ADDIS ABABA UNIVERSITY

Addis Ababa institute of Technology

School of Civil & Environmental Engineering

Comparative Study of Flexible and Composite Pavement Structures

(A Case Study on Addis Ababa - Djibouti Trunk Road Segment)

By

Bereket Tesfaw

A Thesis submitted to School of Graduate Studies in partial fulfillment of the requirements for Degree of Masters of Science in Civil Engineering (Road and Transport Engineering)

Advisor: ***- Dr. Habtamu Melese***

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Addis Ababa, Ethiopia

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DECLARATON

I, the undersigned, declare that this thesis titled "***Comparative Study of Flexible and Composite Pavement Structures*** **(**A Case Study on Addis Ababa - Djibouti Trunk Road Segment)" is my own work performed under the supervision of my research advisor ***Dr. Habtamu Melese.*** The work has not been presented elsewhere for assessment and award of any degree or diploma. All sources of materials used for this thesis have also been duly acknowledged/referred.

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ABSTRACT

Most major road networks in Ethiopia constructed with flexible pavement, this type of pavement used as the only option considered during the pavement type selection (PTS) process for many years. Composite pavement, which is a combination of a rigid base overlaid with a hot-mix asphalt (HMA) surface course, is proposed to be considered as alternative during PTS process in this study. The thesis seeks to find out the better option between **flexible** and **composite pavement** in levels of performance both structurally (technical aspect) and economically viable alternative (economic aspect) over the analysis period of time for Addis Ababa - Djibouti trunk road segment.

The thesis covers design of both flexible and composite pavement, technical analysis and computation of respective costs over an analysis period of 40 (forty) years. Appropriate input parameters used for empirical method of design, linear elastic analysis using Kenpave software and deterministic life cycle cost analysis (LCCA); and the output expressed with graphs and charts. The data required for this study was obtained from journals, design manual, books and reports of various organizations. Material and traffic data used to come up with the pavement design details which, gives respective pavement layers thickness and type of materials used for each layer. These design outputs ultimately used for pavement structures economical and structural comparison.

A mechanistic analysis based on the multi-layer linear elastic theory was performed on different composite structures to understand the structural responses (stress, strain and deflection) and compared with flexible pavement. At the economic level, a deterministic LCCA was performed. Results from technical analysis of this study suggest that composite pavements have potential by mitigate various structural and functional problems that typical flexible pavement tend to present, such as HMA flexural fatigue failure, HMA rutting due to subgrade vertical deformations. The results of the deterministic LCCA suggest that the use of a composite pavement results in cost-effective alternative than flexible pavement. So, composite pavements have structural and economic potential to be considered during the pavement type selection process for major roads.

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ACRONYMS

AADT	Annual Average Daily Traffic
AASHTO	American Association of State Highway and Transportation Officials
AC	Asphalt Concrete
ADT	Average Daily Traffic
AI	Asphalt Institute
CBM	Cement-Bound Material
CBR	California Bearing Ratio
CRCP	Continuously Reinforced Concrete Pavement
CTA	Cement-treated Aggregate
CTB	Cement-treated Base
ERDC	Engineering Research and Development Center
ESAL	Equivalent Single-axle Load
EUAC	Equivalent Uniform Annual Cost
GB	Granular Base
HMA	Hot-mix Asphalt
JRCP	Jointed Reinforced Concrete Pavement
LC	Lean Concrete
LCCA	Life Cycle Cost Analysis
LEA	Linear Elastic Analysis
M&R	Maintenance and Repair
MR	Modulus of Rupture
MSA	Million Single Axles
PCC	Portland cement concrete
PCCP	Portland cement concrete pavement
PSI	Present Serviceability Index
PTS	Pavement Type Selection
PW	Present Worth
RCC	Roller-Compacted Concrete
RUC	Road User Cost(s)
SC	Soil Cement
VDOT	Virginia Department of Transportation
VOC	Vehicle Operating Cost(s)

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1.0 INTRODUCTION

1.1 Background

Road transport is the dominant mode of transport in Ethiopia. The road network plays a significant role in the economy of the country by providing safe freight and passenger movement from one place to the other. The road may be granular, flexible, rigid or composite pavement based on the material used to construct the road. Granular pavement usually used for low traffic volume, the other pavement types recommended for medium and high traffic volume.

Rigid and composite pavements have often been considered for high volume and heavily trafficked roads, because their initial construction cost is very high and their excellent traffic carrying capacity. As a result, the major road network in Ethiopia constructed with flexible pavements. However, several countries have adopted rigid and composite pavements more widely. This is because, they last a long time and their maintenance demands are low so that, in whole life cost terms, they can provide good value for money. Naturally this depends on the relative costs of the materials.

The two most commonly designed and constructed pavement types are flexible and rigid in the road industry throughout the world. In pavement type selection (PTS) process a transportation agency should select the best pavement type for a particular project.

In Ethiopia, almost all highway projects constructed with flexible pavement. Flexible pavement has its own advantage (low initial cost and easy for construction) and disadvantage(less durable and high maintenance demand relative to the others) in different conditions. Other pavement alternatives (rigid and composite pavements) didn't get attention even if the traffic volume increase in higher rate in Ethiopia, as they have advantages over the flexible pavement in some conditions like in case of high traffic volume, heavy traffic, static loading condition. So, the rigid and composite pavements should be considered as alternative during pavement type selection process for particular project. Project with rigid pavement under construction which is found in Afar region (Ditchoto Gulafi Junction – Elidar –Belecho Design-Build Project) and, Oromiya region (Chancho- Derba-Becho rigid project and trial section rigid pavement located at Beseka lake around Methihara) are the initial rigid road sections for future consideration and experience of the country on rigid pavement. The other alternative i.e. composite pavement, which doesn't construct yet in Ethiopia, is the main concern of this study.

A composite pavement, which is a combination of flexible and rigid layers, will potentially meet all design considerations and become a feasible alternative if it is considered during pavement type selection process. Technical and economic evaluations should be performed to justify the consideration of composite pavements in the PTS process. At the technical level, a mechanistic analysis conducted on different composite structures to understand the behavior of composite pavements compared to flexible structure. The multi-layer linear elastic theory used to investigate the behavior and interaction of the flexible and rigid layers of the composite structure. At the economic level, by using a deterministic life cycle cost analysis (LCCA) the cost of flexible pavement and composite pavement computed.

Addis Ababa - Djibouti road section selected for this study because of high volume, heavy traffic load and need frequent maintenance in existing condition, as it has being used as main highway for import and export of freight for Ethiopia. From this road section, segment of the road section which will require consistent pavement layer thickness which can provide sufficient structural support during the expected design period for a given traffic load used for this study.

1.1.1 Location and project Background

Addis Ababa to Djibouti road section is located in the eastern part of the Country which has the highest traffic volume and load in the Country as it is the main corridor of port Djibouti. Large portion of the traffic is trucks with heavy load, transporting import and export goods. One of the lanes has been damaged severely as is serves for the import trucks of which almost all the heavily loaded.

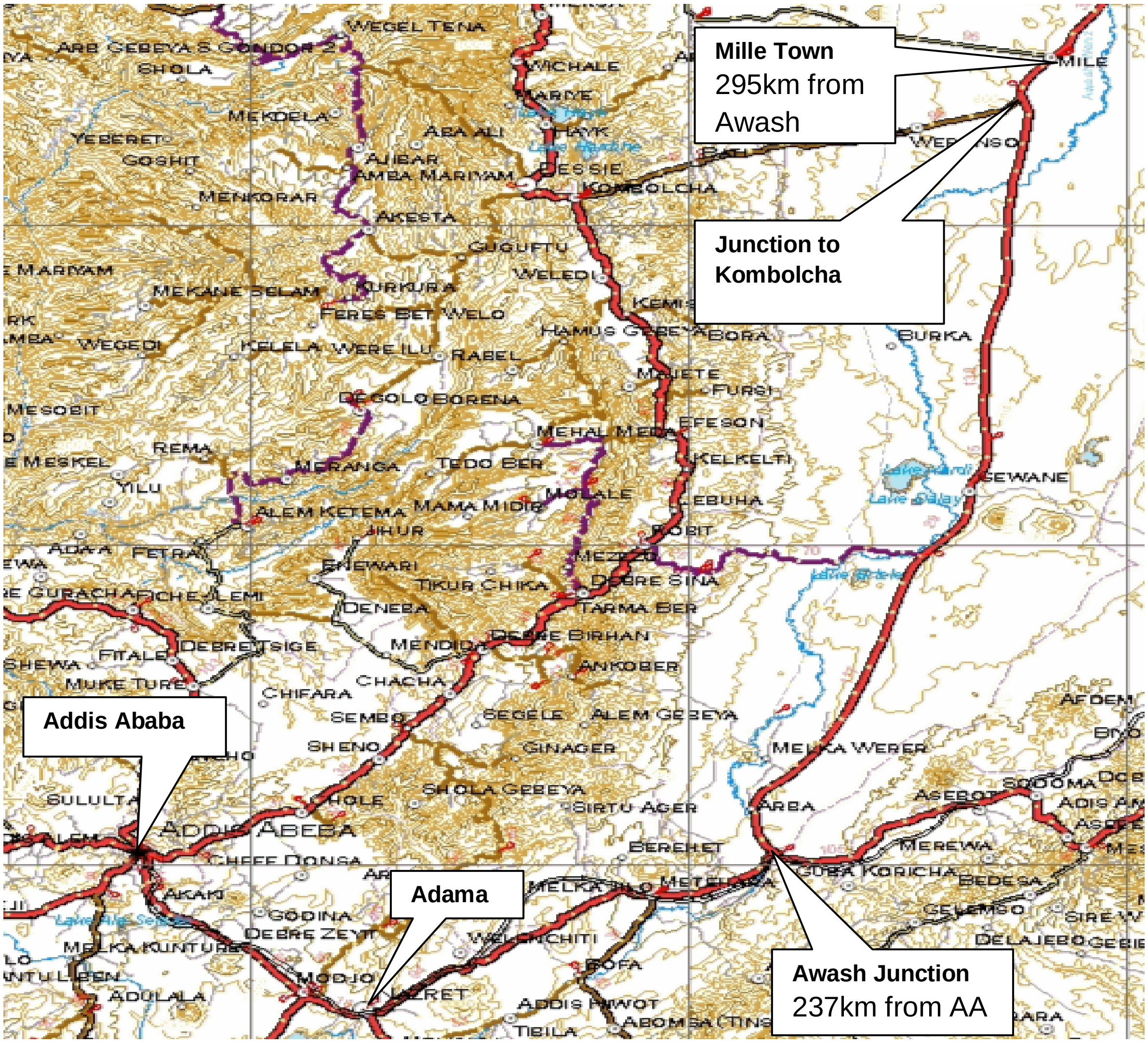


Figure 1.1: Location of study area

Part of the road section, Addis Ababa to Awash, was previously constructed by the Contractor Keangnam Enterprise of Republic of South Korea and supervised by a Consultant Carl Bro in 2002 and it has served for the 8 years, after that the road was not in a good condition and is under rehabilitation i,e overlay since 2011 to bring the required level of service of the road.

The road alignment traverses dominantly through flat to rolling terrain. Generally it gently descends from the starting point to the end point of the project road length. The following table describes the general terrain classification of Adama-Awash road project.

Table 1.1: Terrain Classifications

No	Terrain Type	Coverage (%)
1	Flat	59.38
2	Rolling	40.62
3	Mountainous	0

1.1 Statement of the Problem

Flexible pavement is the only alternative used as highway infrastructure for many years in Ethiopia. However, other pavement options should become available for pavement engineers to consider. Composite pavements, which are a combination of a rigid base layer and a flexible surface course, are increasingly considered a viable alternative on high-volume highway projects worldwide. To assist pavement engineers and transportation agencies in their consideration of this pavement type during the pavement type selection process, a technical and economic study should be performed to understand the advantages, limitations, and feasibility of designing and constructing a composite pavement.

1.2 Research Objective

This study aims to propose the consideration of composite pavements if it is feasible as alternative in the pavement type selection process, supported by a technical and economic investigation.

Specific objectives

- ✓ Design pavement structures
- ✓ Conduct technical analysis to know mechanistic behavior (deflection, stress and strain) of pavement structures
- ✓ Compute cost of the pavements using a deterministic life cycle cost analysis (LCCA) to investigate the economic aspects

1.3 Scope and Limitation of the Study

The study carried out to identify potential of composite pavement by conducting Technical and Economic analysis.

The challenges and the limitation that reveal are;

Challenges

- Getting input parameters and properties to design and analyze different types of pavements was challenging.
- There is no design standard, work schedule and cost of composite pavements in Ethiopian Road Authority manual. So it was the challenge for this study.

Limitation

An engineering analysis such as environmental pollution, non-user impacts such as congestion, tire pavement noise, and road safety for composite pavements is beyond the scope of this study, because some of them are difficult to quantify and collect data.

Cost of composite pavement computed using unit rate cost of "Ditchoto Gulafi Junction – Elidar –Belecho Design-Build Project" rigid pavement under construction in Afar region because, cost breakdown and structural detail of 1 km trial section rigid pavement located at Beseka Lake around Methihara town can't found from client, contractor and consultant side. This may cause cost variation.

1.4 Significance of the Study

This study attempts to contribute to the efforts of development practitioners, project planners and academic accomplishments.

- ✓ Present potential of composite pavements, the findings may encourage and influence transport agencies and project planner to consider it during the pavement type selection process.
- ✓ Enhance the understanding of the performance of composite pavement systems.
- ✓ Through mechanistic modeling, the pavement responses at various stiffness levels of the base course underneath an HMA layer studied.
- ✓ The behavior of the pavement structure and distribution of stresses and strains at different depths investigated.
- ✓ Perform LCCA to understand and assess the economic implications of implementing a composite pavement alternative to typical flexible.

2.0 LITERATURE REVIEW

2.1 Introduction

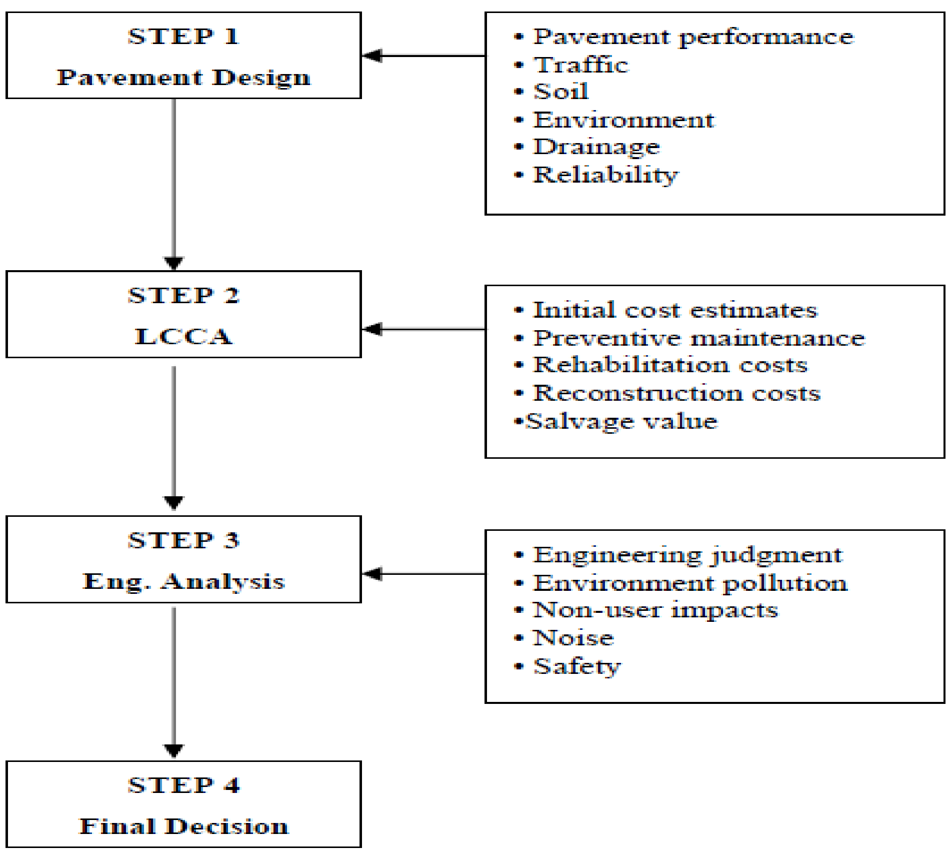
The pavement type selection (PTS) process involves a thorough analysis of the pavement alternatives available for a project. The PTS procedure determines the most cost-effective pavement type capable of supporting the predicted traffic under the prevailing environmental conditions while contributing to safety and driver comfort. When performing a PTS, three primary areas need to be addressed: pavement design analysis, life cycle cost analysis (LCCA), and engineering analysis. Each of these aspects has significant impact on PTS decision making.

The first step in the PTS flowchart involves the review and analysis of several pavement design factors such as sub-grade, traffic, materials, climate/drainage, environment, and construction. In this step, the pavement design is performed following a standard design methodology, either empirically or mechanistically based. Some of the input parameters during the pavement design stage have a higher impact on the final alternatives to be considered. For example, correctly estimating the traffic volume is crucial when selecting an appropriate pavement; good historical records and an accurate prediction of future traffic is of great importance when designing any pavement alternative.

The life cycle cost analysis (LCCA) step provides the pavement engineer with economic models and tools to determine the most cost-effective pavement alternative. Life cycle costs refer to all pavement costs involved with construction, maintenance, rehabilitation, and user-related costs over a given analysis period. They are also defined as the total cost of a pavement alternative until a major reconstruction occurs; at this point, a new life cycle starts. The engineering analysis step involves considerations that are not accounted in the pavement design or LCCA steps such as environmental pollution, non-user impacts such as congestion, tire pavement noise, and road safety. After the engineering analysis step, a decision has to be made considering the technical factors (pavement design,) and the economic factors (LCCA,). This decision should also consider engineering judgment, environmental concerns, non-user impacts, noise, and safety. Even though it is challenging to quantify some of these factors, an effort should be made to minimize negative impacts on environmental and safety factors, among others.

2.2 Pavement Type Selection (PTS) Process

The first step in the PTS flowchart shown in Fig 2.1 involves the review and analysis of several pavement design factors such as subgrade, traffic, materials, climate/drainage, environment, and construction (WSDOT, 2005). In this step, the pavement design is performed following a standard design methodology, either empirically or mechanistically based. Some of the input parameters during the pavement design stage have a higher impact on the final alternatives to be considered. Good historical records and an accurate prediction of future traffic are of great importance when designing any pavement alternative.



Source: WSDOT 2005

Figure 2.1: Typical PTS Flowchart

The life cycle cost analysis (LCCA) step provides the pavement engineer with economic models and tools to determine the most cost-effective pavement alternative. Life cycle costs refer to all pavement costs involved with construction, maintenance, rehabilitation, and user-related costs over a given analysis period. They are also defined as the total cost of a pavement alternative until a major reconstruction occurs; at this point, a new life cycle starts (WSDOT, 2005). The engineering analysis step involves considerations that are not accounted in the pavement design or LCCA steps such as environmental pollution, non-user impacts such as congestion, tire-pavement noise, and road safety. After the engineering analysis step, a decision has to be made considering the technical factors (pavement design) and the economic factors (LCCA). This decision should also consider engineering judgment, environmental concerns, non-user impacts, noise, and safety. Even though it is challenging to quantify some of these factors, an effort should be made to minimize negative impacts on environmental and safety factors, among others. An engineering analysis of pavements is beyond the scope of this study.

2.2.1 Pavement Structure in PTS process

The two most prevalent pavement structures considered as the main options during the PTS process are flexible and rigid pavements. Each of these structures differs from the other in several aspects, including design procedure, service life, deterioration (distress models), construction, and rehabilitation options, among others. As composite pavement is combination of these two pavement structures, it is necessary to refer about flexible and rigid pavement structures.

2.2.1.1 Flexible Pavement Structure

Flexible pavements are constructed of bituminous and granular materials (Huang, 2004). Their structure is composed of several layers that gradually distribute loads from the hot-mix asphalt (HMA) surface layer to the layers underneath (e.g., granular base, subbase, subgrade). A typical flexible pavement cross-section is shown in Fig. 2.2.

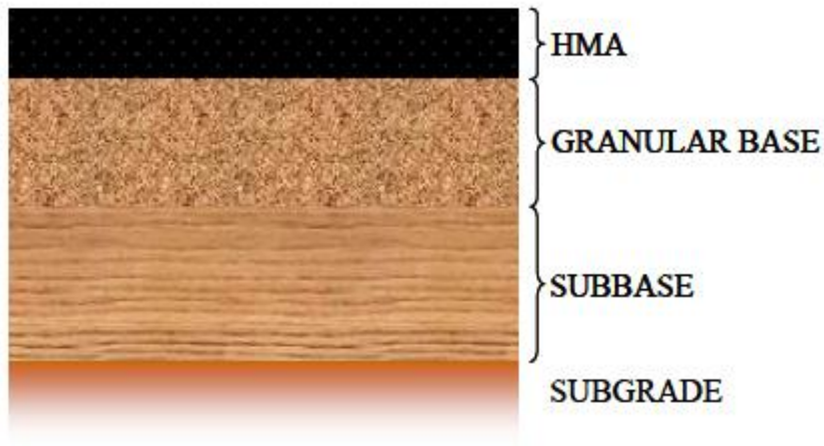


Figure 2.2: Typical Flexible Pavement Structure

Traffic and subgrade property are two important pavement design parameters to design flexible pavement structure using ERA manual and AASHTO 1993 Guide design procedure uses various input parameters to obtain a structural number (SN), which then is used to compute the minimum thicknesses of each of the structural layers that would satisfy the traffic demand of the road. The SN is a method used to quantify the structural capacity of the pavement system.

The surface course material (i.e., HMA) used in composite pavements is the same as that used in purely flexible pavements. As a result, the potential distresses that affect the HMA surface layer are typically investigated using various transfer functions (distress models), which relate structural responses to various types of distresses (Huang, 2004). The two most commonly used transfer functions for HMA are fatigue cracking and rutting.

2.2.1.1.1 ERA Flexible Pavement Design

To design flexible pavement structure using Ethiopian Road Authority (ERA) design manual, two important pavement design parameters are traffic (vehicle class, axle load, volume) which can use the road section and subgrade property along the route. Traffic should be estimated in terms of the cumulative number of equivalent standard axles that will use the road over the selected design life and subgrade soil strength should be characterized over which the road is to be built. The strength of the road subgrade for flexible pavements is commonly assessed in terms of the California Bearing Ratio (CBR) and this is dependent on the type of soil, its density, and its moisture content. Using the input data obtained from traffic and Subgrade to select a suitable structure from the catalogue of pavement structures presented in manual.

Determining an appropriate design period is the first step towards pavement design. Many factors may influence this decision, including budget constraints. However, the designer should follow certain guidelines in choosing an appropriate design period, taking into account the conditions governing the project. Some of the points to consider include:

- Functional importance of the road
- Traffic volume
- Location and terrain of the project
- Financial constraints
- Difficulty in forecasting traffic

According to ERA manual the design period for each functional classification of the road is shown in table below:

Table 2.1: Design period

Road Classification	Design Period (years)
Trunk Road	20
Link Road	20
Main Access Road	15
Other Roads	10

Source: ERA manual 2013

Pavement design requires predicting the expected level of traffic over the design period. There are many vehicles which have different number of axle, axle configuration and axle load. Vehicles having different load and axle configuration affect the pavement differently so, it is necessary to analyze the traffic within some group of vehicle class. Finally the total damage caused by total traffic estimated.

In order to determine the total traffic over the design life of the road, the first step is to estimate initial traffic volumes. The estimate should be the Annual Average Daily Traffic (AADT) currently using the route (or, more specifically, the AADT expected to use the route during the first year the road is placed in service) classified into the thirteen classes of vehicles described above. Adjustments will usually be required between the AADT based on the latest traffic counts and the AADT during the first year of service.

The AADT is defined as the total annual traffic summed for both directions and divided by 365. It is usually obtained by recording actual traffic volumes over a shorter period from which the AADT is then estimated. It should be noted that for structural design purposes the traffic loading in one direction is required and for this reason care is always required when interpreting AADT figures. For long projects, large differences in traffic along the road may make it necessary to estimate the flow at several locations.

The analysis and forecasting of traffic along the proposed road project shall constitute the detailed review and synthesis of national, regional and local socio- economic and other data in relation to the project influence area, in order establish robust traffic growth factors.

Even with stable economic conditions, traffic forecasting is an uncertain process. Although the pavement design engineer may often receive help from specialized professionals at this stage of the traffic evaluation, some general remarks are in order. The most common method of forecasting normal traffic is to extrapolate data on traffic levels and assume that growth will either remain constant in absolute terms i.e. a fixed number of vehicles per year or constant in relative terms i.e. a fixed percentage increase. As a general rule it is only safe to extrapolate forward for as many years as reliable traffic data exist from the past, and for as many years as the same general economic conditions are expected to continue.

As an alternative to time, growth can be related linearly to anticipate Gross Domestic Product (GDP). This is normally preferable since it explicitly takes into account changes in overall economic activity. GDP of Ethiopia from 2006 to 2013 is shown in table below:

Table 2.2: GDP of Ethiopia from 2006 - 2013

Year	2006/07	2007/08	2008/09	2009/10	2010/11	2011/12	2012/13
GDP	11.8	11.2	10	10.4	11.4	8.8	9.7

Source: IMF 2014

From the table an average growth rate of 10.5% is achieved,

This is for computation of the average traffic growth rates and for traffic volume projections.

To forecast traffic growth it is usually necessary to separate traffic into the following three categories:

i. Normal Traffic: traffic which would pass along the existing road or track even if no new pavement were provided.

ii. Diverted traffic: traffic that changes from another route (or mode of transport) to the project road because of the improved pavement, but still travels between the same origin and destination.
iii. Generated traffic: additional traffic which occurs in response to the provision or improvement of the road.

If it is thought that a particular component of the traffic (e.g. a category of trucks, due to the development of an industry) will grow at a different rate to the rest, it should be specifically identified and dealt with separately, i.e. a uniform growth rate among the various traffic classes should not necessarily be assumed a priori.

The damage that vehicles do to a paved road is highly dependent on the axle loads of the vehicles. For pavement design purposes the damaging power of axles is related to a "standard" axle of 8.16 metric tons using empirical equivalency factors. In order to determine the cumulative axle load damage that a pavement will sustain during its design life, it is necessary to express the total number of heavy vehicles that will use the road over this period in terms of the cumulative number of equivalent standard axles (ESAs).

Axle loads can be converted and compared using standard factors to determine the damaging power of different vehicle types. A vehicle's damaging power, or Equivalency Factor (EF), can be expressed as the number of equivalent standard axles (ESAs), in units of 80kN. The design lives of pavements are expressed in terms of the ESAs they are designed to carry.

Axle load surveys must be carried out to determine the axle load distribution of a sample of the heavy vehicles using the road. Data collected from these surveys are used to calculate the mean number of EF for a vehicle in each class. These values are then used in conjunction with traffic volume forecasts to determine the predicted cumulative equivalent standard axles that the road will carry over its design life.

On certain roads it may be necessary to consider whether the axle load distribution of the traffic travelling in one direction is the same as that of the traffic travelling in the opposite direction. Significant differences between the two streams can occur on roads serving ports, quarries, cement works, etc., where the vehicles travelling one way are heavily loaded but are empty on the return journey. In such cases the results from the more heavily trafficked lane should be used when converting volumes to ESA for pavement design.

Once the axle load data has been gathered, the mean equivalency factor for each class of vehicle must be calculated.

The number of equivalent factor (ef) of an axle is related to the axle load as follows:

$ef = (L/8160)^n$ (for loads in kg)………………..…….. equation 2.1

or $ef = (L/80)^n$ (for loads in kN)………………………… equation 2.2

Where:

ef = equivalent factor , L = axle load (in kg or kN) and n = damage exponent (n = 4.5).

The sum of the individual *ef* values for each axle of the vehicle gives the equivalence factor for the vehicle as a whole, EF(*v*). Guidance on the likely average EF(*v*) for different vehicle classes derived from historical data is given in Table 2.8 of ERA flexible pavement design manual 2013. However, ERA recommends using data from any recent axle load survey on the road in question or a similar road in the vicinity is better than using countrywide averages.

The cumulative ESAs over the design period for each vehicle class 'v' is obtained by multiplying EF(*v*) by the cumulative traffic, T(*v*). The total number of cumulative standard axles for all vehicle classes is then obtained by adding together the values of EF(*v*) x T(*v*) for all the classes.

In some cases there will be distinct differences in each direction and separate vehicle damage factors for each direction should be derived. The higher of the two directional values should be used for design.

On narrow roads the traffic tends to be more channelized than on wider two-lane roads. In such cases the effective traffic loading is greater than that for a wider road and the design traffic loading is calculated using the relationships given in Table 2.4.

The pavement design thicknesses required for the design lane are usually applied to the whole carriageway width.

Table 2.3: Factors for design traffic loading

Cross section	Paved width	Corrected traffic design loading (ESA)	Explanatory notes
Single carriageway	<3.5m	Double the sum of ESAs in both directions	The driving pattern on this cross-section is highly channelized.
	Min. 3.5m but less than 4.5m	The sum of ESAs in both Directions	Traffic in both directions uses the same lane
	Min. 4.5m but less than 6m	80% of the ESAs in both Directions	To allow for overlap in the centre section of the road
	6m or wider	Total ESAs in the heaviest loaded direction	Little traffic overlap in the centre section of the road.
More than one lane in each direction		90% of the total ESAs in the studied direction	The majority of vehicles use one lane in each direction.

Source: ERA 2013

Accurate estimates of cumulative traffic are difficult to achieve due to errors in the surveys and uncertainties with regard to traffic growth, axle loads and axle equivalencies. To a reasonable extent, however, pavement thickness design is not very sensitive to cumulative axle loads and the method recommended in ERA manual provides fixed structures of paved roads for ranges of traffic.

The subgrade is the supporting ground beneath a pavement structure. It is located below the base and sub-base courses. It is usually investigated to such depth as may be important to structural design and pavement life, and it may consist of materials forming the natural ground surface or exposed in excavations. In a fill section, the subgrade is the upper part of the embankment.

The California Bearing Ratio (CBR) is an indirect measure of the strength of the Subgrade. It is also the most widely used method for designing pavement structures. The higher the CBR value of a Subgrade, the more strength it has to support the pavement. This means that a thinner pavement structure could be designed on a Subgrade with higher CBR compared to a lower CBR value. However, it should be noted that although the CBR value is directly correlated with strength, the change in pavement thickness needed to carry a given traffic load is not directly proportional to the change in CBR value of the Subgrade soil.

Sands and gravels with high CBR values are often considered as the best Subgrade for formation. In general, a Subgrade having a CBR of 10 or greater can usually support heavy loads and repetitious loading without excessive deformation. The CBR value of the Subgrade can be correlated with resilient modulus and other engineering properties.

The Subgrade soil stiffness is measured by resilient modulus (MR). MR of a material is an estimate of its modulus of elasticity (E). While the modulus of elasticity is stress divided by strain for a slowly applied load, the resilient modulus is stress divided by strain for rapidly applied loads, such as vehicle loads on pavements. MR measures the amount of recoverable deformation at any stress level for a dynamically loaded test specimen. It is defined simply as the ratio of the cyclic axial stress to resilient axial strain as given below.

$$MR = \frac{\Delta\sigma}{\Delta\epsilon} \text{..............................equation 2.3}$$

Where: MR= Resilient Modulus, $\Delta\sigma$ = repeated deviator stress and $\Delta\epsilon$ = recoverable resilient axial strain.

AASHTO recommends the use of a resilient modulus (MR) value obtained from a repeated triaxial test for the design of pavements, especially for heavily trafficked pavements.

Moisture is one of the main factors which affects the strength and stiffness of a Subgrade. It also controls the ease of compaction (density) and the compressibility and swelling/shrinkage characteristics of Subgrade soils.

The road has been divided into sections in which the Subgrade soil is fairly uniform. In each section, the design CBR is determined statistically from all CBR values measured in that section. The design CBR is defined as the CBR value that 90 percent of all test values in the section are equal to or greater than. Based on an Engineering Manual for Highway Pavement Design this is determined using following steps:

1. Arrange all test values in ascending order.

2. For each different test value, beginning with the lowest one, compute the percentage of the total number of values that is equal to, or greater than that value.

3. Plot the results on cross section paper: abscissa = Subgrade CBR value; ordinate = percent of Subgrade CBR values equal to or greater than. Then, draw a smooth best-fit curve through the plotted points.

4. Read from the curve the Subgrade CBR value at 90 percent. This is the Design CBR value

2.2.1.1.2 AASHTO 1993 Flexible Pavement Design

At the conclusion of the AASHTO flexible pavement equation show the relationship traffic loading, material properties with structural number as shown below.

$$\log_{10} W18 = Zr * So + 9.36(\log_{10}(SN+1)) - 0.20 + \frac{\log_{10}\left(\frac{\Delta PSI}{2.7}\right)}{0.40 + \left(\frac{1094}{(SN+1)^{5.19}}\right)}$$

$+\ 2.32 \log_{10} Mr -\ 8.07$……………………………………equation 2.4

Where;

W_{18}= 18kip equivalent single axle load

S_o= overall standard deviation of traffic

SN=structural number

Zr = reliability (z-statistics from the standard normal curve)

ΔPSI= loss in serviceability from the time the pavement is new until it reaches its TSI

Mr= soil resilient modulus of the sub grade in lb/in^2

The following equation can be used to determine the traffic (w_{18}) in the design lane.

$W_{18}= D_D \times D_L \times W*_{18}$ ……………………equation 2.5

Where: D_D= a directional distribution factor, expressed as a ratio, that accounts for the distribution of ESAL units by direction

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

W^*_{18}= a cumulative two directional 18-kip ESAL units predicted for a specific section of highway during the analysis period.

D_D (directional distribution factor) is generally 0.5 (50 percent) for most roadways, there are instances where more weight may be moving in one direction than the other. Thus, the side with heavier vehicles should be designed for a greater number of ESAL units. Experience has shown that D_D can vary from 0.3 to 0.7 depending on which direction is "loaded" and which is "unloaded" (AASHTO, 1993).

According to AASHTO1993 the D_L (lane distribution factor) for one lane highway in each direction is 100%.

Traffic Growth Rate Calculation

GR = $[((ADTf / ADTi)^{(1/(F-I))})-1] \times 100$……………………equation 2.6

Where:

GR = Growth Rate (%) I = Initial year for ADT

ADTf = Average daily traffic for future year F = Future year for ADT

ADTi = Average daily traffic for initial year

Future ADT Calculation

If an ADT and growth rate is provided, then a future ADT can be calculated using the following equation:

$ADT_f = ADTI\ (1+GR)^{(F-I)}$……………………………….equation 2.7

Where:

GR = Growth Rate (%) I = Initial year for ADT

ADTf = Average daily traffic for future year F = Future year for ADT

ADTi = Average daily traffic for initial year (year traffic data is provided)

AASHTO 1993 Pavement Design Guidelines classify highway as interstate, divided primary route, undivided primary route and High Volume Secondary Route. The guide recommends the design variables values as shown in table below for Undivided Primary Route.

Table 2.4: Design variables values of Undivided Primary Route for flexible pavement

Design variable	**Value**
Pavement Design life (year)	20
Initial Overlay Design (year)	10
Lane distribution factor (%)	100
Reliability for rural section (%)	85
Initial serviceability index	4.2
Terminal serviceability index	2.8
Standard Deviation	0.49
Subgrade Resilient modulus (Psi)	10000
Drainage Coefficients (m)	1

To compute a design resilient modulus, analysis of all the soils data should be conducted prior to selecting a value. An average Resilient Modulus (Mr) should not be used as the design Mr if the coefficient of variance (Cv) is greater than 10%. If the Cv is greater than 10%, then the Pavement Designer should look at sections with similar Mr values and design those section based on that average Mr. If no sections clearly exist, then use the average Mr times 67% to obtain the design Mr. For those locations with an actual Mr less than the design Mr, then the pavement designer should consider a separate design for that location or undercutting the area.

If resilient modulus results are not available, then use the following correlations:

For fine-grained soils with a soaked CBR between 5 and 10, use the following equation to correlate CBR to resilient modulus (Mr):

Design Mr (psi) = 1,500 x CBR……………………….equation 2.8

For non fine-grained soils with a soaked CBR greater than 10, use the following equation:

$Mr = 3{,}000 \times CBR^{0.65}$

When FWD testing is conducted and the backcalculated resilient modulus is determined, use the following equation:

Design Mr = C x Backcalculated Mr…………………equation 2.9

Where C = 0.33

If CBR and backcalculated Mr results are available, use the smaller Design Mr for pavement design purposes. If the Design Mr based on CBR is greater than 15,000 psi or if the Design Mr from backcalculation is greater than 15,000 psi, then use a Design Mr value of 15,000 psi.

Once the design structural number (SN) for an initial pavement structure is determined, it is necessary to identify a set of pavement layer thicknesses which, when combined, will provide the load carrying capacity corresponding to the design SN. The following equation provides the basis for converting SN in to the actual thickness of surfacing, base and subbase:

$SN = a_1D_1 + m_2a_2D_2 + m_3a_3D_3$ …………………………………………… equation 2.10

Where:

SN = structural number

a_1, a_2, a_3 = layer coefficients for surface, base and subbase respectively

m_2, m_3 = drainage coefficients for base and subbase respectively

D_1, D_2, D_3 = thickness of each layer (layer 1 = HMA, layer 2 = base, layer 3 = subbase)

Table 2.5: Structural layer coefficient

Pavement component	Coefficient
Wearing surface	
Sand-mix asphaltic concrete	0.35
Hot mix asphaltic concrete	0.44
Base	
Crushed stone	0.14
Dense-graded crushed stone	0.18
Soil cement	0.20
Emulsion/aggregate bituminous	0.30
Portland cement/ aggregate	0.40
Lime-pozzoland/aggregate	0.40
Hot-mix asphalt concrete	0.40
Subbase	
Crushed stone	0.11

The structural number equation does not have a single unique solution; i.e, there are many combinations of layer thicknesses that are satisfactory solutions. The thickness of the flexible pavement layers should be rounded to 1/2 inch when selecting appropriate values for the layer thickness, it is necessary to consider their cost effectiveness along with the construction and maintenance constraints in order to avoid the possibility of producing an impractical design from a cost effective view, if the ratio of costs for layer 1 to layer 2 is less than the corresponding ratio of layer coefficients times the drainage coefficient, then the optimum economical design is one where the minimum base thickness is used. Since it is generally impractical and uneconomical to place surface, base, or subbase course of less than some minimum thickness, the following are provided as minimum practical thickness for each pavement course

Table 2.6: Minimum Thickness of pavement layers

Traffic, ESAL's	Asphalt Concrete (in)	Aggregate Base
Less than 50,000	1.0 (surface treatment)	4
50,001-150,000	2.0	4
150,001-500,000	2.5	4
500,001-2,000,000	3.0	6
2,000,001-7,000,000	3.5	6
Greater than 7,000,000	4.0	6

Source: AASHTO1993

It should be recognized that for flexible pavements, the structure is a layered system and should be designed accordingly the structure should be designed in accordance with the principles shown in figure 2.3. First, the structural number required over the roadbed soil should be computed. In the same way, the structural number required over the subbase layer and the base layer should also be computed, using the applicable strength values for each by working with difference between the computed structural numbers required over each layer, the maximum allowable thickness of any given layer can be computed

It should be recognized that this procedure should not be applied to determine the SN required over the subbase or base materials having the modulus greater than 40,000 psi. for such cases, layer thickness of material above the" high" modulus layer should be established based on cost effectiveness and minimum practical thickness considerations..

