CHAPTER ONE
INTRODUCTORY AND BACKGROUND

1.1 General

The purpose of a pavement is to carry traffic safely, conveniently and economically over its extended life. The pavement must provide smooth riding quality with adequate skid resistance and have adequate thickness to ensure that traffic loads are distributed over an area so that the stresses and strains at all levels in the pavement and sub-grade are within the capabilities of the materials at each level.

There are three major types of pavements, flexible or asphalt pavements, rigid or concrete pavements, and composite pavements.

Flexible pavement provides sufficient thickness for load distribution through a multilayer structure so that the stresses and strains in the sub-grade soil layers are within the required limits.

Rigid pavements are constructed of Portland cement concrete slab PCC placed either directly on the prepared sub-grade or on a single layer of granular or stabilized material.

The composite pavement is composed of both HMA and PCC. The use of PCC as a bottom layer and HMA as a top layer results in an ideal pavement with the most desirable characteristics. However, this type of pavement is very expensive and is rarely used as a new construction.

1.2 Problem Statement and Significance

Selection of the most appropriate pavement type must take into consideration a number of potentially conflicting issues, any of which may limit the range of options that can be considered. The main issues are the cost of investigation, design, construction, and future maintenance of pavement which must be determined in order that resources are not wasted on providing a costly, long life pavement when a less expensive, short term solution is required to allow the most desirable pavement type.

The other issues that might control selection of pavement type are the climate, geomorphology, land use, geometry, construction time, availability and quality of construction materials, level of service and structural capacity and some critical success factors for the road project e.g. timing, funding, practicality, innovation.

In Sudan, flexible pavements are widely used despite some doubts regarding their economics under different conditions. In many cases these pavements experienced poor performance with premature failure especially few years after construction, which needs to be managed and maintained according to its degree of severity.

In the past, lack of research, less construction know how and high concrete price comparing with cheaper asphalt price are the main reasons for absence of concrete pavement in Sudan.

The need for durable, costless and long term good performance pavement is important goal to be adopted as the huge parts of the country need to be connected with road networks.

1.3 Objectives

1.3.1 General Objectives

The general objectives of conducting this study are:

1. Apply the full depth asphalt and AASHTO design methods for flexible pavements instead of using the empirical method of TRL which is widely common used in Sudan. Also the AASHTO and PCA design methods were applied for jointed plane concrete which will be implemented in the near future.

2. Ensure the availability of materials justificatory for construction of concrete pavement in certain states.

1.3.2 Specific Objective

The specific objectives of conducting this study are:

1. Quantify and characterize the loading of the various vehicles that use the case study two roads.

2. Compare between the two flexible pavement design methods, Asphalt institute (AI) and American Association of State Highway Officials (AASHTO) for pavement structure thickness and layers materials to find which the feasible option. The same comparison is done for the two rigid pavement design methods, (AASHTO) and Portland Cement Association (PCA)

3. Carry out the life cycle cost analysis on the case study roads to determine the most feasible pavement alternative.

4. To find if the feasible long term pavement performance can be achieved and good condition pavement can be maintained through using the rigid pavement as a replacement of the practiced flexible pavement.

1.4 Scope of Work

Sudan experienced road network's expansion as advantage of the oil industry. Between 2000 and 2008 the length of roads almost doubled from 3,400 km to over 6,200 km.

The network expansion involved major arterial routes that connect Khartoum with Port Sudan and onward to Egypt. While these north-south links are developed and in relatively good condition, east-west connectivity lags behind as clear from Figure 1.1.

