al-Qadi's design method still showed improvement in cost effective value. By contrast, Perkins' design method suggests that with an increase of subgrade strength from CBR=0.5% to 6%, the cost-effectiveness ratio would decrease.

6.3 CONCLUSIONS

Based on the information evaluated and the inputs established for the LCCA over the 50 year analysis period, the following conclusions are made:

For agency costs, Al-Qadi's design method suggests that there is a 20% reduction among the 25 preventative pavement design alternatives, and Perkins's design method gives from no cost reduction to a 40% cost reduction.

For user costs, Al-Qadi's design method suggests a 70% cost reduction among the 25 representative pavement design alternatives. Perkins' design method gives from almost no cost reduction to a 100% cost reduction.

The cost effectiveness ratio from the two design methods shows that the lowest cost-effectiveness ratio using Al-Qadi's design method is 1.7 and the highest is 3.2. The average is 2.6. For Perkins' design method, the lowest value is 1.01 and the highest value is 5.7. The average is 2.1.

6.4 RECOMMENDATIONS FOR FUTURE WORK

This study is mainly intended to propose a cost-effectiveness analysis framework that can be used to operate similar analyses. It presented a framework on how to quantify engineering benefits into economic benefits by performing life cycle cost analyses. Included in this study are models that predict pavement performance, suggest rehabilitation designs, and predict user costs and accident rates at work zones. Many of these models need to be improved by using more reliable parameters and be calibrated to specific local conditions. This is especially applicable to the pavement performance models. Research should be undertaken to replace these models and to improve the predictive qualities of the framework using mechanistic approach.

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