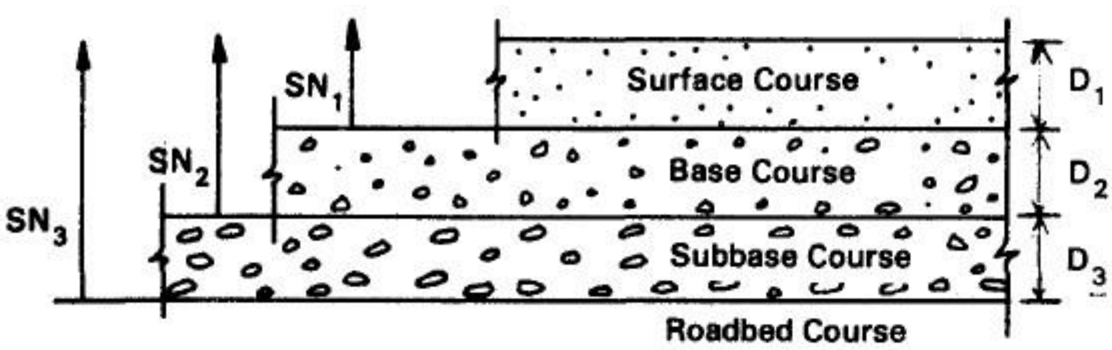


Figure 2.3: Procedure for Determining Thickness of Layers using a Layered Analysis Approach

D*1 $\geq \frac{SN1}{a1}$

SN*1=a1 D*1≥SN1

D*2≥$\frac{SN2-SN*1}{a1m2}$

SN*1+SN*2 ≥SN2

D*3≥$\frac{SN3-(SN*1+SN*2)}{a3m3}$

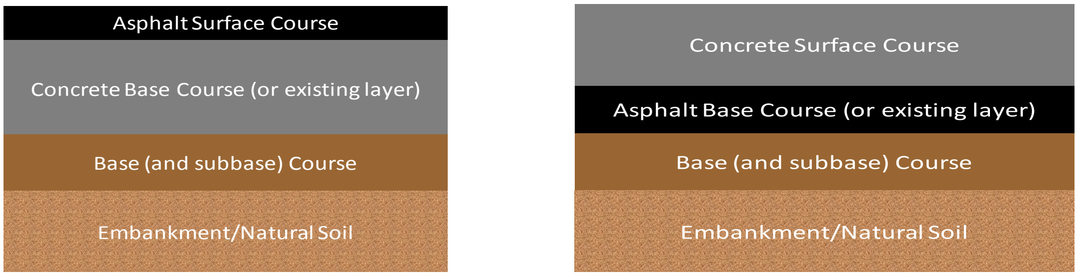
D, a, m and SN are minimum required values

An asterisk with D or SN indicates that it represents the value actually used, which must be equal to or greater than the required value.

Full-depth shoulder (same design as the mainline pavement) is also recommended for high-volume non-interstate routes.

2.2.1.2 Composite Pavement Structure

A composite pavement structure is one that combines the elements of both asphalt and concrete pavement systems and acts as one composite material. While most composite pavements consist of an HMA surface placed over a concrete layer (either a new base or an existing concrete pavement surface) (see Fig. 2.4a), they may also consist of a concrete surface placed on an HMA layer (either a new base or an existing HMA pavement surface) (see Fig. 2.4b). The former structure takes advantage of the strong support provided by the rigid base and the latter structure benefits from a stronger, less erodible bound base (compared to an unbound base).



a. Asphalt over concrete b. Concrete over Asphalt Composite pavements.

Figure 2.4: Composite Pavements

Most composite pavements consist of an HMA surface placed over a concrete layer

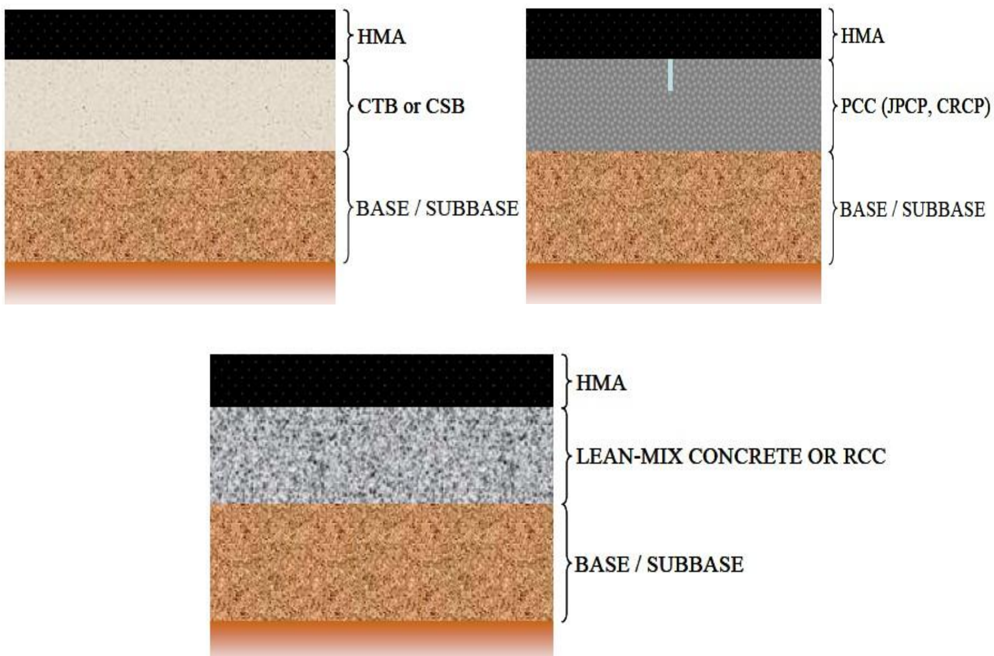


Figure 2.5: Typical Cross-sections of Composite Pavements

2.2.1.2.1 Benefits of Composite Pavement

Most composite pavements consist of an HMA surface placed over a concrete layer. Composite pavements, when compared to traditional flexible or rigid pavements, have the potential to provide better levels of performance both structurally and functionally (technical aspects) while being an economically viable alternative (economic aspect). Some of the benefits that composite pavements provide are (Donald, 2003).

- Strong support to the HMA layer provided by the rigid base layer.

- Improvement of the rideability of the pavement and driver comfort by providing a smooth and quiet driving surface.
- Ensuring adequate pavement skid resistance (i.e., friction).
- Preservation of the structural integrity of the rigid base to ensure a long-life pavement system by performing preventive maintenance on the HMA surface course.
- Prevention of the intrusion of deicing salts and surface water to the rigid base due to the impermeable characteristic of the asphalt layer.
- Reduction of the temperature gradient in the rigid layer because of the overlaying asphalt layer.

(Donald, 2003) Discusses how the traditional heavy-duty pavement type is a thick AC or HMA on an unbound aggregate base and granular subbase course. This type of conventional flexible pavement structure relies principally on the HMA for stiffness—the HMA is the layer that provides the majority of the structural capacity. Therefore, tensile strains at the bottom of the HMA layer need to be analyzed when designing a flexible pavement. This means that the risk of fatigue cracking (flexural fatigue) that initiates at the bottom of the HMA layer and propagates upward needs to be considered. Permanent deformation (rutting) is also a distress that affects typical flexible pavements. The compressive strains at the top of the subgrade could potentially cause rutting to show on the flexible layer. Therefore, compressive strain at this location (top of subgrade) needs to be considered a critical strain for rutting just as the tensile strain at the bottom of the HMA is considered for fatigue. In a composite structure, as shown in Fig. 2.6, the critical strain location for flexural fatigue (tensile strain) is shifted to a tensile stress location at the bottom of the rigid layer.

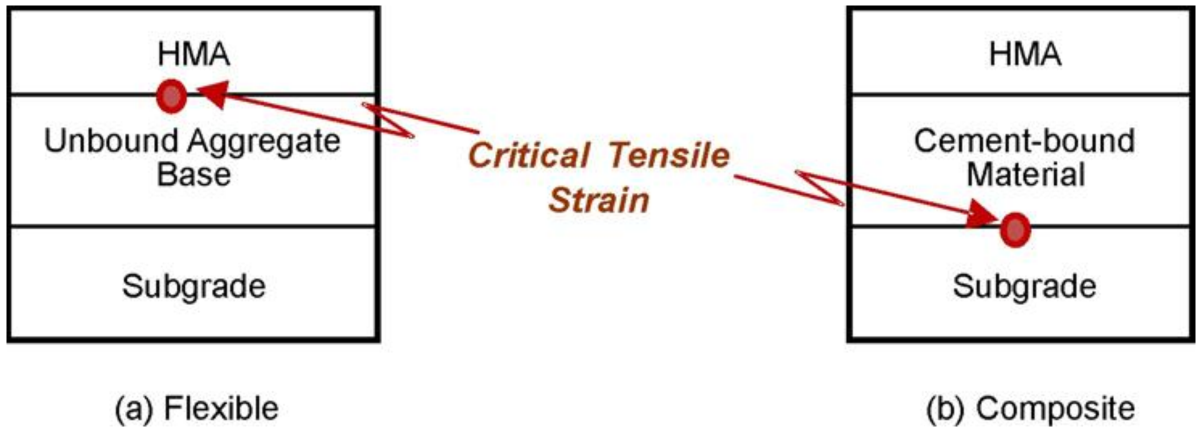


Figure 2.6: Shift in Critical Strain Location from a Typical Flexible Pavement to a Semi-Rigid one

It is expected that the traffic loading would tend to generate high tensile strains in conventional flexible pavements. A stiff base is needed to reduce these critical strains (Donald, 2003)

2.2.1.2.2 Past Performance of Composite Pavement System

Composite pavements have been implemented worldwide in the last decades. In Europe, composite pavements have been used extensively; countries such as Germany, France, and Spain are known for their wide use (30 to 50 percent) of long-life semi-rigid structures in their main road network (Thogersen, 2004)

A study by (Merrill, 2006) reviewed the experiences of composite pavements in Europe. The authors commented that composite pavements from the U.K., the Netherlands, and Hungary were performing satisfactorily in terms of rutting, cracking, and deflections. Some of the conclusions mentioned that the expected life of a semi-rigid pavement structure is statistically longer than that of a flexible one, and that semi-rigid structures with relatively thin layers (250 mm total thickness) could achieve a long-life performance even under heavy traffic.

Moreover, field observations confirmed that composite structures tend to have longer lives (i.e., they may be classified as long-life pavements). Because of this longevity, these structures may provide highway and transportation authorities with potential economic benefits.

In the United States, composite pavements usually have been the result of PCCP rehabilitation, which consists of a bituminous surface being overlaid onto the deteriorated rigid pavement, thus creating a composite structure. This rehabilitation task has been widely accepted and used to restore the functional and structural performance of an existing pavement; it has also been used to increase the structural capacity in order to improve performance and handle additional and heavier traffic. The performance of composite pavements may vary due to different factors such as design of rigid base, selection of adequate HMA type, constructability, and maintainability. A study of composite pavements presented by (Hein, 2002) concluded that (1) there is a difficulty in predicting the condition of the concrete base based on the surface conditions of the HMA, (2) faulting and spalling were effectively hidden from view, (3) the use of an open-graded HMA interlayer does not mitigate reflection cracking, (4) there is an early (3 to 5 years) deterioration due to reflective cracking on the HMA from the underlying rigid layer's discontinuities, and (5) the pavement condition ratings based only on the HMA surface do not accurately reflect the condition of the overall pavement structure and/or concrete base.

2.2.1.2.3 Composite pavement Design

There is no current ERA design manual or AASHTO 1993 Guide standard in designing composite pavement structures. AASHTO 1993 guide for PCC rehabilitation (AC overlays of PCC pavement) principle can be used for AC overlays of PCC design of new composite pavement.

The 1993 AASHTO Guide for Design of Pavement Structures could be used to design two different composite pavements:

(1) A flexible pavement with a cement-treated (or soil-cement) base and

(2) A rehabilitated PCC pavement using a specific section in the guide for the design of AC overlays of PCC (both jointed plain concrete pavement [JPCP] and continuously reinforced concrete pavement [CRCP]).

The first alternative, a flexible pavement with a cement-treated base, includes the design of a conventional flexible pavement with a cement-stabilized soil base. The most important characteristic of this design would be an adequate layer coefficient, a2, for the stabilized base from the flexible SN design equation:

$SN = a_1D_1 + m_2a_2D_2 + m_3a_3D_3$ ……………………….. equation 2.11

Where: SN = structural number m_2, m_3 = drainage coefficients

a_1, a_2, a_3 = layer coefficients

D_1, D_2, D_3 = thickness of each layer (layer 1 = HMA, layer 2 = base, layer 3 = subbase)

The second alternative using the AASHTO 1993 guide is based on the procedure for designing rehabilitation of PCC pavements with an AC overlay. In this case, the first step is to design a conventional PCC pavement, in other words, compute the thickness to satisfy the traffic demand, Df. Once the slab thickness has been obtained, it could be assumed that placing an AC layer with a thickness slightly greater than 2 in. would allow for the decrease of 1 in. of PCC layer. This is because the guide's "AC Overlay of PCC Pavement" procedure indicates that the required thickness, Dol, of an AC overlay of PCC is calculated using the following equation:

$D_{ol} = A(D_f - D_{eff})$…………………………………..….. equation 2.12

Where: A = factor to convert PCC thickness deficiency to AC overlay thickness

Df = slab thickness to carry future traffic (in.)

Deff = effective thickness of existing slab (in.)

Therefore, two assumptions are made. First, in a new composite pavement design, Deff is equal to Df because it is appropriate to assume that a newly constructed PCCP would not have any distresses, thus none of the adjustment factors shown in Equation 2.13 would be applicable.

$Deff = Fjc * Fdur * Ffat * D$……………………………………...…. equation 2.13

Where: D = original slab thickness (this would be equal to Df)

Fjc, Fdur, Ffat = adjustment factors for joints and cracks, durability, and fatigue = 1

The second assumption involves the A factor in Equation 2.12. According to the guide, the A factor is computed using the following equation:

$A = 2.2233 + 0.0099 (D_f - D_{eff})^2 - 0.1534 (D_f - D_{eff})$ ………………… equation 2.14

Because it was established that Df = Deff, A = 2.2233 would be obtained. This implies that because the A factor is 2.2233, for every 1 in. of PCC slab thickness, 2.2233 in. of HMA should be used as overlay. Once the overlay thickness is computed, it is rounded up to the nearest 0.5 in. Therefore 2.2233in. HMA can provide sufficient structural support as 1in. PCC for a certain amount of traffic.

A rigid pavement structure is constructed of a Portland cement concrete (PCC) slab on top of a base, subbase, or directly over compacted subgrade. This type of structure is designed to support loads through flexural action. Rigid pavements have often been recommended for heavily trafficked roads because of their initial high cost and their excellent traffic carrying capacity. The deflections under a loaded wheel are very small compared with the deflections observed in flexible pavements and the stresses within the underlying sub-base and subgrade are also comparatively small. Rigid pavements therefore deteriorate through quite different mechanisms from those that affect flexible pavements.

The initial cost of a rigid pavement is often considered a disadvantage but, depending on the relative costs of materials, their whole life costs can be considerably less than flexible pavements and they can therefore provide a more sustainable option.
A typical rigid pavement structure is shown in fig. 2.7

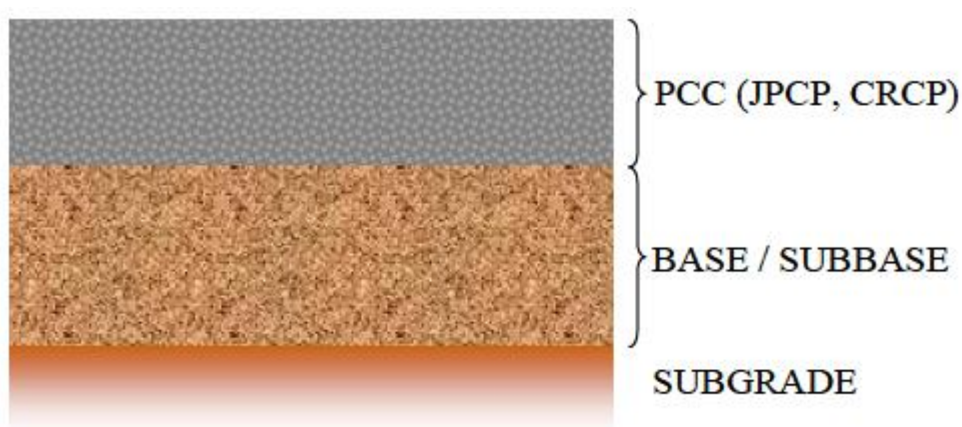


Figure 2.7: Typical Rigid Pavement Structure

The design procedures used for rigid pavement for this study are ERA manual and AASHTO 1993 guide. During the design process, various parameters (e.g., CBR, reliability) and concrete properties, such as elastic modulus and modulus of rupture (MR), are taken into consideration to compute the required slab thickness that would satisfy the traffic demand.

2.2.1.2.3.1 ERA Rigid Pavement Design

The durability of concrete is very high and therefore rigid pavements can be designed for periods of up to 60 years, but 40 years is the most common design period. If properly constructed, the pavement will last a long time with a high level of serviceability and low maintenance requirements. However, as with all pavements, the maintenance must not be neglected.

The factors which control the performance of a rigid pavement and for which design criteria are required are as follows:

- Quality of the concrete and steel for constructing the pavement slabs.
- Strength of the subgrade.
- Quality of the sub-base.
- Environment (moisture and temperature).
- Traffic and design life.

For the design procedure recommended in ERA rigid pavement design manual, the assumptions are made that the materials used for construction meet the following requirements:

- ✓ 28-day characteristic compressive strength of **30**, **35**, **40**, 45 or 50 MPa for the concrete.
- ✓ Yield strength of the steel reinforcement bars is greater than 400 MPa.

There are three basic types of rigid pavement:

i) Jointed Unreinforced Concrete Pavements (JUCP)

ii) Jointed Reinforced Concrete Pavements (JRCP)

iii) Continuously Reinforced Concrete Pavements (CRCP)

There is usually very little data available for accurately estimating whole life costs. Furthermore, there is little experience of rigid pavements in Ethiopia. Hence there is no foolproof way of determining the best choice of rigid pavement type for any particular situation.

General guidelines concerning the relative merits of each type are as follows:

- JUCP is suitable for all levels of traffic and whenever the risk of Subgrade movement is low.
- JRCP is suitable for all levels of traffic and is used when the risk of settlements of the subgrade cannot be neglected.
- CRCP should be considered only for high levels of traffic (>30 million ESAs) and where the low cost of maintenance is paramount; they are particularly suitable were settlement of the sub soil is an acknowledged risk.
- Less concrete is required if there is more reinforcement.
- Although CRCP potentially provides the best quality pavement overall, it requires an experienced contractor and is the most expensive.

When there is no overriding factor, which is the normal situation, it is standard practice to design typical sections of the road using each of the available options and then to compare them on an economical basis using engineering judgment where data are unavailable.

Unavoidably, there will be situations where only construction costs will determine the type of rigid pavement to build, even though higher maintenance or repair costs may be involved at a later date.

Where circumstances permit, a more realistic economical evaluation has to take into account all expected costs including the initial cost of construction, the cost of subsequent stages or corrective works, anticipated life, maintenance cost and salvages value. Costs to road users during periods of reconstruction or maintenance operations should also be considered. This is particularly important in Ethiopia because alternative or by-pass routes may be very long.

Although pavement structures are based on an initial design period, few are abandoned at the end of this period and continue to serve as part of the future pavement structure. For this reason, the analysis period should be of sufficient duration to include a representative reconstruction of all pavement types.

In ERA manual a reliability of 85% is used for added safety in view of the lower quality of concrete that is achievable in local practice and to cover uncertainties in the design models at lower strengths.

The method of computing cumulative equivalent standard axle loads over the design life of the rigid pavement is the same as flexible pavement but the damage exponent value used to compute equivalent factor is 4.3 and 4.5 for rigid and flexible pavement respectively which causes different pavement damage on structure: The relationship between pavement damage and axle load for rigid pavements is slightly different from flexible pavement.

$ef = (L/8160)^n$ (for loads in kg………………………equation 2.15 or

$ef = (L/80)^n$ (for loads in kN) ………………………equation 2.16

Where: *ef* = number of equivalent standard axles (ESAs), L = axle load (in kg or kN) & n = damage exponent (n = 4.3)

Thickness of rigid pavement layers which have the capacity to carry the corresponding traffic over the design period under a given subgrade and other pavement layer properties. Capping, Sub-base, Concrete Slab thickness and Reinforcement can be determined as follows.

A capping layer is required if the design CBR of the subgrade is less than 15%. The required thickness of capping layer and sub-base thickness is shown in table below;

Table 2.7: Thickness of Sub-base and Capping Layers

Subgrade Class	CBR range %	Sub-base thickness (mm)	Capping layer thickness (mm)
S1	2	200	400
S2	3,4	175	350
S3	5-8	150	250
S4	8-15	150	200
S5	15-30	175	0
S6	>30	0	0

Source:-ERA Manual 2013

A sub-base layer is required whenever the subgrade material does not comply with the requirement for a sub-base (CBR is less than 30%) but it is usually provided in all cases because the sub-base and capping layers are primarily designed to provide a good working platform for construction activities. This enables construction levels to be more easily achieved within the tolerances required.

For good performance of the rigid pavement the sub-base material should be very resistant to erosion. To ensure this it should, ideally, be stabilized with cement or lime (Class CS with unconfined compressive strength in the range 0.75-1.5 MPa), especially if the traffic level is high (i.e. the higher classes of road).

If the sub-base is not stabilized the thickness of the concrete pavement must be increased as shown in ERA manual for unreinforced and reinforced pavements.

Based on the design traffic volume expressed in Equivalent Standard Axles, the strength of the concrete, the type of rigid pavement, the shoulder design, and the thickness of the pavement are determined from chart.

2.2.1.2.3.2 AASHTO1993 Rigid Pavement Design

The design procedure for rigid pavements is based on a selected reduction in serviceability and similar to the procedure for flexible pavements. However, instead of measuring pavement strength by using a structural number, the thickness of the PCC slab is the measure of strength as shown in the equation below.

$$\log_{10} W18 = Z_r\, S_o + 7.35\left[\log_{10}(D+1)\right] - 0.06 + \frac{\log_{10}\left[\frac{\Delta PSI}{3.0}\right]}{1+\left[1.624*10^7 / (D+1)^{8.46}\right]} + (4.22 - 0.32TSI)\log_{10}\left[\frac{s_c\, c_d\left[D^{0.75}-1.132\right]}{215.63J\left\{D^{0.75}-\left[18.42/\left(^{Ec}/_{K}\right)^{0.25}\right]\right\}}\right] \text{.............................equation 2.17}$$

Where; W_{18}= 18kip (80.1KN) equivalent single axle load

Zr = reliability (z-statistics from the standard normal curve)

S_o= overall standard deviation of traffic

TSI= pavement's terminal serviceability index

C_d= drainage coefficient

S_c= concrete modulus of rupture in lb/in^2

D= PCC slab thickness in inches, J= Load transfer coefficient

E_c=concrete elastic modulus in lb/in^2 K= modulus of Subgrade reaction in lb/in^3

ΔPSI= loss in serviceability from the time the pavement is new until it reaches its TSI

AASHTO 1993 Pavement Design Guidelines classify highway as interstate, divided primary route, undivided primary route and High Volume Secondary Route. The guide recommends the design variables values as shown in table below for Undivided Primary Route

Table 2.8: Design variables values of Undivided Primary Route for Rigid pavement

Design variable	Value
Pavement Design life (year)	30
Initial AC Overlay Design (year)	10
Initial PCC Overlay Design (year)	30
Lane distribution factor (%)	100
Reliability for rural section (%)	85
Initial serviceability index	4.5
Terminal serviceability index	2.8
Standard Deviation	0.39
28-day mean PCC modulus of rupture(Psi)	650
28-day mean PCC modulus of elasticity(Psi)	5,000,000
Mean effective K-value (Psi/inch)	500
Subdrainage coefficient	1
Load transfer factor (J-factor) for Jointed Reinforced concrete with Asphalt shoulder	3.2
Load transfer factor for Jointed Reinforced concrete with Tied PCC shoulder or wide lane	2.8

Mean modulus of Subgrade reaction(k-value) in lb/in^3 can be computed using the following equation If the subgrade resilient modulus is known or obtained from the CBR

k-value = Mr / 19.4…………………………equation 2.18

Caution must be used when selecting a design k-value based on resilient modulus and CBR. An analysis of all the soils data should be conducted prior to selecting a value. An average Resilient Modulus (Mr) should not be used as the design Mr if the coefficient of variance (Cv) is greater than 10%. If the Cv is greater than 10%, then the Pavement Designer should look at sections with similar Mr values and design those section based on that average Mr. If no sections clearly exist, then use the average Mr times 67% to obtain the design Mr. For those locations with an actual Mr less than the design Mr, then the pavement designer should consider a separate design for that location or undercutting the area.

If the k-value is obtained from backcalculation, then use this value.
If k-value (based on backcalculation or subgrade resilient modulus) is larger than 500, then use 500 as the design value.

Two types of shoulders are designed for Portland cement concrete highways – full-width concrete shoulders, narrow-width concrete section with an asphalt concrete extension, or an asphalt shoulder. For full-width concrete shoulders, the pavement shoulder shall have the same design as the mainline pavement. This will allow the shoulder to support extended periods of traffic loading as well as provide additional support to the mainline structure.

A narrow-width concrete section with an asphalt concrete extension shoulder is constructed when a wide concrete lane (14 feet) is part of the mainline pavement. Twelve feet of the fourteen-foot wide slab is part of the outside travel lane, the remaining two feet is striped and designated as part of the shoulder. The two-foot section of concrete has the same structure as the twelve-foot section; therefore, no separate pavement design is necessary. For the asphalt concrete portion of the shoulder and other asphalt concrete shoulders, the shoulder's pavement structure should be based on 2.5% of the design ESALs (minimum) for the project following the AASHTO pavement design methodology. A minimum of two AC layers must be designed for the shoulder. The AC layers must be placed on an aggregate or cement stabilized aggregate layer, not directly on subgrade, to provide adequate support and drainage for the shoulder structure. To help ensure positive subsurface drainage, the total pavement depth of the shoulder should be equal to the mainline structure (i.e. mainline pavement structure thickness above the subgrade is 20 inches, shoulder pavement structure thickness above the subgrade is 20 inches).

2.2.1.2.4 Composite Pavement Distresses

A composite pavement structure, throughout its service life, may present different type of distresses. The distresses that affect composite pavement, according to (Von Quintus, 1979), are very similar to those of flexible pavements because of the exposure of the HMA layer in the composite structure. Three types of distresses that have been modeled extensively include: fatigue cracking in the HMA layer (both bottom-up and top-down) and the rigid layer, rutting due to permanent (plastic strain) deformations within the HMA layer and subgrade, and reflective cracking.

Numerous publications have agreed that reflective cracking (also called reflection cracking) is a major type of distress in composite pavement systems. Reflective cracks are cracks that occur in the asphalt surface course of the composite pavement system and that coincide with cracks of appreciable width or joints in the underlying layers; they are caused by relative horizontal and vertical movements of these cracks or joints caused by temperature cycles and/or traffic loading (Oowusu-Antwi, 1998).

Geosynthetic materials may be used to retard or control reflection cracking, to provide separation between layers to prevent migration of fines into the base, or to provide additional structure or load-carrying capacity over soft subgrade soils (ERA, 2013).

2.2.2 Life Cycle Cost Analysis (LCCA) in PTS Process

Life Cycle Cost Analysis is an economic method to compare alternative pavement structures that satisfy a need in order to determine the lowest cost alternative. According to Chapter 3 of the AASHTO Guide for Design of Pavement Structures, life cycle costs "refer to all costs which are involved in the provision of a pavement during its complete life cycle."

These costs borne by the agency include the costs associated with initial construction and future maintenance and rehabilitation. Additionally, costs are borne by the traveling public and overall economy in terms of user delay. The life cycle starts when the project is initiated and opened to traffic and ends when the initial pavement structure is no longer serviceable and reconstruction is necessary (VDOT, 2011).

During a PTS process, from a life cycle cost perspective, pavement costs can generally be categorized as agency costs and user costs. Agency costs include initial construction cost, rehabilitation, maintenance costs, and salvage value. User costs include indirect costs such as time delay costs at work zones, vehicle operating costs (VOC), accident costs, and discomfort costs (Beg, 1998). It is important to determine whether pavement costs are categorized as initial or future costs. The cost of an initial reconstruction or new construction are part of the initial costs, whereas any rehabilitation and maintenance activity performed during the life cycle of the pavement is categorized as a future cost (VDOT, 2011).

Vehicle operating costs depend on the condition of the road surface. The road surface deterioration, hence its condition, depends on the nature of the traffic, the properties of the pavement layers materials, the environment, and the maintenance strategy adopted (VDOT, 2011).

In Ethiopia at the present time there is insufficient experience of many of the possible structures and their likely maintenance requirements for this to be done with much accuracy. However data is being accumulated to assist with this in future and more roads need to be built using alternative structures. The lack of data is an impediment/obstacle to accurate analysis (ERA, 2013).

For the pavement structures recommended ERA design manual, the level of deterioration that is reached by the end of the design period should be limited to levels which yield acceptable economic designs under most anticipated conditions. Routine and periodic maintenance activities are assumed to be performed at a reasonable and not excessive level (ERA, 2013).

2.2.2.1.1 Economic Analysis Components

Analysis Period

To maintain consistency with the FHWA Technical Bulletin, Life Cycle Cost Analysis in Pavement Design, LCCA periods should be sufficiently long to reflect long-term differences associated with reasonable maintenance strategies. The analysis period should generally be longer than the pavement design period. As a rule of thumb, the analysis period should be long enough to incorporate at least one complete cycle of rehabilitation activity. The FHWA's September 1996 Final LCCA Policy Statement recommends an analysis period of at least 35 years for all pavement projects, including new or total reconstruction projects and rehabilitation, restoration, and resurfacing projects. For VDOT's LCCA procedure, a 50-year analysis period was selected for new construction and reconstruction type projects. This period is sufficiently long to reflect the service lives of several rehabilitation activities. For major rehab type projects where multiple pavement types are considered and LCCA is required, the analysis period is taken to be the design life of the rehab design (VDOT, 2011). The ***analysis period*** is the period of time during which the initial and any future costs for the project alternatives will be evaluated. Table 2.11 provides the common analysis periods to be used when comparing alternatives of a given design life or lives. For example, a minimum analysis period of 35 years should be used if 10-year and 20-year design life alternatives are compared or if two different design alternatives with the same 20-year design life are compared (Caltrans, 2008).

Table 2.9: LCCA Analysis Periods

Alternative Design Life	CAPM	10-Yr	years15 or 20-Yr	years25 to 40-Yr
CAPM	20 years	20 years	20 years	
10-Yr	20 years	20 years	35 years	55 years
15 or 20-Yr	20 years	35 years	35 years	55 years
25 to 40-Yr		55 years	55 years	55 years

Source: State of California, Department of Transportation

In order to account for the cost related to future activities, the time value of money must be considered. In LCCA, the discount rate is used. The *discount rate* is defined as the difference between interest and inflation rates (VDOT, 2011). Historically, this value has ranged from 2% to 5%; for LCCA purposes, a value of 4% used. This value is consistent with the values recommended in the FHWA Interim Technical Bulletin and practices by many other state agencies. The discount rate accounts not only for the increased cost associated with performing an activity in the future but also for the economic benefit the agency would receive if those funds were instead invested in an interest-bearing account (VDOT, 2011).

Discount rate is the interest rate by which future costs (in dollars) will be converted to present value. In other words, it is the percentage by which the cost of future benefits will be reduced to present value (as if the future benefit takes place in the present day). Real discount rates (as opposed to nominal discount rates) reflect only the true time value of money without including the general rate of inflation (Caltrans, 2008). Real discount rates typically range from 3% to 5% and represent the prevailing interest of U.S. Government 10-year Treasury Notes. Caltrans currently uses a discount rate of 4% in the LCCA of pavement structures.

The following two equations are employed to convert future cost into present cost and vice versa.

F= $P(1+r)^n$……………………………equation 2.19

P= $F[\frac{1}{(1+r)^n}]$……………………………..equation 2.20

Where: *P* = the present-day cost or value; the present sum of money.

F = the cost sum at a future date, n periods from the present; the sum is equivalent to *P* with compound interest at *r* (discount rate) over n periods

r = Value in decimals representing a specific change over time periods; discount rate per period of time; it could be in this sense nominal or real depending on the nature of analysis.

n = Number of discount periods; it is mostly expressed in years

Further, the real discount rate can be estimated using the derived formula.

$r^* = \frac{i-f}{1+f}$ ……………………………..equation 2.21

Where: *f* = Inflation rate   *i* = Nominal interest rate

*r**= Real discount rate—an interest rate that has been adjusted to remove the effect of expected or actual inflation.

Numerous economic analysis methods can be used to evaluate pavement alternatives, including the benefit/cost ratio method, internal rate of return, equivalent uniform annual costs (EUAC), cost effectiveness method, and the present worth (PW) method or net present value (NPV) method (Haas, 2003). From all of the mentioned models, the PW method and the EUAC method are the two most commonly used economic indicators by many state highway agencies in the United States (NCHRP, Guide for Mechanistic -Emperical Design of New and Rehabilitated Pavement Structures-NCHRP1-37A,National CooperativeHighway Research program, 2004).

The EUAC method describes the average cost an agency will pay per year over the analysis period. All costs including initial construction and future maintenance are distributed evenly.

Although this dollar value may not seem realistic in years when little pavement action is required, it can be used to evaluate and compare alternatives (VDOT, 2011).

The PW method reports initial and future pavement costs as a lump sum amount in today's dollar value. For activities that occur in the initial year of the analysis period, the PW cost is the same as the actual cost, i.e., no adjustment for inflation and interest. For future maintenance and rehabilitation activities, the PW cost is less than the actual cost (based on today's unit prices) since total costs are discounted. Please note that for two identical actions that occur 30 years apart, the later action will cost much less. This is because of the number of years that are discounted. The PW method is the more widely used approach for pavement LCCA. It gives an indication of how much a pavement alternative will cost over the analysis period and can be used to clearly compare alternatives for lowest cost. The formula to compute both PW and EUAC are provided below (VDOT, 2011).

$$PW = \text{Initial cost} + \sum_{k=1}^{n} UpkeepCost_{k} * \left[\frac{1}{(1+i)^{k}}\right] \text{..........................equation 2.22}$$

Where: PW=present worth

Initial cost= initial cost of construction i = discount rate

k = year of activity/expenditure n = analysis period

Upkeep Cost=cost of maintenance/Rehabilitation activity at year K.

$$EUAC = PW * \left[\frac{i(1+i)^{n}}{(1+i)^{n}-1}\right] \text{...equation 2.23}$$

Where: EUAC= equivalent uniform annual costs, i = discount rate and n = analysis period

2.2.2.1.2 Cost Factors

Numerous costs are included in LCCA for pavements, ranging from initial costs associated with new construction to future maintenance costs associated with patching, sealing, and other activities.

LCCA comparisons are always made between mutually exclusive competing alternatives and LCCA need only consider differential costs between alternatives. Costs common to all alternatives cancel out; these cost factors are generally noted and excluded from LCCA calculations (FHWA, 1998).

To conduct an LCCA for comparing pavement alternatives, the initial cost is a major percentage of the PW or EUAC over the analysis period. The initial cost is determined at Year 0(zero) of the analysis period.

Although numerous activities are performed during the construction, reconstruction, or major rehabilitation of a pavement, only those activities that are specific to a pavement alternative should be included in the initial costs. By focusing on those activities, the engineer can concentrate on estimating the quantities and costs related to those activities. According to VDOT, 2011 actions dependent on pavement type include, but are not limited to the following.

- Milling
- Pavement removal
- Asphalt concrete paving
- Portland cement concrete paving
- Fracturing Portland cement concrete slabs

After viable project alternatives are identified and the project information is gathered, a pavement maintenance and rehabilitation schedule for each alternative must be determined. The schedules identify the sequence and timing of future activities that are required to maintain and rehabilitate the pavement over the analysis period. To determine the applicable pavement maintenance and rehabilitation schedule for a project the following information should be considered (Caltrans, 2008):

- Existing pavement type; these are flexible, rigid, and composite
- Pavement climate region
- Pavement design life
- Maintenance service level
- Volume and type of traffic use the pavement

For all pavement options, the initial pavement life is designed to support traffic for estimated years. At around the end of the design year period, the pavement must be rehabilitated. For flexible pavements, this rehabilitation generally includes removing AC surface and intermediate materials and replacing with new AC material. For rigid pavements, concrete pavement restoration (CPR) is generally conducted and an AC overlay may be placed. However, wherever feasible, concrete overlays could also be considered on both asphalt and concrete pavements. Rehabilitation activities may include but are not limited to the following (VDOT, 2011):

- Milling
- AC paving
- PCC and AC patching
- Jointcleaning

Structural/functional improvement activities are performed during the life of a pavement in order to maintain a smooth, safe, durable pavement surface. Structural/functional improvements are designed to last 10 years. Typical improvement activities include the following (VDOT, 2011):

- Milling
- AC and PCC patching
- AC paving
- PCC grinding
- Joint cleaning and sealing

All pavement types require preventive and corrective maintenance during their service life. The timing and extent of these activities vary from year to year. Routine reactive type maintenance cost data are normally not available except on a very general, area wide type cost per lane mile. Fortunately, routine reactive type maintenance costs are generally not very high due to the relatively high performance levels maintained on major highway facilities. Further, state highway agencies that do report routine reactive maintenance costs note little difference between most alternative pavement strategies. When discounted to the present, small reactive maintenance cost differences have negligible effect on PW and can generally be ignored. Therefore, they are not included in this LCCA procedure (VDOT, 2011).