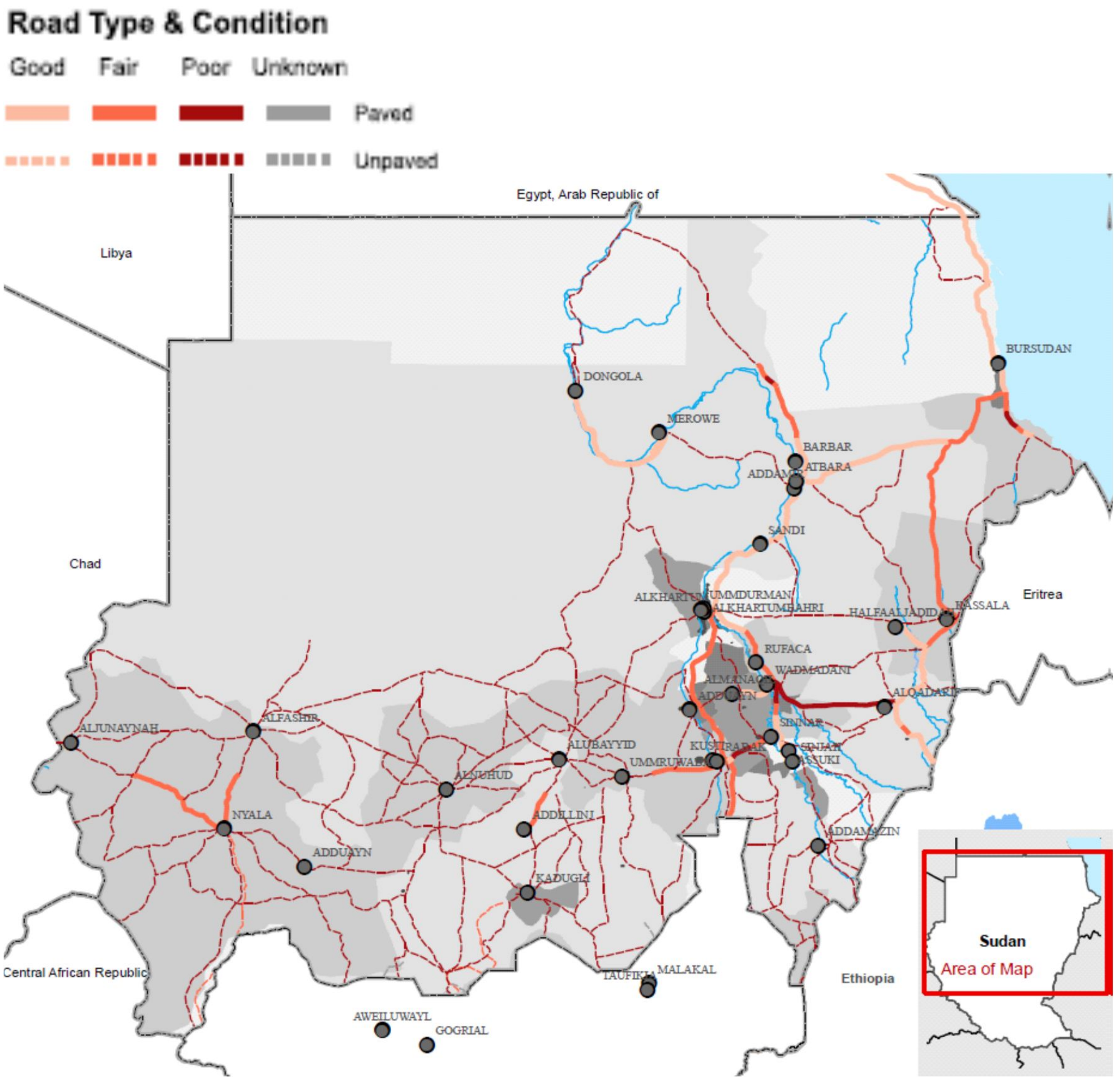

Figure 1.1: Sudan Road Network

Despite this expansion a large share of Sudan is unconnected or lacks paved roads. The existing networks consist of nearly 2,500 miles of single track railroad with a feeder line (supplemented with limited river steamers) of about 1,200 miles of gravel road primarily in greater Khartoum and Port Sudan. In addition some roads in the north-south direction have been built.

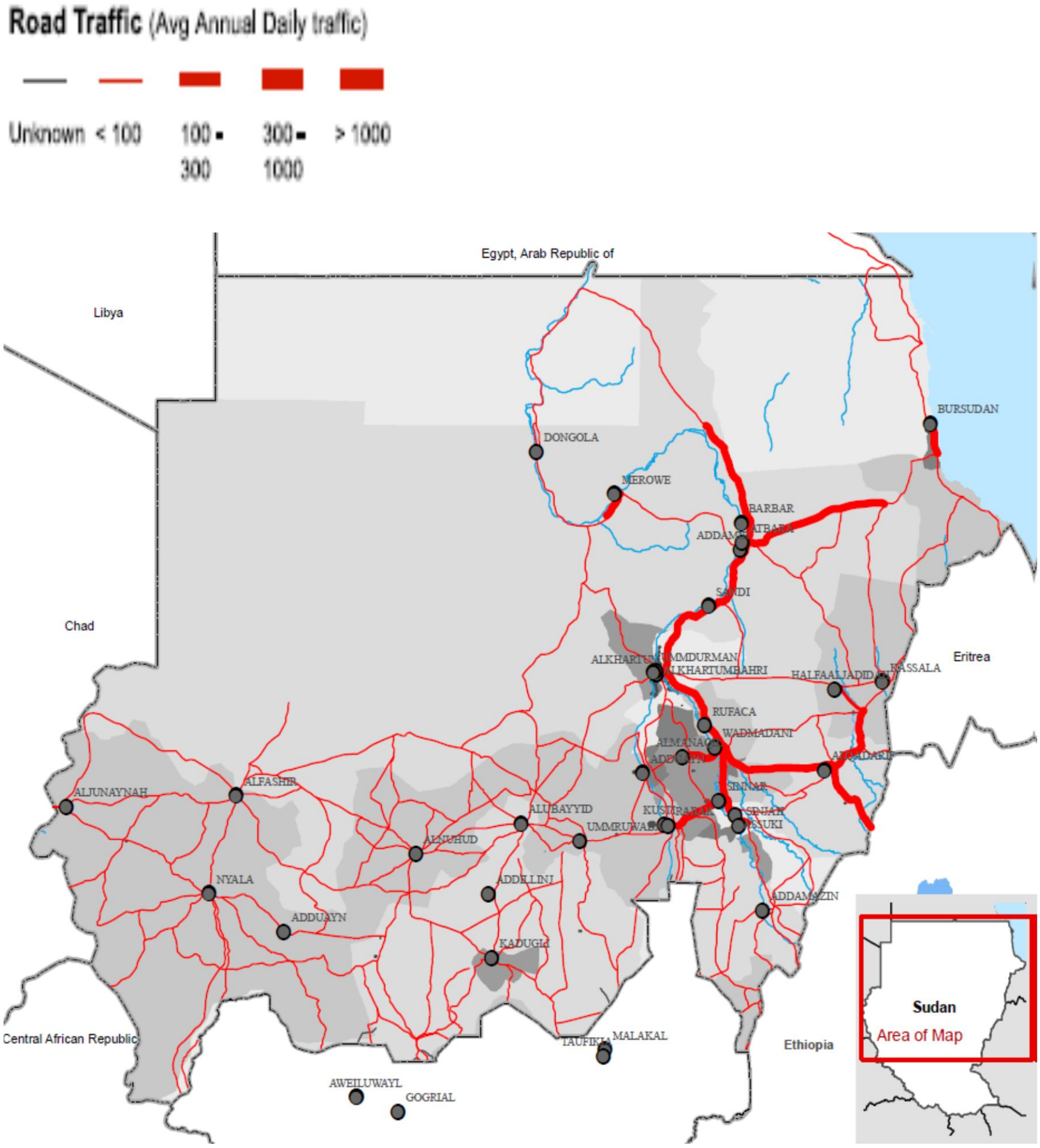

Figure 1.2: Sudan Road Network Traffic Volume

The trading artery in Sudan is the route that connects Kosti to Port Sudan via Khartoum as this road records the greatest traffic volumes in Sudan and boasts overall good-quality road, particularly from Khartoum to Port Sudan as indicated in Figure 1.2.

Another corridor connects Sudan to the Djibouti Corridor offering connectivity to the Port of Djibouti and Addis Ababa. While systematic data on these routes are not available, traffic volumes from Sudan along this corridor are expected to be very low, and the quality of roads range from fair to poor. Connectivity with South Sudan is practically nonexistent and was never a strategic priority.

The regional corridor connecting to South Sudan is in bad condition and records very low traffic volumes. During the rainy season (April/May to October/November), a majority of the roads particularly in South Sudan are impenetrable. These low traffic volumes raise questions about the extent to which roads in Sudan meet traffic thresholds that justify paving.

1.5 Methodology

The study included design of flexible and rigid pavements for the two road projects A and B, for the purpose of comparing the costs.