At the end of the LCCA period, the pavement structure may be defined as having some remaining value to the managing agency, known as the salvage value (VDOT, 2011). Different pavement types attain different condition at the end of the analysis period. If the condition of the pavement at the end of the analysis period is such that a complete removal and replacement is warranted, then the salvage value would have been the cost of any residual materials obtained from the pavement system (materialized by the agency). However, in most situations and depending on the timing and extent of the last maintenance treatment, the pavement either continues to remain in service or some kind of rehabilitation treatment is performed on the existing pavement (which may involve partial removal of the pavement materials or reclamation type treatment combined with overlays). So, pavements typically offer some sort of remaining life at the end of analysis period. In such cases, the residual value of the materials is not realized. Therefore, the remaining life essentially represents the salvage value of the pavement for practical purpose. Estimating a dollar figure for this component could be complex. Fortunately, the dollar figures for the 'salvage value' for the competing pavement types when discounted 50 years to PW are not expected to be significantly different. For simplicity, disregards the salvage value for the competing pavement types in its LCCA process (VDOT, 2011).

2.2.2.1.3 Users Costs

Road user costs (RUC) are the costs that the highway user must incur over the lifetime of the project. RUC are composed of three other separate cost factors: vehicle operating costs (VOC), user delay costs, and crash-related costs. The AASHTO Red Book (AASHTO 1960) defines VOC as the mileage-dependent cost of using any type of motor vehicle, including vehicle depreciation attributed to mileage increase and the maintenance costs due to vehicle wear such as tire, engine oil, and fuel expenses. Travel time has been found to have the greatest impact on VOC; nevertheless, user travel time is highly dependent on who is traveling and for what purposes, making it extremely variable. User delay costs can be defined as the cost incurred by the user traveling on the highway and by those who are denied access to the highway by detours due to maintenance, rehabilitation work, or other agency requirements. User costs also have an added level of complexity due to the non-homogeneity of highway users and vehicle types. Non-user costs and accident costs are very difficult to estimate in numerical terms; for this reason they are usually omitted from an LCCA (Vadakpat et al. 2000).

3.0 METHODOLOGY

To achieve the objective of this study various data collected for design and evaluation of pavement alternatives. Both Ethiopian Road Authority (ERA) and AASHTO 1993 pavement design manual procedures used to design pavement structures. These empirically-based methods include equations, catalogue, monograph and procedures to design new flexible and rigid pavement alternatives as well as their rehabilitation options. But, there are no various design methods, equations, procedures, and recommendations for composite pavements which can be easily accessed from website of different transport agencies.

The evaluation of flexible and composite pavement systems was based on a technical and economic analysis. The design of flexible and composite pavements using available applicable guide was the first step. An economic evaluation perform by, a deterministic approach. Typical agency values should be obtained for material cost, rehabilitation procedures, and discount rates. In this economic evaluation flexible and composite pavements considered.

3.1 Data Collection

The data required for this study was obtained from journals, design manual, books and reports of various organizations. Traffic data, Subgrade soil property and material properties of pavement structure layers collected from different sources.

3.1.1 Traffic data

A review of available historical traffic data was done. This included data from historical records made available from the ERA from counts conducted from 2002 to 2014 for the road section. Analysis of these past traffic flows was undertaken in an attempt to gain an overall traffic growth pattern.

3.1.1.1 Traffic Growth Rates

To determine the likely future traffic growth rates for the road project, an analysis of the available historical traffic data and the recorded recent year traffic flows was undertaken. For the undertaking of this analysis, previous and recent recorded traffic flows were tabulated and annual growth rates between these years derived for the corridor. This exercise was undertaken through comparison of growth changes for total traffic volume.

Traffic growth rate can be computed from traffic volume historical data of the study area using the following equation.

$GR = [((ADTf / ADTi)^{(1/(F-I))})-1] \times 100$…………………………..equation 3.1

Where:

GR = Growth Rate (%)

ADTf = Average daily traffic for future year

ADTi = Average daily traffic for initial year

I = Initial year for ADT

F = Future year for ADT

Traffic volume historical data from 2002 up to 2014 from ERA database of the road sections. Adama – Awash and Awash – Mille road sections traffic annual average daily (AADT) shown for different year in table below.

Table 3.1: Traffic Volume Historical Data of the Road sections

Year	Adama-Awash Total AADT
2009	3671
2014	5755

Year	Awash-Mille Total AADT
2002	625
2014	1,662

The traffic growth rate from historical data computed using equation 3.1.

$GR = [((5755 / 3671)^{(1/(2014-2009)}})-1] \times 100 = 9.4\%$ which is growth rate of Adama-Awash road section

$GR = [((5755 / 3671)^{(1/(2014-2002))})-1] \times 100=8.5\%$ which is growth rate of Awash-Mille road section. This estimated growth rate of road sections utilized for this study.

In addition traffic growth rate determined form real gross domestic product (GDP) growth of Ethiopia as estimated by International Monetary Fund (IMF) is shown in table below:

Table 3.2: GDP of Ethiopia from 2006-2013 GC.

Year	2006/07	2007/08	2008/09	2009/10	2010/11	2011/12	2012/13
GDP %	11.8	11.2	10	10.4	11.4	8.8	9.7

Source: IMF

From the above, an average growth rate of seven year GDP is 10.5%.

3.1.1.2 Initial Traffic Volume

Traffic volume count conducted by ERA in 2014 used as recent year initial traffic for the road section of this study. There was a significant change in volume of traffic between Adama - Awash and Awash - Mille road sections that could require different pavement layer thickness and material to be used for each section. So, the traffic analyzed and forecasted separately for each section. Traffic volume in terms of AADT from ERA traffic volume database used for both road sections which are shown in table below:

Table 3.3: AADT of Adama - Awash road section in 2014 GC

Vehicle class	car	Land rover	Small bus	Large bus	Small truck	Medium Truck	Heavy truck	Truck and Trailer	Total
AADT	263	588	686	366	573	819	976	1,484	5,755

Source: ERA

Table 3.4: AADT of Awash - Mille road section in 2014 GC

Vehicle class	car	Land rover	Small bus	Large bus	Small truck	Medium Truck	Heavy truck	Truck and Trailer	Total
AADT	7	71	76	12	69	71	185	1,171	1,662

Source: ERA

3.1.1.3 Axle Load Data

From Awash to Adama direction of the road section used as design lane because, tuck and truck trailers heavily loaded from port of Djibouti.

Table 3.5: Average equivalency factors for different vehicle class

Direction	Car	4x4	small Bus	Large Bus	Small Truck	Medium Truck	Heavy Truck	Truck Trailer	Total
Awash - Adama lane	0.05	0.06	0.01	0.05	0.14	5.72	9.34	11.45	27.44

Source: Adama-Awash road overlay project engineering report

3.1.2 Subgrade data

California Bearing Ratio (CBR) value of Subgrade soil along the road section of this study used from Adama-Awash overlay project engineering report,. The CBR values are shown in Appendix A of this document. Then the design CBR is calculated as shown in Appendix A.

3.1.3 Design variable of AASHTO 1993

Road section of this study considered as Undivided Primary Route and most of the route is rural section, VDOT recommend the following design variable values for flexible and rigid pavement structures design using AASHTO 1993 method for this type of routes.

Table 3.6: Design Variable values for flexible pavement

Design variable	Value
Reliability for rural section (%)	85
Initial serviceability index	4.2
Terminal serviceability index	2.8
Standard Deviation	0.49
Subgrade Resilient modulus (Psi)	10000
Drainage Coefficients (m)	1

The Subgrade Resilient modulus computed from CBR value as shown in Appendix A.

Table 3.7: Design Variable for rigid pavement

Design variable	Value
Reliability for rural section (%)	85
Initial serviceability index	4.5
Terminal serviceability index	2.8
Standard Deviation	0.39
28-day mean PCC modulus of rupture(Psi)	650
28-day mean PCC modulus of elasticity(Psi)	5,000,000
Mean effective K-value (Psi/inch)	500
Subdrainage coefficient	1
Load transfer factor (J-factor) for Jointed Reinforced concrete with Asphalt shoulder	3.2
Load transfer factor for Jointed Reinforced concrete with Tied PCC shoulder or wide lane	2.8

The modulus of Subgrade reaction computed from CBR value of Adama-Awash road section.

3.1.4 Pavement Material property data for technical analysis

Material characteristics such as elastic modulus, Poisson ratio of each pavement layer, design traffic in Million Single Axles (MSA), wheel load, tire pressure and coordinates at which stress, strain and deflection are required.

Table 3.8: Elastic Modulus for Different Materials

Material	Range (Psi)	Typical
Portland cement concrete	$3x10^6$ to$6x10^6$	$4x10^6$
Cement-treated bases	$1x10^6$ to$3x10^6$	$2x10^6$
Soil cement materials	$5X10^4$ to$2x10^6$	$1x10^6$
Lime-flyash materials	5×10^5 to $2 .5 \times 10^6$	$1x10^6$
Stiff clay	7600 to 17,000	12,000
Medium clay	4700 to 12,300	8000
Soft clay	1800 to 7700	5000
Very soft clay	1000 to 5700	3000

Source: Huang 2004

Table 3.9: Poisson Ratios for Different Materials

Material	Range	Typical
Hot mix asphalt	0 .30–0 .40	0 .35
Portland cement concrete	0 .15–0 .20	0 .15
Untreated granular materials	0 .30–0 .40	0 .35
Cement-treated granular materials	0 .10–0 .20	0 .1 5
Cement-treated fine-grained soils	0 .15–0 .35	0 .25
Lime-stabilized materials	0 .10–0 .25	0 .20
Lime–flyash mixtures	0 .10–0 .15	0 .1 5
Loose sand or silty sand	0 .20–0 .40	0.3 0
Dense sand	0 .30–0 .45	0 .35
Fine-grained soils	0 .30–0 .50	0.40
Saturated soft clays	0 .40–0 .50	0.45

Source: Huang 2004

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Table 3.10: Range of elastic/resilient modulus used for base or sub-base

Type of Material	Range of elastic/resilient modulus (E)
Cement treated granular base	1,000,000 to 2,000,000psi
Cement aggregate mixture	500,000 to 1,000,000psi
Asphalt treated base	350,000 to 1,000,000psi
Bituminous stabilized mixture	40,000 to 300,000psi
Lime stabilized	20,000 to 70,000psi
Unbound granular material	15,000 to 45,000psi
Fine grained or natural Subgrade material	3,000 to 40,000psi

Source: AASHTO 1993 guide

Table 3.11: Typical Material Properties for the Composite Pavement Layers

Layer	Material	Thickness (inch)	Elastic Modulus (psi)	Poisson's Ratio	MR
Surface course	HMA	7	**550,000**	**0.35**	-
Base course	PCC	8	4,100,000	0.15	650
	RCC		3,600,000	0.15	600
	Lean concrete		2,100,000	0.15	450
	CTB		1,100,000	0.20	200
	Soil cement		550,000	0.20	100
	Granular base		**35,000**	**0.32**	-
Sub-base	Aggregate sub base	10	25,000	**0.40**	-
Subgrade	Soft clay	-	**10,000**	**0.45**	-

Awash-Mille road section flexible pavement layer thickness designed with AASHTO 1993 method and the corresponding material properties selected from Table3.10 range for KENPAVE software input as shown in table above.

The use of a layered elastic analysis computer program will allow one to calculate the theoretical stress, strain and deflections anywhere in a pavement structure. However, a few critical locations are often used in pavement analysis as shown in table below:

Table 3.12: Critical location of pavement structure for pavement analysis

Location	Response	Reason for use
Pavement surface	Deflection	Used in imposing load restrictions during spring thaw and overlay design
Bottom of HMA layer	Horizontal Tensile Strain	Used to predict fatigue failure in the HMA
Top of intermediate layer(Base or Subbase)	Vertical compressive strain	Used to predict rutting failure in base or sub base
Top of Subgrade	Vertical compressive strain	Used to predict rutting failure in the sub-grade

3.1.5 Cost Data

The bills of quantities for the two pavements used in this study are meant to cover a 1km road length. The rates were derived from two contractors namely Defense Construction Enterprise who bid for a section of Ditchoto Gulafi Junction – Elidar –Belecho rigid pavement Project and Ethiopian Construction Works Corporation who bid a section Awash - Mille Road Asphalt Overlay Project, Contract 2.

Cost data on concrete pavement contract, the case of, Ditchoto Gulafi Junction – Elidar – Belecho project, and on flexible pavement the case of, Awash - Mille Road Asphalt Overlay Project, Contract 2shown in table below:

Table 3.13: Awash-Mille Overlay Project contracts unit rate cost

Contractor-Ethiopian Construction Works Corporation

	Awash - Mille Road Asphalt Overlay Project, Contract 2: From km 72+500 - km 145+500		Client-ERA
Bill No.	Work Item	Unit	Unit Rate cost (birr)
1	ROAD BASES		
1.1	Crushed aggregate base course compacted to 100% of AASHTO T180 density, compacted thickness not more than 200mm	m^3	750.91
1.2	Crushed aggregate base course compacted to 100% of AASHTO T180 density in restricted area, compacted thickness not more than 200mm	m^3	788.46
2	BITUMINOUS ROAD BASES AND SURFACINGS		
2.1	Continuously graded asphaltic surfacing course, 180mm thickness (three layers), using 5.5 percent 40/50 penetration grade bitumen	m^2	1098.17
2.2	Hot mix asphaltic layer constructed for overlay purposes in accordance with the provisions of sub-clause 6413(f) , 55-75mm thickness in one layer (number of layers as per the design drawing), using 5.0 percent 40/50 penetration grade bitumen	m^3	6100.95
3	PRE-OVERLAY REPAIR IN EXISTING BITUMINOIS SURFACING		
3.1	Backfilling of Excavated, milled , cut or sawed area by patching with Continuously graded asphaltic surfacing material, one layer not greater than 75mm thickness (layers to accommodate the depth of milling), using 5.0 percent 40/50 penetration grade bitumen	m^3	6406

Source:ERA

Table 3.14: Ditchoto Gulafi Junction – Elidar Rigid pavement contract data

General Information of the Project		
S/N	**Item**	**Description**
1	Employer / Client	Ethiopian Roads Authority
2	Contractor	Defense Construction Enterprise
3	Project	Ditchoto Gulafi Junction – Elidar –Belecho Design-Build Project
4	Location	Afar Region, Ethiopia
4	Date of Award	-
5	Thickness of Concrete Slab	400 mm
6	Total contract Amount Without VAT	2,316,229,584.13 Birr
7	Length of Project	80.537 km

Unit Rate Cost			
Bill No.	Work Item	Unit	Unit Rate cost (birr)
1	C-35, 400mm thick concrete slab	m^2	1744.32
2	14mm dia. for reinforcement bar, 6mm dia.for dowel bar and 12mm dia. for tie bar.	kg	45.55
3	Granular Sub-base (unstabilized)	m^3	283.98
4	Capping Layer	m^3	228.10
5	Shoulder	m^3	283.98

Source: Defense Construction Enterprise

3.2 Pavement Structures Design

Ethiopian Road Authority (ERA) design manual and American Association of State Highway and Transportation Officials (AASHTO) 1993 guide for design of pavement structures have been used for this study to design the pavement for flexible and composite pavement structures.

3.2.1 Flexible pavement

To design flexible pavement structure using Ethiopian Road Authority (ERA) design manual, two important pavement design parameters were; traffic expected to use the route over the design year and subgrade property along the route. Traffic estimated in terms of the cumulative number of equivalent standard axles that expected to use the road over the selected design life and subgrade soil strength characterized over which the road is to be built. The strength of the road subgrade for flexible pavements is commonly assessed in terms of the California Bearing Ratio (CBR). Using the input data obtained from traffic and Subgrade to select a suitable structure from the catalogue of pavement structures from ERA manual.

ERA procedure used to determine cumulative number of vehicles and finally cumulative number of equivalent standard axles (ESAs) analyzed.

3.2.1.1 Subgrade

A flexible pavement is one which has low flexural strength. Thus, the external load is largely transmitted to the sub grade by the lateral distribution with increasing depth. Because of low flexural strength, the pavement deflects if the sub grade deflects. The pavement thickness is so designed that the stresses on the sub grade soil are kept within bearing power and the sub grade is prevented from excessive deformations.

Thus strength of road subgrade determined & the determination of sub grade strength is assessed in terms of CBR (Californian Bearing Ratio) which depends on the types of soil, its density and moisture content.

From Adama-Awash overlay project, engineering report, CBR value of the road section used for this study. The CBR values are shown in Appendix A of this document. Then the design CBR is calculated.

According to Asphalt Institute Manual (1970), the design CBR is defined as the CBR value that 90 percent of all test values in the section are equal to or greater than. Based on an Engineering Manual for Highway Pavement Design this is determined as shown in Appendix A. Using the design CBR, Subgrade class selected from ERA 2013 design manual.

3.2.1.2 Flexible Pavement Design Using AASHTO 1993 Guide

At the conclusion of the AASHTO flexible pavement equation show the relationship traffic loading, material properties with structural number. AASHTO flexible pavement design equation used to determined structural number.

Adama –Awash road section considered as Undivided Primary Route and most of the route is rural section, design variable recommended by VDOT used to solve the equation.

The subgrade resilient modulus of Adama-Awash road section obtained by correlation of CBR value of test pit values and Falling Weight Deflictometer(FWD) result using equation shown in Appendix A. For design, resilient modulus from CBR and backcalculated Mr results are available, according to VDOT use the smaller Design Mr for pavement design purposes.

3.2.2 Composite Pavement

To compare the composite pavement output with the flexible pavement, it is important to design the composite pavement systems. There is no design procedure for composite pavement in ERA manual as well as AASHTO 1993 Guide, but AASHTO 1993 Guide "AC Overlay of PCC Pavement" principle can be used in this study to design composite pavement structure.

In this case, the first step is to design a conventional PCC pavement, in other words, compute the thickness to satisfy the traffic demand, Df. Once the slab thickness has been obtained, it could be assumed that placing an AC layer with a thickness slightly greater than 2 in. would allow for the decrease of 1 in. of PCC layer. This is because the guide's "AC Overlay of PCC Pavement" procedure indicates that the required thickness, Dol, of an AC overlay of PCC is calculated using the following equation:

$$D_{ol} = A(D_f - D_{eff}) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{equation 3.2}$$

Where:

A = factor to convert PCC thickness deficiency to AC overlay thickness

Df = slab thickness to carry future traffic (in.)

Deff = effective thickness of existing slab (in.)

Therefore, two assumptions are made. First, in a new composite pavement design, Deff is equal to Df because it is appropriate to assume that a newly constructed PCCP would not have any distresses, thus none of the adjustment factors shown in Equation (3.2) would be applicable.

Deff = Fjc *Fdur *Ffat *D………………………equation 3.3

Where:

D = original slab thickness (this would be equal to Df)

Fjc, Fdur, Ffat = adjustment factors for joints and cracks, durability, and fatigue = 1

The second assumption involves the A factor in Equation (3.2). According to the guide, the A factor is computed using the following equation:

$A = 2.2233 + 0.0099\,(D_f - D_{eff})^2 - 0.1534\,(D_f - D_{eff})$ …………equation 3.4

Because it was established that Df = Deff, A = 2.2233 would be obtained. This implies that, for every 1 in. of PCC thickness; 2.2233 in. of HMA should be used as overlay, because the conversion factor (A) is 2.2233. Once the overlay thickness is computed, it is rounded up to the nearest 0.5 in. Therefore, 2.2233in. HMA can provide sufficient structural support as 1in. PCC for a certain amount of traffic. First the rigid pavements designed and then apply the above principle to design the composite pavement.

The PCC slab thickness reduced with relationship of 2.2233in. HMA can provide sufficient structural support as 1in. PCC for a certain amount of traffic.

3.2.2.1 Rigid Pavement Design Using ERA Design manual

If rigid pavements properly constructed, the pavement will last a long time with a high level of serviceability and low maintenance requirements, because durability of concrete is very high. As previously stated the design period of 15 and 30years were selected for flexible and composite pavements respectively.

In order to determine the total traffic over the design period of composite pavement, the same procedure follows as flexible pavement.

Growth rate of 9.4% and 8.5% for Adama-Awash and Awash-Mille road sections respectively used to forecast the traffic

3.2.2.2 Rigid Pavement Design Using AASHTO 1993 Guide

The design procedure for rigid pavements is based on a selected reduction in serviceability and similar to the procedure for flexible pavements. However, instead of measuring pavement strength by using a structural number, the thickness of the PCC slab is the measure of strength using AASHTO equation for rigid pavement.

Adama –Awash road section considered as Undivided Primary Route and most of the route is rural section, VDOT recommend the following design variable values for rigid pavement structure design using AASHTO 1993 method for this type of routes.

The modulus of Subgrade reaction is known or obtained from the CBR, then use the following equation:

k-value = Mr / 19.4…………………………….equation 3.5

= 10000/19.4= 515.46, which is greater than 500

VDOT recommend to use k-value of 500 as the design value if k-value (based on backcalculation or subgrade resilient modulus) is larger than 500.

3.3 Technical Analysis of Pavement Structure

To understand the technical advantages of composite pavements, a technical evaluation performed. This evaluation involves a mechanistic modeling of a typical composite pavement structure that is designed with AASHTO (1993) guide for Design of Pavement Structures.

3.3.1 Mechanistic Analysis

A mechanistic-based analysis will be performed in order to understand and model pavement behavior and responses using KENPAVE software such as;

- ✓ Deflections
- ✓ Horizontal Stresses and Strains
- ✓ Vertical Stresses and Strains

KENPAVE software used to model the response of the pavement in this study, based on the nonlinear elastic analysis theory, the mechanistic responses of pavement structures. This is done by changing the rigidity of base course layer then, the corresponding structure response i.e stress strain and deflection compared.

Material characteristics such as elastic modulus, Poisson ratio of individual layer, design traffic in Msa, wheel load, tire pressure and coordinates at which stress, strain and deflection are required and these values were used from AASHTO 1993 and Huang 2004.

KENPAVE software output used to examine the deflection of asphalt concrete at the surface, starting from center of wheel with radial interval 0f 1.5m up to 13.5m used and analyzed for different base course material.

Horizontal stresses have been investigated. The stresses obtained for the plot were computed using the KENPAVE computer software at center of wheel of different depth of pavement structure.

Vertical strains have been used in the past to determine how much deformation is likely to occur on top of the subgrade and analyzed based on the software output at center of wheel for different depth of structure.

3.4 Economic Analysis of Pavement Structures

Design period of 15 and 30 years used for flexible and composite pavements respectively. Type of work activities and timing determined first to compute life cycle cost of each alternative.

The bills of quantities for the two pavements used in this study are meant to cover a 1km road length. The rates were derived from two contractors namely Defense Construction Enterprise who bid for a section of Ditchoto Gulafi Junction – Elidar –Belecho rigid pavement Project and Ethiopian Construction Works Corporation who bid a section Awash -Mille Road Asphalt Overlay Project, Contract 2.

Two methods, present worth (PW) and equivalent uniform annual costs (EUAC) used to compare cost of alternative pavement structures.

4.0 ANALYSIS AND DISCUSSION

The evaluation of composite pavement systems is based on a technical and economic analysis. To perform a technical analysis, the technical benefits of a composite structure should be investigated first. In addition, various mechanistic-based analyses will conduct on various composite pavements, mainly varying the type of rigid base, to understand the behavior of the structure under various base rigidity scenarios. Design of typical composite pavements perform using available applicable guide and compared to flexible pavement. An economic evaluation perform by, a deterministic approach. Typical agency values should be obtained for material cost, rehabilitation procedures, and discount rates.

4.1 Design Pavement Structures

This study concerns on comparative study of flexible and composite pavement structures, to do this comparison, first pavement structures should be designed with appropriate design procedure. Ethiopian Road Authority (ERA) design manual and American Association of State Highway and Transportation Officials (AASHTO) 1993 guide for design of pavement structures have been used for this study to design the pavement for both pavement structures.

4.1.1 Flexible pavement

To design flexible pavement structure using Ethiopian Road Authority (ERA) design manual, two important pavement design parameters are; traffic expected to use the route over the design year and subgrade property along the route. Traffic should be estimated in terms of the cumulative number of equivalent standard axles that will use the road over the selected design life and subgrade soil strength should be characterized over which the road is to be built. The strength of the road subgrade for flexible pavements is commonly assessed in terms of the California Bearing Ratio (CBR) and this is dependent on the type of soil, its density, and its moisture content. Using the input data obtained from traffic and Subgrade to select a suitable structure from the catalogue of pavement structures from ERA manual.

The first step in this design process of new pavement structure, selection of the route, for this study the route has already known, because the study concerns on existing road. The next steps analysis of traffic and material, these steps will be analyzed in next section of this document

4.1.1.1 Traffic

Determining an appropriate design period is the first step towards pavement design. Many factors may influence this decision, including budget constraints. However, the designer should follow certain guidelines in choosing an appropriate design period, taking into account the conditions governing the project. Some of the points to consider include: Functional importance of the road, Traffic volume, Location and terrain of the project, financial constraints and Difficulty in forecasting traffic.

Addis Ababa – Djibouti road section classify as trunk road and ERA recommends design year of 20 for flexible pavement and 40 year for rigid pavement for such heavily traffic road sections.

Practically from the observation, flexible pavement fail early from design year of 20 and due to uncertainty of traffic and Subgrade data for this study, design period of 15 and 30 year for flexible and composite pavement respectively used.

Table 4.1: Design period of pavement structures used for this study

Pavement Type	Design Period (year)
Flexible Pavement	15
Composite Pavement	30

To forecast traffic growth it is usually necessary to separate traffic into normal, diverted and generated traffic.

In order to determine the total traffic over the design period of the road, the first step is to estimate initial traffic volumes. The traffic count conducted by ERA in 2014 is the recent traffic data used as initial traffic for this study. To design and construct the road assumed to take 4years (1 year for design and 3 years for construction)

There is a significant change in volume of traffic between Adama - Awash and Awash - Mille road sections that will require different pavement layer thickness and material to be used for each section, so it is better to analyze and forecast the traffic separately for each section. Traffic volume in terms of AADT has been gained from ERA traffic volume count conducted in 2014.

Even with stable economic conditions, traffic forecasting is an uncertain process. Although the pavement design engineer may often receive help from specialized professionals at this stage of the traffic evaluation. As an alternative to time, growth can be related linearly to anticipate Gross Domestic Product (GDP). This is normally preferable since it explicitly takes into account changes in overall economic activity. An average growth rate of 10.5% was achieved from average GDP of Ethiopia.

Growth rate value computed from GDP is higher than historical data rate. For purposes of this study it is better to use the traffic growth rate computed from historical data, because there are another complementary mode of transport other than road transport, i.e., railway transport already constructed and pipe transport for gas and oil transport proposed to be constructed between Addis Ababa and Djibouti, so the probability of traffic to grow linearly equivalent to GDP of the country will be less for this road sections. An estimated growth rate of **9.4% and 8.5 %** are utilized for traffic volume projections of Adama-Awash and Awash-Mille road sections respectively for this study.

Adjustments will usually be required between AADT of latest base year traffic counts and the AADT during the opening year of the road. Assume that the construction of the proposed road will be completed and open to traffic in 2018.

In order to determine the likely normal traffic volume of road opening year i.e in 2018, the recently year traffic count, i.e., base year traffic in 2014 will be forecasted with the growth rate estimated above. The time duration between traffic count year (2014) and road opening year (2018) is 4 years; the volume of normal traffic in 2018 can be estimated using the following equation and the result of computation shown in tables below:

$AADT(v)1 = AADT(v)0\ (1+i)^n$……………………..equation 4.1

Where;

AADT(v)1 = The AADT of vehicle class v in road opening year (2018)

AADT(v)0 = The AADT of vehicle class v in recent traffic count year(2014)

i= anticipated traffic growth rate (**9.4% and 8.5 %)**

n=time duration in year between traffic count and road opening year (4years)

Table 4.2: AADT of Adama - Awash road section in 2018

Vehicle class	Car	Land rover	Small bus	Large bus	Small truck	Medium Truck	Heavy truck	Truck and Trailer	Total
AADT(v)o	263	588	686	366	573	819	976	1,484	5,755
AADT(v)1	377	842	983	524	821	1173	1398	2126	8244

Table 4.3: AADT of Awash - Mille road section in 2018

Vehicle class	Car	Land rover	Small bus	Large bus	Small truck	Medium Truck	Heavy truck	Truck and Trailer	Total
AADT(v)o	20	135	193	79	94	158	118	112	904
AADT(v)1	10	98	105	17	96	98	256	1623	2303

Traffic volume of each vehicle class estimated for 2018 is only normal traffic. So, this volume should be adjusted by adding the estimate for diverted traffic and for generated traffic, and subtract the estimated traffic due to modal change to other mode of transport if any are anticipated.

According to the Ethiopian Railway corporation study the realization of the Addis - Djibouti railway project would relieve the ever increasing costs associated to both infrastructure development and operation of the road transport sector due to the current global trends manifested in the form of international crude price movements that is becoming highly volatile; high increase in the costs of bitumen, lubricants and spares that are directly linked to the road transport sector. Specifically the country's economy would benefit from the transit time and vehicle operating cost savings resulting from the rail transport mode as compared to the road transport mode due to the shorter route distances and faster speeds. According to a recent feasibility study conducted on the Addis – Djibouti Railway Project1 the proposed new and more efficient railway system is expected to increase the average travel speed to 80-90 km ph that is almost two fold compared to the 40-50 km ph of the road mode at an average.

The railway line would serve as an alternative main international access route to Ethiopia through Djibouti port and it complements the existing ever busy Addis – Djibouti road, which is the only import/export corridor as well as international access at the moment of which the project road is the main section of the this import and export corridor.

The results of the Origin Destination (OD) survey conducted by Transport Construction Design Share Company (TCD) along the Adama – Awash road project for overlay activity of the section in 2011 indicate that most of the goods traffic represent import and export traffic using this section of the road corridor that is leading to Djibouti port. According to these results the origin and destination of most of the truck trailers has been identified to be Djibouti Port, which accounts for 74.9 and 91.1 percent of the total traffic, respectively. The modal split and distribution in this study has considered the results of the OD survey analyzed in terms of passenger and goods traffic separately.

The Ethiopian Railway corporation feasibility study of Addis Djibouti Railway project2 has considered 80 percent as approximate proportion of the total traffic diverting/shifting to the rail transport mode. This proportion of the modal shift is considered in the estimation of the modal shift of freight traffic after the realization of the Railway project in 2015 and assume the railway be able to give complete service in 2018 .

AADT of the road section after the adjustment due to modal shift is estimated as shown in table below for road opening *yea*r:

Table 4.4: AADT of Adama - Awash road section in 2018 including modal shift

Vehicle class	Car	Land rover	Small bus	Large bus	Small truck	Medium Truck	Heavy truck	Truck and Trailer	Total
AADT(v)o	263	588	686	366	573	819	976	1,484	5,755
AADT(*v*)1	377	842	983	524	821	1173	1398	2126	8244
AADT(*v*)1	377	842	786	210	780	939	280	425	4639

Table 4.5 : AADT of Awash - Mille road section in 2018 including modal shift

Vehicle class	Car	Land rover	Small bus	Large bus	Small truck	Medium Truck	Heavy truck	Truck and Trailer	Total
AADT(v)o	20	135	193	79	94	158	118	112	904
AADT(*v*)1	10	98	105	17	96	98	256	1623	2303
AADT(*v*)1	10	98	84	7	91	79	51	325	745

Based on the traffic projections as per ERA design manual 2013 the design standard of the Road is taken as DC7 and DC5 for Adama to Awash and Awash to Mille road sections respectively.

Table 4.6: Design standard of the road sections

Design Standard	Design Traffic (AADT)	Surface Type	Carriageway Width (m)	Flat terrain each side of carriageways Shoulder width (m)
DC7	3,000-10,000	Paved	7.3	3
DC5	300-1,000	Paved	7	1.5

Source: ERA manual 2013

For structural pavement design the cumulative traffic loading of each of the vehicle classes over the design life of the road in one direction is required. For a given vehicle class, v, this can be computed by the following equation:

$T(v) = 0.5 \times 365 \times AADT(v)1\ [(1+i/100)^N - 1]/(i/100)$ ………………….equation 4.2

Where:

$T(v)$ = the cumulative traffic of vehicle class v

AADT(v)1 = The AADT of vehicle class v in the first year/road opening year

N = the design period in years =15 years

i = the annual growth rate of traffic in percent = 9.4% & 8.5%

Design period of 15 years, the cumulative number of vehicles in one direction over the design period is calculated as follows:

Cumulative Land Rove over 15 year in one direction=$0.5*365*842*[(1.094)^{15}-1]/0.094$= 4657524

Table 4.7: Cumulative Traffic for each vehicle class of Adama - Awash road section

Vehicle class	Car	Land rover	Small bus	Large bus	Small truck	Medium Truck	Heavy truck	Truck and Trailer	Total
AADT (v)1	378	844	789	210	782	941	280	426	4650
T(v)	2083212	4657524	4347022	1159628	4311774	5189812	1546171	2350941	25646084

Table 4.8: Cumulative Traffic for each vehicle class of Awash-Mille road section

Vehicle class	Car	Land rover	Small bus	Large bus	Small truck	Medium Truck	Heavy truck	Truck and Trailer	Total
AADT(v)1	29	194	222	45	128	176	51	48	893
T(v)	85648	868719	743917	58730	802035	694975	452713	2865549	6572286

The damage that vehicles do to a paved road is highly dependent on the axle loads of the vehicles. For pavement design purposes the damaging power of axles is related to a "standard" axle of 8.16 metric tons (80KN) using empirical equivalency factors. In order to determine the cumulative axle load damage that a pavement will sustain during its design life, it is necessary to express the total number of vehicles that will use the road over this period in terms of the cumulative number of equivalent standard axles (ESAs). Equivalent factor of each vehicle class used for this study shown in table below:

Table 4.9: Average equivalency factors for different vehicle class

Direction	Car	4x4	small Bus	Large Bus	Small Truck	Medium Truck	Heavy Truck	Truck Trailer	Total
Awash - Adama lane	0.05	0.06	0.01	0.05	0.14	5.72	9.34	11.45	27.44

Source: Adama-Awash road overlay project engineering report

The cumulative ESAs over the design period for each vehicle class is obtained by multiplying EF(v) by the cumulative traffic, T(v). Finally, the total number of cumulative standard axles for all vehicle classes is then obtained by adding together the values of EF(v) x T(v) for all the classes.

Table 4.10: Total equivalent standard axles (ESAs) of Adama - Awash road section

Vehicle Class	Cumulative number of vehicle (T(*v*)	Equivalent factor	Total ESAL (10^6)
Car	2083212	0.05	0.1
Land rover	4657524	0.06	0.28
Small bus	4347022	0.01	0.04
Large bus	1159628	0.67	0.78
Small Truck	4311774	0.14	0.6
Medium truck	5189812	5.72	29.69
Heavy truck	1546171	9.34	14.44
Truck and Trailer	2350941	11.45	26.92
Cumulative ESAs over 15year			**72.85**

Table 4.11: Total equivalent standard axles (ESAL) of Awash - Mille road section

Vehicle Class	Cumulative number of vehicle (T(*v*)	Equivalent factor	Total ESAL (10^6)
Car	49983	0.05	0
Land rover	506974	0.06	0.03
Small bus	434141	0.01	0
Large bus	34274	0.67	0.02
Small Truck	468059	0.14	0.06
Medium truck	405579	5.72	2.32
Heavy truck	264198	9.34	2.47
Truck and Trailer	1672301	11.45	19.15
Cumulative ESAs over 15year			**24.05**

Accurate estimates of cumulative traffic are difficult to achieve due to errors in the surveys and uncertainties with regard to traffic growth, axle loads and axle equivalencies. To a reasonable extent, however, pavement thickness design is not very sensitive to cumulative axle loads and the method recommended in ERA manual provides fixed structures of paved roads for ranges of traffic as shown in table below:.

Table 4.12: Traffic Classes for Flexible Pavement Design

Traffic Classes	Range of ESAs (millions)
LV1	< 0.01
LV2	0.01 – 0.1
T1/LV3 (see note)	0.1 – 0.3
T2/LV4 (see note)	0.3 – 0.5
T2/LV5 (see note)	0.5-0.7
T3	0.7 – 1.5
T4	1.5 – 3.0
T5	3.0 – 6.0
T6	6.0 – 10
T7	10 – 17
T8	17 – 30
T9	30 – 50
T10*	50 – 80
T11	>80

Source: ERA Manual 2013

Based on the above analysis, the Trunk road under study belongs to the traffic class **T10** and **T8** for Adama-Awash road and Awash-Mille road sections respectively of flexible pavement design.

4.1.1.2 Subgrade

A flexible pavement is one which has low flexural strength. Thus, the external load is largely transmitted to the sub grade by the lateral distribution with increasing depth. Because of low flexural strength, the pavement deflects if the sub grade deflects. The pavement thickness is so designed that the stresses on the sub grade soil are kept within bearing power and the sub grade is prevented from excessive deformations.

This implies that in a flexible pavement, the sub grade plays an important role as it carries the vehicle loads transmitted to it through the pavement. If the pavement itself is very strong, but it is constructed on loose and poor sub grade, it can fail. Thus, before any construction the study of the strength of the soil is very much important.

The types & strength of sub grades are largely determined by the location of the road. The soil within the corridor of the road usually varies significantly in strength from place to place.