The main pavement design factor is design traffic in term of cumulative equivalent standard axle load (ESAL). Data were collected for the two road projects including traffic reports. Traffic analysis was conducted according to the AASHTO procedure to determine the ESAL for flexible and rigid pavements during design life.

The other important design factor is the load bearing capacity of sub-grade soil support. Flexible pavement design uses soil sub-grade strength in terms of resilient modulus; M_R. Rigid pavement design applies modulus of sub-grade reaction, k as sub-grade soil strength.

Material reports for the two road project are appended to this thesis. Bearing capacities of two road projects were measured in term of California Bearing Ratio (CBR). The design CBR was determined using Asphalt Institute design percentile value. For flexible pavement design the correlation suggested by AI for fine-grained soils with soaked CBR of 10 or less was applied to obtain the resilient modulus M_R, And the correlation for the soil with CBR more than 10.

AASHTO modification of the modulus of sub-grade reaction k was applied to obtain the design k for rigid pavement design.

The design had been conducted for the flexible pavement through two methods, the Asphalt Institute (AI) and AASHTO methods while the AASHTO and PCA methods were adopted for two types of rigid pavement, jointed plain concrete and continuous reinforced concrete.

The study sheds light on the economical status of the recent global and local oil and cement production and rates, and the near future expected scenario to encourage the feasible selection of pavement type.

Work sheets of the two road projects for flexible and jointed plain concrete pavement were prepared via AASHTO design method. The rates were filled according the 2014 Sudanese pounds for both initial and maintenance costs. Life cost analysis for the two roads pavement types was done to compose the cost comparison via the total costs present worth.

1.6 Contents

This study is divided in six chapters. The first chapter is introductory and background presents the objectives of the research, problem statement and significant, scope of work, methodology and the description of the chapters.

The second chapter is the literature review, flexible pavement, rigid pavement and design parameters. This chapter focuses at pavements definitions, types, structure, design methods, construction and maintenance. It presents all parameters involved in pavement design in details.

The third chapter describes the two case study roads A and B and prepares the flexible and jointed plain concrete pavement structural design by applying AI and AASHTO methods for flexible pavement. The AASHTO and PCA methods were used to design concrete pavement structure.

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The fourth chapter conducts the life circle cost analysis for Road A and Road B flexible and concrete pavement. The construction costs, net present maintenance cost and the residual values were calculated. Comparison was made based on total net present costs of the alternative pavements.

The fifth chapter shows the obtained results and discussion.

The sixth chapter presents the summary, conclusions, final remarks and recommendations for future studies involving the application and implementation of jointed plain concrete.

CHAPTER TWO
LITRETURE REVIEW,
FLEXIBLE PAVEMENT, RIGID PAVEMENT
AND DESIGN PARAMETERS

2.1 Literature Review

Roads are considered having high construction cost comparing with the other infrastructures which can make a great influence in decision making. Also the global increment in materials and products and limitation of projects funding to fulfill the needs for sustainable development makes the planners search to minimize the construction cost and maintain the required quality to involve in many projects.

In Sudan, the entire road network had been constructed with flexible pavement. Only few concrete pavements were constructed manually by private sector, especially in oil production areas internal road networks, due to its higher paving cost compared with asphalt pavements which can be constructed in lower prices.

Sudan moved to invest forward in cement production according to the studies that indicate availability of huge percents of lime stone the main cement's raw material in the country especially at the state of Nile River. About eight factories are working in cement production industry now and many others are proposed and under establishment. This leads to increment in production and reduction in cement prices the main ingredient of rigid pavement.

Ali, G (2013) and Qasim, F (2014) made comparative studies for rigid versus pavements for Sudan highways under different soil strength and traffic conditions. It was found that replacing of jointed plain concrete pavement instead of practiced flexible pavements is more feasible and achieves long term good pavement performance.

2.2 Flexible Pavement

The structural capacity of flexible pavements is attained by combined action of the different layers of the pavement. The load is directly applied on the wearing course and it gets dispersed with depth in the base, sub-base and sub-grade layers and then ultimately to the ground. The sub-grade layer is responsible for transferring the load from top layers to the ground. Flexible pavements are designed in such a way that the load transmitted to the sub-grade does not exceed its bearing capacity.

A. Flexible Pavement Types

1. Conventional Flexible Pavements

They are layered systems with better materials on top where the intensity of stress is high and inferior materials at the bottom where the intensity is low Figure 2.1 shows the cross section of a conventional flexible pavement. Starting from the top, the pavement consists of seal coat, surface course, tack coat, binder course, prime coat, base course, sub-base course, compacted sub-grade, and natural sub grade. The use of the various courses is based on either necessity or economy, and some of the courses may be omitted.

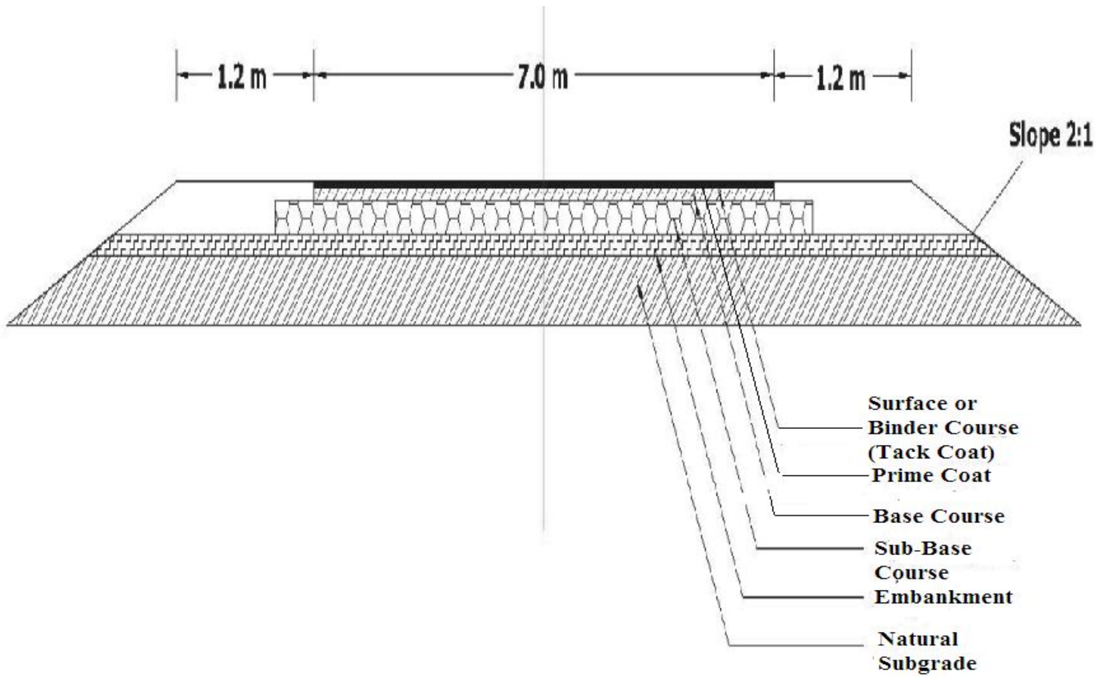

Figure 2.1: Typical Conventional Flexible Pavement Cross Section

2. Full-Depth Asphalt Pavements

Full-depth asphalt pavements are constructed by placing one or more layers of HMA directly on the sub-grade or improved sub-grade.
This type of construction is quite popular in areas where local materials are not available. It is more convenient to purchase only one material, i.e. HMA, rather than several materials from different sources, thus minimizing the administration and equipment costs. Figure 2.2 shows the typical cross section for a full-depth asphalt pavement.

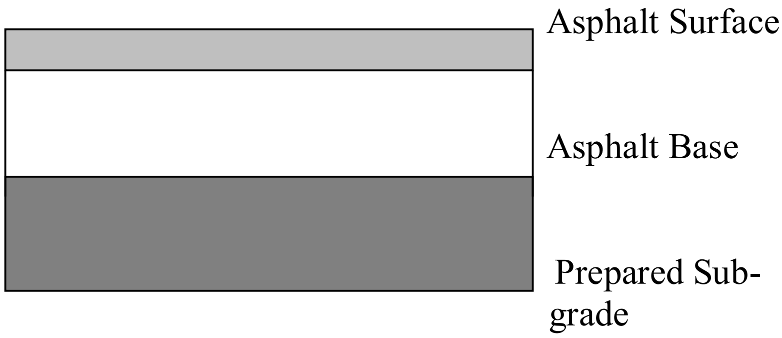

Figure 2.2: Typical Full Depth Pavement Cross Section

B. Flexible Pavement Structure

1. Surface Course

The surface course usually consists of one or two hot mix asphalt layers, a wearing course and a binder course. To provide a durable, watertight, smooth-riding, and skid-resistant traveled surface, the wearing course is often constructed of dense-graded hot mix asphalt with polish-resistant aggregate. The binder course generally has larger aggregates and less asphalt. The tack coat is usually applied between the wearing and binder course to be bonded together.

2. Base Course and Sub-base Course

The base course is the layer of material immediately beneath the surface or binder course. It can be composed of crushed stone, crushed slag, or other untreated or stabilized materials. The sub-base course is the layer of material beneath the base course. The reason that two different granular materials are used is for economy. Instead of using more expensive base course material for the entire layer, local and cheaper materials can be used as a sub-base course on top of the sub-grade.

C. Flexible Pavement Design

The thickness design of flexible pavements is a complex engineering problem involving a large number of variables. The main design variables control the design method is the design traffic which is expressed in term of equivalent single axle load (ESAL) and sub-grade strength measured by the resilient modulus (M_R).

This section presents the methods of the Asphalt Institute and AASHTO for flexible highway pavements design

1. Asphalt Institute Design Method

The Asphalt Institute promotes the use of full-depth pavements in which asphalt mixtures are employed for all courses above the sub-grade. Potential benefits of full-depth pavements derive from the higher load bearing and spreading capability and moisture resistance of asphalt mixtures as compared to granular aggregates. Thickness design charts are provided for full-depth pavements, pavements with emulsified asphalt base, and untreated aggregate base. These charts are developed based on two design criteria:

1) Maximum tensile strains induced at the underside of the lowest asphalt-bound layer.
2) Maximum vertical strains induced at the top of the sub-grade layer.

The design curves have incorporated the effects of seasonal variations of temperature and moisture on the sub-grade and granular base materials.

1.1 Full Depth Asphalt Concrete

Pavement of this type uses asphalt mixtures for all the courses above the sup-grade see the design chart of this type presented in Figure A-1 of Appendix A.

1.2 Asphalt Concrete Surface and Emulsified Asphalt Base

Theses pavements have asphalt concrete as surface course and emulsified asphalt as base courses. Depending on aggregate types, three types of mixes are specified:

1. Type I: mixes with processed dense graded aggregates, which should be mixed in a plant and have properties similar to HMA, the design chart is presented in Figure A-2 of Appendix A.

2. Type II: mixes with semi processed, crusher run, pit run, or bank run aggregates, the design chart of this type is presented in Figure A-3 of Appendix A.

3. Type III: mixes with sands or silty sands, the design chart is available in Figure A-4 of Appendix A.

Minimum thickness of HMA over emulsified asphalt bases are presented in Table A-1of Appendix A

1.3 Asphalt Concrete Surface and Untreated Aggregate Base

These pavements consist of layer of asphalt concrete over untreated aggregate base and sub-base of 6, 8, 10, 12, and 18 in thickness respectively are covered in the design charts in Figures A-5 to A-9 in Appendix A.

2. AASHTO Design Method

The AASHTO design method is based on the empirical methodology obtained from the AASHO Road Test (Highway research board 1962). It defines pavement performance in terms of the present serviceability index (PSI).

The original equation is applicable only to the specific environmental and soil conditions at the test site. To make it applicable to other areas, the equation was modified by introducing an effective roadbed soil resilient modulus M_R and two drainage coefficients m_2 and m_3 for granular base and sub-base, respectively.

The nomograph in Figure A-10 of Appendix A is solving the design equation to obtain the pavement layers thickness. The input design variables had been discussed below.

2.1 Design Factors

Several general variables related to AASHTO design are presented as follows:

1. Traffic W_{18}:

The design procedure is considering the cumulative expected 18-kip (80-kN equivalent single-axle load (ESAL) as a major variable controlling the design.

2. Reliability R%:

Reliability indicates the probability that the pavement designed will not reach the terminal serviceability level before the end of the design period. The level of reliability to be used for design should increase as the volume of traffic, difficulty of diverting traffic, and public expectation of availability increase.