Thus strength of road sub grade should be determined &the determination of sub grade strength is assessed in terms of ***CBR*** (***Californian Bearing Ratio***) which depends on the types of soil, its density and moisture content.

The Transport Construction Design Share Company (TCD) has carried out soil investigation both in the field and laboratory. The CBR values shown in Appendix A of this document

For particular road project the CBR value of the sub grade is determined from laboratory test result taken at a certain intervals along the route which is assumed to be a representative CBR for that particular interval. Then the design CBR is calculated.

The road has been divided into sections in which the subgrade soil is fairly uniform. In each section, the design CBR is determined statistically from all CBR values measured in that section. According to Asphalt Institute Manual (1970), the design CBR is defined as the CBR value that 90 percent of all test values in the section are equal to or greater than. Based on an Engineering Manual for Highway Pavement Design this is determined as shown in Appendix A.

From the calculation of the design subgrade CBR value at 90 percent ranges between 4 and 7% CBR value those are at 100 and 96.3 percent respectively. By linear interpolation the design CBR is 6.2% which is approximately 6%. Subgrade CBR value of 6% is used as design CBR throughout the road section for this study.

The structural catalogue given in ERA manual requires that the subgrade strength for design be assigned to one of six strength classes reflecting the sensitivity of thickness design to subgrade strength.

Table 4.13: Subgrade Strength Classes

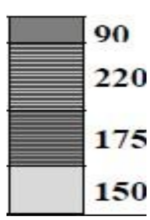
Subgrade Class	CBR Range %
S1	2
S2	3,4
S3	5-8
S4	8-15
S5	15-30
S6	>30

Source: ERA Manual 2013

For the Design CBR value of sub grade in this study, which is 6%, the subgrade strength class is S3 from the above table.

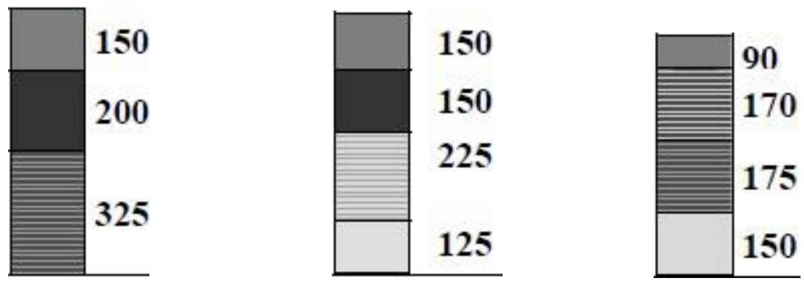
From the analysis of traffic and Subgrade data of the study route, the traffic class and Subgrade class of the road sections which are necessary to select the appropriate pavement structure from flexible pavement design catalogue has already determined. The analysis results are traffic class of T10 and Subgrade class of S3 for Adama-Awash road section; and T8 & S3 for Awash-Mille road section.

Using traffic and subgrade class of each road section Adama-Awash road section has only one pavement structure, but Awash-Mille road section has three alternatives which can sustain the expected traffic over the design year effectively as shown in figures below.



Source: ERA Manual 1013

Figure 4.1: Adama-Awash Road Section pavement Structure



Alternative 1 **Alternative 2** **Alternative 3**

Figure 4.2: Awash-Mille Road Section Pavement Structure Alternatives

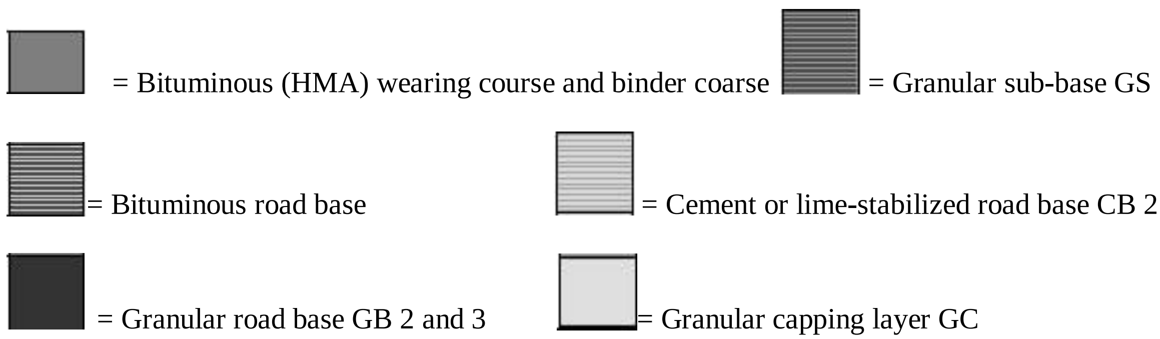


Figure 4.3: Key for Pavement Materials

Table 4.14: Awash-Mille Road Section alternative Pavement Structures

Pavement Components	Design Chart No. and Alternative structure		
	C1 and structure 1	C2 and Structure 2	D and Structure 3
Surfacing:			
Asphalt Concrete	150mm	150mm	90mm
Roadbase:			
Asphalt Concrete roadbase	-	-	170mm
Crushed Stone	200mm	150mm	-
Cement Stabilized(2.5MPaUCS)	-	225mm	-
Granular sub-base	325mm	125mm	175mm
Selected Fill/ capping	-	-	150mm

All of these three structures able to perform well but they are not expected to deteriorate in the same way. Local experience is required for each type to calibrate performance models and allow more accurate whole life cost principles to be used to identify the true best value options.

The initial choice from these alternative pavement structures should be based on minimum total transport costs, i.e., local costs of the feasible option.

Analyses of recent contracts, production costs, hauling distances and associated costs have established relative costs for the various alternate pavement layers. With these elements, the relative costs of the possible alternate pavement structures are should be evaluated.

There is no information about the timing and the way these pavement structures deteriorate to compare the life cycle cost of each pavement structure, this makes difficult to select the best among these three alternatives.

Alternative 1 is selected for this study, because structure 2 has Cement Stabilized subbase layer ; and structure 3 has asphalt Concrete roadbase and Selected Fill/ capping layer, due to these layers the initial construction cost of alternative 2 and alternative 3 expected to be higher.

4.1.1.3 Flexible Pavement Design Using AASHTO 1993 Guide

At the conclusion of the AASHTO flexible pavement equation show the relationship traffic loading, material properties with structural number as shown below.

$$\log_{10} W18 = Zr * So + 9.36(\log_{10}(SN+1)) - 0.20 + \frac{\log_{10}\left(\frac{\Delta PSI}{2.7}\right)}{0.40 + \left(\frac{1094}{(SN+1)^{5.19}}\right)}$$

$+\ 2.32 \log_{10} Mr -\ 8.07$……………………….equation 4.3

Where;

W_{18}= 18kip equivalent single axle load

S_o= overall standard deviation of traffic

SN=structural number

Zr = reliability (z-statistics from the standard normal curve)

ΔPSI= loss in serviceability from the time the pavement is new until it reaches its TSI.

Mr= soil resilient modulus of the sub grade in lb/in^2

Adama –Awash road section considered as Undivided Primary Route and most of the route is rural section, VDOT recommend the following design variable values for flexible pavement structure design using AASHTO 1993 method for this type of routes. The subgrade resilient modulus of Adama-Awash road section obtained by correlation of CBR value of test pit values and Falling Weight Deflictometer(FWD) result using equation shown below.

For fine-grained soils with a soaked CBR between 5 and 10, use the following equation to correlate CBR to resilient modulus (Mr):

Mr (psi) = 1,500 x CBR………………………..equation 4.4

For non fine-grained soils with a soaked CBR greater than 10:

Mr = 3,000 x CBR $^{0.65}$…………………………equation 4.5

After Adama- Awash road section, test pit CBR values results changed to the equivalent resilient modulus using the above equation the average resilient modulus taken as design resilient modulus i.e 22678psi

FWD testing is conducted and the backcalculated resilient modulus is determined, uses the following equation:

Design Mr = C x Backcalculated Mr………..equation 4.6

Where C = 0.33

The average back-calculated Mr = 30288.1

Based on the above equation Design Mr= 0.33 x 30288.1=9995.1≈ 10000psi

Two design resilient modulus from CBR and backcalculated Mr results are available, according to VDOT use the smaller Design Mr for pavement design purposes. So, subgrade design resilient modulus of 10000psi used for this study.

Table 4.15: Structural layer coefficient

Pavement component	Coefficient
Wearing surface	
Hot mix asphaltic concrete	0.44
Base Course	
Crushed stone	0.14
Subbase Course	
Gravel	0.11

From traffic analysis of Adama- Awash and Awash-Mille road sections, cumulative ESAL (W18) of 72.9 and 24.1million expected to use and using the above design variable, structural number required to sustain the estimated cumulative ESAL over the design year are 6.17 and 5.29 respectively. These structural numbers computed using simple spread sheet to solve AASHTO flexible pavement design equation.

AASHTO 1993 guide for design of pavement structures recommends the minimum thickness of Asphalt concrete and aggregate base pavement layers of 4 and 6in respectively for cumulative ESAL greater than 7 million. Initially asphalt concrete and base course layer thickness selected, and then subbase layer computed.

Table 4.16: Flexible Pavement Layer thickness (Adama-Awash road section)

Layer	Material	Layer Coefficient	Resilient modulus	Required SN	Actual used SN	thickness used(D*)
surface course	HMA	0.44		SN1=3.60	SN*1=3.74	8.5in.
Base	Crushed stone	0.14	45000psi	SN2=4.91	SN*2=5.14	10in.
Subbase	Aggregate	0.11	20000psi	SN3=6.17	SN*3=6.17	9.5in.

Subbase course pavement layer thicknesses which, when combined, will provide the load carrying capacity corresponding to the design SN=6.17 can be computed from the following equation.

$$SN = a_1D_1 + m_2a_2D_2 + m_3a_3D_3 \text{……………………equation 4.7}$$

$$6.17 = 0.44\times8.5 + 1\times0.14\times10 + 1\times0.11\times D3$$

$$D_3 = 9.36\text{in.} \approx 9.5\text{in.}$$

The appropriateness of these thicknesses can be checked by layer design analysis.

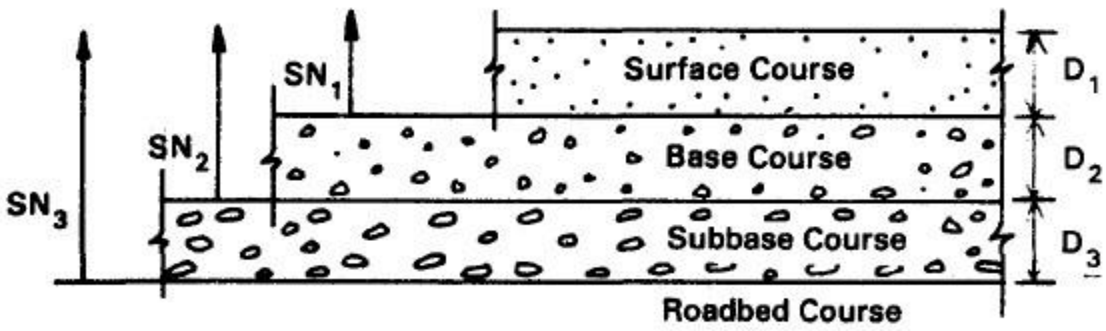


Figure 4.4: procedure for Determining Thickness of Layers using a layered Analysis Approach

The appropriateness of these thicknesses can be checked by layer design analysis. The required structural number over subbase(SN2) and base course (SN1)computed using the AASHTO 1993 flexible pavement design equation and actual SN computed as follows:

SN*1=a1×D*1=0.44×8.5=3.74

SN*2=a1×D*1+ a2×D*2= 0.44×8.5+0.14×10 = 5.14

SN*3=a1×D*1+ a2×D*2+a3×D*3= 0.44×8.5+0.14×10+0.11×9.36 = 6.17

D, a, m and SN are minimum required values

An asterisk with D or SN indicates the value actually used, which must be equal to or greater than the required value.

From required and actual SN comparison

SN*1>SN1, SN*2>SN2 and SN*3=SN3, this shows that the actual pavement thicknesses are appropriate (safe) to sustain the traffic.

Table 4.17: Flexible Pavement Layer thickness (Awash-Mille road section)

Layer	Material	Layer Coefficient	Resilient modulus	Required SN	Actual used SN	thickness used
surface course	HMA	0.44		SN1=2.95	SN*1=3.08	7in.
Base	Crushed stone	0.14	45000psi	SN2=4.12	SN*2=4.2	8in.
Subbase	Aggregate	0.11	20000psi	SN3=5.29	SN*3=5.29	10in.

Subbase course pavement layer thicknesses which, when combined, will provide the load carrying capacity corresponding to the design SN=5.29 can be computed from the following equation.

$$SN = a_1 \times D_1 + m_2 \times a_2 \times D_2 + m_3 \times a_3 \times D_3$$

$$5.29 = 0.44 \times 7 + 1 \times 0.14 \times 8 + 1 \times 0.11 \times D_3$$

$$D_3 = 9.91\text{in.} \approx 10\text{in.}$$

The appropriateness of these thicknesses can be checked by layer design analysis. The required structural number over subbase(SN2) and base course (SN1)computed using the AASHTO 1993 flexible pavement design equation and actual SN computed as follows:

$$SN^*1 = a1D^*1 = 0.44 \times 7 = 3.08$$

$$SN^*2 = a1 \times D^*1 + a2 \times D^*2 = 0.44 \times 7 + 0.14 \times 8 = 4.2$$

$$N^*3 = a1 \times D^*1 + a2 \times D^*2 + a3 \times D^*3 = 0.44 \times 7 + 0.14 \times 8 + 0.11 \times 9.91 = 5.29$$

From required and actual SN comparison

SN*1>SN1, SN*2>SN2 and SN*3=SN3, this shows that the actual pavement thicknesses are appropriate (safe) to sustain the traffic.

4.1.2 Composite Pavement

To compare the composite pavement output with the flexible pavement, it is important to design the composite pavement systems. There is no design procedure for composite pavement in ERA manual as well as AASHTO 1993 Guide, but AASHTO 1993 Guide "AC Overlay of PCC Pavement" principle can be used in this study to design composite pavement structure.

In this case, the first step is to design a conventional PCC pavement, in other words, compute the thickness to satisfy the traffic demand, Df. Once the slab thickness has been obtained, it could be assumed that placing an AC layer with a thickness slightly greater than 2 in. would allow for the decrease of 1 in. of PCC layer. This is because the guide's "AC Overlay of PCC Pavement" procedure indicates that the required thickness, Dol, of an AC overlay of PCC is calculated using the following equation:

$D_{ol}= A(D_f - D_{eff})$…………………………equation 4.8

Where:

A = factor to convert PCC thickness deficiency to AC overlay thickness

Df = slab thickness to carry future traffic (in.)

Deff = effective thickness of existing slab (in.)

Therefore, two assumptions are made. First, in a new composite pavement design, Deff is equal to Df because it is appropriate to assume that a newly constructed PCCP would not have any distresses, thus none of the adjustment factors shown in Equation 4.9 would be applicable.

Deff = Fjc *Fdur *Ffat *D………………equation 4.9

Where:

D = original slab thickness (this would be equal to Df)

Fjc, Fdur, Ffat = adjustment factors for joints and cracks, durability, and fatigue = 1

The second assumption involves the A factor in Equation 4.8. According to the guide, the A factor is computed using the following equation:

$A = 2.2233+ 0.0099 (D_f - D_{eff})^2 - 0.1534 (D_f - D_{eff})$ ………………equation 4.10

Because it was established that Df = Deff, A = 2.2233 would be obtained. This implies that because the A factor is 2.2233, for every 1 in. of PCC thickness; 2.2233 in. of HMA should be used as overlay. Once the overlay thickness is computed, it is rounded up to the nearest 0.5 in. Therefore, 2.2233in. HMA can provide sufficient structural support as 1in. PCC for a certain amount of traffic. First the rigid pavement should be designed and then apply the above principle to design the composite pavement.

4.1.2.1 Rigid Pavement Design Using ERA Design manual

If rigid pavements properly constructed, the pavement will last a long time with a high level of serviceability and low maintenance requirements, because durability of concrete is very high. As previously stated the design period of 15 and 30years are selected for flexible and composite pavements respectively

In order to determine the total traffic over the design period of composite pavement, the same procedure follows as flexible pavement.

Growth rate of 9.4% and 8.5% for Adama-Awash and Awash-Mille road sections respectively use used d to forecast the traffic, the traffic analysis results over 30 years are shown in tables below for both road sections.

Table 4.18: Total equivalent standard axles (ESAs) of Adama - Awash road section

Vehicle Class	Cumulative number of vehicle (T(*v*)	Equivalent factor	Total ESAL (10^6)
Car	10099870	0.05	0.51
Land rover	22580697	0.06	1.35
Small bus	21075317	0.01	0.21
Large bus	5622133	0.67	3.77
Small Truck	20904426	0.14	2.93
Medium truck	25161348	5.72	143.92
Heavy truck	7496177	9.34	70.01
Truck and Trailer	11397876	11.45	130.51
Cumulative ESAs over 30year			353.21

Table 4.19: Total equivalent standard axles (ESAL) of Awash - Mille road section

Vehicle Class	Cumulative number of vehicle (T(*v*)	Equivalent factor	Total ESAL (10^6)
Car	219914	0.05	0.01
Land rover	2230557	0.06	0.13
Small bus	1910110	0.01	0.02
Large bus	150798	0.67	0.1
Small Truck	2059338	0.14	0.29
Medium truck	1784445	5.72	10.21
Heavy truck	1162403	9.34	10.86
Truck and Trailer	7357695	11.45	84.25
Cumulative ESAs over 30year			105.87

For the design procedure recommended in ERA rigid pavement design manual, the assumptions are made that the materials used for construction meet the following requirements:

- ✓ 28-day characteristic compressive strength of **30**, **35**, **40**, 45 or 50 MPa for the concrete.
- ✓ Yield strength of the steel reinforcement bars is greater than 400 MPa.

Based on this recommendation Jointed Reinforced Concrete Pavements (JRCP) with 28-day compressive strength of 35MPa used for this study.

Thickness of rigid pavement layers which have the capacity to carry the corresponding traffic over the design period under a given subgrade and other pavement layer properties. Capping, Sub-base, Concrete Slab thickness and Reinforcement can be determined from ERA manual.

A capping layer is required if the design CBR of the subgrade is less than 15%. The required thickness of capping layer and sub-base thickness to the corresponding sub grade class is shown in table below;

Table 4.20: Thickness of Sub-base and Capping Layers

Subgrade Class	CBR range %	Sub-base thickness (mm)	Capping layer thickness (mm)
S1	2	200	400
S2	3,4	175	350
S3	5-8	150	250
S4	8-15	150	200
S5	15-30	175	0
S6	>30	0	0

Source:-ERA Manual 2013

The design CBR in this study as estimated for flexible pavement is 6% which is in range of Subgrade class of S3. So, the thickness of subbase and capping layer is 150mm and 250mm respectively from the above table.

Based on the design traffic volume expressed in Equivalent Standard Axles, the strength of the concrete, the type of rigid pavement, the shoulder design, and the thickness of the pavement are determined as described below.

Figure 4.5: Design Thicknesses for JRCP (35 MPa concrete and tied shoulders)

From figure 4.5, concrete slab thickness required to sustain the cumulative ESAL over 30 year are 368 and 284mm for Adama-awash and Awash-Mille road section respectively.

If the base course for rigid pavement is unsterilized, ERA recommends using additional thickness of concrete slab. Based on this recommendation using subgrade class and traffic class of the road sections, 30mm for Adama-Awash and 25 mm for Awash-Mille road section additional slab thickness should be used. So, the total slab thickness of 398 and 309mm required for Adama-Awash and Awash-Mille road section respectively.

4.1.2.2 Rigid Pavement Design Using AASHTO 1993 Guide

The design procedure for rigid pavements is based on a selected reduction in serviceability and similar to the procedure for flexible pavements. However, instead of measuring pavement strength by using a structural number, the thickness of the PCC slab is the measure of strength as shown in the equation below.

$$\log_{10} W18 = Z_r\, S_o + 7.35[\log_{10}(D+1)] - 0.06 + \frac{\log_{10}\left[\frac{\Delta PSI}{3.0}\right]}{1+\left[1.624*10^7/(D+1)^{8.46}\right]} + (4.22 - 0.32TSI)\log_{10}\left[\frac{s_c\, c_d[D^{0.75}-1.132]}{215.63J\left\{D^{0.75}-\left[18.42/\left(Ec/K\right)^{0.25}\right]\right\}}\right] \quad \text{……………………….equation 4.10}$$

Where;

W_{18}= 18kip (80.1KN) equivalent single axle load

Zr = reliability (z-statistics from the standard normal curve)

S_o= overall standard deviation of traffic

D= PCC slab thickness in inches,

E_c=concrete elastic modulus in lb/in^2

TSI= pavement's terminal serviceability index

C_d= drainage coefficient

S_c= concrete modulus of rupture in lb/in^2

J= Load transfer coefficient

K= modulus of Subgrade reaction in lb/in^3

ΔPSI= loss in serviceability from the time the pavement is new until it reaches its TSI

Adama –Awash road section considered as Undivided Primary Route and most of the route is rural section, VDOT recommend the following design variable values for rigid pavement structure design using AASHTO 1993 method for this type of routes.

If the subgrade resilient modulus is known or obtained from the CBR, then use the following equation:

k-value = Mr / 19.4……………………….equation 4.11

= 10000/19.4= 515.46, which is greater than 500

VDOT recommend to use k-value of 500 as the design value if k-value (based on backcalculation or subgrade resilient modulus) is larger than 500.

Based on the traffic analysis of Adama-Awash and Awash-Mille road section 353.21 and 105.9 million ESAL expected to use the route throughout the design period of 30 years.

Using the above design variable values and the corresponding W18, the PCC slab thickness required to sustain the estimated cumulative ESAL over the design year :

D= 365.25mm=14.38in≈14.5in for Adama-Awash road section

D= 301.75mm= 11.88in≈ 12in for Awash-Mille road section

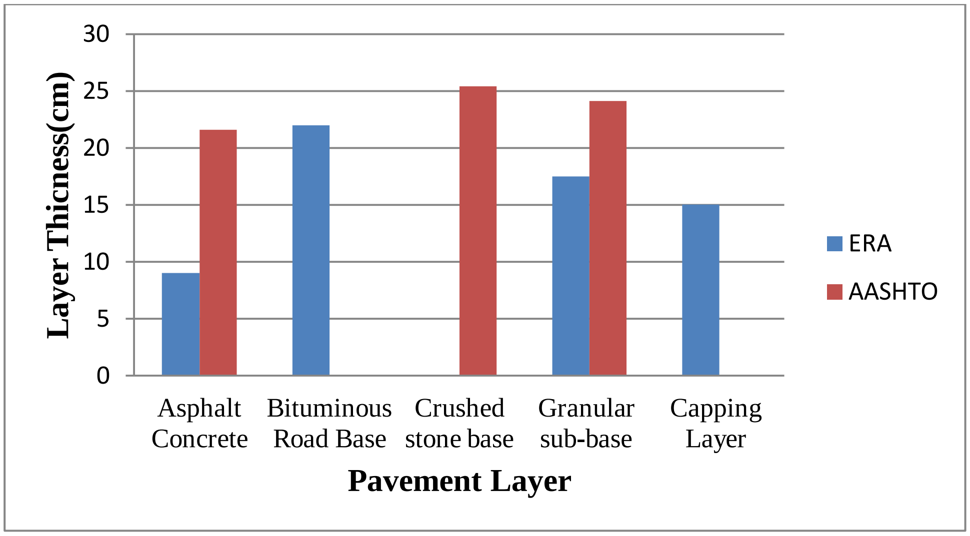


Figure 4.6: Adama-Awash Flexible Pavement Layer thickness

From the above figure, that shows pavement layer thickness design results of ERA2013 and AASHTO 1993 method. There a pavement material difference recommended by ERA 1993 and the material used to design using AASHTO1993. ERA recommend to use Bituminous Road Base ,but crushed stone base used to design with AASHTO method and ERA recommend Capping layer, but capping didn't use for AASHTO method. So, it is difficult to compare flexible pavement layer thickness of Adama-Awash road section.

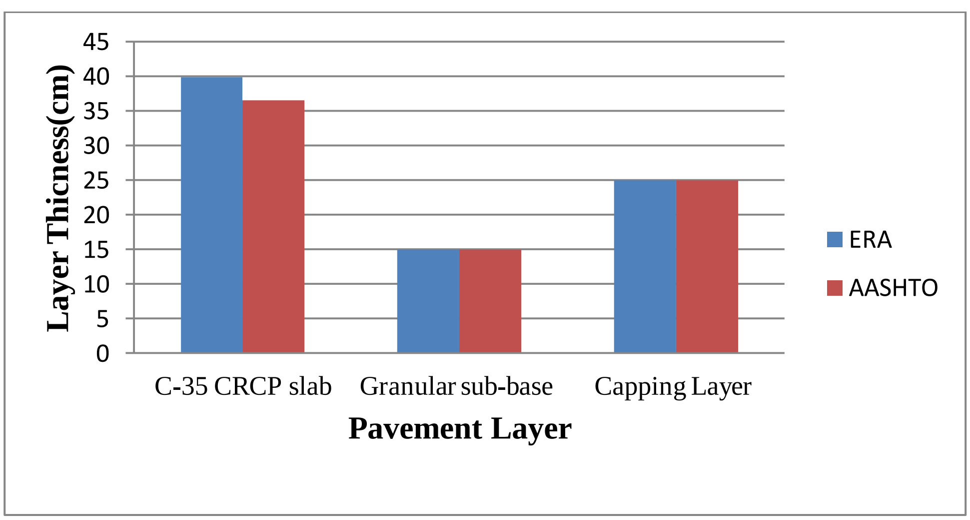


Figure 4.7: Adama-Awash Rigid Pavement Layer thickness

PCC layer is the main load carrying layer for rigid pavement structure, from the above graph PCC layer thickness design value based on ERA is larger than by 3.3cm. The reason for this difference could be material property variation because ERA method doesn't require many design variables as AASHTO method.

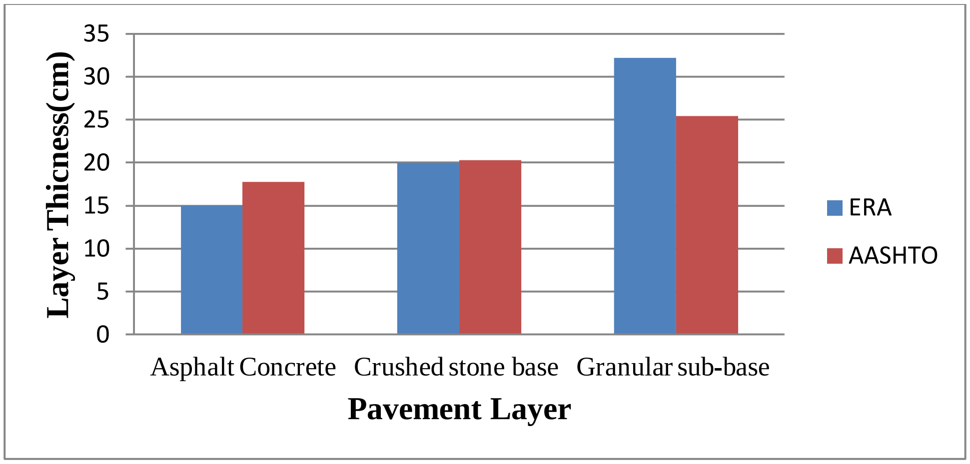


Figure 4.8: Awash-Mille Flexible Pavement Layer thickness

The comparison of Awash-mille flexible pavement layer thickness of ERA and AASHTO methods, crushed stone base layer thickness had no significant difference i.e. 0.32cm between the two methods as shown on above graph. Asphalt concrete thickness based on AASHTO method is higher than ERA method by 2.78cm, and for granular sub-base layer in opposite of asphalt layer, ERA method thickness value was higher than AASHTO method by 6.8cm. This thickness difference of pavement layers may cause the whole pavement structure to be equivalent for two methods for the two methods.

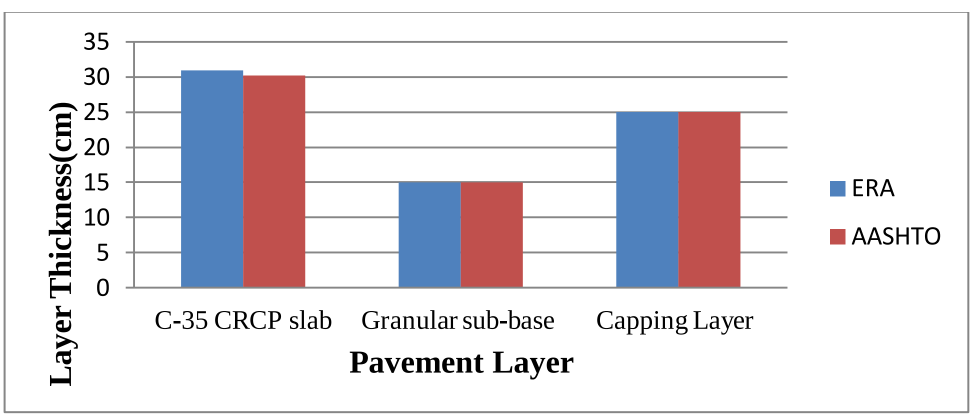


Figure 4.9: Awash-Mille Rigid Pavement layer thickness

PCC layer is the main load carrying layer for rigid pavement structure, from the above graph PCC layer thickness design value based on ERA and AASHTO has no significant difference which was 0.8cm. Cumulative ESAL was the only design variable between Adama-Awash and Awash-Mille road sections rigid pavement design.

The PCC layer thickness difference of Adama-Awash road section for the two methods is higher, but PCC layer thickness difference of Awash-Mille road section was not significant. From this either ERA design method is more sensitive to ESAL or AASHTO method less sensitive to ESAL.

8cm asphalt concrete layer thickness selected for this study and the PCC slab thickness reduced with relationship of 2.2233in. HMA can provide sufficient structural support as 1in. PCC for a certain amount of traffic.

4.2 Technical Analysis of Pavement Structure

To understand the technical advantages of composite pavements, a technical evaluation should be performed. This evaluation involves a mechanistic modeling of a typical composite pavement structure that is designed with AASHTO (1993) guide for Design of Pavement Structures.

4.2.1 Mechanistic Analysis

A mechanistic-based analysis will be performed in order to understand and model pavement behavior and responses such as;

- ✓ Deflections
- ✓ Horizontal Stresses and Strains
- ✓ Vertical Stresses and Strains

KENPAVE is the software used to model the response of the pavement in this study, based on the nonlinear elastic analysis theory, the mechanistic responses of the composite structures. This is done by changing the rigidity of base course layer.

KENPAVE software applies to calculate stresses, strains, deflections in flexible and composite pavements and consists of two well-known computer programs for pavement analysis and design:.

Material characteristics such as elastic modulus, Poisson ratio of individual layer, design traffic in Msa, wheel load, tire pressure and coordinates at which stress, strain and deflection are required.

Awash-Mille road section flexible pavement layer thickness designed with AASHTO 1993 method and the corresponding material properties selected from the above range for KENPAVE software input as shown in table below.

Table 4.21: Typical Material Properties for the Composite Pavement Layers

Layer	Material	Thickness(inch)	Elastic Modulus(psi)	Poisson's Ratio
Surface course	HMA	7	550,000	0.35
Base course	PCC	8	4,100,000	0.15
	RCC		3,600,000	0.15
	Lean concrete		2,100,000	0.15
	CTB		1,100,000	0.20
	Soil cement		550,000	0.20
	Granular base		35,000	0.32
Sub-base	Aggregate sub base	10	25,000	0.40
Subgrade	Soft clay	-	10,000	0.45

The use of a layered elastic analysis computer program will allow one to calculate the theoretical stress, strain and deflections anywhere in a pavement structure

4.2.1.1 Deflections

Composite pavements have been known to provide greater structural support than traditional flexible pavements in addition to the quiet, smooth, comfortable riding surface. High structural support of a pavement structure has been traditionally associated with low deflections at the surface (i.e., deflection measurements are known to be reduced when the bearing capacity of the road is high). In addition, a reduction of deflection under an applied load to the pavement reduces the traffic-induced stresses and strains within the layers of the structure (Nunn, 1997).Therefore, a structure that provides lower deflection measurements would tend to reduce the layers' state of stress and strain, causing the pavement structure to be less affected (damaged) by the loading conditions.

The deflection analysis performed is shown in Fig. 4.10. The deflection data was obtained using the KENPAVE computer software.

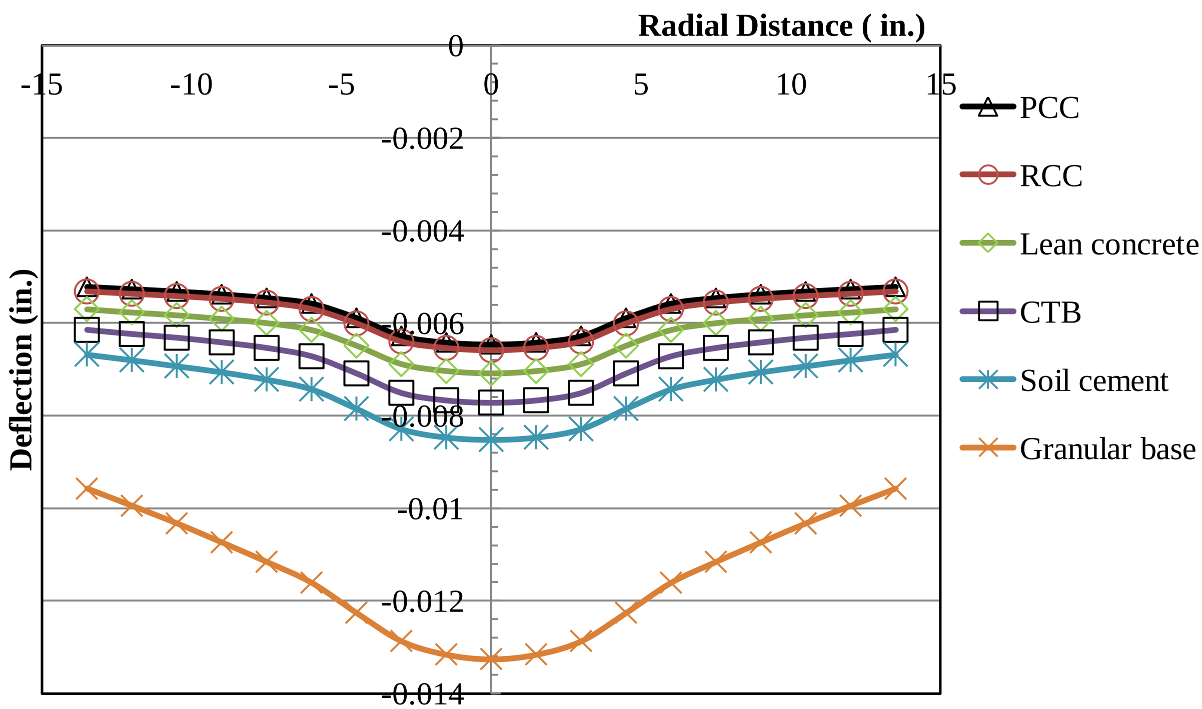


Figure 4.10: Surface Pavement Deflections Varying Base course Stiffness

As confirmed by the mechanistic model, the deflections measured at the pavement surface are greatly reduced as the stiffness of the base increases. In this case, the stiffness or elastic modulus (E) of the base increased from granular base (E = **35,000**psi) to PCC (E = 4,100,000 psi). The maximum deflection obtained when the granular base was used was 1.33×10^{-2} in. The table below shows the percent reduction of deflections, when comparing rigid bases to the granular one, as stiffer base layers were used in the pavement structure.

Table 4.22: Maximum Deflection of Pavement Surface with Different Base-course Material

Base Layer	Max. Deflection (in.)	Percent Reduction (%)
Granular base	0.01326	0
Soil cement	0.00844	36
CTB	0.00782	41
Lean concrete	0.00724	45
RCC	0.00675	49
RCC	0.00662	50

The table shows that as the rigidity of the base increases, the deflections of the pavement structure decrease. This suggests a reduction of stresses and strains in the various pavement layers, especially in the HMA.

4.2.1.2 Horizontal Stresses and Strains

A pavement structure, when subjected to a load, presents stress and strain responses that are a function of the load force, load location, pressure, and material properties, among other factors.

Horizontal stresses have been investigated to understand their effect on failure of HMA and cement-bound materials (e.g., soil cement, CTB, lean mix, RCC, PCC) (Kennedy, 1983). In addition, horizontal strains have also been investigated to predict HMA and cement-bound material failure (Kennedy, 1983).

The stress analysis performed is shown in Fig. 4.11. The stresses obtained for the plot were computed using the KENPAVE computer software.

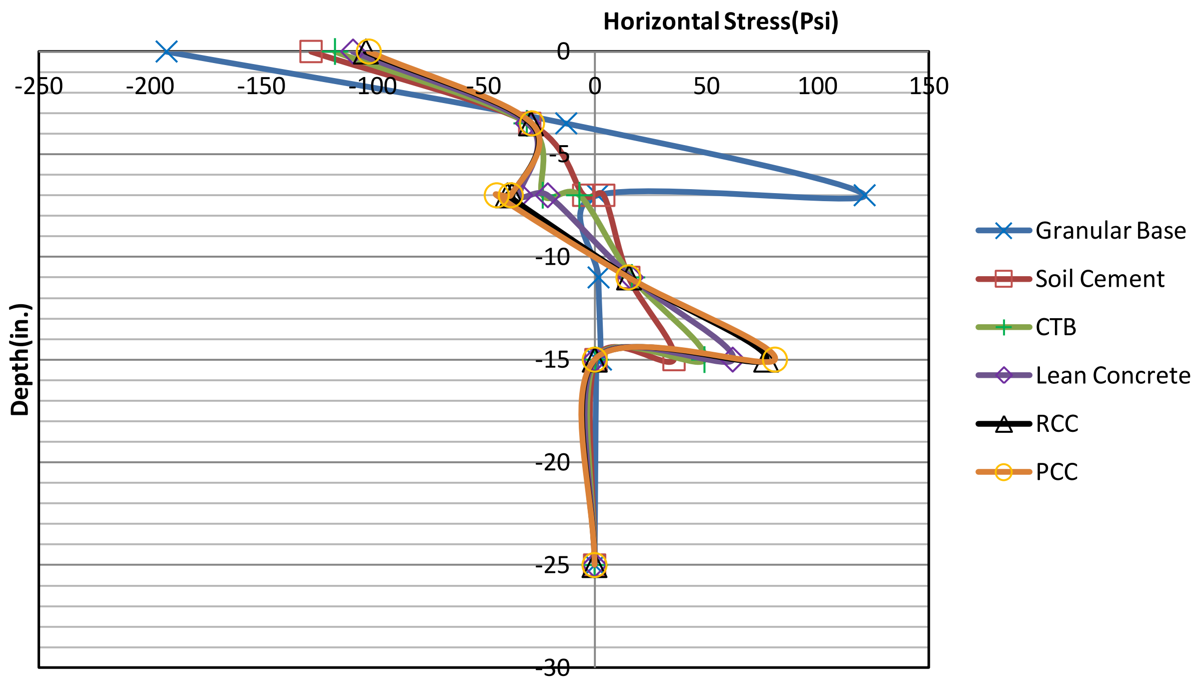


Figure 4.11: Horizontal Stresses Analysis Varying Base course Stiffness

Two observations from the horizontal stresses output of the mechanistic model can be discussed.

First, considerably higher compressive and tensile stresses can be observed in the HMA layer of the typical pavement structure (granular base scenario). In the case of rigid bases, the magnitude of both compressive and tensile stresses is significantly reduced. For a flexible pavement structure, the highest compressive stress is located at the top of the HMA layer, whereas the highest tensile stress is located at the bottom of the HMA layer. For the case of composite pavements, the stresses at the top and bottom of the HMA are in compression due to the rigidity of the base as compared to a granular base.

Second, in the base layer from depths of 7 to 15 in. of the typical flexible pavement structure, the stresses inside the granular base are small because of its low modulus. In the case of composite pavements, the stress distribution goes from compression (top of rigid layer) to tension (bottom of rigid layer). The magnitude of both of these stresses increments as the stiffness of the base increases. According to the literature, the tensile stress at the bottom of the rigid layer criteria is the one used to predict fatigue failure of this layer.

The strain analysis performed is shown in Fig. 4.12. The strains obtained for this plot were computed using the KENPAVE computer software.

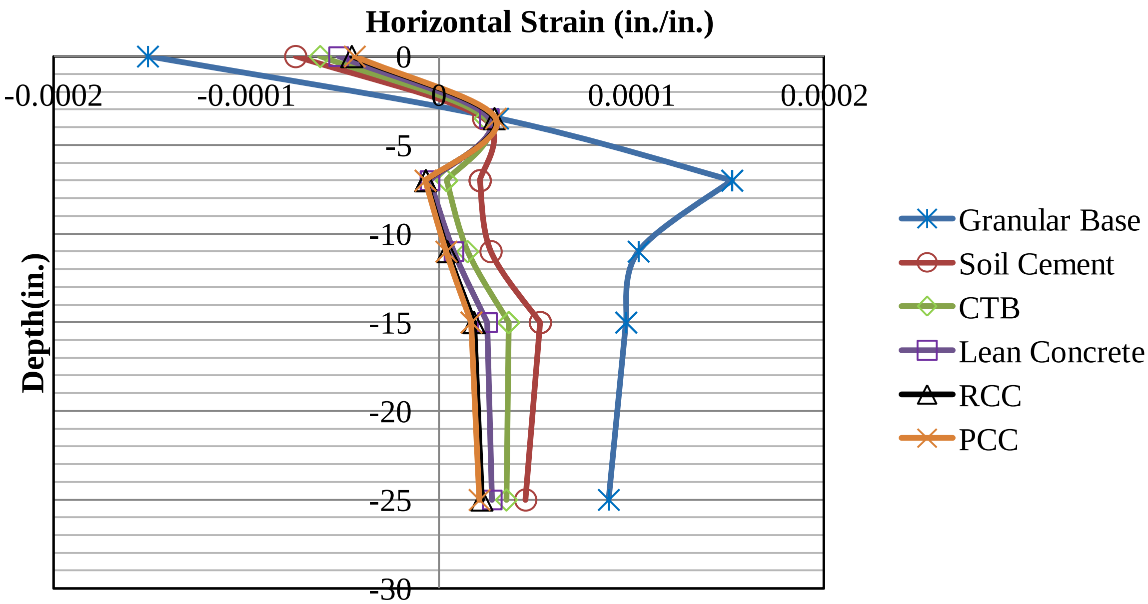


Figure 4.12: Horizontal Strain Analysis Varying Base course Stiffness

The horizontal strains output obtained from the mechanistic modeling are consistent with the results from the horizontal stresses. In this case, it can be observed that the tensile strain at the bottom of the HMA, which is the most commonly used point of interest when investigating flexural fatigue damage, is significantly larger in the granular base case than in any of the ones obtained when a rigid base was used. This suggests that the chance of having fatigue failure in the HMA when using a granular base is much higher than that with any composite pavement structure. Furthermore, the tensile strain at the bottom of the HMA only occurs for granular, soil cement, and CTB bases; when lean concrete, RCC, and PCC are used as bases, the strains become compressive in nature. Thus the likelihood of fatigue cracking is greatly minimized. This phenomenon was also noted in previous publications (NCHRP, 2003).

In the rigid base analysis, due to the rigidity of the base, the highest stresses shown in Fig. 4.11 correspond to the smallest strain responses in Fig. 4.12. In other words, in Fig. 4.11, the rigid layer with the highest stress at the bottom was PCC, whereas in Fig. 4.12, PCC was the layer with the smallest tensile strain magnitude

4.2.1.3 Vertical Strains

Vertical strains have been used in the past to determine how much deformation is likely to occur on top of the subgrade and thus help determine rutting due to subgrade permanent deformation (Huang, 2004). In addition, vertical strains are used in the permanent deformation model proposed by the proposed MEPDG; in this model, resilient vertical strain responses are computed to obtain plastic strain accumulations that are then used to compute the rutting within the HMA layer (NCHRP, Guide for Mechanistic -Emperical Design of New and Rehabilitated Pavement Structures-NCHRP1-37A,National CooperativeHighway Research program, 2004).The vertical strain analysis performed is shown in Fig. 4.13. The strains obtained for the plot were computed using the KENPAVE computer software.

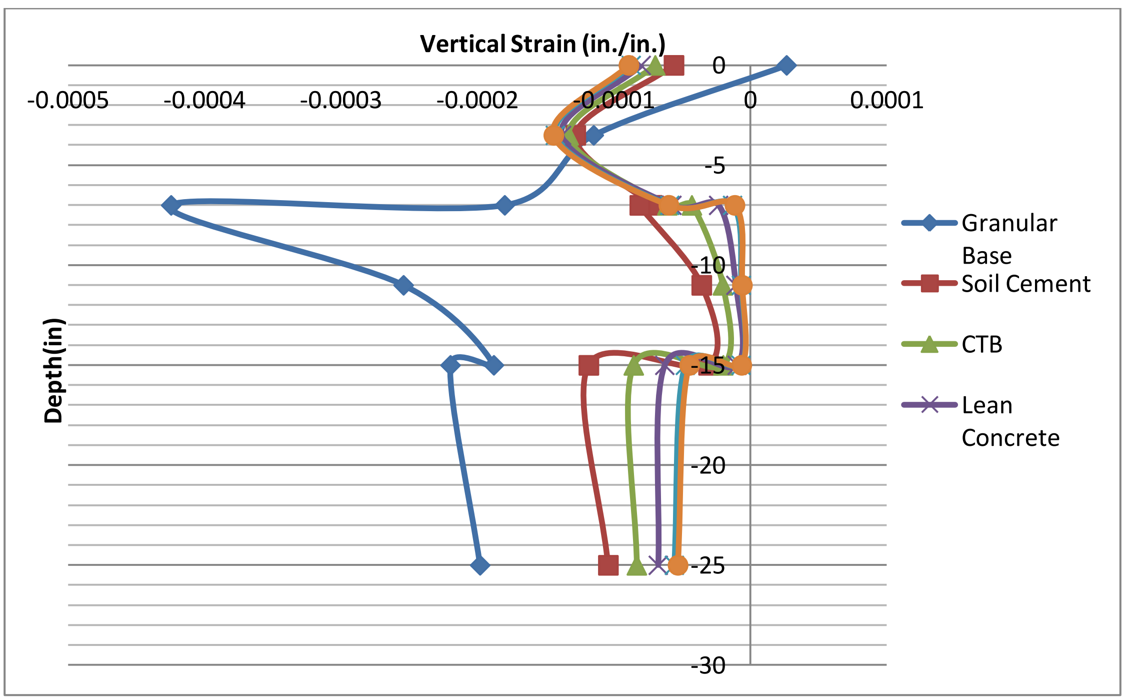


Figure 4.13: Vertical Strain Analysis Varying Base course Stiffness

The mechanistic model output shows an interesting vertical strain distribution especially in the HMA layer (0 to 7 in.). In the granular case, vertical strains at the top region (0 to 0.5 in.) are tensile in nature. The remainder of the strain distribution (granular case) suggests that the rest of the HMA is in compression with the lower region (4 to 7 in.) presenting a greater magnitude of compressive responses. This lower region, in the case of composite pavements, instead of showing an increasing compressive magnitude as the granular case,

shows a reduction of compressive strains. This suggests that higher vertical deformations presented in the HMA are prone to occur in the upper region of the layer (0 to 4 in.)
As the stiffness of the base increases, the compressive strains in the unbound layers (subbase and subgrade) noticeably decrease. The significant reduction of vertical strains at top of the subgrade—at a depth just below 25 inches—suggests that rutting due to permanent deformation of the subgrade is greatly minimized or even unlikely to occur.

4.2.1.4 Effect of Base course layer thickness on Pavement structure

Material variability is one cause for pavement design uncertainty. This variability directly affects pavement performance. Pavement layers material and Subgrade soil properties are important to select the appropriate layer thickness to support load of traffic without excessive failures within the design year.

Depending on the strength of material, thickness of the pavement layer affect the performance of pavement. A relatively thin cement concrete pavement layer distributes the load over a wide area due to its high rigidity. Sensitivity of structural responses (deformation, stress and strain) due to variation of basecourse layer thickness for different material performed in this study.

4.2.1.4.1 Deflection

When the stiffness of basecourse material increases it provide greater structural. Low deflections at the surface (i.e., deflection measurements are known to be reduced when the bearing capacity of the road is high). Stresses and strains within the layers of the structure induced by traffic reduce due to low deflection. Therefore, a structure that provides lower deflection measurements would tend to reduce the layers' state of stress and strain, causing the pavement structure to be less affected (damaged) by the loading conditions.

To analyze the effect of basecourse layer thickness on surface deflection thickness from 3in. to 13in. with 2in. interval used for Granular Base(GB), Soil Cement (SC), Cement Treated Base (CTB), Lean Concrete (LC), Roller-Compacted Concrete (RCC) and Portland Cement Concrete (PCC) materials.

The deflection analysis performed is shown from figure 4.14 to figure 4.19 for different basecourse material and thickness. The deflection data was obtained using the KENPAVE computer software.

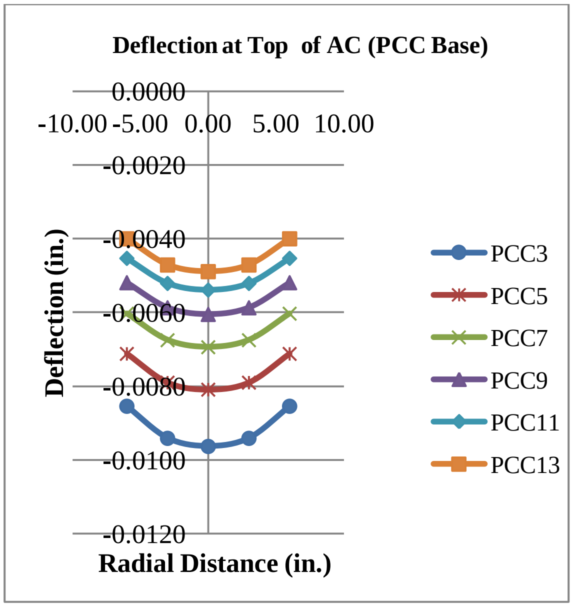


Figure 4.14: Deflection of AC Surface Using various PCC Basecourse Layer Thikness

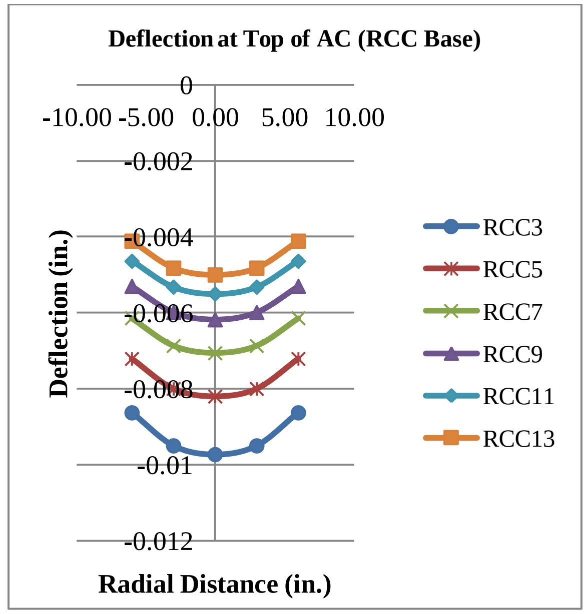


Figure 4.16: Deflection of AC Surface Using various RCC Basecourse Layer Thikness

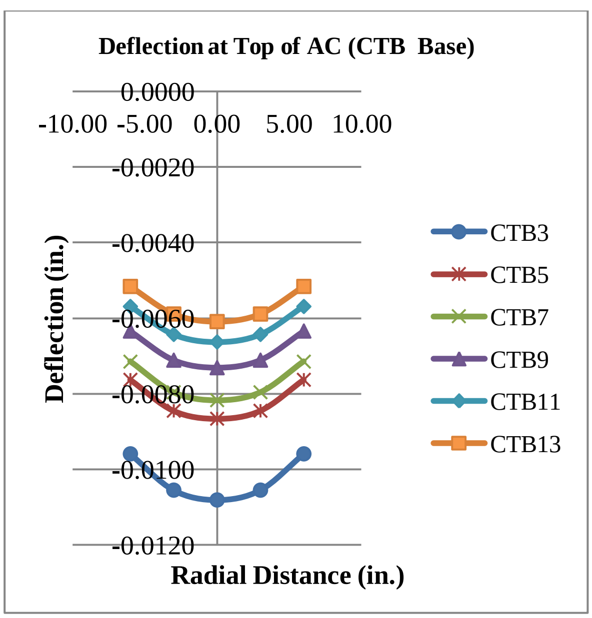


Figure 4.15: Deflection of AC Surface Using various CTB Basecourse Layer Thikness

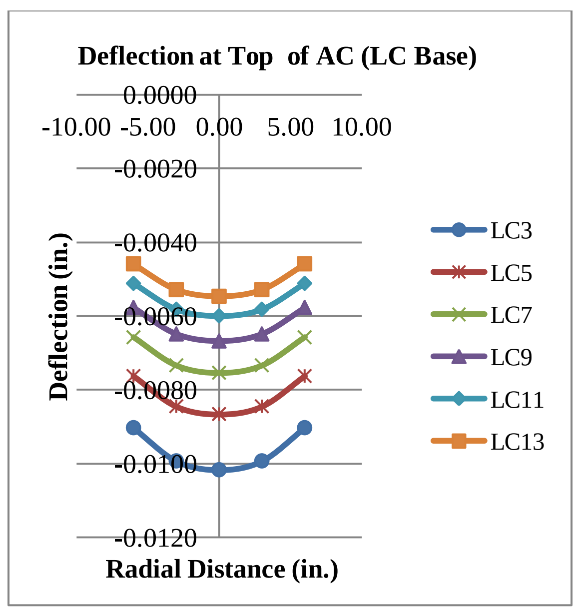


Figure 4.17: Deflection of AC Surface Using various LC Basecourse Layer Thikness

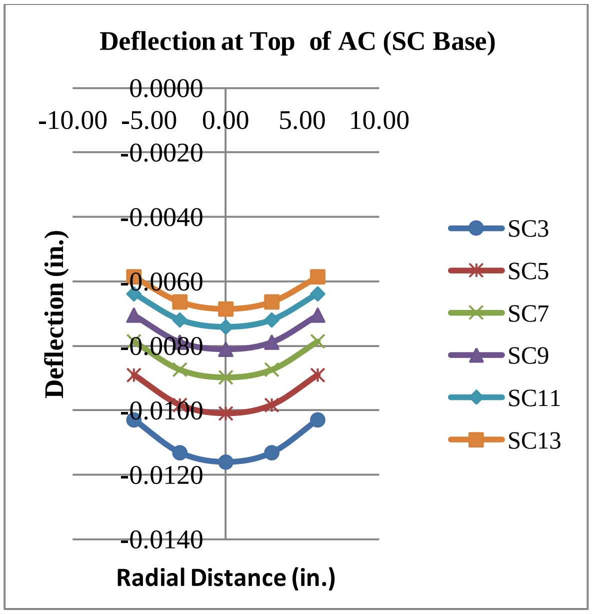


Figure 4.18: Deflection of AC Surface Using various SC Basecourse Layer Thikness

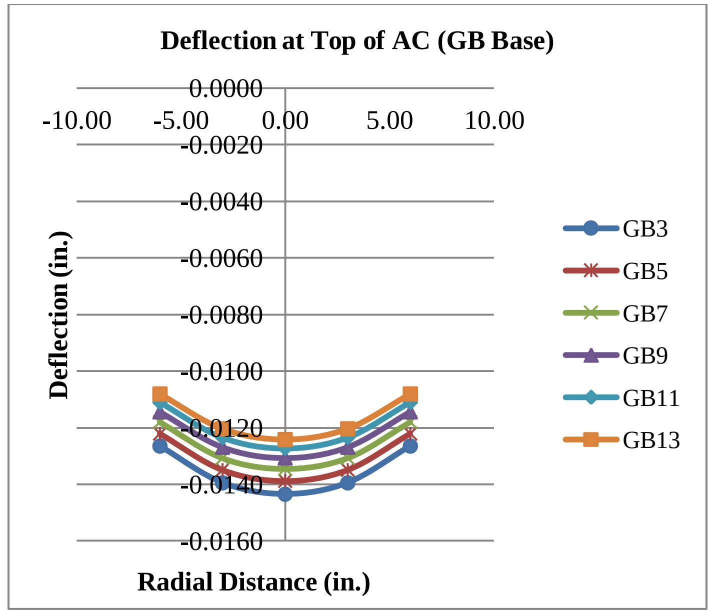


Figure 4.19: Deflection of AC Surface Using various GB Basecourse Layer Thikness

Deflections measured at the pavement surface are greatly reduced as the stiffness of the base increases. When the thickness of basecourse layer increases, surface deflection reduced in higher amount for stiffer materials. The table below shows the percent reduction of deflections, when the thickness increase by 2in. for each basecourse material

Table 4.23: Maximum Deflection with Different Base-course Material and Thickness

Base course-material	Radial Distance	Base-course Layer Thickness(in.)					
		3	5	7	9	11	13
PCC	0.00	-0.0096	-0.0081	-0.0069	-0.0061	-0.0054	-0.0049
Deflection Change (%)			-15.99	-14.22	-12.82	-10.91	-9.28
RCC	0.00	-0.00973	-0.0082	-0.0071	-0.0062	-0.0055	-0.0050
Deflection Change (%)			-15.72	-13.90	-12.46	-11.00	-9.09
Lean concrete	0.00	-0.0102	-0.0087	-0.0075	-0.0067	-0.0060	-0.0055
Deflection Change (%)			-14.85	-12.93	-11.41	-10.18	-8.83
CTB	0.00	-0.0108	-0.0093	-0.0082	-0.0073	-0.0066	-0.0061
Deflection Change (%)			-14.15	-11.96	-10.53	-9.30	-8.14
Soil cement	0.00	-0.0116	-0.0101	-0.0090	-0.0081	-0.0074	-0.0069
Deflection Change (%)			-13.01	-11.09	-9.69	-8.51	-7.55
Granular base	0.00	-0.0144	-0.0139	-0.0135	-0.0131	-0.0127	-0.0124
Deflection Change (%)			-3.28	-3.10	-2.83	-2.60	-2.44

To analyze the effect of basecourse layer thickness on surface deflection thickness from 3in. to 13in. increased by 2in. interval used for Granular Base(GB), Soil Cement (SC), Cement Treated Base (CTB), Lean Concrete (LC), Roller-Compacted Concrete (RCC) and Portland Cement Concrete (PCC) materials.

The table shows that as the rigidity and thickness of the base increases, the deflections of the pavement structure decrease. Increment of base thickness by 2in., cause the reduction of surface deflection in higher amount for stiffer materials as shown in table above. This suggests a reduction of stresses and strains in the various pavement layers.

4.2.1.4.2 Horizontal Stress and Strain

Pavement stress and strain responses that are a function of the load magnitude, load location, pressure, and material properties when the pavement structure subjected to traffic load.

Failure of HMA and cement-bound materials (e.g., soil cement, CTB, lean mix, RCC, PCC) caused by horizontal stress should be investigated. To predict HA and cement-bound material failure horizontal strains investigated for different bascourse material and thickness.

The stress analysis performed is shown in Fig. 4.20 and 4.21 at bottom of basecourse and HMA respectively using various basecourse material and thickness.

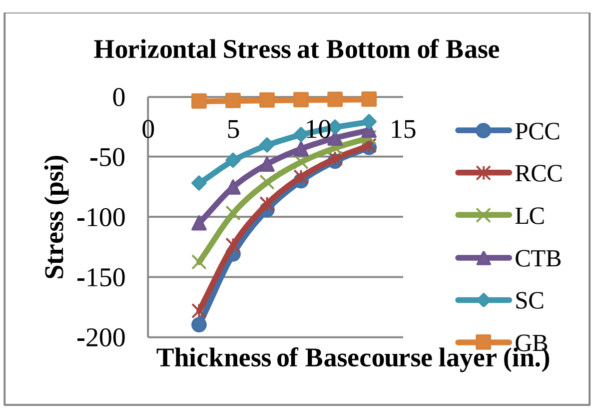


Figure4.20**:** Horizontal Stress with Different Base-course Material and thickness

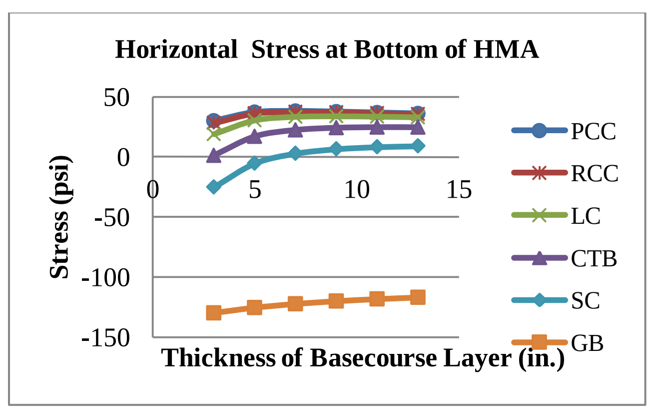


Figure4.21: Horizontal Stress with Different Base-course Material and thickness

High tensile stress at bottom of HMA when the base course material was Granular as shown in Figure 4.21 which causes fatigue crack. This high tensile stress reduced when the stiffness of the base layer material increase.

Tensile stress at bottom of bascourse layer increase when the stiffness of basecourse material increase as shown in fig. 4.20. so, the critical points to design of pavement structure are bottom of HMA layer and bottom of basecourse layer for a pavement with granular base and rigid(PCC) base layer respectively.

Table 4.24: Horizontal Stress at Bottom of HMA layer

Base course-material	Base-course Layer Thickness(in.)					
	3	5	7	9	11	13
PCC	29.904	37.069	37.976	37.461	36.68	35.936
Stress Change (%)		23.96	2.45	-1.36	-2.08	-2.03
RCC	28.209	36.141	37.381	37.012	36.289	35.562
Stress Change (%)		28.12	3.43	-0.99	-1.95	-2.00
Lean Concrete	18.781	30.255	33.182	33.64	33.326	32.79
Stress Change (%)		61.09	9.67	1.38	-0.93	-1.61
CTB	1.089	16.917	22.277	24.114	24.609	24.557
Stress Change (%)		1453.44	31.68	8.25	2.05	-0.21
Soil cement	-25.207	-5.378	2.832	6.53	8.251	9.026
Stress Change (%)		-78.66	-152.66	130.58	26.36	9.39
Granular base	130.025	-125.602	-122.468	-120.153	-118.398	-117.043
Stress Change (%)		-3.40	-2.50	-1.89	-1.46	-1.14

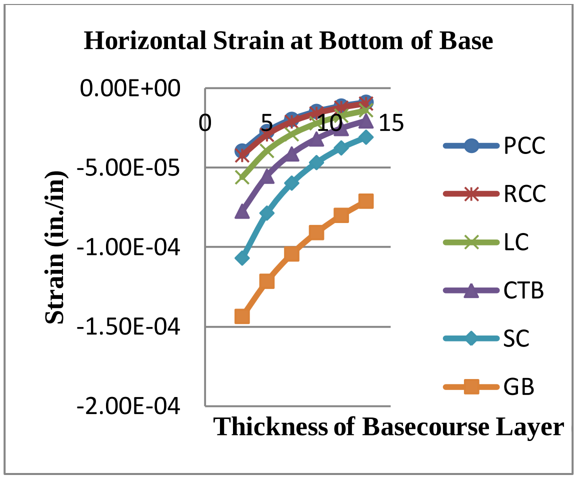
Table 4.25: Horizontal Stress at Bottom of Base-course layer

Base course-material	Base-course Layer Thickness(in.) b					
	3	5	7	9	11	13
PCC	-189.925	-131.068	-94.138	-70.146	-53.83	-42.313
Stress Change (%)		-30.99	-28.18	-25.49	-23.26	-21.40
RCC	-178.358	-123.697	-89.33	-66.897	-51.57	-40.696
Stress Change (%)		-30.65	-27.78	-25.11	-22.91	-21.09
Lean Concrete	-137.686	-96.93	-71.247	-54.261	-42.496	-34.027
Stress Change (%)		-29.60	-26.50	-23.84	-21.68	-19.93
CTB	-105.129	-75.276	-56.234	-43.468	-34.522	-28.014
Stress Change (%)		-28.40	-25.30	-22.70	-20.58	-18.85
Soil cement	-25.207	-5.378	2.832	6.53	8.251	9.026
Stress Change (%)		-78.66	-152.66	130.58	26.36	9.39
Granular base	-3.705	-3.26	-2.89	-2.588	-2.336	-2.122
Stress Change (%)		-12.01	-11.35	-10.45	-9.74	-9.16

Thickness increment cause horizontal compressive and tensile stress decrease and increase at bottom of HMA layer and bottom of base layer as shown in tables above. At bottom of HMA compressive stress increase up 7in. PCC base layer thickness and the stress decrease when the thickness was greater than 7in. High tensile stress at bottom of HMA for granular base layer reduce in smaller amount when the thickness of base layer increase as shown in Table 4.24.

Tensile stress at bottom of basecorse layer reduced in higher amount when the thickness of base increase in case of PCC base layer as shown in Table 4.25 .

The strain analysis performed is shown in Fig. 4.22 and 4.23 for various basecourse layer and thickness.



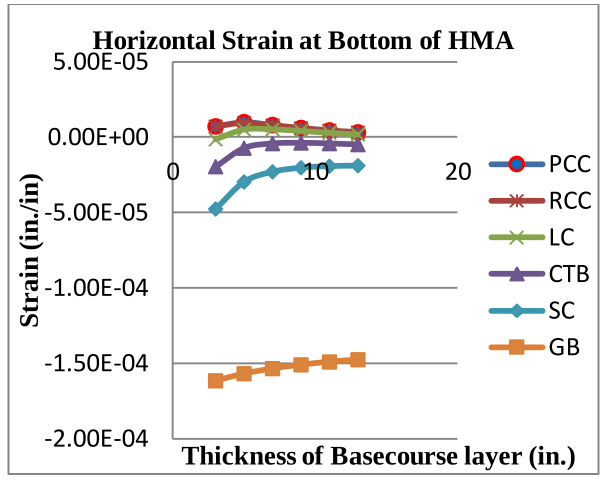


Figure 4.22: Horizontal Strain with Different Base-course Material and thickness

Figure 4.23: Horizontal Strain with Different Base-course Material and thickness

Table 4.26: Horizontal Strain at Bottom of HMA layer

Base course-material	Base-course Layer Thickness(in.)					
	3	5	7	9	11	13
PCC	7.10E-06	9.90E-06	8.00E-06	6.01E-06	4.38E-06	3.11E-06
Strain Change (%)		39.39	-19.22	-24.87	-27.09	-28.93
RCC	7.10E-06	9.38E-06	7.78E-06	5.90E-06	4.31E-06	3.04E-06
Strain Change (%)		32.08	-17.00	-24.15	-27.05	-29.53
Lean Concrete	-1.59E-06	4.97E-06	5.13E-06	4.02E-06	2.76E-06	1.62E-06
Strain Change (%)		-412.07	3.32	-21.60	-31.28	-41.28
CTB	-1.96E-05	-7.40E-06	-4.37E-06	-3.95E-06	-4.36E-06	-5.01E-06
Strain Change (%)		-62.34	-40.94	-9.48	10.22	14.92
Soil cement	-4.77E-05	-2.98E-05	-2.30E-05	-2.03E-05	-1.93E-05	-1.90E-05
Strain Change (%)		-37.66	-22.55	-11.72	-5.21	-1.66
Granular base	-1.62E-04	-1.57E-04	-1.54E-04	-1.51E-04	-1.49E-04	-1.48E-04
Strain Change (%)		-2.91	-2.17	-1.63	-1.26	-1.01

Table 4.27: Horizontal Strain at Bottom of Base- course layer

Base course-material	Base-course Layer Thickness(in.) b					
	3	5	7	9	11	13
PCC	-3.95E-05	-2.73E-05	-1.96E-05	-1.46E-05	-1.12E-05	-8.79E-06
Strain Change (%)		-15.99	-14.22	-12.82	-10.91	-9.28
RCC	-4.23E-05	-2.93E-05	-2.12E-05	-1.58E-05	-1.22E-05	-9.63E-06
Strain Change (%)		-15.72	-13.90	-12.46	-11.00	-9.09
Lean Concrete	-5.60E-05	-3.94E-05	-2.90E-05	-2.21E-05	-1.73E-05	-1.38E-05
Strain Change (%)		-14.85	-12.93	-11.41	-10.18	-8.83
CTB	-7.74E-05	-5.54E-05	-4.13E-05	-3.19E-05	-2.54E-05	-2.06E-05
Strain Change (%)		-14.15	-11.96	-10.53	-9.30	-8.14
Soil cement	-1.07E-04	-7.86E-05	-5.97E-05	-4.68E-05	-3.76E-05	-3.08E-05
Strain Change (%)		-13.01	-11.09	-9.69	-8.51	-7.55
Granular base	-1.44E-04	-1.22E-04	-1.04E-04	-9.08E-05	-8.00E-05	-7.11E-05
Strain Change (%)		-3.28	-3.10	-2.83	-2.60	-2.44

The horizontal strains output obtained from the mechanistic modeling are consistent with the results from the horizontal stresses. In this case, it can be observed that the tensile strain at the bottom of the HMA is significantly larger in the granular base case than in any of the ones obtained when a rigid base was used.

As granular base thickness increase tensile strain at bottom of HMA layer reduce in smaller amount, so, increasing granular base thickness didn’t significantly reduce the strain as shown in table 4.26. Tensile strain at bottom of basecourse layer highly reduced due to PCC layer thickness increment as shown in table 4.27.

In the rigid base analysis, due to the rigidity of the base, the highest stresses correspond to the smallest strain responses as shown in tables and figures above.

4.3 Economic Analysis of Pavement Structures

There is usually very little data available for accurately estimating whole life costs. Furthermore, there is little experience of rigid pavements in Ethiopia. Hence there is no foolproof way of determining the best choice of rigid pavement type for any particular situation.

Design period of 15 and 30 years used for flexible and composite pavements respectively and the quantities have been calculated based on final design.

The bills of quantities for the two pavements used in this study are meant to cover a 1km road length. The rates were derived from two contractors namely Defense Construction Enterprise who bid for a section of Ditchoto Gulafi Junction – Elidar –Belecho rigid pavement Project and Ethiopian Construction Works Corporation who bid a section Awash - Mille Road Asphalt Overlay Project, Contract 2. Unit rate cost of materials collected from two contractors in 2017 but, the base year (first year of construction for this study) is 2015. Unit rate cost of materials varies in different year due to inflation so, the cost rate of 2017 adjusted to 2015 by using 6.5% inflation rate.

It is expected that the different work items for the two pavements would be similar such as:

- Preliminary and supervisors/support service
- Site clearances and topsoil stripping
- Earth works
- Excavation and filling for structures
- Culverts and drainage work
- Passage of traffic
- Day works
- Road furniture

Work items which can cause significance for difference initial cost of construction for Composite and flexible pavements are:

- Construction material for capping layer, sub-base and base
- Bituminous mixes
- Concrete for pavement works
- Construction material for shoulder
- Steel bars for pavement work

The geometric characteristics of the designed road section were as follows:

Adama-Awash Road section

Carriageway Width=7.3m

Shoulder Width=1.5m

Length of Road= 1km

Carriageway area= 7.3*1000=7300m^2

Shoulder Area=1.5*1000=1500 m^2

Awash-Mille Road Section

Carriageway Width=7m

Shoulder Width=1.5m

Length of Road= 1km

Carriageway Area=7*1000=7000 m^2

Shoulder Area=1.5*1000=1500 m^2

Literatures show that 4 in. or 10.16cm HMA surface course is considered as minimum thickness to mitigate or retard reflective cracking distress of composite pavement structure. Due to economy 8cm HMA surface course used for composite pavement for this study.

Guidelines for AASHTO 1993 Pavement Design recommends a full-depth shoulder (same design as the mainline pavement) for high-volume routes. For this study, full depth shoulder for both flexible and composite pavement structures used and the depth of mainline layers shown in table below.

Table 4.28: Layer Thickness (cm.) of Pavement Structures used for LCCA

Layer	Adama-Awash Road Section		Awash-Mille Road Section	
	Flexible Pavement	Composite Pavement	Flexible Pavement	Composite Pavement
HMA	21.59	8	17.78	8
JRCP	-	34.849	-	28.549
Crushed Stone Base	25.4	-	20.32	-
Granular sub-base	24.13	15	25.4	15
Capping Layer	-	25	-	25

As shown in Table 4.23, two types of pavements for two road sections were analyzed. In order to obtain accurate agency costs for each of the pavements, thicknesses had to be computed. The cross-sections of all the pavements used were obtained using the AASHTO 1993 pavement structure design method.

The analysis period should generally be longer than the pavement design period. A 40-year analysis period was used as recommended by VDOT; at least one complete cycle of rehabilitation activity should be incorporated after the design period. From the total 40 year analysis period 3 years (from 2015-2017) taken as construction year, 30 years design period (from 2018 -2018) for composite pavement by assuming the road open for traffic in 2018 and 7 years rehabilitation period after design period.

The discount rate for economic analysis of pavement structures used by Ethiopian road Authority is 10.23 %, this discount rate used for this study. 6.5% inflation rate per year for pavement material used. Using the nominal discount rate 10.23% and inflation rate 6.5%, real discount is 3.5% which is computed using equation 2.21. According to VDOT the real discount rate range from 3% to 5% for pavement structure life cycle cost analysis.

Routine reactive type maintenance cost data and the salvage value i.e the residual values of materials for the competing pavement types normally not available. Fortunately, routine reactive type maintenance costs and salvages value for the competing pavement types when discounted 40 years to PW are not expected to be significantly. VDOT disregards the routine reactive type maintenance costs and salvage value for the competing pavement types in its LCCA process. So, initial construction and rehabilitation cost of alternative pavement structures used to compute life cycle cost for this study.

From Adama-Awash road section overlay project engineering report the road constructed in 2002 and it has served for 8 years, after that the road is not in a good condition, required to overlay in 20011. From this, time schedule of 8 and 7 year overlay activity after construction and rehabilitation activity respectively for flexible pavement. For composite pavement structure 7 and 6 year overlay activity after construction and rehabilitation activity respectively used for this study. Composite pavement requires overlay early relative to flexible pavement due to high compressive stress on the top of composite pavement wearing course.

The following equation is employed to convert future cost into present cost.

$P = F[\frac{1}{(1+r)^n}]$……………………………..equation 4.12

Where:

P = the present-day cost or value; the present sum of money.

F = the cost sum at a future date, n periods from the present; the sum is equivalent to P with compound interest at r (discount rate) over n periods

r = Value in decimals representing a specific change over time periods; discount rate per period of time; it could be in this sense nominal or real depending on the nature of analysis.
n = Number of discount periods; it is mostly expressed in years

The general cost of pavement structures shown in figure below and detail bill of quantities is given in Appendix D of this document.

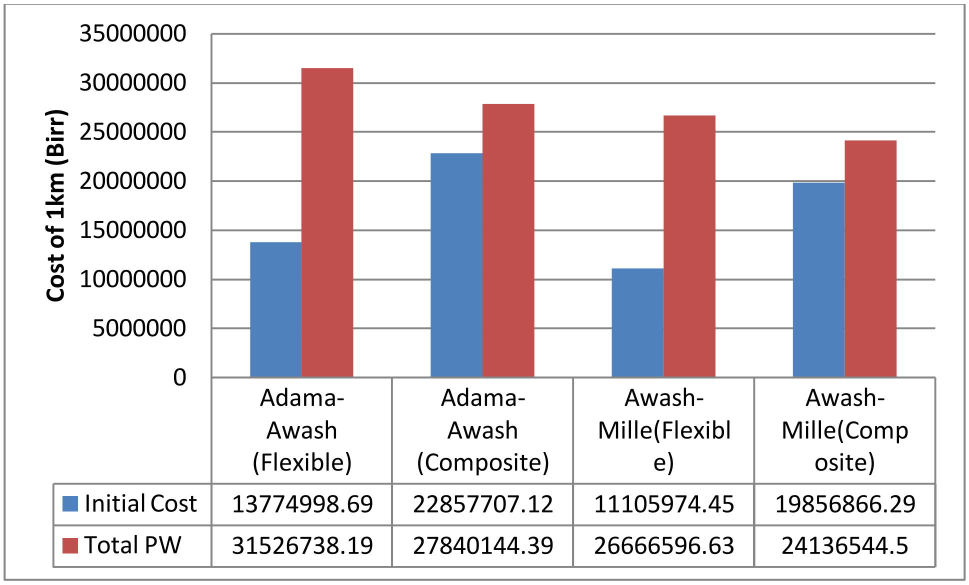


Figure 4.24: Initial Construction and Total PW Costs for pavement Alternatives

From the above figure the initial construction cost of composite pavement were higher than by 39.74 and 44.07% from flexible pavement alternative for Adama-awash and Awash-Mille road sections respectively. So, the initial construction cost of composite pavement is more expensive than flexible pavement alternative.