Table A-2 of Appendix A presents recommended levels of reliability for various functional classifications.

3. Effective Roadbed Soil & Pavement structure Resilient Modulus M_R:

The total pavement thickness requirement is a function of the resilient modulus M_R of sub-grade soil.

To account for seasonal variations of sub-grade soil resilient modulus, AASHTO defines an effective roadbed soil M_R to represent the combined effect of all the seasonal modulus values. This effective M_R is a weighted value that would give the correct equivalent annual pavement damage for design purpose.

The correlation was made to obtain M_R for granular materials and fine grained soils composing the base and sub-base layers as suggested by AI with soaked CBR 10% or less and Asphalt Institute for soaked CBR more than 10% values.

For bituminous pavement layers, M_R may be tested by the repeated load indirect tensile test described in ASTM Test D-4123, Figure A-10 of Appendix A shows a chart developed by Van Til et al. [1972] relating M_R of hot-mix asphalt mixtures to other properties (see Figure A-11 in Appendix A).

4. Serviceability PSI:

The serviceability index PSI, is a function of pavement type and construction quality, which varies from 0 to 5. The PSI of newly constructed flexible pavements and rigid pavements were found to be about 4.2 and 4.5, respectively. For pavements of major highways, the end of service life is considered to be reached when PSI = 2.5. A terminal value of PSI = 2.0 may be used for secondary roads. Serviceability loss ΔPSI, given by the difference of the initial and terminal serviceability, is required as an input Parameter. Typical initial PSI values from the AASHO Road Test were 4.2 for flexible pavements and 4.5 for rigid pavement.

2.2 Design Equation

$$\log w_{t18} = Z_R S_0 \log(SN+1) - 0.2 + \{[\log[(4.2-pt)] \div (4.2-1.5)] \div [0.4 + 1094/(SN+1)^{\wedge}5.19]\} + 2.32\log Mr - 8.07 \quad \textbf{(2.1)}$$

Where:

Log W_{t18} = Axle load application at end of time t.

Z_R = Normal deviate for a given reliability R. Can be determining from, Table A-2 Appendix A.

S_0 = Standard deviation.

SN = Structural number of pavement, which was computed as

$$SN = a_1D_1 + a_2m_2D_2 + a_3m_3D_3 \quad \textbf{(2.2)}$$

Which: a_1, a_2, and a_3 are layer coefficients for the surface, base, and sub-base, respectively, and m_2, m_3 are drainage coefficients for base and sub-base, and D_1, D_2, and D_3 are the thicknesses of the surface, base, and sub-base, respectively.

a. Structure Number

Structural number is a function of layer thicknesses, layer coefficients, and drainage coefficients.

1. Layer Coefficient

The layer coefficient a, is a measure of the relative ability of a unit thickness of a given material to function as a structural component of the pavement.
Layer coefficients can be determined from test roads or satellite sections, or from correlations with material properties. Resilient modulus is the fundamental material property that the layers coefficient can be correlated from.

1.1 Asphalt Concrete Surface Course Layer Coefficient (a_1)

Figure A-12 of Appendix A is a chart relating the layer coefficient of a dense-graded HMA, a_1 to its resilient modulus at 70°F (21°C).

1.2 Untreated and Stabilized Base Courses Layer Coefficient (a_2)

Many charts had been designed to estimate a_2 for an untreated and stabilized base course presented in AASHTO guide for pavement structure design. The following equation can also be used to estimate a_2 from its resilient modulus E_{BS}:

$$a_2 = 0.249 (\log E_{BS}) — 0.977 \quad (2.3)$$

1.3 Granular Sub-base Course Layer Coefficient (a_3)

There is a chart to estimate a3 for an untreated sub-base presented in AASHTO guide for pavement structure design.
The relationship between a_3 and E_{SB} can be expressed as:

$$a_3 = 0.227(\log E_{BS}) — 0.839 \quad (2.4)$$

2. Drainage Coefficients

Depending on the quality of drainage and the availability of moisture, drainage coefficients m_2 and m_3 should be applied to granular bases and sub-bases to modify the layer coefficients. Table A-3 of Appendix A shows the recommended drainage coefficients for untreated base and sub-base materials in flexible pavements.

2.3 Determination of Layer Thicknesses.

Using the input parameters described in the preceding sections, the total pavement thickness requirement is obtained in terms of the structural number **SN**.

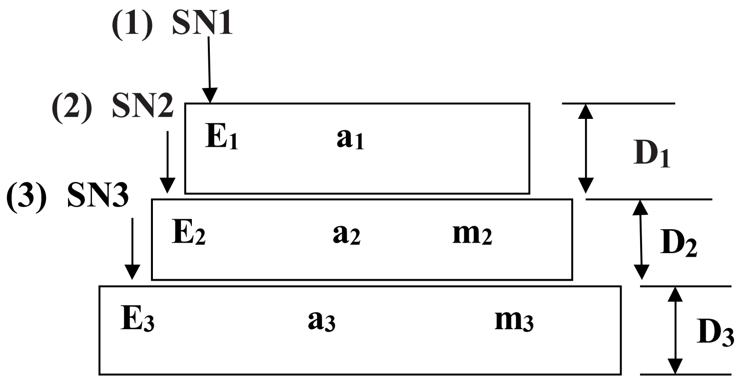

Figure 2.3: Determination of layer thicknesses

2.4 Minimum Thickness

It is generally impractical and uneconomical to use layers of material that are less than some minimum thickness. Furthermore, traffic considerations may dictate the use of a certain minimum thickness for stability .Table A-4 of Appendix A, shows the minimum thicknesses of asphalt surface and aggregate base. Because such minimums depend somewhat on local practices and conditions, they may be changed if needed.