When the total life cycle cost of the two alternatives compared, flexible pavement structures were more expensive than composite pavement by11.69% and 9.49% for Adama-awash and Awash-Mille road sections respectively. The reason for these cost difference were rehabilitation and reconstruction cost during the analysis period were higher for flexible pavement alternatives. Adama-Awash road section has higher traffic volume than Awash-Mille road section, the economic analysis computation shows that constructing high trafficked road sections with flexible result higher life cycle cost because LCC in the form of PW difference between flexible and composite pavement structures is higher for Adama-Awash road section i.e 11.69%.

Using the following equation equivalent uniform annual costs (EUAC) of each alternative can be computed to know how much each pavement will cost per throughout the analysis period

$$EUAC = PW * [\frac{i(1+i)^n}{(1+i)^n-1}] \text{……………………equation 4.13}$$

Where: EUAC= equivalent uniform annual costs, i = discount rate =10.23% and n = analysis period= 40 year

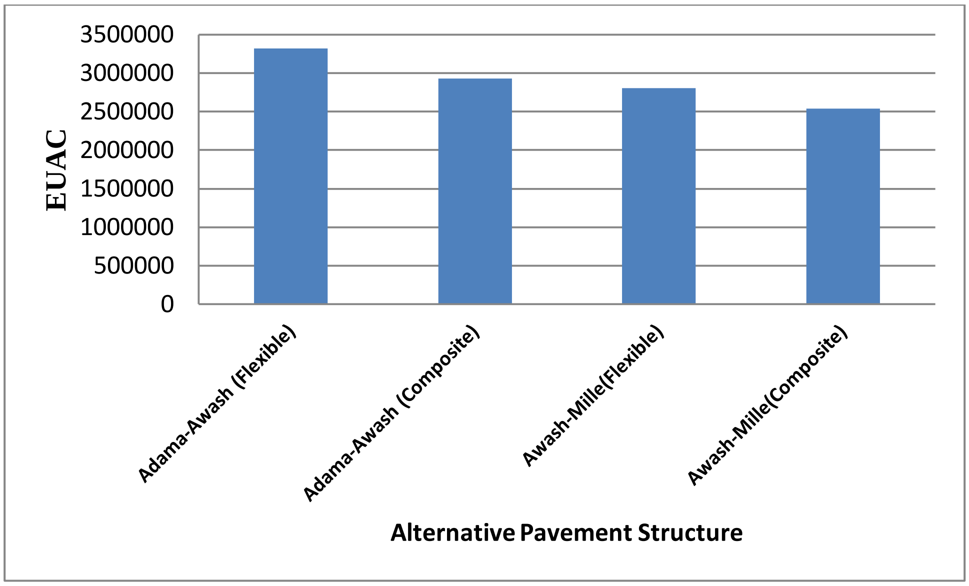


Figure 4.25: EUAC for pavement Alternatives

EUAC computation result shows that Adama-awash road section cost around 3.3 and 2.9 million birr for flexible and composite pavement structures respectively, and Awash-Mille road section cost 2.8 and 2.5 million birr for flexible and composite pavement structures respectively per year throughout the analysis of 40 year.

5.0 CONCLUSIONS AND RECOMMENDETIONS

5.1 Summary

Composite pavements were investigated in this study using technical and economic evaluations. Various potential technical and economic benefits have been proposed and supported by modeling of pavement responses and performing an LCCA. The following section summarizes the main findings and conclusions of this thesis and provides recommendations for implementation of these results and for future research to further investigate this topic.

5.2 Findings

The main findings of this study concerning the technical and economic evaluations of composite pavement systems to be used during the PTS process are the following:

• According to the literature many countries (e.g., the U.K., Spain) that have used composite pavement systems in their main road network have had a positive experience in terms of functional and structural performance.

• Although a composite pavement involves the combination of a flexible and a rigid layer in no particular placement order (i.e., HMA on top or PCC on top), the most commonly accepted definition of a composite pavement system includes the combination of a flexible (HMA) layer on top of a rigid base course. A cement-bound layer (e.g., CTB, lean mix, PCC) could be considered as rigid base, mainly because a rigid base would tend to behave like a concrete slab as it hardens during the pavement service life.

•The use of some kind of reflective cracking treatment such as a stress-relief interlayer, geocomposite, or geotextile is always strongly recommended as documented by the literature. The thickness of the HMA layer has a significant impact on the mitigation of reflective cracking; however, the mechanism of this particular distress is still not well understood. Therefore it is believed that the combination of a thick HMA overlay and a reflective cracking mitigation technique such as the aforementioned would increase the possibility of preventing reflective cracks.

• The use of a high-stiffness base layer for the HMA surface course provided the following benefits:

- ✓ Deflections at the HMA surface are significantly reduced as the stiffness of the base layer increases.
- ✓ Flexural fatigue (bottom-up) in the HMA due to high tensile strain concentration at the bottom of the layer is greatly minimized, and in some cases the number of repetitions to fatigue cracking was determined to be unlimited.
- ✓ The HMA layer's top-down fatigue life generally increases as the rigidity of the base course increases.
- ✓ Permanent deformations (rutting) due to vertical compressive strains and stresses in the unbound subbase and, most importantly, subgrade layer are significantly minimized.

• Permanent deformations within the HMA layer tend to increase as the stiffness of the base increases.

• The deterministic agency-cost LCCA of the composite pavement resulted in the lower cost when compared to the flexible pavement alternatives.

• A sensitivity analysis of the agency-costs over the life-cycle of the pavements, suggest that "true" composite pavements (i.e., composite pavement with CRCP base) have the potential to become a cost-effective alternative for high-traffic high-priority highways.

5.3 Conclusions

Composite pavement systems have the potential to become a cost-effective pavement alternative during the PTS process of transportation agencies because of the functional, structural, and economic benefits they provide during their service life. A long-life pavement that offers good serviceability levels and cost-effective maintenance operations as part of an infrastructure network is much desired, especially for high traffic volume roads.

At the technical level, composite pavements mitigate various structural and functional problems that typical flexible pavement tend to present, such as HMA flexural fatigue failure, HMA rutting due to subgrade vertical deformations. At the same time, though, other types of distresses such as reflective cracking and rutting within the HMA layer need to be considered because they affect composite pavement systems more than the flexible pavement structure. Treated HMA surfaces may be required to mitigate these potential problems.

At the economic level, the results of the deterministic LCCA suggest that the use of a composite pavement results in cost-effective alternative than flexible pavement. The life cycle work schedule for the composite pavement suggests that as long as functional milling and overlaying maintenance operations are performed on the surface course, the pavement will provide good levels of serviceability and structural adequacy for a longtime, as is mentioned in literatures.

5.4 Contribution

The potential of composite pavements was presented to encourage their consideration in the PTS process. With advances of pavement technology, the PTS protocol should be broadened to include different types of pavement alternatives so that the most cost-effective option may be selected. This study proposed the inclusion of composite pavements in the PTS process and showed the potential benefits that nontraditional pavement structures offer to transportation agencies.

Through mechanistic modeling, the pavement responses at various (increasing) stiffness levels of the base course underneath an HMA layer were studied. The behavior of the pavement structure and distribution of stresses and strains at different depths were investigated. These analyses enhance the understanding of the performance of composite pavement systems.

LCCA was performed to understand and assess the economic implications of implementing a composite pavement alternative to typical flexible pavement. This economic analysis found that these types of pavement system can be cost-effective for high-traffic highways.

5.5 Recommendations of the Study

The traditional PTS process should include more alternatives than flexible pavements. As this study demonstrates, composite pavements can be a potential candidate for high-priority, high-volume roads in the Ethiopia to obtain long-lasting roads in infrastructure network.

This study shows the use of a composite pavement to be cost effective and thus recommendable as a highly viable pavement alternative. It is important to mention that as new technologies and studies emerge, appropriate methods should be used to mitigate reflective cracking on the HMA surface course because it is considered a major type of distress in any composite structure.

5.6 Recommendations for Future Research

The modeling of composite pavements is difficult due to the very different behaviors inherent in asphalt and concrete. Only stress, strain deflection response of composite pavement investigated under linear elastic behavior of pavement layers total bonding of layers and single wheel load in this study. Further study can be conducted for different loading condition, bonding and type of material (nonlinear or viscoelastic). In addition Damage analysis by using different elastic modulus values for various periods can be conducted to predict the design life of pavement structure.

Research studies that model reflective cracking distress are particularly important. Such modeling could include the applicability of Paris' crack growth law, temperature variation effects on the initiation of cracking (bottom-up and top-down), and effectiveness of interlayer systems on the mitigation of reflective cracking.

LCCA similar to the one performed in this study should be performed to for more investigation the cost implications of any variations (e.g., layer thicknesses) on the composite pavement design and performance. In terms of the economic analysis, LCCA that includes the computation of user costs should be conducted. There is no time schedule and cost for maintenance and rehabilitation activity of pavement structures in Ethiopia, study should be carried out on this. In addition, the use of a probabilistic-based LCCA should be considered as well because it would provide a more realistic evaluation of the alternatives to be compared at the economic level.

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APPENDIX A- Design CBR, resilient modulus Table

CBR value of Adama-Awash road section from station 20+500 to 40+500 obtained from engineering report of this road section overly project in 2011.

For fine-grained soils with a soaked CBR between 5 and 10 , use the following equation to correlate CBR to resilient modulus (Mr):

$$Mr\ (psi) = 1{,}500 \times CBR$$

For non fine-grained soils with a soaked CBR greater than 10, use the following equation:

$$Mr = 3{,}000 \times CBR^{0.65}$$

Station	Material Description	CBR %	Resilient Modulus(Mr) (Psi)	**CBR % (Ascending order)**	**No. equal to or greater than**	**% equal to or greater than**
20+500	Light grey Clay	13	15892	4	27	=(27/27/)*100=**100**
21+000	Dark Brown Clay with few Gravels	13	15892	7	26	=(26/27)*100=**96.3**
21+500	Dark Brown Clay with few Gravels	9	13500	8	25	92.6
22+000	Black Clay	5	7500	8	25	92.6
22+500	Dark Brown Clay with few Gravels	8	12000	9	23	85.2
23+000	Yellowish Clay	22	22372	12	22	81.5
23+500	Grayish Brown Clay	6	9000	12	22	81.5
24+000	Dark Silty Clay	9	13500	13	20	74.1
24+500	Grayish Clay	31	27958	14	19	70.4
25+000	Dark Silty Clay	20	21028	23	18	66.7
25+500	Gravely Clay	36	30812	24	17	63.0
26+000	Dark Silty Clay	60	42946	26	16	59.3
26+500	Dark Clay			26	16	59.3
27+000	Dark Silty Clay	12	15087	27	14	51.9
27+500	Dark Silty Clay	8	12000	31	13	48.1
28+000	Brown Silty Clay	41	33530	31	13	48.1
28+500	Grayish Gravely Clay	24	23673	36	11	40.7

29+000	Cinder with pumice	12	15087	37	10	37.0
29+500	Grayish Gravely Clay	31	27958	38	9	33.3
30+000	Silt with pumice	9	13500	41	8	29.6
30+500	Oversized Bsaltic Gravel			41	8	29.6
31+000	Brownish clay with cinder	37	31366	42	6	22.2
31+500	Dark Gravely Clay	26	24938	43	5	18.5
32+000	Cinder with soil	52	39131	47	4	14.8
32+500	Oversized Basaltic Gravel	27	25557	48	3	11.1
33+000	Basalt boulders			52	2	7.4
33+500	Light Brown Gravelly Clay	26	24938	60	1	3.7
34+000	Dark Clay	4	6000			
34+500	Dark Silty Clay with Few Gravel	47	36642			
35+000	Dark Clay	7	10500			
35+500	Dark Silty Clay with Few Gravel	41	33530			
36+000	Grayish Clay	23	23027			
36+500	Dark Silty Clay with Few Gravel	43	34584			
37+000	Dark Silty Clay	13	15892			
37+500	Black Cinder	31	27958			
38+000	Silty Clay with basalt rock	42	34059			
38+500	Weathered Cinder	38	31914			
39+000	Silty Clay with basalt rock	48	37147			
39+500	Grayish Brown Clay	14	16676			
40+000	Dark Clay					
40+500	Redidsh Brown Cinder	8	12000			
		Sum	839092.864			
		Average	22678.186			

Source : Engineering report of Adama-Awash road overlay project

The average Mr value taken as design resilient modulus which is to 22678.186Psi.

Resilient modulus value of Adama-Awash road section from station 20,001 to 39,999.50 obtained from engineering report of this road section overly project of 2011.

No.	Chainage	Right Side Mr. (MPa)	Left Side Mr. (MPa)	Average Mr.(MPa)	Average Mr. in (Psi)
1	20,001.00	112.04482	112.04482	112.04482	16250.98069
2	20,500.00	114.28571	114.28571	114.28571	16575.99938
3	21,000.20	119.04762	107.81671	113.432165	16452.20121
4	21,504.60	98.522167	105.82011	102.1711385	14818.90193
5	22,000.20	98.522167	105.82011	102.1711385	14818.90193
6	22,509.20	114.28571	96.8523	105.569005	15311.72849
7	22,999.90	150.37594	146.52015	148.448045	21530.90445
8	23,500.70	112.04482	105.82011	108.932465	15799.56472
9	23,999.70	272.10884	126.98413	199.546485	28942.22218
10	24,500.70	142.85714	184.3318	163.59447	23727.74193
11	25,000.00	114.28571	89.285714	101.785712	14762.99967
12	25,500.70	634.92063	714.28571	674.60317	97844.44378
13	26,000.00	158.73016	163.26531	160.997735	23351.11148
14	26,500.20	124.2236	129.87013	127.046865	18426.8773
15	27,000.50	952.38095	1904.7619	1428.571425	207199.9995
16	27,504.10	439.56044	285.71429	362.637365	52596.92342
17	28,000.00	197.04433	168.06723	182.55578	26477.89033
18	28,500.30	238.09524	129.87013	183.982685	26684.84863
19	29,000.90	139.37282	211.64021	175.506515	25455.46494
20	29,499.80	154.44015	168.06723	161.25369	23388.2352
21	29,999.50	228.57143	173.16017	200.8658	29133.57563
22	30,500.20	102.04082	77.220077	89.6304485	13000.00025
23	31,003.00	272.10884	204.08163	238.095235	34533.33288
24	31,501.20	126.98413	139.37282	133.178475	19316.20601
25	31,999.90	121.58055	136.05442	128.817485	18683.68802
26	32,499.80	146.52015	126.98413	136.75214	19834.53039
27	33,001.40	119.04762	197.04433	158.045975	22922.98821
28	33,500.70	146.52015	136.05442	141.287285	20492.30782
29	33,999.50	136.05442	163.26531	149.659865	21706.66682
30	34,500.60	105.82011	100.25063	103.03537	14944.25006
31	34,999.50	119.04762	142.85714	130.95238	18993.3332
32	35,500.30	272.10884	272.10884	272.10884	39466.66615
33	36,003.70	178.57143	272.10884	225.340135	32683.33318
34	36,500.30	272.10884	178.57143	225.340135	32683.33318
35	37,001.40	136.05442	168.06723	152.060825	22054.90206
36	37,500.60	439.56044	519.48052	479.52048	69549.65042
37	38,000.00	92.165899	77.220077	84.692988	12283.87098
38	38,501.20	136.05442	105.82011	120.937265	17540.74092
39	38,999.50	114.28571	132.89037	123.58804	17925.20932
40	39,503.10	121.58055	146.52015	134.05035	19442.66276
41	39,999.50	380.95238	228.57143	304.761905	44202.6667
				Sum	1241811.856
				Average	30288.09405

1 MPa = 145.04 psi

Design Mr = C x Backcalculated Mr
Where C = 0.33
Design Mr = 0.33*30288.09405 = 9995.071Psi ≈ 10000PSi

If CBR and backcalculated Mr results are available, according to Guidelines for AASHTO 1993 pavement design use the smaller Design Mr for pavement design purposes from CBR and backcalculated Mr results. So, the Subgrade design Mr for this study used is 10000 Psi.

APPENDIX B - Excel Spreadsheets used for Designing Flexible and Rigid pavements

Adama-Awash road section (Flexible Pavement structure) Design

AASHTO FLEXIBLE PAVEMENT DESIGN

SN Determination

Design Inputs

W18 =	72852915		ESALs Applications Over Design Period	Typ. Range 0.1to 80 million
R =	85	%	Reliability	Typ. Range 80 to 95%
So =	0.49		Standard Deviation	Typ. Range 0.3 to 0.5
MR =	10,000	psi	Subgrade Resilient Modulus	Typ. Range 3000 to 9000 psi
Pi =	4.2		Initial Serviceability	Typ. Range 4.4 to 4.8
Pt =	2.8		Terminal Serviceability	Typ. Range 2.0 to 3.0

DESIGN SN = 6.17

Awash-Mille road section (Flexible Pavement structure) Design

AASHTO FLEXIBLE PAVEMENT DESIGN

SN Determination

Design Inputs

W18 =	24066949		ESALs Applications Over Design Period	Typ. Range 0.1to 80 million
R =	85	%	Reliability	Typ. Range 80 to 95%
So =	0.49		Standard Deviation	Typ. Range 0.3 to 0.5
MR =	10,000	psi	Subgrade Resilient Modulus	Typ. Range 3000 to 9000 psi
Pi =	4.2		Initial Serviceability	Typ. Range 4.4 to 4.8
Pt =	2.8		Terminal Serviceability	Typ. Range 2.0 to 3.0

DESIGN SN = 5.29

Adama-Awash road section (Rigid Pavement structure) Design

AASHTO RIGID PAVEMENT DESIGN

Design Inputs

W18 =	353206919		*ESALs Applications Over Design Period*	Typ. Range 0.5 to 100 million
PCC MR =	650	psi	*Concrete Modulus of Rupture*	Typ. Range 550 to 750 psi
E =	5,000,000	psi	*Concrete Elastic Modulus*	Typ. Range 3 to 6 million psi
k-value =	500	psi/in	*Modulus of Subgrade Reaction*	Typ. Range 100 to 500 psi/in
R =	85	%	*Reliability*	Typ. Range 80 to 95%
So =	0.39		*Standard Deviation*	Typ. Range 0.3 to 0.5
J =	3.2		*Load Transfer Coefficient*	Typ. Range 2.2 to 4.4
Cd =	1		*Drainage Coefficient*	Typ. Range 0.9 to 1.1
Pi =	4.5		*Initial Serviceability*	Typ. Range 4.5 to 4.8
Pt =	2.8		*Terminal Serviceability*	Typ. Range 2.0 to 3.0

DESIGN D (In) =	**14.38**
DESIGN D (mm) =	**365.25**

Awash- Mille road section (Rigid Pavement structure) Design

AASHTO RIGID PAVEMENT DESIGN

Design Inputs

W18 =	105862751.6		*ESALs Applications Over Design Period*	Typ. Range 0.5 to 100 million
PCC MR =	650	psi	*Concrete Modulus of Rupture*	Typ. Range 550 to 750 psi
E =	5,000,000	psi	*Concrete Elastic Modulus*	Typ. Range 3 to 6 million psi
k-value =	500	psi/in	*Modulus of Subgrade Reaction*	Typ. Range 100 to 500 psi/in
R =	85	%	*Reliability*	Typ. Range 80 to 95%
So =	0.39		*Standard Deviation*	Typ. Range 0.3 to 0.5
J =	3.2		*Load Transfer Coefficient*	Typ. Range 2.2 to 4.4
Cd =	1		*Drainage Coefficient*	Typ. Range 0.9 to 1.1
Pi =	4.5		*Initial Serviceability*	Typ. Range 4.5 to 4.8
Pt =	2.8		*Terminal Serviceability*	Typ. Range 2.0 to 3.0

DESIGN D (In) =	**11.88**
DESIGN D (mm) =	**301.75**

APPENDIX C - Pavement Mechanistic Responses with Various Rigid Bases

KENPAVE software output value of deflection, stress and strain shows positive values for "compressive" and negative values for "tensile", since the +Z axis is in the downward direction.

Tangential and radial stress values are equal at radial distance of zero i.e. at the center of wheel of pavement structure.

Portland Cement Concrete(PCC)

```
INPUT FILE NAME  -C:\Users\toshiba\Desktop\Pavement Analysis2\PCC.DAT
NUMBER OF PROBLEMS TO BE SOLVED =  1
TITLE -PCC
MATL = 1 FOR LINEAR ELASTIC LAYERED SYSTEM
NDAMA = 0, SO DAMAGE ANALYSIS WILL NOT BE PERFORMED
NUMBER OF PERIODS PER YEAR (NPY) =  1
NUMBER OF LOAD GROUPS (NLG) =  1
TOLERANCE FOR INTEGRATION (DEL) -- =  0.001
NUMBER OF LAYERS (NL)------------- =  4
NUMBER OF Z COORDINATES (NZ)------ =  8
LIMIT OF INTEGRATION CYCLES (ICL)- =  80
COMPUTING CODE (NSTD)------------- =  9
SYSTEM OF UNITS (NUNIT)------------=  0

Length and displacement in in., stress and modulus in psi
unit weight in pcf, and temperature in F

THICKNESSES OF LAYERS (TH) ARE : 7  8  10
POISSON'S RATIOS OF LAYERS (PR) ARE : 0.35  0.15  0.4  0.45
VERTICAL COORDINATES OF POINTS (ZC) ARE:  0.001  3.5  6.999  7.001  11
 14.999  15.001  25.001
ALL INTERFACES ARE FULLY BONDED
FOR PERIOD NO. 1 LAYER NO. AND MODULUS ARE :    1  5.500E+05   2  4.100E+06
3  2.500E+04   4  1.000E+04
LOAD GROUP NO. 1 HAS 1 CONTACT AREA
CONTACT RADIUS (CR)--------------- =  4.5
CONTACT PRESSURE (CP)------------- =  120
RADIAL COORDINATES OF  10  POINT(S) (RC) ARE :  0  1.5  3  4.5  6  7.5  9
 10.5  12  13.5

PERIOD NO.  1   LOAD GROUP NO.  1

  RADIAL     VERTICAL     VERTICAL    VERTICAL    RADIAL  TANGENTIAL   SHEAR
COORDINATE  COORDINATE  DISPLACEMENT   STRESS      STRESS    STRESS     STRESS
                                      (STRAIN)   (STRAIN)   (STRAIN)   (STRAIN)
 0.00000     0.00100     0.00646     120.037    101.609    101.609     0.000
(STRAIN)                            8.893E-05  4.370E-05  4.370E-05  .000E+00
 0.00000     3.50000     0.00599      98.959     28.255     28.255     0.000
(STRAIN)                            1.440E-04 -2.958E-05 -2.958E-05  .000E+00
 0.00000     6.99900     0.00561      59.239     37.790     37.790     0.000
(STRAIN)                            5.961E-05  6.963E-06  6.963E-06  .000E+00
 0.00000     7.00100     0.00561      59.214     44.135     44.135     0.000
```

(STRAIN)			1.121E-05	6.984E-06	6.984E-06	.000E+00
0.00000	11.00000	0.00558	19.130	-15.048	-15.048	0.000
(STRAIN)			5.767E-06	-3.819E-06	-3.819E-06	.000E+00
0.00000	14.99900	0.00556	1.177	-80.927	-80.927	0.000
(STRAIN)			6.209E-06	-1.682E-05	-1.682E-05	.000E+00
0.00000	15.00100	0.00556	1.177	0.084	0.084	0.000
(STRAIN)			4.441E-05	-1.682E-05	-1.682E-05	.000E+00
0.00000	25.00100	0.00516	0.697	0.187	0.187	0.000
(STRAIN)			5.291E-05	-2.109E-05	-2.109E-05	.000E+00
1.50000	0.00100	0.00642	120.368	102.823	102.377	-0.007
(STRAIN)			8.827E-05	4.520E-05	4.411E-05	-.328E-07
1.50000	3.50000	0.00596	93.642	27.729	26.800	9.266
(STRAIN)			1.356E-04	-2.623E-05	-2.851E-05	.455E-04
1.50000	6.99900	0.00561	55.705	36.026	35.973	8.825
(STRAIN)			5.546E-05	7.162E-06	7.030E-06	.433E-04
1.50000	7.00100	0.00561	55.682	44.374	43.911	8.827
(STRAIN)			1.035E-05	7.179E-06	7.049E-06	.495E-05
1.50000	11.00000	0.00558	18.247	-14.541	-14.897	5.584
(STRAIN)			5.527E-06	-3.669E-06	-3.769E-06	.313E-05
1.50000	14.99900	0.00556	1.167	-79.518	-80.277	0.064
(STRAIN)			6.131E-06	-1.650E-05	-1.671E-05	.361E-07
1.50000	15.00100	0.00556	1.167	0.088	0.084	0.062
(STRAIN)			4.394E-05	-1.650E-05	-1.672E-05	.695E-05
1.50000	25.00100	0.00516	0.696	0.188	0.187	0.014
(STRAIN)			5.275E-05	-2.098E-05	-2.106E-05	.419E-05
3.00000	0.00100	0.00628	120.218	105.105	103.797	0.021
(STRAIN)			8.564E-05	4.855E-05	4.533E-05	.103E-06
3.00000	3.50000	0.00586	75.994	27.720	22.818	18.517
(STRAIN)			1.060E-04	-1.248E-05	-2.451E-05	.909E-04
3.00000	6.99900	0.00559	46.015	31.174	30.985	15.546
(STRAIN)			4.411E-05	7.680E-06	7.216E-06	.763E-04
3.00000	7.00100	0.00559	45.998	44.914	43.290	15.549
(STRAIN)			7.992E-06	7.688E-06	7.232E-06	.872E-05
3.00000	11.00000	0.00556	15.829	-13.157	-14.464	10.314
(STRAIN)			4.871E-06	-3.259E-06	-3.626E-06	.579E-05
3.00000	14.99900	0.00554	1.139	-75.514	-78.403	0.124
(STRAIN)			5.909E-06	-1.559E-05	-1.640E-05	.694E-07
3.00000	15.00100	0.00554	1.139	0.100	0.085	0.119
(STRAIN)			4.259E-05	-1.560E-05	-1.641E-05	.134E-04
3.00000	25.00100	0.00515	0.692	0.189	0.187	0.029
(STRAIN)			5.227E-05	-2.065E-05	-2.095E-05	.828E-05
4.50000	0.00100	0.00589	60.000	48.417	63.688	0.991
(STRAIN)			3.775E-05	9.321E-06	4.680E-05	.486E-05
4.50000	3.50000	0.00571	46.836	30.012	17.901	22.013
(STRAIN)			5.467E-05	1.337E-05	-1.636E-05	.108E-03
4.50000	6.99900	0.00555	32.943	24.543	24.228	18.613
(STRAIN)			2.886E-05	8.242E-06	7.469E-06	.914E-04
4.50000	7.00100	0.00555	32.934	45.075	42.371	18.617
(STRAIN)			4.833E-06	8.239E-06	7.480E-06	.104E-04
4.50000	11.00000	0.00554	12.480	-11.241	-13.802	13.595
(STRAIN)			3.960E-06	-2.693E-06	-3.412E-06	.763E-05
4.50000	14.99900	0.00552	1.096	-69.516	-75.515	0.174
(STRAIN)			5.573E-06	-1.423E-05	-1.592E-05	.975E-07
4.50000	15.00100	0.00552	1.096	0.117	0.087	0.168

(STRAIN)			4.055E-05	-1.424E-05	-1.592E-05	.188E-04
4.50000	25.00100	0.00514	0.685	0.191	0.186	0.042
(STRAIN)			5.150E-05	-2.012E-05	-2.076E-05	.122E-04
6.00000	0.00100	0.00558	-0.290	-0.649	16.098	-0.021
(STRAIN)			-1.036E-05	-1.124E-05	2.987E-05	-.103E-06
6.00000	3.50000	0.00562	20.666	29.684	14.170	16.043
(STRAIN)			9.668E-06	3.180E-05	-6.277E-06	.788E-04
6.00000	6.99900	0.00556	20.256	17.880	17.569	17.977
(STRAIN)			1.427E-05	8.439E-06	7.676E-06	.883E-04
6.00000	7.00100	0.00556	20.253	43.753	41.099	17.982
(STRAIN)			1.835E-06	8.427E-06	7.683E-06	.101E-04
6.00000	11.00000	0.00555	8.946	-9.183	-12.952	15.246
(STRAIN)			2.992E-06	-2.093E-06	-3.150E-06	.855E-05
6.00000	14.99900	0.00554	1.046	-62.148	-71.719	0.212
(STRAIN)			5.153E-06	-1.257E-05	-1.526E-05	.119E-06
6.00000	15.00100	0.00554	1.046	0.141	0.093	0.206
(STRAIN)			3.808E-05	-1.258E-05	-1.526E-05	.230E-04
6.00000	25.00100	0.00517	0.679	0.198	0.190	0.054
(STRAIN)			5.045E-05	-1.934E-05	-2.044E-05	.157E-04
7.50000	0.00100	0.00546	-0.746	5.365	14.287	-0.004
(STRAIN)			-1.386E-05	1.137E-06	2.304E-05	-.216E-07
7.50000	3.50000	0.00551	7.212	24.838	12.114	8.825
(STRAIN)			-1.040E-05	3.286E-05	1.630E-06	.433E-04
7.50000	6.99900	0.00552	10.667	12.530	12.404	15.139
(STRAIN)			3.528E-06	8.100E-06	7.791E-06	.743E-04
7.50000	7.00100	0.00552	10.668	40.701	39.661	15.144
(STRAIN)			-3.382E-07	8.086E-06	7.794E-06	.850E-05
7.50000	11.00000	0.00552	5.842	-7.374	-12.045	15.480
(STRAIN)			2.135E-06	-1.572E-06	-2.882E-06	.868E-05
7.50000	14.99900	0.00551	0.986	-54.491	-67.598	0.240
(STRAIN)			4.707E-06	-1.085E-05	-1.453E-05	.134E-06
7.50000	15.00100	0.00551	0.986	0.161	0.096	0.233
(STRAIN)			3.533E-05	-1.086E-05	-1.453E-05	.261E-04
7.50000	25.00100	0.00515	0.666	0.201	0.189	0.066
(STRAIN)			4.907E-05	-1.843E-05	-2.008E-05	.190E-04
9.00000	0.00100	0.00538	-0.557	9.423	14.174	0.000
(STRAIN)			-1.603E-05	8.467E-06	2.013E-05	.581E-09
9.00000	3.50000	0.00544	1.884	19.495	10.999	4.892
(STRAIN)			-1.598E-05	2.725E-05	6.393E-06	.240E-04
9.00000	6.99900	0.00547	4.741	8.856	9.039	11.857
(STRAIN)			-2.768E-06	7.333E-06	7.782E-06	.582E-04
9.00000	7.00100	0.00547	4.742	36.434	38.083	11.862
(STRAIN)			-1.570E-06	7.320E-06	7.782E-06	.665E-05
9.00000	11.00000	0.00547	3.490	-5.970	-11.149	14.728
(STRAIN)			1.478E-06	-1.176E-06	-2.628E-06	.826E-05
9.00000	14.99900	0.00546	0.927	-47.194	-63.430	0.257
(STRAIN)			4.273E-06	-9.224E-06	-1.378E-05	.144E-06
9.00000	15.00100	0.00546	0.927	0.179	0.098	0.250
(STRAIN)			3.264E-05	-9.227E-06	-1.378E-05	.280E-04
9.00000	25.00100	0.00512	0.653	0.204	0.188	0.076
(STRAIN)			4.763E-05	-1.747E-05	-1.972E-05	.219E-04
10.50000	0.00100	0.00532	0.106	12.145	14.640	-0.003
(STRAIN)			-1.685E-05	1.270E-05	1.882E-05	-.151E-07
10.50000	3.50000	0.00538	0.001	15.386	10.305	3.295

(STRAIN)			-1.635E-05	2.142E-05	8.944E-06	.162E-04
10.50000	6.99900	0.00542	1.626	6.546	7.068	9.120
(STRAIN)			-5.707E-06	6.369E-06	7.651E-06	.448E-04
10.50000	7.00100	0.00542	1.627	31.769	36.371	9.124
(STRAIN)			-2.096E-06	6.358E-06	7.649E-06	.512E-05
10.50000	11.00000	0.00542	1.932	-4.962	-10.297	13.449
(STRAIN)			1.029E-06	-9.042E-07	-2.400E-06	.754E-05
10.50000	14.99900	0.00541	0.871	-40.592	-59.348	0.266
(STRAIN)			3.869E-06	-7.761E-06	-1.302E-05	.149E-06
10.50000	15.00100	0.00541	0.871	0.194	0.100	0.260
(STRAIN)			3.011E-05	-7.764E-06	-1.303E-05	.291E-04
10.50000	25.00100	0.00509	0.638	0.207	0.187	0.084
(STRAIN)			4.609E-05	-1.645E-05	-1.933E-05	.245E-04
12.00000	0.00100	0.00527	0.126	12.957	14.567	-0.002
(STRAIN)			-1.729E-05	1.421E-05	1.816E-05	-.825E-08
12.00000	3.50000	0.00533	-0.537	12.505	9.802	2.743
(STRAIN)			-1.517E-05	1.684E-05	1.020E-05	.135E-04
12.00000	6.99900	0.00537	0.228	5.137	5.964	7.152
(STRAIN)			-6.650E-06	5.399E-06	7.430E-06	.351E-04
12.00000	7.00100	0.00537	0.229	27.329	34.585	7.156
(STRAIN)			-2.209E-06	5.392E-06	7.427E-06	.401E-05
12.00000	11.00000	0.00537	1.018	-4.254	-9.512	12.000
(STRAIN)			7.519E-07	-7.268E-07	-2.202E-06	.673E-05
12.00000	14.99900	0.00536	0.819	-34.848	-55.469	0.268
(STRAIN)			3.504E-06	-6.500E-06	-1.228E-05	.151E-06
12.00000	15.00100	0.00536	0.819	0.206	0.103	0.263
(STRAIN)			2.781E-05	-6.502E-06	-1.229E-05	.295E-04
12.00000	25.00100	0.00506	0.623	0.210	0.186	0.092
(STRAIN)			4.446E-05	-1.538E-05	-1.890E-05	.266E-04
13.50000	0.00100	0.00522	-0.366	12.417	13.967	0.000
(STRAIN)			-1.745E-05	1.392E-05	1.773E-05	-.123E-08
13.50000	3.50000	0.00528	-0.564	10.491	9.384	2.566
(STRAIN)			-1.367E-05	1.346E-05	1.074E-05	.126E-04
13.50000	6.99900	0.00531	-0.271	4.260	5.331	5.829
(STRAIN)			-6.596E-06	4.525E-06	7.155E-06	.286E-04
13.50000	7.00100	0.00531	-0.270	23.409	32.793	5.833
(STRAIN)			-2.122E-06	4.520E-06	7.152E-06	.327E-05
13.50000	11.00000	0.00532	0.546	-3.738	-8.806	10.593
(STRAIN)			5.921E-07	-6.096E-07	-2.031E-06	.594E-05
13.50000	14.99900	0.00531	0.772	-29.968	-51.862	0.267
(STRAIN)			3.182E-06	-5.440E-06	-1.158E-05	.150E-06
13.50000	15.00100	0.00531	0.772	0.215	0.105	0.263
(STRAIN)			2.576E-05	-5.442E-06	-1.158E-05	.294E-04
13.50000	25.00100	0.00502	0.606	0.213	0.184	0.098
(STRAIN)			4.279E-05	-1.431E-05	-1.845E-05	.285E-04

Roller Compacted Concrete (RCC)

POISSON'S RATIOS OF LAYERS (PR) ARE : 0.35 0.15 0.4 0.45
FOR PERIOD NO. 1 LAYER NO. AND MODULUS ARE : 1 5.500E+05 2 3.600E+06 3 2.500E+04 4 1.000E+04

RADIAL COORDINATE	VERTICAL COORDINATE	VERTICAL DISPLACEMENT	VERTICAL STRESS (STRAIN)	RADIAL STRESS (STRAIN)	TANGENTIAL STRESS (STRAIN)	SHEAR STRESS (STRAIN)
0.00000	0.00100	0.00659	120.037	102.922	102.922	0.000
(STRAIN)			8.726E-05	4.525E-05	4.525E-05	.000E+00
0.00000	3.50000	0.00612	98.672	28.717	28.717	0.000
(STRAIN)			1.429E-04	-2.885E-05	-2.885E-05	.000E+00
0.00000	6.99900	0.00574	58.521	37.283	37.283	0.000
(STRAIN)			5.895E-05	6.821E-06	6.821E-06	.000E+00
0.00000	7.00100	0.00574	58.496	39.296	39.296	0.000
(STRAIN)			1.297E-05	6.841E-06	6.841E-06	.000E+00
0.00000	11.00000	0.00571	18.792	-15.346	-15.346	0.000
(STRAIN)			6.499E-06	-4.406E-06	-4.406E-06	.000E+00
0.00000	14.99900	0.00568	1.246	-76.991	-76.991	0.000
(STRAIN)			6.762E-06	-1.823E-05	-1.823E-05	.000E+00
0.00000	15.00100	0.00568	1.246	0.071	0.071	0.000
(STRAIN)			4.757E-05	-1.823E-05	-1.823E-05	.000E+00
0.00000	25.00100	0.00526	0.726	0.188	0.188	0.000
(STRAIN)			5.562E-05	-2.230E-05	-2.230E-05	.000E+00
1.50000	0.00100	0.00654	120.368	104.102	103.669	-0.007
(STRAIN)			8.663E-05	4.671E-05	4.564E-05	-.326E-07
1.50000	3.50000	0.00609	93.373	28.185	27.263	9.363
(STRAIN)			1.345E-04	-2.552E-05	-2.779E-05	.460E-04
1.50000	6.99900	0.00574	55.028	35.570	35.505	8.863
(STRAIN)			5.482E-05	7.061E-06	6.902E-06	.435E-04
1.50000	7.00100	0.00574	55.005	39.596	39.104	8.864
(STRAIN)			1.200E-05	7.078E-06	6.920E-06	.566E-05
1.50000	11.00000	0.00570	17.925	-14.839	-15.193	5.472
(STRAIN)			6.231E-06	-4.236E-06	-4.349E-06	.350E-05
1.50000	14.99900	0.00568	1.235	-75.622	-76.360	0.070
(STRAIN)			6.676E-06	-1.788E-05	-1.811E-05	.448E-07
1.50000	15.00100	0.00568	1.235	0.076	0.071	0.068
(STRAIN)			4.705E-05	-1.788E-05	-1.812E-05	.760E-05
1.50000	25.00100	0.00526	0.724	0.189	0.188	0.015
(STRAIN)			5.545E-05	-2.218E-05	-2.226E-05	.449E-05
3.00000	0.00100	0.00640	120.218	106.291	105.031	0.021
(STRAIN)			8.410E-05	4.992E-05	4.682E-05	.104E-06
3.00000	3.50000	0.00598	75.775	28.161	23.281	18.693
(STRAIN)			1.050E-04	-1.183E-05	-2.381E-05	.918E-04
3.00000	6.99900	0.00571	45.451	30.856	30.627	15.624
(STRAIN)			4.351E-05	7.689E-06	7.127E-06	.767E-04
3.00000	7.00100	0.00571	45.434	40.310	38.575	15.626
(STRAIN)			9.334E-06	7.697E-06	7.143E-06	.998E-05
3.00000	11.00000	0.00568	15.550	-13.453	-14.756	10.104
(STRAIN)			5.495E-06	-3.770E-06	-4.186E-06	.646E-05
3.00000	14.99900	0.00566	1.204	-71.734	-74.540	0.135
(STRAIN)			6.429E-06	-1.687E-05	-1.777E-05	.860E-07
3.00000	15.00100	0.00566	1.204	0.089	0.073	0.130

(STRAIN)			4.557E-05	-1.687E-05	-1.777E-05	.146E-04
3.00000	25.00100	0.00525	0.719	0.190	0.188	0.031
(STRAIN)			5.493E-05	-2.182E-05	-2.214E-05	.886E-05
4.50000	0.00100	0.00601	60.000	49.469	64.837	0.991
(STRAIN)			3.635E-05	1.050E-05	4.822E-05	.486E-05
4.50000	3.50000	0.00583	46.682	30.429	18.361	22.238
(STRAIN)			5.383E-05	1.393E-05	-1.569E-05	.109E-03
4.50000	6.99900	0.00568	32.531	24.405	24.017	18.736
(STRAIN)			2.833E-05	8.388E-06	7.435E-06	.920E-04
4.50000	7.00100	0.00568	32.521	40.731	37.795	18.740
(STRAIN)			5.762E-06	8.384E-06	7.446E-06	.120E-04
4.50000	11.00000	0.00566	12.261	-11.532	-14.087	13.312
(STRAIN)			4.473E-06	-3.127E-06	-3.943E-06	.851E-05
4.50000	14.99900	0.00564	1.156	-65.912	-71.737	0.189
(STRAIN)			6.057E-06	-1.537E-05	-1.723E-05	.121E-06
4.50000	15.00100	0.00564	1.156	0.108	0.075	0.183
(STRAIN)			4.332E-05	-1.537E-05	-1.723E-05	.205E-04
4.50000	25.00100	0.00523	0.712	0.192	0.187	0.045
(STRAIN)			5.410E-05	-2.124E-05	-2.194E-05	.130E-04
6.00000	0.00100	0.00570	-0.290	0.246	17.140	-0.021
(STRAIN)			-1.159E-05	-1.028E-05	3.119E-05	-.102E-06
6.00000	3.50000	0.00573	20.576	30.069	14.618	16.286
(STRAIN)			8.974E-06	3.227E-05	-5.651E-06	.799E-04
6.00000	6.99900	0.00568	19.991	17.898	17.496	18.150
(STRAIN)			1.382E-05	8.687E-06	7.699E-06	.891E-04
6.00000	7.00100	0.00568	19.988	39.732	36.699	18.154
(STRAIN)			2.368E-06	8.675E-06	7.706E-06	.116E-04
6.00000	11.00000	0.00567	8.790	-9.460	-13.225	14.919
(STRAIN)			3.387E-06	-2.443E-06	-3.646E-06	.953E-05
6.00000	14.99900	0.00565	1.101	-58.764	-68.050	0.231
(STRAIN)			5.590E-06	-1.353E-05	-1.650E-05	.148E-06
6.00000	15.00100	0.00565	1.100	0.134	0.081	0.224
(STRAIN)			4.057E-05	-1.354E-05	-1.650E-05	.251E-04
6.00000	25.00100	0.00526	0.705	0.199	0.191	0.058
(STRAIN)			5.293E-05	-2.039E-05	-2.158E-05	.168E-04
7.50000	0.00100	0.00556	-0.746	6.116	15.230	-0.004
(STRAIN)			-1.494E-05	1.904E-06	2.427E-05	-.210E-07
7.50000	3.50000	0.00563	7.170	25.196	12.549	9.062
(STRAIN)			-1.098E-05	3.326E-05	2.220E-06	.445E-04
7.50000	6.99900	0.00564	10.517	12.644	12.433	15.352
(STRAIN)			3.163E-06	8.386E-06	7.866E-06	.754E-04
7.50000	7.00100	0.00564	10.517	37.033	35.458	15.357
(STRAIN)			-9.910E-08	8.371E-06	7.868E-06	.981E-05
7.50000	11.00000	0.00563	5.743	-7.633	-12.308	15.134
(STRAIN)			2.426E-06	-1.847E-06	-3.340E-06	.967E-05
7.50000	14.99900	0.00562	1.035	-51.361	-64.069	0.260
(STRAIN)			5.097E-06	-1.164E-05	-1.570E-05	.166E-06
7.50000	15.00100	0.00562	1.035	0.157	0.084	0.254
(STRAIN)			3.756E-05	-1.164E-05	-1.570E-05	.284E-04
7.50000	25.00100	0.00524	0.691	0.202	0.190	0.070
(STRAIN)			5.145E-05	-1.941E-05	-2.120E-05	.203E-04
9.00000	0.00100	0.00548	-0.557	10.049	15.023	0.000
(STRAIN)			-1.697E-05	9.065E-06	2.127E-05	.109E-08
9.00000	3.50000	0.00555	1.871	19.824	11.414	5.109

(STRAIN)			-1.648E-05	2.759E-05	6.947E-06	.251E-04
9.00000	6.99900	0.00558	4.662	9.002	9.123	12.095
(STRAIN)			-3.057E-06	7.595E-06	7.891E-06	.594E-04
9.00000	7.00100	0.00558	4.664	33.107	34.072	12.100
(STRAIN)			-1.504E-06	7.582E-06	7.891E-06	.773E-05
9.00000	11.00000	0.00558	3.435	-6.204	-11.400	14.382
(STRAIN)			1.688E-06	-1.392E-06	-3.051E-06	.919E-05
9.00000	14.99900	0.00557	0.971	-44.311	-60.036	0.278
(STRAIN)			4.618E-06	-9.848E-06	-1.487E-05	.178E-06
9.00000	15.00100	0.00557	0.971	0.177	0.087	0.272
(STRAIN)			3.461E-05	-9.850E-06	-1.487E-05	.305E-04
9.00000	25.00100	0.00521	0.676	0.206	0.189	0.081
(STRAIN)			4.990E-05	-1.838E-05	-2.081E-05	.234E-04
10.50000	0.00100	0.00542	0.106	12.666	15.403	-0.003
(STRAIN)			-1.767E-05	1.316E-05	1.988E-05	-.147E-07
10.50000	3.50000	0.00548	0.002	15.684	10.696	3.488
(STRAIN)			-1.678E-05	2.171E-05	9.465E-06	.171E-04
10.50000	6.99900	0.00553	1.588	6.684	7.173	9.366
(STRAIN)			-5.931E-06	6.577E-06	7.778E-06	.460E-04
10.50000	7.00100	0.00553	1.589	28.761	32.546	9.370
(STRAIN)			-2.113E-06	6.567E-06	7.776E-06	.599E-05
10.50000	11.00000	0.00553	1.907	-5.166	-10.534	13.115
(STRAIN)			1.184E-06	-1.075E-06	-2.790E-06	.838E-05
10.50000	14.99900	0.00552	0.910	-37.945	-56.090	0.287
(STRAIN)			4.171E-06	-8.241E-06	-1.404E-05	.184E-06
10.50000	15.00100	0.00552	0.910	0.194	0.091	0.282
(STRAIN)			3.184E-05	-8.244E-06	-1.404E-05	.315E-04
10.50000	25.00100	0.00518	0.661	0.209	0.188	0.090
(STRAIN)			4.823E-05	-1.727E-05	-2.039E-05	.261E-04
12.00000	0.00100	0.00537	0.126	13.396	15.257	-0.002
(STRAIN)			-1.800E-05	1.457E-05	1.914E-05	-.791E-08
12.00000	3.50000	0.00543	-0.532	12.771	10.165	2.914
(STRAIN)			-1.556E-05	1.709E-05	1.069E-05	.143E-04
12.00000	6.99900	0.00547	0.211	5.249	6.071	7.393
(STRAIN)			-6.820E-06	5.547E-06	7.563E-06	.363E-04
12.00000	7.00100	0.00547	0.212	24.614	30.941	7.397
(STRAIN)			-2.256E-06	5.539E-06	7.560E-06	.473E-05
12.00000	11.00000	0.00547	1.012	-4.427	-9.736	11.682
(STRAIN)			8.712E-07	-8.662E-07	-2.562E-06	.746E-05
12.00000	14.99900	0.00546	0.854	-32.422	-52.345	0.290
(STRAIN)			3.769E-06	-6.861E-06	-1.322E-05	.185E-06
12.00000	15.00100	0.00546	0.854	0.207	0.094	0.285
(STRAIN)			2.933E-05	-6.863E-06	-1.323E-05	.319E-04
12.00000	25.00100	0.00514	0.644	0.213	0.186	0.098
(STRAIN)			4.649E-05	-1.613E-05	-1.993E-05	.284E-04
13.50000	0.00100	0.00532	-0.366	12.790	14.594	0.000
(STRAIN)			-1.809E-05	1.420E-05	1.863E-05	-.948E-09
13.50000	3.50000	0.00538	-0.560	10.724	9.720	2.718
(STRAIN)			-1.403E-05	1.367E-05	1.121E-05	.133E-04
13.50000	6.99900	0.00541	-0.277	4.343	5.431	6.056
(STRAIN)			-6.724E-06	4.618E-06	7.286E-06	.297E-04
13.50000	7.00100	0.00541	-0.277	20.961	29.322	6.060
(STRAIN)			-2.172E-06	4.612E-06	7.283E-06	.387E-05
13.50000	11.00000	0.00542	0.551	-3.881	-9.015	10.295

(STRAIN)			6.905E-07	-7.254E-07	-2.365E-06	.658E-05
13.50000	14.99900	0.00541	0.803	-27.748	-48.868	0.288
(STRAIN)			3.415E-06	-5.705E-06	-1.245E-05	.184E-06
13.50000	15.00100	0.00541	0.803	0.217	0.097	0.284
(STRAIN)			2.709E-05	-5.707E-06	-1.246E-05	.318E-04
13.50000	25.00100	0.00510	0.627	0.216	0.185	0.104
(STRAIN)			4.469E-05	-1.497E-05	-1.944E-05	.303E-04

Lean Concrete (LC)

POISSON'S RATIOS OF LAYERS (PR) ARE : 0.35 0.15 0.4 0.45
FOR PERIOD NO. 1 LAYER NO. AND MODULUS ARE : 1 5.500E+05 2 2.100E+06
3 2.500E+04 4 1.000E+04

RADIAL COORDINATE	VERTICAL COORDINATE	VERTICAL DISPLACEMENT	VERTICAL STRESS (STRAIN)	RADIAL STRESS (STRAIN)	TANGENTIAL STRESS (STRAIN)	SHEAR STRESS (STRAIN)
0.00000	0.00100	0.00709	120.037	108.695	108.695	0.000
(STRAIN)			7.991E-05	5.207E-05	5.207E-05	.000E+00
0.00000	3.50000	0.00664	97.208	30.208	30.208	0.000
(STRAIN)			1.383E-04	-2.616E-05	-2.616E-05	.000E+00
0.00000	6.99900	0.00628	55.068	33.570	33.570	0.000
(STRAIN)			5.740E-05	4.630E-06	4.630E-06	.000E+00
0.00000	7.00100	0.00628	55.044	21.193	21.193	0.000
(STRAIN)			2.318E-05	4.646E-06	4.646E-06	.000E+00
0.00000	11.00000	0.00621	17.276	-16.244	-16.244	0.000
(STRAIN)			1.055E-05	-7.809E-06	-7.809E-06	.000E+00
0.00000	14.99900	0.00618	1.557	-61.932	-61.932	0.000
(STRAIN)			9.589E-06	-2.518E-05	-2.518E-05	.000E+00
0.00000	15.00100	0.00618	1.557	-0.012	-0.012	0.000
(STRAIN)			6.264E-05	-2.518E-05	-2.518E-05	.000E+00
0.00000	25.00100	0.00564	0.841	0.185	0.185	0.000
(STRAIN)			6.742E-05	-2.766E-05	-2.766E-05	.000E+00
1.50000	0.00100	0.00704	120.368	109.701	109.335	-0.006
(STRAIN)			7.946E-05	5.328E-05	5.238E-05	-.313E-07
1.50000	3.50000	0.00660	92.008	29.653	28.760	9.846
(STRAIN)			1.301E-04	-2.294E-05	-2.513E-05	.483E-04
1.50000	6.99900	0.00626	51.787	32.157	32.020	8.943
(STRAIN)			5.332E-05	5.136E-06	4.800E-06	.439E-04
1.50000	7.00100	0.00626	51.764	21.751	21.140	8.944
(STRAIN)			2.159E-05	5.150E-06	4.816E-06	.980E-05
1.50000	11.00000	0.00620	16.485	-15.743	-16.087	4.959
(STRAIN)			1.012E-05	-7.525E-06	-7.714E-06	.543E-05
1.50000	14.99900	0.00617	1.541	-60.745	-61.385	0.098
(STRAIN)			9.457E-06	-2.465E-05	-2.500E-05	.108E-06
1.50000	15.00100	0.00617	1.541	-0.004	-0.011	0.096
(STRAIN)			6.187E-05	-2.466E-05	-2.501E-05	.108E-04
1.50000	25.00100	0.00564	0.839	0.186	0.185	0.020
(STRAIN)			6.719E-05	-2.749E-05	-2.760E-05	.584E-05
3.00000	0.00100	0.00689	120.218	111.404	110.395	0.022
(STRAIN)			7.743E-05	5.580E-05	5.332E-05	.106E-06
3.00000	3.50000	0.00649	74.674	29.567	24.793	19.565
(STRAIN)			1.012E-04	-9.539E-06	-2.126E-05	.960E-04
3.00000	6.99900	0.00623	42.787	28.261	27.770	15.817

(STRAIN)			4.214E-05	6.484E-06	5.279E-06	.776E-04
3.00000	7.00100	0.00623	42.769	23.196	21.007	15.818
(STRAIN)			1.721E-05	6.490E-06	5.292E-06	.173E-04
3.00000	11.00000	0.00618	14.318	-14.365	-15.635	9.150
(STRAIN)			8.961E-06	-6.746E-06	-7.442E-06	.100E-04
3.00000	14.99900	0.00615	1.496	-57.373	-59.809	0.189
(STRAIN)			9.083E-06	-2.316E-05	-2.449E-05	.207E-06
3.00000	15.00100	0.00615	1.496	0.016	-0.008	0.185
(STRAIN)			5.969E-05	-2.316E-05	-2.449E-05	.207E-04
3.00000	25.00100	0.00563	0.833	0.188	0.185	0.040
(STRAIN)			6.649E-05	-2.700E-05	-2.744E-05	.115E-04
4.50000	0.00100	0.00649	60.000	53.888	69.758	0.991
(STRAIN)			3.041E-05	1.541E-05	5.436E-05	.487E-05
4.50000	3.50000	0.00633	45.932	31.747	19.882	23.344
(STRAIN)			5.066E-05	1.584E-05	-1.328E-05	.115E-03
4.50000	6.99900	0.00618	30.643	22.882	22.008	19.098
(STRAIN)			2.715E-05	8.099E-06	5.954E-06	.938E-04
4.50000	7.00100	0.00618	30.633	24.716	20.823	19.100
(STRAIN)			1.133E-05	8.094E-06	5.962E-06	.209E-04
4.50000	11.00000	0.00615	11.316	-12.441	-14.942	12.039
(STRAIN)			7.345E-06	-5.666E-06	-7.035E-06	.132E-04
4.50000	14.99900	0.00612	1.427	-52.337	-57.384	0.264
(STRAIN)			8.517E-06	-2.093E-05	-2.369E-05	.289E-06
4.50000	15.00100	0.00612	1.427	0.046	-0.003	0.259
(STRAIN)			5.639E-05	-2.093E-05	-2.369E-05	.290E-04
4.50000	25.00100	0.00561	0.823	0.191	0.185	0.058
(STRAIN)			6.536E-05	-2.621E-05	-2.717E-05	.169E-04
6.00000	0.00100	0.00615	-0.290	3.880	21.534	-0.020
(STRAIN)			-1.670E-05	-6.464E-06	3.687E-05	-.990E-07
6.00000	3.50000	0.00620	20.162	31.280	16.118	17.460
(STRAIN)			6.496E-06	3.378E-05	-3.430E-06	.857E-04
6.00000	6.99900	0.00616	18.853	17.338	16.305	18.721
(STRAIN)			1.287E-05	9.150E-06	6.616E-06	.919E-04
6.00000	7.00100	0.00616	18.849	25.091	20.493	18.723
(STRAIN)			5.720E-06	9.138E-06	6.620E-06	.205E-04
6.00000	11.00000	0.00613	8.147	-10.337	-14.046	13.465
(STRAIN)			5.621E-06	-4.501E-06	-6.532E-06	.147E-04
6.00000	14.99900	0.00611	1.344	-46.185	-54.213	0.322
(STRAIN)			7.811E-06	-1.822E-05	-2.261E-05	.352E-06
6.00000	15.00100	0.00611	1.344	0.084	0.006	0.316
(STRAIN)			5.231E-05	-1.822E-05	-2.262E-05	.354E-04
6.00000	25.00100	0.00563	0.810	0.199	0.187	0.075
(STRAIN)			6.367E-05	-2.504E-05	-2.667E-05	.218E-04
7.50000	0.00100	0.00600	-0.746	9.031	19.117	-0.004
(STRAIN)			-1.927E-05	4.728E-06	2.949E-05	-.182E-07
7.50000	3.50000	0.00608	7.008	26.314	14.013	10.179
(STRAIN)			-1.292E-05	3.447E-05	4.274E-06	.500E-04
7.50000	6.99900	0.00609	9.940	12.704	11.853	16.116
(STRAIN)			2.447E-06	9.230E-06	7.140E-06	.791E-04
7.50000	7.00100	0.00609	9.940	23.852	20.065	16.120
(STRAIN)			1.596E-06	9.215E-06	7.141E-06	.177E-04
7.50000	11.00000	0.00608	5.364	-8.454	-13.098	13.621
(STRAIN)			4.094E-06	-3.473E-06	-6.017E-06	.149E-04
7.50000	14.99900	0.00606	1.252	-39.867	-50.821	0.361

(STRAIN)			7.074E-06	-1.544E-05	-2.144E-05	.395E-06
7.50000	15.00100	0.00606	1.252	0.120	0.012	0.355
(STRAIN)			4.796E-05	-1.545E-05	-2.145E-05	.397E-04
7.50000	25.00100	0.00560	0.793	0.203	0.186	0.090
(STRAIN)			6.175E-05	-2.375E-05	-2.618E-05	.262E-04
9.00000	0.00100	0.00591	-0.557	12.345	18.437	0.001
(STRAIN)			-2.060E-05	1.107E-05	2.602E-05	.352E-08
9.00000	3.50000	0.00598	1.856	20.843	12.808	6.103
(STRAIN)			-1.804E-05	2.857E-05	8.842E-06	.300E-04
9.00000	6.99900	0.00602	4.429	9.322	8.902	12.986
(STRAIN)			-3.544E-06	8.466E-06	7.434E-06	.638E-04
9.00000	7.00100	0.00602	4.430	21.338	19.474	12.990
(STRAIN)			-8.054E-07	8.453E-06	7.433E-06	.142E-04
9.00000	11.00000	0.00601	3.256	-6.933	-12.149	12.896
(STRAIN)			2.913E-06	-2.666E-06	-5.523E-06	.141E-04
9.00000	14.99900	0.00600	1.161	-33.866	-47.372	0.383
(STRAIN)			6.356E-06	-1.283E-05	-2.022E-05	.420E-06
9.00000	15.00100	0.00600	1.161	0.152	0.020	0.378
(STRAIN)			4.372E-05	-1.283E-05	-2.023E-05	.423E-04
9.00000	25.00100	0.00556	0.774	0.208	0.185	0.104
(STRAIN)			5.969E-05	-2.236E-05	-2.566E-05	.301E-04
10.50000	0.00100	0.00583	0.106	14.473	18.405	-0.003
(STRAIN)			-2.073E-05	1.453E-05	2.419E-05	-.127E-07
10.50000	3.50000	0.00590	0.049	16.598	11.995	4.344
(STRAIN)			-1.811E-05	2.251E-05	1.122E-05	.213E-04
10.50000	6.99900	0.00595	1.531	7.032	7.122	10.303
(STRAIN)			-6.223E-06	7.279E-06	7.500E-06	.506E-04
10.50000	7.00100	0.00595	1.532	18.302	18.719	10.307
(STRAIN)			-1.915E-06	7.269E-06	7.497E-06	.113E-04
10.50000	11.00000	0.00595	1.859	-5.782	-11.237	11.706
(STRAIN)			2.101E-06	-2.083E-06	-5.071E-06	.128E-04
10.50000	14.99900	0.00593	1.076	-28.490	-44.006	0.393
(STRAIN)			5.691E-06	-1.050E-05	-1.900E-05	.431E-06
10.50000	15.00100	0.00593	1.076	0.179	0.027	0.388
(STRAIN)			3.976E-05	-1.050E-05	-1.900E-05	.435E-04
10.50000	25.00100	0.00552	0.754	0.213	0.184	0.115
(STRAIN)			5.748E-05	-2.088E-05	-2.508E-05	.335E-04
12.00000	0.00100	0.00577	0.126	14.832	17.911	-0.001
(STRAIN)			-2.061E-05	1.549E-05	2.305E-05	-.636E-08
12.00000	3.50000	0.00583	-0.472	13.572	11.359	3.643
(STRAIN)			-1.672E-05	1.775E-05	1.232E-05	.179E-04
12.00000	6.99900	0.00588	0.232	5.520	6.079	8.313
(STRAIN)			-6.960E-06	6.020E-06	7.393E-06	.408E-04
12.00000	7.00100	0.00588	0.233	15.339	17.854	8.315
(STRAIN)			-2.260E-06	6.013E-06	7.390E-06	.911E-05
12.00000	11.00000	0.00588	1.041	-4.923	-10.391	10.372
(STRAIN)			1.590E-06	-1.677E-06	-4.671E-06	.114E-04
12.00000	14.99900	0.00586	0.999	-23.877	-40.829	0.394
(STRAIN)			5.098E-06	-8.525E-06	-1.781E-05	.431E-06
12.00000	15.00100	0.00586	0.999	0.200	0.034	0.390
(STRAIN)			3.621E-05	-8.528E-06	-1.781E-05	.436E-04
12.00000	25.00100	0.00548	0.733	0.219	0.183	0.125
(STRAIN)			5.516E-05	-1.935E-05	-2.446E-05	.363E-04
13.50000	0.00100	0.00570	-0.366	13.948	16.957	0.000

(STRAIN)			-2.033E-05	1.480E-05	2.219E-05	.297E-09
13.50000	3.50000	0.00576	-0.509	11.409	10.807	3.343
(STRAIN)			-1.506E-05	1.419E-05	1.271E-05	.164E-04
13.50000	6.99900	0.00580	-0.227	4.509	5.445	6.918
(STRAIN)			-6.748E-06	4.878E-06	7.176E-06	.340E-04
13.50000	7.00100	0.00580	-0.227	12.738	16.937	6.920
(STRAIN)			-2.228E-06	4.872E-06	7.172E-06	.758E-05
13.50000	11.00000	0.00581	0.620	-4.262	-9.621	9.087
(STRAIN)			1.287E-06	-1.387E-06	-4.321E-06	.995E-05
13.50000	14.99900	0.00579	0.930	-20.027	-37.900	0.389
(STRAIN)			4.581E-06	-6.896E-06	-1.668E-05	.426E-06
13.50000	15.00100	0.00579	0.930	0.216	0.041	0.385
(STRAIN)			3.309E-05	-6.898E-06	-1.669E-05	.431E-04
13.50000	25.00100	0.00543	0.711	0.224	0.182	0.133
(STRAIN)			5.279E-05	-1.782E-05	-2.381E-05	.386E-04

Cement Treated Base (CTB)

POISSON'S RATIOS OF LAYERS (PR) ARE : 0.35 0.2 0.4 0.45
FOR PERIOD NO. 1 LAYER NO. AND MODULUS ARE : 1 5.500E+05 2 1.100E+06 3 2.500E+04 4 1.000E+04

RADIAL COORDINATE	VERTICAL COORDINATE	VERTICAL DISPLACEMENT	VERTICAL STRESS (STRAIN)	RADIAL STRESS (STRAIN)	TANGENTIAL STRESS (STRAIN)	SHEAR STRESS (STRAIN)
0.00000	0.00100	0.00772	120.037	116.818	116.818	0.000
(STRAIN)			6.957E-05	6.167E-05	6.167E-05	.000E+00
0.00000	3.50000	0.00729	94.622	30.607	30.607	0.000
(STRAIN)			1.331E-04	-2.404E-05	-2.404E-05	.000E+00
0.00000	6.99900	0.00694	49.822	23.448	23.448	0.000
(STRAIN)			6.074E-05	-3.993E-06	-3.993E-06	.000E+00
0.00000	7.00100	0.00694	49.801	6.973	6.973	0.000
(STRAIN)			4.274E-05	-3.984E-06	-3.984E-06	.000E+00
0.00000	11.00000	0.00682	15.410	-16.625	-16.625	0.000
(STRAIN)			2.005E-05	-1.489E-05	-1.489E-05	.000E+00
0.00000	14.99900	0.00675	1.990	-49.253	-49.253	0.000
(STRAIN)			1.972E-05	-3.618E-05	-3.618E-05	.000E+00
0.00000	15.00100	0.00675	1.990	-0.181	-0.181	0.000
(STRAIN)			8.539E-05	-3.619E-05	-3.619E-05	.000E+00
0.00000	25.00100	0.00607	0.980	0.165	0.165	0.000
(STRAIN)			8.312E-05	-3.501E-05	-3.501E-05	.000E+00
1.50000	0.00100	0.00767	120.368	117.500	117.259	-0.006
(STRAIN)			6.946E-05	6.242E-05	6.183E-05	-.287E-07
1.50000	3.50000	0.00725	89.610	30.041	29.191	10.663
(STRAIN)			1.252E-04	-2.098E-05	-2.307E-05	.523E-04
1.50000	6.99900	0.00692	46.900	22.728	22.398	8.647
(STRAIN)			5.656E-05	-2.775E-06	-3.584E-06	.424E-04
1.50000	7.00100	0.00692	46.878	7.731	6.990	8.646
(STRAIN)			3.994E-05	-2.766E-06	-3.574E-06	.189E-04
1.50000	11.00000	0.00681	14.727	-16.151	-16.472	4.281
(STRAIN)			1.932E-05	-1.437E-05	-1.472E-05	.934E-05
1.50000	14.99900	0.00674	1.967	-48.276	-48.766	0.143
(STRAIN)			1.943E-05	-3.538E-05	-3.591E-05	.311E-06
1.50000	15.00100	0.00674	1.966	-0.170	-0.179	0.141
(STRAIN)			8.424E-05	-3.538E-05	-3.592E-05	.158E-04

1.50000	25.00100	0.00606	0.977	0.166	0.165	0.027
(STRAIN)			8.279E-05	-3.478E-05	-3.493E-05	.774E-05
3.00000	0.00100	0.00751	120.218	118.309	117.763	0.023
(STRAIN)			6.835E-05	6.366E-05	6.232E-05	.111E-06
3.00000	3.50000	0.00713	72.782	29.922	25.305	21.024
(STRAIN)			9.719E-05	-8.016E-06	-1.935E-05	.103E-03
3.00000	6.99900	0.00687	38.882	20.721	19.526	15.354
(STRAIN)			4.508E-05	5.064E-07	-2.427E-06	.754E-04
3.00000	7.00100	0.00687	38.865	9.745	7.062	15.352
(STRAIN)			3.228E-05	5.089E-07	-2.418E-06	.335E-04
3.00000	11.00000	0.00678	12.853	-14.842	-16.029	7.901
(STRAIN)			1.730E-05	-1.292E-05	-1.421E-05	.172E-04
3.00000	14.99900	0.00671	1.900	-45.503	-47.365	0.273
(STRAIN)			1.861E-05	-3.310E-05	-3.513E-05	.596E-06
3.00000	15.00100	0.00671	1.900	-0.137	-0.173	0.270
(STRAIN)			8.096E-05	-3.311E-05	-3.514E-05	.303E-04
3.00000	25.00100	0.00605	0.969	0.169	0.165	0.053
(STRAIN)			8.183E-05	-3.409E-05	-3.470E-05	.152E-04
4.50000	0.00100	0.00709	60.000	59.532	76.318	0.992
(STRAIN)			2.264E-05	2.149E-05	6.269E-05	.487E-05
4.50000	3.50000	0.00694	44.702	32.065	20.492	25.154
(STRAIN)			4.783E-05	1.681E-05	-1.159E-05	.123E-03
4.50000	6.99900	0.00680	28.047	17.846	15.645	18.694
(STRAIN)			2.968E-05	4.643E-06	-7.600E-07	.918E-04
4.50000	7.00100	0.00680	28.037	12.149	7.206	18.693
(STRAIN)			2.197E-05	4.637E-06	-7.552E-07	.408E-04
4.50000	11.00000	0.00673	10.253	-12.998	-15.345	10.399
(STRAIN)			1.447E-05	-1.089E-05	-1.345E-05	.227E-04
4.50000	14.99900	0.00667	1.800	-41.361	-45.217	0.382
(STRAIN)			1.738E-05	-2.971E-05	-3.391E-05	.833E-06
4.50000	15.00100	0.00667	1.799	-0.088	-0.164	0.378
(STRAIN)			7.601E-05	-2.971E-05	-3.392E-05	.423E-04
4.50000	25.00100	0.00603	0.956	0.174	0.165	0.077
(STRAIN)			8.027E-05	-3.300E-05	-3.432E-05	.223E-04
6.00000	0.00100	0.00672	-0.290	8.152	27.175	-0.019
(STRAIN)			-2.301E-05	-2.286E-06	4.441E-05	-.934E-07
6.00000	3.50000	0.00678	19.562	31.577	16.810	19.321
(STRAIN)			4.776E-06	3.427E-05	-1.979E-06	.948E-04
6.00000	6.99900	0.00673	17.496	14.627	11.803	18.587
(STRAIN)			1.499E-05	7.950E-06	1.019E-06	.912E-04
6.00000	7.00100	0.00673	17.492	13.702	7.361	18.587
(STRAIN)			1.207E-05	7.938E-06	1.020E-06	.406E-04
6.00000	11.00000	0.00669	7.497	-10.956	-14.456	11.635
(STRAIN)			1.144E-05	-8.694E-06	-1.251E-05	.254E-04
6.00000	14.99900	0.00664	1.676	-36.330	-42.456	0.463
(STRAIN)			1.585E-05	-2.561E-05	-3.230E-05	.101E-05
6.00000	15.00100	0.00664	1.676	-0.030	-0.149	0.458
(STRAIN)			6.990E-05	-2.562E-05	-3.230E-05	.513E-04
6.00000	25.00100	0.00602	0.937	0.183	0.167	0.099
(STRAIN)			7.794E-05	-3.140E-05	-3.366E-05	.288E-04
7.50000	0.00100	0.00654	-0.746	12.045	23.858	-0.003
(STRAIN)			-2.420E-05	7.193E-06	3.619E-05	-.132E-07
7.50000	3.50000	0.00662	6.868	26.609	14.744	11.868
(STRAIN)			-1.383E-05	3.463E-05	5.504E-06	.583E-04

7.50000	6.99900	0.00664	9.471	11.591	8.793	16.327
(STRAIN)			4.248E-06	9.453E-06	2.584E-06	.801E-04
7.50000	7.00100	0.00664	9.470	13.774	7.490	16.328
(STRAIN)			4.743E-06	9.438E-06	2.583E-06	.356E-04
7.50000	11.00000	0.00661	5.064	-9.086	-13.503	11.772
(STRAIN)			8.710E-06	-6.725E-06	-1.154E-05	.257E-04
7.50000	14.99900	0.00656	1.544	-31.163	-39.508	0.517
(STRAIN)			1.425E-05	-2.143E-05	-3.053E-05	.113E-05
7.50000	15.00100	0.00656	1.544	0.028	-0.135	0.512
(STRAIN)			6.346E-05	-2.143E-05	-3.054E-05	.574E-04
7.50000	25.00100	0.00598	0.914	0.190	0.167	0.119
(STRAIN)			7.538E-05	-2.965E-05	-3.301E-05	.345E-04
9.00000	0.00100	0.00642	-0.557	14.331	22.365	0.002
(STRAIN)			-2.436E-05	1.218E-05	3.190E-05	.749E-08
9.00000	3.50000	0.00650	1.972	21.134	13.521	7.515
(STRAIN)			-1.847E-05	2.857E-05	9.880E-06	.369E-04
9.00000	6.99900	0.00654	4.456	9.033	6.774	13.479
(STRAIN)			-1.957E-06	9.277E-06	3.733E-06	.662E-04
9.00000	7.00100	0.00654	4.457	12.584	7.511	13.480
(STRAIN)			3.980E-07	9.264E-06	3.729E-06	.294E-04
9.00000	11.00000	0.00652	3.205	-7.524	-12.529	11.146
(STRAIN)			6.560E-06	-5.145E-06	-1.060E-05	.243E-04
9.00000	14.99900	0.00649	1.414	-26.251	-36.518	0.546
(STRAIN)			1.270E-05	-1.748E-05	-2.868E-05	.119E-05
9.00000	15.00100	0.00649	1.414	0.081	-0.119	0.542
(STRAIN)			5.717E-05	-1.749E-05	-2.869E-05	.607E-04
9.00000	25.00100	0.00594	0.890	0.198	0.167	0.136
(STRAIN)			7.258E-05	-2.774E-05	-3.230E-05	.396E-04
10.50000	0.00100	0.00632	0.106	15.704	21.654	-0.002
(STRAIN)			-2.358E-05	1.471E-05	2.931E-05	-.974E-08
10.50000	3.50000	0.00640	0.255	16.869	12.648	5.467
(STRAIN)			-1.832E-05	2.246E-05	1.210E-05	.268E-04
10.50000	6.99900	0.00644	1.763	7.038	5.529	10.955
(STRAIN)			-4.791E-06	8.156E-06	4.452E-06	.538E-04
10.50000	7.00100	0.00644	1.764	10.795	7.404	10.957
(STRAIN)			-1.705E-06	8.146E-06	4.447E-06	.239E-04
10.50000	11.00000	0.00643	1.956	-6.292	-11.578	10.112
(STRAIN)			5.027E-06	-3.971E-06	-9.737E-06	.221E-04
10.50000	14.99900	0.00640	1.293	-21.861	-33.625	0.557
(STRAIN)			1.126E-05	-1.399E-05	-2.683E-05	.121E-05
10.50000	15.00100	0.00640	1.293	0.126	-0.103	0.553
(STRAIN)			5.135E-05	-1.400E-05	-2.683E-05	.619E-04
10.50000	25.00100	0.00589	0.864	0.206	0.166	0.151
(STRAIN)			6.959E-05	-2.572E-05	-3.151E-05	.438E-04
12.00000	0.00100	0.00624	0.126	15.552	20.619	-0.001
(STRAIN)			-2.279E-05	1.508E-05	2.751E-05	-.423E-08
12.00000	3.50000	0.00631	-0.272	13.799	11.930	4.520
(STRAIN)			-1.687E-05	1.767E-05	1.308E-05	.222E-04
12.00000	6.99900	0.00635	0.508	5.553	4.776	9.011
(STRAIN)			-5.649E-06	6.733E-06	4.827E-06	.442E-04
12.00000	7.00100	0.00635	0.509	8.939	7.193	9.013
(STRAIN)			-2.470E-06	6.726E-06	4.822E-06	.197E-04
12.00000	11.00000	0.00634	1.209	-5.334	-10.682	8.950
(STRAIN)			4.011E-06	-3.127E-06	-8.961E-06	.195E-04