D. Flexible Pavement Construction

1. Sub-grade Preparation

In order to provide maximum structural support, a sub-grade soil must be compacted to an adequate density, if it is not, the sub grade will continue to compress, deform or erode after construction, causing pavement cracks and deformation. Generally, adequate density is specified as a relative density for the top 150 mm (6 inches) of sub-grade of not less than 95 percent of maximum density determined in the laboratory. In fill areas, sub-grade below the top 150 mm (6 inches) is often considered adequate if it is compacted to 90 percent relative density. In order to achieve these densities the sub-grade must be at or near its optimum moisture content. Usually compaction of in situ or fill sub-grade will result in adequate structural support. If the structural support offered by the in situ compacted sub-grade is or is estimated to be inadequate, there are two options (any one or combination of the two can be used):

a) Stabilization. The binding characteristics of these materials generally increase sub-grade load-bearing capacity. Typically, lime is used with highly plastic soils (plasticity index greater than 10), cement is used with less plastic soils (plasticity index less than 10) and emulsified asphalt can be used with sandy soils.

b) Over excavation. The general principle is to replace poor load-bearing in situ sub-grade with better load-bearing fill. Typically, 0.3 - 0.6 m (1 - 2 ft.) of poor soil may be excavated and replaced with better load-bearing fill such as gravel borrows.

2. Construction of Granular Sub-base and Base Course

Construction of both sub-base and base layers starts with hauling and placing the well grading granular materials which follow the required specifications and contains amount of moisture required for compaction. Granular material of sub-base shall be compacted three days prior to replacement of base layer.

Maximum layer thickness to be compacted is 150 mm (6 in), if the thickness is more than the required the layer shall be placed in two layers to achieve adequate compaction.

Finally after the layer has adjusted to the design level, guardian and cross slopes, it shall be moisturized and a steel drum roller makes number of passes for final layer finishing.

3. Construction of Asphalt Concrete Surface Course

Liquid asphalt prime coat shall then be applied to the aggregate base course at the rate of 0.25 gallons per square yard unless otherwise directed. After the liquid asphalt has penetrated the base course, any excess standing on the surface shall be absorbed to the satisfaction of the Engineer with a suitable coating of clean sand. Tack coat shall be applied to all vertical surfaces of existing pavement, curbs, gutters, catch basins, manhole frames, and construction joints in the surfacing to the horizontal surface of all existing pavements to be resurfaced and other surfaces designated. Asphaltic paint binder shall be provided in sufficient quantity to produce a thin, uniform black, glossy coat of asphalt. Prior to placing asphalt over existing pavement, sweep the pavement clean of loose dirt to the satisfaction of the Engineer.

Asphalt courses shall be placed by means of an approved self-propelled asphalt paving machine. When placing asphalt over existing pavement, repair large cracks, spalls, and chuck-holes, and clean the pavement surface.
Asphalt concrete shall be rolled such that compaction after rolling shall be 95% of the density obtained with the California Test 304

E. Flexible Pavement Maintenance

1. Preventive Maintenance

Preventive maintenance activities can include conventional treatments such as crack sealing, chip sealing, fog sealing, rut filling, and thin overlays. They can also include emerging technologies such as ultra-thin wearing courses, very thin overlays, and micro surfacing applications. Aside from crack treatments, all of these treatments leave the pavement with a new wearing surface.

Often, preventive maintenance methods are designed to repair damage caused by the environment. Periodic renewal of the pavement surface can provide several benefits, including sealing the pavement surface (which prevents water from penetrating into the pavement structure), and controlling the effects of oxidation, raveling, and surface cracking. Environmental conditions remain fairly consistent over time, so the maximum time between preventive maintenance treatments should be based on time, rather than the amount of traffic on a roadway section.

2. Corrective Maintenance

Corrective maintenance is much more reactive than preventive maintenance, and is performed to correct a specific pavement or area of distress.

Delays in maintenance increase pavement defects and their severity so that, when corrected, the cost is much greater. Consequently, the life cycle costs of the pavement will be considerably increased when corrective maintenance is performed.

Corrective maintenance activities include structural overlays, mill and overlays, pothole repair, patching, and crack repair.

3. Emergency Maintenance

This maintenance activity may be performed during an emergency situation, such as when a blowout or severe pothole must be repaired immediately, generally for safety reasons, or to allow for traffic to use the roadway.

Emergency maintenance also describes those treatments that hold the surface together until a more extensive rehabilitation or reconstruction treatment can be accomplished.

When emergency maintenance is needed, some of the typical considerations for choosing a treatment method are no longer important. Cost may be the least important consideration, after safety and time of application are considered. Materials that may not be acceptable when used in preventive or corrective maintenance activities, for cost or long-term performance reasons, may be highly acceptable when used in an emergency situation.

2.3 Rigid Pavement

Rigid pavements are named so because of the high flexural rigidity of the concrete slab. The concrete slab is capable of distributing the traffic load into a large area with small depth which minimizes the need for a number of layers to help reduce the stress. Generally, depending on the strength of soil, a layer is provided as base/sub-base immediately above the sub-grade. Figure 3.1 shows a typical cross section of a rigid pavement.

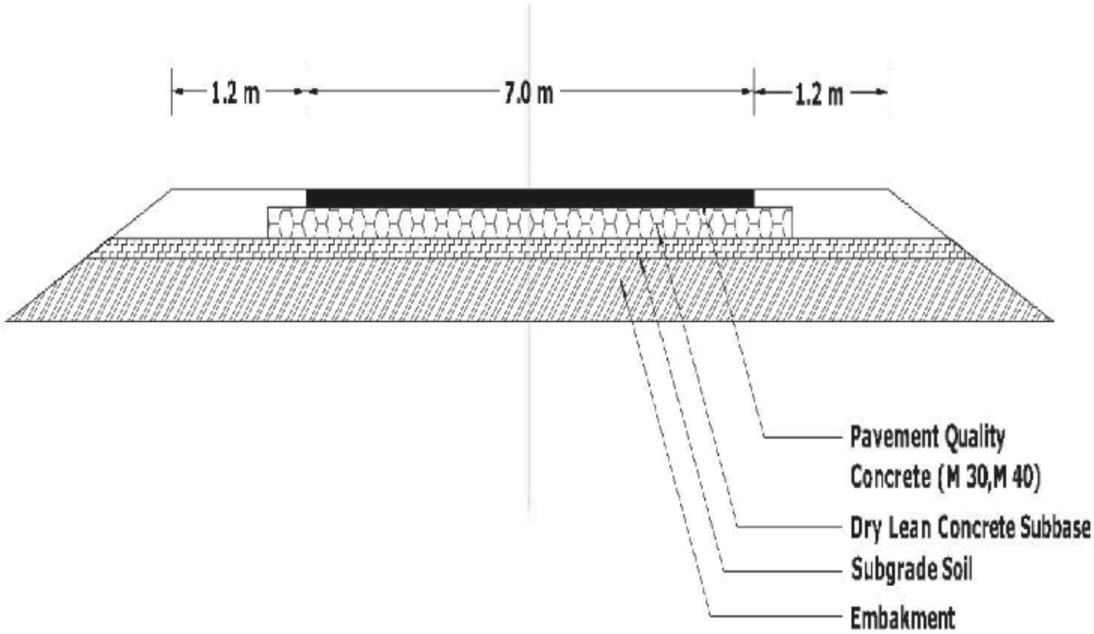

Figure 2.4: Typical Cross Section of Rigid Pavement

A. Rigid Pavement Types

1. Jointed Plain Concrete Pavements (JPCP)

All plain concrete pavements should be constructed with closely spaced contraction joints. Dowels or aggregate interlocks may be used for load transfer across the joints.