12.00000	14.99900	0.00632	1.184	-18.112	-30.925	0.554
(STRAIN)			9.992E-06	-1.106E-05	-2.504E-05	.121E-05
12.00000	15.00100	0.00632	1.184	0.162	-0.087	0.550
(STRAIN)			4.617E-05	-1.106E-05	-2.504E-05	.616E-04
12.00000	25.00100	0.00584	0.836	0.215	0.166	0.163
(STRAIN)			6.646E-05	-2.364E-05	-3.065E-05	.472E-04
13.50000	0.00100	0.00615	-0.366	14.339	19.240	0.000
(STRAIN)			-2.203E-05	1.406E-05	2.609E-05	.182E-08
13.50000	3.50000	0.00621	-0.352	11.572	11.287	4.027
(STRAIN)			-1.519E-05	1.408E-05	1.338E-05	.198E-04
13.50000	6.99900	0.00625	0.023	4.468	4.301	7.589
(STRAIN)			-5.538E-06	5.372E-06	4.961E-06	.373E-04
13.50000	7.00100	0.00625	0.024	7.290	6.915	7.590
(STRAIN)			-2.561E-06	5.366E-06	4.956E-06	.166E-04
13.50000	11.00000	0.00625	0.809	-4.572	-9.857	7.828
(STRAIN)			3.359E-06	-2.511E-06	-8.277E-06	.171E-04
13.50000	14.99900	0.00623	1.089	-15.004	-28.462	0.542
(STRAIN)			8.893E-06	-8.663E-06	-2.334E-05	.118E-05
13.50000	15.00100	0.00623	1.089	0.190	-0.072	0.539
(STRAIN)			4.167E-05	-8.666E-06	-2.335E-05	.604E-04
13.50000	25.00100	0.00578	0.807	0.222	0.166	0.172
(STRAIN)			6.329E-05	-2.157E-05	-2.976E-05	.500E-04

Soil Cement(SC)

POISSON'S RATIOS OF LAYERS (PR) ARE : 0.35 0.2 0.4 0.45
FOR PERIOD NO. 1 LAYER NO. AND MODULUS ARE : 1 5.500E+05 2 5.500E+05
3 2.500E+04 4 1.000E+04

RADIAL COORDINATE	VERTICAL COORDINATE	VERTICAL DISPLACEMENT	VERTICAL STRESS (STRAIN)	RADIAL STRESS (STRAIN)	TANGENTIAL STRESS (STRAIN)	SHEAR STRESS (STRAIN)
0.00000	0.00100	0.00852	120.037	127.573	127.573	0.000
(STRAIN)			5.588E-05	7.438E-05	7.438E-05	.000E+00
0.00000	3.50000	0.00813	90.858	29.286	29.286	0.000
(STRAIN)			1.279E-04	-2.321E-05	-2.321E-05	.000E+00
0.00000	6.99900	0.00777	42.955	5.033	5.033	0.000
(STRAIN)			7.170E-05	-2.139E-05	-2.139E-05	.000E+00
0.00000	7.00100	0.00777	42.934	-3.968	-3.968	0.000
(STRAIN)			8.095E-05	-2.138E-05	-2.138E-05	.000E+00
0.00000	11.00000	0.00756	13.446	-15.233	-15.233	0.000
(STRAIN)			3.553E-05	-2.705E-05	-2.705E-05	.000E+00
0.00000	14.99900	0.00744	2.578	-35.560	-35.560	0.000
(STRAIN)			3.055E-05	-5.266E-05	-5.266E-05	.000E+00
0.00000	15.00100	0.00744	2.578	-0.476	-0.476	0.000
(STRAIN)			1.183E-04	-5.267E-05	-5.267E-05	.000E+00
0.00000	25.00100	0.00654	1.150	0.122	0.122	0.000
(STRAIN)			1.040E-04	-4.504E-05	-4.504E-05	.000E+00
1.50000	0.00100	0.00847	120.368	127.771	127.717	-0.005
(STRAIN)			5.627E-05	7.444E-05	7.431E-05	-.250E-07
1.50000	3.50000	0.00808	86.125	28.726	27.931	11.819
(STRAIN)			1.205E-04	-2.035E-05	-2.230E-05	.580E-04
1.50000	6.99900	0.00775	40.531	5.441	4.780	7.839
(STRAIN)			6.719E-05	-1.894E-05	-2.056E-05	.385E-04
1.50000	7.00100	0.00775	40.510	-3.079	-3.822	7.836

(STRAIN)			7.616E-05	-1.894E-05	-2.056E-05	.342E-04
1.50000	11.00000	0.00754	12.893	-14.813	-15.091	3.486
(STRAIN)			3.432E-05	-2.613E-05	-2.674E-05	.152E-04
1.50000	14.99900	0.00743	2.545	-34.836	-35.201	0.207
(STRAIN)			3.010E-05	-5.146E-05	-5.226E-05	.903E-06
1.50000	15.00100	0.00743	2.545	-0.458	-0.472	0.206
(STRAIN)			1.167E-04	-5.147E-05	-5.227E-05	.230E-04
1.50000	25.00100	0.00653	1.147	0.124	0.123	0.036
(STRAIN)			1.036E-04	-4.471E-05	-4.493E-05	.104E-04
3.00000	0.00100	0.00829	120.218	127.245	127.390	0.024
(STRAIN)			5.654E-05	7.379E-05	7.414E-05	.117E-06
3.00000	3.50000	0.00793	70.045	28.633	24.211	23.078
(STRAIN)			9.373E-05	-7.922E-06	-1.877E-05	.113E-03
3.00000	6.99900	0.00767	33.875	6.524	4.111	13.981
(STRAIN)			5.482E-05	-1.231E-05	-1.823E-05	.686E-04
3.00000	7.00100	0.00767	33.859	-0.679	-3.392	13.976
(STRAIN)			6.304E-05	-1.231E-05	-1.823E-05	.610E-04
3.00000	11.00000	0.00750	11.375	-13.648	-14.680	6.445
(STRAIN)			3.098E-05	-2.361E-05	-2.586E-05	.281E-04
3.00000	14.99900	0.00739	2.450	-32.780	-34.169	0.396
(STRAIN)			2.880E-05	-4.807E-05	-5.110E-05	.173E-05
3.00000	15.00100	0.00739	2.449	-0.406	-0.460	0.394
(STRAIN)			1.118E-04	-4.807E-05	-5.110E-05	.441E-04
3.00000	25.00100	0.00652	1.136	0.129	0.123	0.070
(STRAIN)			1.022E-04	-4.376E-05	-4.461E-05	.204E-04
4.50000	0.00100	0.00785	60.000	66.585	84.739	0.994
(STRAIN)			1.279E-05	2.896E-05	7.352E-05	.488E-05
4.50000	3.50000	0.00772	42.948	30.834	19.615	27.685
(STRAIN)			4.598E-05	1.625E-05	-1.129E-05	.136E-03
4.50000	6.99900	0.00757	24.853	7.782	3.259	17.181
(STRAIN)			3.816E-05	-3.740E-06	-1.484E-05	.843E-04
4.50000	7.00100	0.00757	24.844	2.362	-2.723	17.177
(STRAIN)			4.530E-05	-3.749E-06	-1.484E-05	.750E-04
4.50000	11.00000	0.00743	9.257	-11.994	-14.045	8.511
(STRAIN)			2.630E-05	-2.007E-05	-2.454E-05	.371E-04
4.50000	14.99900	0.00733	2.306	-29.706	-32.583	0.553
(STRAIN)			2.684E-05	-4.300E-05	-4.928E-05	.241E-05
4.50000	15.00100	0.00733	2.305	-0.330	-0.442	0.550
(STRAIN)			1.046E-04	-4.301E-05	-4.928E-05	.616E-04
4.50000	25.00100	0.00649	1.118	0.136	0.124	0.103
(STRAIN)			1.001E-04	-4.224E-05	-4.409E-05	.298E-04
6.00000	0.00100	0.00743	-0.290	13.174	34.239	-0.017
(STRAIN)			-3.070E-05	2.350E-06	5.405E-05	-.851E-07
6.00000	3.50000	0.00750	18.743	30.464	16.152	21.888
(STRAIN)			4.414E-06	3.318E-05	-1.946E-06	.107E-03
6.00000	6.99900	0.00745	16.005	8.502	2.477	17.344
(STRAIN)			2.211E-05	3.698E-06	-1.109E-05	.851E-04
6.00000	7.00100	0.00745	16.000	4.840	-1.934	17.340
(STRAIN)			2.803E-05	3.685E-06	-1.110E-05	.757E-04
6.00000	11.00000	0.00735	6.989	-10.142	-13.229	9.567
(STRAIN)			2.121E-05	-1.617E-05	-2.291E-05	.417E-04
6.00000	14.99900	0.00726	2.128	-25.987	-30.565	0.670
(STRAIN)			2.443E-05	-3.691E-05	-4.690E-05	.292E-05
6.00000	15.00100	0.00726	2.128	-0.238	-0.417	0.667

(STRAIN)			9.560E-05	-3.691E-05	-4.690E-05	.747E-04
6.00000	25.00100	0.00647	1.093	0.148	0.126	0.132
(STRAIN)			9.698E-05	-4.009E-05	-4.323E-05	.384E-04
7.50000	0.00100	0.00722	-0.746	15.193	29.575	-0.001
(STRAIN)			-2.984E-05	9.278E-06	4.458E-05	-.592E-08
7.50000	3.50000	0.00731	6.735	25.644	14.241	14.149
(STRAIN)			-1.314E-05	3.328E-05	5.287E-06	.695E-04
7.50000	6.99900	0.00731	9.183	8.323	1.924	15.554
(STRAIN)			1.018E-05	8.064E-06	-7.643E-06	.764E-04
7.50000	7.00100	0.00731	9.182	6.032	-1.164	15.552
(STRAIN)			1.492E-05	8.051E-06	-7.648E-06	.679E-04
7.50000	11.00000	0.00724	4.959	-8.405	-12.345	9.735
(STRAIN)			1.656E-05	-1.260E-05	-2.119E-05	.425E-04
7.50000	14.99900	0.00717	1.940	-22.143	-28.388	0.747
(STRAIN)			2.190E-05	-3.064E-05	-4.427E-05	.326E-05
7.50000	15.00100	0.00717	1.940	-0.146	-0.389	0.744
(STRAIN)			8.615E-05	-3.065E-05	-4.427E-05	.833E-04
7.50000	25.00100	0.00642	1.064	0.159	0.126	0.159
(STRAIN)			9.354E-05	-3.770E-05	-4.235E-05	.460E-04
9.00000	0.00100	0.00706	-0.557	15.965	26.879	0.003
(STRAIN)			-2.828E-05	1.228E-05	3.906E-05	.133E-07
9.00000	3.50000	0.00715	2.216	20.321	13.096	9.359
(STRAIN)			-1.724E-05	2.720E-05	9.470E-06	.459E-04
9.00000	6.99900	0.00717	4.806	7.429	1.600	13.145
(STRAIN)			2.992E-06	9.431E-06	-4.877E-06	.645E-04
9.00000	7.00100	0.00717	4.806	6.039	-0.517	13.145
(STRAIN)			6.731E-06	9.419E-06	-4.883E-06	.574E-04
9.00000	11.00000	0.00713	3.370	-6.911	-11.436	9.278
(STRAIN)			1.280E-05	-9.633E-06	-1.951E-05	.405E-04
9.00000	14.99900	0.00707	1.755	-18.470	-26.167	0.788
(STRAIN)			1.942E-05	-2.470E-05	-4.150E-05	.344E-05
9.00000	15.00100	0.00707	1.755	-0.060	-0.360	0.785
(STRAIN)			7.690E-05	-2.471E-05	-4.150E-05	.879E-04
9.00000	25.00100	0.00636	1.032	0.171	0.128	0.181
(STRAIN)			8.973E-05	-3.506E-05	-4.137E-05	.526E-04
10.50000	0.00100	0.00693	0.106	16.252	25.174	-0.001
(STRAIN)			-2.617E-05	1.346E-05	3.536E-05	-.547E-08
10.50000	3.50000	0.00700	0.632	16.183	12.244	6.862
(STRAIN)			-1.694E-05	2.123E-05	1.156E-05	.337E-04
10.50000	6.99900	0.00704	2.344	6.245	1.437	10.920
(STRAIN)			-6.265E-07	8.948E-06	-2.853E-06	.536E-04
10.50000	7.00100	0.00704	2.345	5.380	-0.028	10.920
(STRAIN)			2.317E-06	8.939E-06	-2.860E-06	.477E-04
10.50000	11.00000	0.00701	2.263	-5.696	-10.545	8.476
(STRAIN)			1.002E-05	-7.345E-06	-1.792E-05	.370E-04
10.50000	14.99900	0.00696	1.583	-15.179	-24.013	0.801
(STRAIN)			1.713E-05	-1.944E-05	-3.872E-05	.350E-05
10.50000	15.00100	0.00696	1.583	0.016	-0.328	0.798
(STRAIN)			6.833E-05	-1.945E-05	-3.872E-05	.894E-04
10.50000	25.00100	0.00630	0.997	0.184	0.129	0.200
(STRAIN)			8.564E-05	-3.227E-05	-4.027E-05	.580E-04
12.00000	0.00100	0.00680	0.126	15.395	23.361	0.000
(STRAIN)			-2.443E-05	1.305E-05	3.260E-05	-.125E-08
12.00000	3.50000	0.00687	0.091	13.197	11.508	5.531

(STRAIN)			-1.556E-05	1.661E-05	1.247E-05	.272E-04
12.00000	6.99900	0.00691	1.095	5.091	1.366	9.130
(STRAIN)			-2.118E-06	7.690E-06	-1.452E-06	.448E-04
12.00000	7.00100	0.00691	1.095	4.508	0.318	9.130
(STRAIN)			2.365E-07	7.683E-06	-1.459E-06	.398E-04
12.00000	11.00000	0.00689	1.561	-4.725	-9.703	7.552
(STRAIN)			8.084E-06	-5.631E-06	-1.649E-05	.330E-04
12.00000	14.99900	0.00684	1.430	-12.363	-21.998	0.794
(STRAIN)			1.510E-05	-1.500E-05	-3.602E-05	.347E-05
12.00000	15.00100	0.00684	1.430	0.078	-0.297	0.792
(STRAIN)			6.071E-05	-1.500E-05	-3.603E-05	.886E-04
12.00000	25.00100	0.00624	0.961	0.197	0.130	0.215
(STRAIN)			8.139E-05	-2.943E-05	-3.909E-05	.624E-04
13.50000	0.00100	0.00668	-0.366	13.767	21.389	0.001
(STRAIN)			-2.304E-05	1.165E-05	3.036E-05	.379E-08
13.50000	3.50000	0.00675	-0.063	11.011	10.832	4.746
(STRAIN)			-1.401E-05	1.317E-05	1.273E-05	.233E-04
13.50000	6.99900	0.00678	0.521	4.112	1.338	7.756
(STRAIN)			-2.520E-06	6.292E-06	-5.148E-07	.381E-04
13.50000	7.00100	0.00678	0.522	3.673	0.552	7.756
(STRAIN)			-5.875E-07	6.287E-06	-5.219E-07	.338E-04
13.50000	11.00000	0.00677	1.148	-3.941	-8.924	6.643
(STRAIN)			6.765E-06	-4.338E-06	-1.521E-05	.290E-04
13.50000	14.99900	0.00673	1.297	-10.027	-20.158	0.774
(STRAIN)			1.333E-05	-1.137E-05	-3.348E-05	.338E-05
13.50000	15.00100	0.00673	1.297	0.128	-0.267	0.772
(STRAIN)			5.411E-05	-1.137E-05	-3.348E-05	.865E-04
13.50000	25.00100	0.00616	0.924	0.209	0.131	0.227
(STRAIN)			7.709E-05	-2.660E-05	-3.786E-05	.658E-04

Granular Base (GB)

POISSON'S RATIOS OF LAYERS (PR) ARE : 0.35 0.32 0.4 0.45
FOR PERIOD NO. 1 LAYER NO. AND MODULUS ARE : 1 5.500E+05 2 3.500E+04
3 2.500E+04 4 1.000E+04

RADIAL COORDINATE	VERTICAL COORDINATE	VERTICAL DISPLACEMENT	VERTICAL STRESS (STRAIN)	RADIAL STRESS (STRAIN)	TANGENTIAL STRESS (STRAIN)	SHEAR STRESS (STRAIN)
0.00000	0.00100	0.01326	120.037	192.489	192.489	0.000
(STRAIN)			-2.674E-05	1.511E-04	1.511E-04	.000E+00
0.00000	3.50000	0.01301	72.116	12.866	12.866	0.000
(STRAIN)			1.147E-04	-3.069E-05	-3.069E-05	.000E+00
0.00000	6.99900	0.01255	14.089	-121.228	-121.228	0.000
(STRAIN)			1.799E-04	-1.522E-04	-1.522E-04	.000E+00
0.00000	7.00100	0.01255	14.085	-1.210	-1.210	0.000
(STRAIN)			4.245E-04	-1.523E-04	-1.523E-04	.000E+00
0.00000	11.00000	0.01124	7.837	-1.649	-1.649	0.000
(STRAIN)			2.541E-04	-1.037E-04	-1.037E-04	.000E+00
0.00000	14.99900	0.01038	4.827	-2.732	-2.732	0.000
(STRAIN)			1.879E-04	-9.721E-05	-9.721E-05	.000E+00
0.00000	15.00100	0.01038	4.826	-0.833	-0.833	0.000
(STRAIN)			2.197E-04	-9.721E-05	-9.721E-05	.000E+00
0.00000	25.00100	0.00868	2.035	0.061	0.061	0.000

(STRAIN)			1.980E-04	-8.821E-05	-8.821E-05	.000E+00
1.50000	0.00100	0.01316	120.368	190.213	191.115	-0.001
(STRAIN)			-2.381E-05	1.476E-04	1.498E-04	-.599E-08
1.50000	3.50000	0.01291	68.643	12.520	11.900	17.336
(STRAIN)			1.093E-04	-2.849E-05	-3.001E-05	.851E-04
1.50000	6.99900	0.01248	13.630	-114.884	-117.431	1.671
(STRAIN)			1.726E-04	-1.428E-04	-1.491E-04	.820E-05
1.50000	7.00100	0.01248	13.625	-1.019	-1.185	1.661
(STRAIN)			4.094E-04	-1.429E-04	-1.491E-04	.125E-03
1.50000	11.00000	0.01120	7.691	-1.578	-1.635	0.772
(STRAIN)			2.491E-04	-1.004E-04	-1.026E-04	.582E-04
1.50000	14.99900	0.01036	4.773	-2.680	-2.711	0.368
(STRAIN)			1.857E-04	-9.542E-05	-9.661E-05	.277E-04
1.50000	15.00100	0.01036	4.772	-0.808	-0.829	0.368
(STRAIN)			2.171E-04	-9.542E-05	-9.661E-05	.412E-04
1.50000	25.00100	0.00867	2.027	0.064	0.061	0.076
(STRAIN)			1.970E-04	-8.753E-05	-8.798E-05	.221E-04
3.00000	0.00100	0.01287	120.218	182.785	186.501	0.031
(STRAIN)			-1.642E-05	1.372E-04	1.463E-04	.152E-06
3.00000	3.50000	0.01264	55.883	13.106	9.263	33.004
(STRAIN)			8.737E-05	-1.763E-05	-2.706E-05	.162E-03
3.00000	6.99900	0.01227	12.352	-97.398	-106.798	3.046
(STRAIN)			1.524E-04	-1.170E-04	-1.401E-04	.150E-04
3.00000	7.00100	0.01227	12.349	-0.500	-1.112	3.027
(STRAIN)			3.676E-04	-1.170E-04	-1.401E-04	.228E-03
3.00000	11.00000	0.01110	7.278	-1.378	-1.593	1.458
(STRAIN)			2.351E-04	-9.134E-05	-9.946E-05	.110E-03
3.00000	14.99900	0.01029	4.619	-2.530	-2.652	0.711
(STRAIN)			1.793E-04	-9.028E-05	-9.486E-05	.537E-04
3.00000	15.00100	0.01029	4.618	-0.737	-0.819	0.711
(STRAIN)			2.096E-04	-9.027E-05	-9.485E-05	.796E-04
3.00000	25.00100	0.00863	2.003	0.074	0.062	0.150
(STRAIN)			1.942E-04	-8.554E-05	-8.731E-05	.435E-04
4.50000	0.00100	0.01226	60.000	112.171	137.515	1.003
(STRAIN)			-4.980E-05	7.826E-05	1.405E-04	.492E-05
4.50000	3.50000	0.01223	33.230	16.414	6.182	40.178
(STRAIN)			4.604E-05	4.764E-06	-2.035E-05	.197E-03
4.50000	6.99900	0.01195	10.560	-73.538	-91.706	3.902
(STRAIN)			1.244E-04	-8.207E-05	-1.267E-04	.192E-04
4.50000	7.00100	0.01195	10.557	0.186	-0.996	3.881
(STRAIN)			3.090E-04	-8.210E-05	-1.267E-04	.293E-03
4.50000	11.00000	0.01092	6.668	-1.092	-1.528	1.998
(STRAIN)			2.145E-04	-7.818E-05	-9.464E-05	.151E-03
4.50000	14.99900	0.01017	4.380	-2.302	-2.558	1.011
(STRAIN)			1.696E-04	-8.243E-05	-9.209E-05	.762E-04
4.50000	15.00100	0.01017	4.379	-0.630	-0.803	1.010
(STRAIN)			1.981E-04	-8.243E-05	-9.208E-05	.113E-03
4.50000	25.00100	0.00857	1.965	0.089	0.062	0.220
(STRAIN)			1.897E-04	-8.233E-05	-8.622E-05	.638E-04
6.00000	0.00100	0.01161	-0.290	47.574	79.633	-0.008
(STRAIN)			-8.148E-05	3.601E-05	1.147E-04	-.391E-07
6.00000	3.50000	0.01176	13.396	17.573	4.384	34.983
(STRAIN)			1.038E-05	2.064E-05	-1.174E-05	.172E-03
6.00000	6.99900	0.01158	8.667	-49.831	-75.539	4.207

(STRAIN)			9.554E-05	-4.805E-05	-1.111E-04	.207E-04
6.00000	7.00100	0.01158	8.665	0.816	-0.857	4.188
(STRAIN)			2.480E-04	-4.807E-05	-1.112E-04	.316E-03
6.00000	11.00000	0.01070	5.950	-0.771	-1.446	2.364
(STRAIN)			1.903E-04	-6.322E-05	-8.866E-05	.178E-03
6.00000	14.99900	0.01002	4.079	-2.021	-2.438	1.253
(STRAIN)			1.573E-04	-7.275E-05	-8.847E-05	.945E-04
6.00000	15.00100	0.01002	4.078	-0.500	-0.780	1.253
(STRAIN)			1.836E-04	-7.275E-05	-8.847E-05	.140E-03
6.00000	25.00100	0.00850	1.912	0.109	0.063	0.284
(STRAIN)			1.835E-04	-7.799E-05	-8.467E-05	.824E-04
7.50000	0.00100	0.01116	-0.746	38.688	67.347	0.008
(STRAIN)			-6.883E-05	2.796E-05	9.830E-05	.372E-07
7.50000	3.50000	0.01129	4.798	14.521	3.976	26.317
(STRAIN)			-3.047E-06	2.082E-05	-5.065E-06	.129E-03
7.50000	6.99900	0.01117	6.995	-30.953	-61.052	4.121
(STRAIN)			7.127E-05	-2.188E-05	-9.576E-05	.202E-04
7.50000	7.00100	0.01117	6.994	1.242	-0.717	4.106
(STRAIN)			1.950E-04	-2.189E-05	-9.579E-05	.310E-03
7.50000	11.00000	0.01045	5.211	-0.464	-1.354	2.568
(STRAIN)			1.655E-04	-4.852E-05	-8.208E-05	.194E-03
7.50000	14.99900	0.00984	3.743	-1.716	-2.301	1.435
(STRAIN)			1.437E-04	-6.222E-05	-8.428E-05	.108E-03
7.50000	15.00100	0.00983	3.742	-0.360	-0.754	1.434
(STRAIN)			1.675E-04	-6.222E-05	-8.428E-05	.161E-03
7.50000	25.00100	0.00840	1.849	0.132	0.063	0.342
(STRAIN)			1.761E-04	-7.285E-05	-8.284E-05	.990E-04
9.00000	0.00100	0.01074	-0.557	29.977	57.515	0.010
(STRAIN)			-5.669E-05	1.826E-05	8.585E-05	.504E-07
9.00000	3.50000	0.01085	2.426	11.022	4.074	19.755
(STRAIN)			-5.196E-06	1.590E-05	-1.151E-06	.970E-04
9.00000	6.99900	0.01076	5.655	-17.891	-49.330	3.853
(STRAIN)			5.306E-05	-4.737E-06	-8.190E-05	.189E-04
9.00000	7.00100	0.01076	5.655	1.454	-0.593	3.842
(STRAIN)			1.537E-04	-4.754E-06	-8.193E-05	.290E-03
9.00000	11.00000	0.01017	4.509	-0.198	-1.258	2.640
(STRAIN)			1.421E-04	-3.538E-05	-7.537E-05	.199E-03
9.00000	14.99900	0.00963	3.394	-1.410	-2.155	1.557
(STRAIN)			1.296E-04	-5.162E-05	-7.972E-05	.117E-03
9.00000	15.00100	0.00962	3.394	-0.223	-0.725	1.557
(STRAIN)			1.509E-04	-5.163E-05	-7.972E-05	.174E-03
9.00000	25.00100	0.00829	1.777	0.158	0.064	0.391
(STRAIN)			1.677E-04	-6.705E-05	-8.070E-05	.113E-03
10.50000	0.00100	0.01033	0.106	22.690	49.549	0.005
(STRAIN)			-4.578E-05	9.656E-06	7.558E-05	.245E-07
10.50000	3.50000	0.01043	1.904	8.563	4.194	15.241
(STRAIN)			-4.656E-06	1.169E-05	9.652E-07	.748E-04
10.50000	6.99900	0.01036	4.613	-9.332	-40.192	3.540
(STRAIN)			3.990E-05	5.675E-06	-7.007E-05	.174E-04
10.50000	7.00100	0.01036	4.613	1.517	-0.492	3.533
(STRAIN)			1.224E-04	5.655E-06	-7.010E-05	.266E-03
10.50000	11.00000	0.00987	3.877	0.016	-1.164	2.621
(STRAIN)			1.213E-04	-2.434E-05	-6.884E-05	.198E-03
10.50000	14.99900	0.00940	3.050	-1.120	-2.007	1.629

(STRAIN)			1.157E-04	-4.155E-05	-7.498E-05	.123E-03
10.50000	15.00100	0.00940	3.050	-0.096	-0.693	1.628
(STRAIN)			1.346E-04	-4.156E-05	-7.498E-05	.182E-03
10.50000	25.00100	0.00816	1.698	0.185	0.064	0.433
(STRAIN)			1.586E-04	-6.081E-05	-7.831E-05	.125E-03
12.00000	0.00100	0.00995	0.126	16.171	42.472	0.004
(STRAIN)			-3.709E-05	2.294E-06	6.685E-05	.219E-07
12.00000	3.50000	0.01002	1.703	6.994	4.211	12.009
(STRAIN)			-4.033E-06	8.953E-06	2.122E-06	.590E-04
12.00000	6.99900	0.00997	3.796	-3.702	-33.068	3.238
(STRAIN)			3.030E-05	1.190E-05	-6.018E-05	.159E-04
12.00000	7.00100	0.00997	3.799	1.496	-0.413	3.231
(STRAIN)			9.863E-05	1.179E-05	-6.020E-05	.244E-03
12.00000	11.00000	0.00957	3.324	0.182	-1.073	2.543
(STRAIN)			1.031E-04	-1.540E-05	-6.270E-05	.192E-03
12.00000	14.99900	0.00916	2.723	-0.857	-1.860	1.657
(STRAIN)			1.026E-04	-3.236E-05	-7.021E-05	.125E-03
12.00000	15.00100	0.00916	2.723	0.016	-0.660	1.657
(STRAIN)			1.192E-04	-3.237E-05	-7.022E-05	.186E-03
12.00000	25.00100	0.00802	1.614	0.212	0.065	0.466
(STRAIN)			1.490E-04	-5.432E-05	-7.572E-05	.135E-03
13.50000	0.00100	0.00958	-0.366	10.525	36.194	0.004
(STRAIN)			-3.039E-05	-3.663E-06	5.934E-05	.212E-07
13.50000	3.50000	0.00964	1.507	5.932	4.131	9.598
(STRAIN)			-3.663E-06	7.197E-06	2.776E-06	.471E-04
13.50000	6.99900	0.00960	3.143	0.105	-27.435	2.960
(STRAIN)			2.311E-05	1.565E-05	-5.195E-05	.145E-04
13.50000	7.00100	0.00960	3.142	1.441	-0.352	2.955
(STRAIN)			7.983E-05	1.566E-05	-5.197E-05	.223E-03
13.50000	11.00000	0.00926	2.849	0.305	-0.987	2.431
(STRAIN)			8.762E-05	-8.300E-06	-5.703E-05	.183E-03
13.50000	14.99900	0.00891	2.421	-0.623	-1.719	1.653
(STRAIN)			9.058E-05	-2.423E-05	-6.555E-05	.125E-03
13.50000	15.00100	0.00891	2.420	0.112	-0.626	1.653
(STRAIN)			1.050E-04	-2.423E-05	-6.555E-05	.185E-03
13.50000	25.00100	0.00787	1.528	0.239	0.065	0.491
(STRAIN)			1.391E-04	-4.779E-05	-7.298E-05	.142E-03

APPENDIX D - Economic Evaluation Spreadsheets

Reinforcement Bar

Longitudinal Reinforcement (ERA) = 600mm^2/m

Diameter of bar(mm)	Area of bar ((π*D2)/4)	no. of bar/m	no. of bar in 3.65 wide lane	no. of bar in 3.5 wide lane
Ø14	153.86	3.899649032	14.23371897	13.64877161

For JRCP, contraction joints are generally at a standard distance of 25 meters.
1km=1000m

1000/25=40 bays in/lane/km=80 bays/carriageway/km

Formula to specific bar length to kg= ((0.222*D^2)/36)*L

D= diameter of bar(mm)

L= length of bar(m)

Carriageway width	Longitudinal bar	Total number of bar /km	Total bar weight(kg)
7.3m	Ø14,25m long	2240	67685.33333
7m	Ø14,25m long	2080	62850.66667

Transverse reinforcement consisting of 12 mm diameter steel bars at 600 mm spacing

Reinforcement bar(Transverse)	Number transverse bar within 25m bay	Total number of bar/carriageway/km	Total bar weight(kg)
Ø12,3.65m	41.66666667	6560	21262.272
Ø12,3.5m	41.66666667	6560	20388.48

Dowel bars was 25 mm in diameter at 300 mm spacing, 400 mm long

Tie bars was 12 mm in diameter at 600 mm spacing, 1000 mm long.

Carriageway width	Joint bar	Total no./km	Total bar weight(kg)
7.3m	Dowel bar Ø12,0.4m long	1922.333333	682.6944
	Tie bars Ø12,1m long	1666.666667	1479.408
7m	Dowel bar Ø12,0.4m long	1843.333333	654.6336
	Tie bars Ø12,1m long	1666.666667	1479.408

Shoulder Reinforcement

Longitudinal

no. of bar/m	no. of bar in 1.5 wide Shoulder	Total number longitudinal of bar /km	Total bar weight(kg)
3.899649032	5.849473547	960	29008

Transverse

Reinforcement bar(Transverse)	Number transverse bar within 25m bay	Total number of bar/shoulders/km	Total bar weight(kg)
Ø12,1.5m	41.66666667	6560	8737.92

Joint	Joint bar	Total no./km	Total bar weight(kg)
Contraction	Dowel bar Ø12,0.4m long	790	280.608

Carriageway Width	Total bar weight(kg)
7.3m	129136.2357
7m	123399.7163

Adama-Awash road section (Flexible Pavement structure) Economic analysis

Carriageway Width=7.3m Shoulder Width=1.5m Length of Road= 1km

Carriageway Area=7.3*1000=7300m^2 Shoulder Area=1.5*1000=1500m^2

Discount Rate =10.23% Inflation Rate = 6.5% per year

Analysis year	Calendar Year	Activity	Thickness (cm)	Quantity	Unit	Unit Cost(birr)	Total(birr)	PW
0	2015	Asphalt Concrete	21.59	1576	m^3	5206.16	8205279.97	8205280
		Crushed stone base	25.4	1854	m^3	618.00	1145895.07	1145895
		Granular sub-base	24.13	1761	m^3	233.72	411688.11	411688.1
		Shoulders AC	21.59	647.7	m^3	5206.16	3372032.86	3372033
		Shoulders Crushed stone base	25.4	762	m^3	618.00	470915.78	470915.8
		Shoulders Granular sub-base	24.13	723.9	m^3	233.72	169186.89	169186.9
						Sub Total	13774998.69	
11	2026	Overlay	9	657	m^3	10469.22	6878278.37	2356036
18	2033	Overlay	17.78	1298	m^3	16269.03	21116223.49	3657798
		Crushed stone base	10	730		1931.22	1409791.43	327082.8
25	2040	Reconstruction						
		Asphalt Concrete	29.21	2132	m^3	25133.80	53593548.49	6288061
		Crushed stone base	27.94	2040	m^3	2983.52	6085240.25	713973.3
		Granular sub-base	31.75	2318	m^3	1128.31	2615139.88	306830.9
		Asphalt Concrete	29.21	876.3	m^3	25133.80	22024745.96	2584135
		Crushed stone base	27.94	838.2	m^3	2983.52	2500783.67	293413.7
		Granular sub-base	31.75	952.5	m^3	1128.31	1074715.02	126094.9
33	2048	Overlay	9	657	m^3	41596.32	27328785.24	1098314
40	2055	Salvage Value						
							Total PW	31526738

Adama-Awash road section (Composite Pavement structure) Economic analysis

Carriageway Width=7.3m Shoulder Width=1.5m Length of Road= 1km

Carriageway Area=7.3*1000=7300m^2 Shoulder Area=1.5*1000=1500m^2

Discount Rate =10.23% Inflation Rate = 6.5% per year

Analysis year	Calendar Year	Activity	Thickness (cm)	Quantity	Unit	Unit Cost(birr)	Total(birr)	PW
0	2015	Asphalt Concrete	8	584	m^3	5206.16	3040400	3040400
		C-35 JRCP slab	34.85	2543.98	m^3	3588.94	9130188	9130188
		Reinforcement Bar	-	129136.2	kg	37.49	4841020	4841020
		Granular sub-base	15	1095	m^3	233.72	255918.8	255918.8
		Capping Layer	25	1825	m^3	187.73	342600.9	342600.9
		Shoulders AC	8	240	m^3	5206.16	1249480	1249480
		Shoulders JRCP slab	34.85	1045.47	m^3	3588.94	3752132	3752132
		Shoulders Granular sub-base	15	450	m^3	233.72	105172.1	105172.1
		Shoulders Capping Layer	25	750	m^3	187.73	140794.9	140794.9
						Sub Total	22857707	
10	2025	Overlay	3	219	m^3	9772.69	2140218	808090.5
16	2031	Overlay	6	438	m^3	14259.74	6245766	1314585
22	2037	Overlay	8	584	m^3	20806.99	12151283	1425694
28	2043	Overlay	4	292	m^3	30360.36	8865225	579822.6
33	2049	Overlay	7	511	m^3	41596.32	21255722	854244.5
40	2055	Salvage Value						
							Total PW	27840144

Awash-Mille road section (Flexible Pavement structure) Economic analysis

Carriageway Width=7m Shoulder Width=1.5m Length of Road= 1km

Carriageway Area=7*1000=7000m^2 Shoulder Area=1.5*1000=1500m^2

Discount Rate =10.23% Inflation Rate = 6.5% per year

Analysis year	Calendar Year	Activity	Thickness (cm)	Quantity	Unit	Unit Cost(birr)	Total(birr)	PW
0	2015	Asphalt Concrete	17.78	1245	m^3	5206.16	6479592.56	6479593
		Crushed stone base	20.32	1422	m^3	618.00	879042.79	879042.8
		Granular sub-base	25.4	1778	m^3	233.72	415546.76	415546.8
		Shoulders AC	17.78	533.4	m^3	5206.16	2776968.24	2776968
		Shoulders Crushed stone base	20.32	609.6	m^3	618.00	376732.62	376732.6
		Shoulders Granular sub-base	25.4	762	m^3	233.72	178091.47	178091.5
						Sub Total	11105974.45	
11	2026	Overlay	9	630	m^3	10469.22	6595609.40	2259212
18	2033	Overlay	17.78	1245	m^3	16269.03	20248433.49	3507478
		Crushed stone base	10	700		1931.22	1239.00	287.4582
25	2040	Reconstruction						0
		Asphalt Concrete	24.13	1689	m^3	25133.80	42453495.83	4981013
		Crushed stone base	25.4	1778	m^3	2983.52	5304692.62	622392.7
		Granular sub-base	27.94	1956	m^3	1128.31	2206748.18	258914.9
		Asphalt Concrete	24.13	723.9	m^3	25133.80	18194355.36	2134720
		Crushed stone base	25.4	762	m^3	2983.517	2273439.70	266739.7
		Granular sub-base	27.94	838.2	m^3	1128.31	945749.22	110963.5
33	2048	Overlay	9	630	m^3	41841.36	26360054.22	1418901
40	2055	Salvage Value						
							Total PW	26666597

Awash-Mille road section (Composite Pavement structure) Economic analysis

Carriageway Width=7m Shoulder Width=1.5m Length of Road= 1km

Carriageway Area=7*1000=7000m^2 Shoulder Area=1.5*1000=1500m^2

Discount Rate =10.23% Inflation Rate = 6.5% per year

Analysis year	Calendar Year	Activity	Thickness (cm)	Quantity	Unit	Unit Cost(birr)	Total(birr)	PW
0	2015	Asphalt Concrete	8	560	m^3	5206.16	2915452	2915452
		C-35 JRCP slab	28.55	1998.43	m^3	3588.94	7172251	7172251
		Reinforcement Bar	-	123399.7	kg	37.49	4625971	4625971
		Granular sub-base	15	1050	m^3	233.72	245401.6	245401.6
		Capping Layer	25	1750	m^3	187.73	328521.4	328521.4
		Shoulders AC	8	240	m^3	5206.16	1249480	1249480
		Shoulders JRCP slab	28.55	856.47	m^3	3588.94	3073822	3073822
		Shoulders Granular sub-base	15	450	m^3	233.72	105172.1	105172.1
		Shoulders Capping Layer	25	750	m^3	187.73	140794.9	140794.9
						Sub Total	19856866	
10	2025	Overlay	3	210	m^3	9772.69	2052264	774881.3
16	2031	Overlay	5	350	m^3	14259.74	4990909	1050468
22	2037	Overlay	7	490	m^3	20806.99	10195425	1196216
28	2043	Overlay	4	280	m^3	30360.36	8500901	555994.3
33	2049	Overlay	6	420	m^3	41596.32	17470456	702118.8
40	2055	Salvage Value						
							Total PW	24136545