Depending on the type of aggregate, climate, and prior experience, joint spacing between 15 and 30 ft (4.6 and 9.1 m) has been used. However, as the joint spacing increases, the aggregate interlock decreases, and there is also an increased risk of cracking.

Based on the results of a performance survey, Nussbaum and Lokken (1978) recommended maximum joint spacing of 20 ft (6 .1 m) for doweled joints and 15 ft (4 .6 m) for un doweled joints.

2. Jointed Reinforced Concrete Pavements (JRCP)

Steel reinforcements in the form of wire mesh or deformed bars do not increase the structural capacity of pavements but allow the use of longer joint spacing. Joint spacing vary from 30 to 100 ft (9 .1 to 30 m). Because of the longer panel length, dowels are required for load transfer across the joints. The amount of distributed steel in JRCP increases with the increase in joint spacing and is designed to hold the slab together after cracking.

3. Continuously Reinforced Concrete Pavements (CRCP)

It was the elimination of joints that prompted the first experimental use of CRCP and the advantages of the joint-free design make it widely in use.
It was originally reasoned that joints were the weak spots in rigid pavements and that the elimination of joints would decrease the thickness of pavement required. As a result, the thickness of CRCP has been empirically reduced by 1 to 2 in. (25 to 50 mm) or arbitrarily taken as 70 to 80% of the conventional pavement. The formation of transverse cracks at relatively close intervals is a distinctive characteristic of CRCP. These cracks are held tightly by the reinforcements and should be of no concern as long as they are uniformly spaced. The distress that occurs most frequently in CRCP is punch out at the pavement edge. This type of distress takes place between two parallel random transverse cracks or at the intersection of Y cracks. If failure occurs at the pavement edge instead of at the joint, there is no reason for a thinner CRCP to be used

4. Pre Stressed Concrete Pavements

Concrete is weak in tension but strong in compression. The thickness of concrete pavement required is governed by its modulus of rupture, which varies with the tensile strength of the concrete. The pre application of a Compressive stress to the concrete greatly reduces the tensile stress caused by the traffic loads and thus decreases the thickness of concrete required. The pre stressed concrete pavements have less probability of cracking and fewer transverse joints and therefore result in less maintenance and longer pavement life.

Pre stressed concrete has been used more frequently for airport pavements than for highway pavements because the saving in thickness for airport pavements is much greater than that for highway pavements. The thickness of pre stressed highway pavements has generally been selected as the minimum necessary to provide sufficient cover for the pre stressing steel.

B. Jointed Plain Concrete Pavement Components

A typical JPCP is constructed with the following components as shown in Figure 2.3:

- Concrete slabs with a determined thickness.
- Tie bars.
- Dowel bars.
- Joints (both transverse and longitudinal).
- Base layer.
- Sub-base layer (if required).
- Sub-grade.

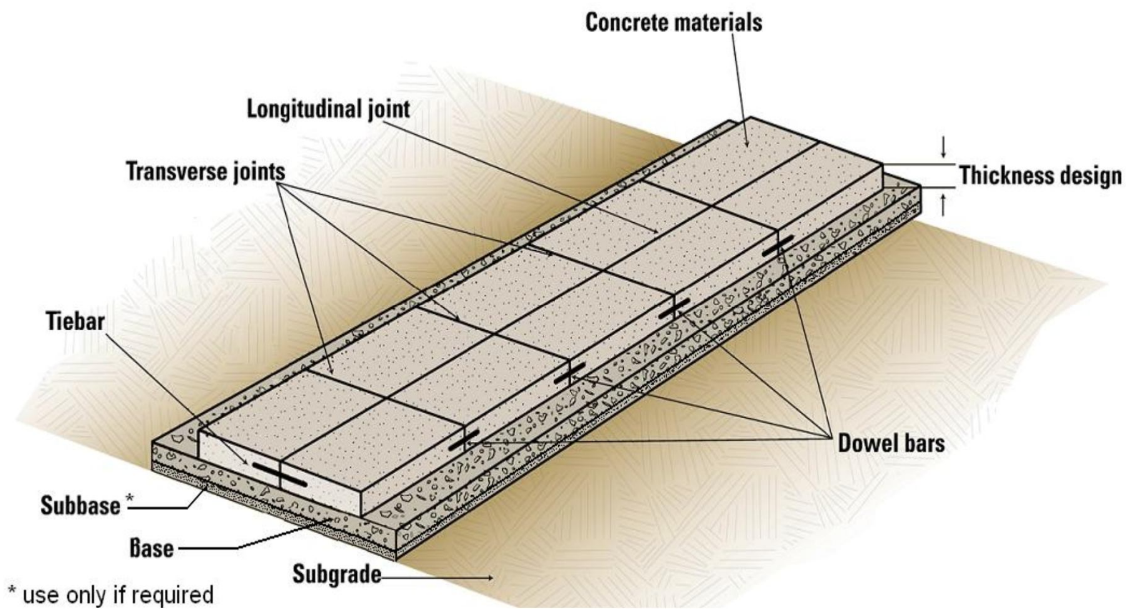

Figure 2.5: Jointed Plain Concrete Pavement Components

1. Joint Opening, (Slab length).

The spacing of joints in plain concrete pavements depends more on the shrinkage characteristics of the concrete rather than on the stress in the concrete.
The opening of a joint can be computed approximately by (Darter and Barenberg, 1977).

$$\Delta L = CL (\alpha_t \Delta T + \varepsilon) \qquad (2.5)$$

Where:

ΔL = is the joint opening caused by temperature change and drying shrinkage of concrete.

C = is the adjustment factor due to slab-sub base friction, 0.65 for stabilized base and 0 .8 for granular sub base.

L = is the joint spacing or slab length

α_t = is the coefficient of thermal expansion of concrete, generally from 5 to 6 x 10-6 /°F (9 to 10 .8 x 10^{-6}/°C).

ΔT = is the temperature range, which is the temperature at placement minus the lowest mean monthly temperature.

ε = is the drying shrinkage coefficient of concrete, approximately 0 .5 to 2 .5 x 10^{-4}.

2. Tie Bars

Tie bars are placed along the longitudinal joint to tie the two slabs together so that the joint will be tightly closed and the load transfer across the joint can be ensured.

The amount of steel required for tie bars can be determined in the same way as the longitudinal or transverse reinforcements by

$$A_S = \gamma_c h L' f_a/f_s \qquad (2.6)$$

Where:

A_S = area of steel per unit length of slab.

γ_c = is the unit weight of the concrete.

h = is the slab thickness.

L' = is the distance from the longitudinal joint to the free edge where no tie bars exist.

For two or three lane highways, L' is the lane width. If tie bars are used in all three longitudinal joints of a four-lane highway, L' is equal to the lane width for the two outer joints and twice the lane width for the inner joint.

F_a = is the average coefficient of friction between slab and sub-grade, usually taken as 1.5

F_s = is the allowable stress in steel.

The length of the tie bar should be based on the full strength of the bar, namely,

$$t = \frac{1}{2}\left(\frac{fs\, d}{\mu}\right) \quad \textbf{(2.7)}$$

t = is the length of bar.

d = is the diameter of bar.

,μ = is the allowable bond stress.

It should be noted that many agencies use a standard tie bar design to simplify the construction. Tie bars 0 .5 in. (13 mm) in diameter by 36 in. (914 mm) long spaced at intervals of 30 to 40 in. (762 to 1016 mm) are most commonly used.

3. Dowel bars

The size of dowels to be used depends on the thickness of slab. Table B-1 of Appendix B shows the size and length of dowels for different slab thicknesses as recommended by PCA (1975). It can be seen that the diameter of dowels is equal to one-eighth of the slab thickness.

4. Joints

Joints should be provided in concrete pavements so premature cracks due to temperature or moisture changes will not occur. There are four types of joints in common use, contraction, expansion, construction, and longitudinal joints.

4.1 Contraction Joints

Contraction joints are transverse joints used to relieve tensile stresses. The joint spacing in feet for plain concrete pavements should not greatly exceed twice the slab thickness in inches. Also, as a general guideline, the ratio of slab width to length should not exceed 1 .25 (AASHTO, 1986).

4.2 Expansion Joints

Expansion joints are transverse joints for the relief of compressive stress. Because expansion joints are difficult to maintain and susceptible to pumping, they are no longer in use today except at the connection between pavement and structure.

Experience has shown that the blowups of concrete pavements are related to a certain source and type of coarse aggregates. If proper precaution is exercised in selecting the aggregates, distress due to blowups can be minimized. The plastic flow of concrete can gradually relieve compressive stress, if any, so it is not necessary to install an expansion joint except at bridge ends.

4.3 Construction Joints

A construction joint is either a transverse joint that joins together two consecutive slabs constructed at two different times, or a longitudinal joint that joins two lanes that are paved in two separate passes. For none dowelled JPCPs (when permitted), tie bars are usually used to connect the two adjoining slabs together so as to act as one slab. It is important to have an adequate slab section to tie into as shown on the plans. Construction joint for doweled pavement shall coincide with the new joint spacing.

4.4 Longitudinal Joints

Longitudinal joints are necessary to control cracking in the longitudinal direction where two or more lane widths are placed at one time. They are constructed at lane lines, typically in multiples of 12 feet. Tie bars are placed at these joints to hold two abutting rigid pavement faces in contact.

C. Rigid Pavement Thickness Design Methods

Structural design of rigid pavements includes thickness and reinforcement designs. Two major forms of thickness design methods are being used in this study for concrete pavements.

The first form is an approach that relies on empirical relationships derived from performance of full-scale test pavements and in-service Pavements performed by AASHTO [1993] design method. The second form develops relationships in terms of the properties of pavement materials as well as load-induced and thermal stresses and calibrates these relationships with pavement performance data. The PCA [1984] method of design adopts this approach.

1. AASHTO Method

The AASHTO design procedure for rigid pavements [AASHTO, 1993] is similar to the design procedure for flexible pavements. Some design variables such as reliability information and serviceability loss are the same in both AASHTO designs. The other variables associated to rigid pavements and required for design purpose will be discussed farther in this chapter.

1.1 Design Equation and Factors

The design equation for rigid pavement is:

$$\log w_{t18} = Z_R S_0 + 7.35\log(D+1) - 0.06 + \left\{\begin{bmatrix} \log[\Delta PSI/(4.5-1.5)] \div 1 + \\ 1.624\times 10^{7}/(D+1)^{8.46} \end{bmatrix}\right\} + (4.22 - 0.32\,Pt)\log\left\{\frac{S_C\ C_d\,(D^{0.75}-1.132)}{215.36} \times J\left[D^{0.75} - \frac{18.42}{\left(\frac{E_C}{K}\right)^{0.23}}\right]\right\} \qquad (2.8)$$

Figure B-1of Appendix B is a nomograph for solving the design equation

The design factors associated to rigid pavement had been presented as follows:

1.1.1 Modulus of Sub-grade Reaction k

The property of roadbed soil to be used for rigid pavement design is the modulus of sub-grade reaction k, rather than the resilient modulus M_R. It is therefore necessary to convert M_R to *k*. As with M_R, the values of k also vary with the season of the year, and the relative damage caused by the change of *k* needs to be evaluated.

The composite modulus of sub-grade reaction (k_∞) and the modified modulus of sub-grade reaction due to rigid foundation near the surface (k) had been determined for the two road projects A & B, results are presented within this study.

1.1.2. Elastic modulus of concrete E_c

The elastic modulus of concrete E_c is determined by the procedure specified in ASTM C469. It could also be estimated using the following correlation with the concrete compressive strength f_c recommended by ACI [1977]:

$$\mathbf{E_c = 57{,}000(f_c)^{0.5}} \qquad \textbf{(2.9)}$$

1.1.3. Concrete Modulus of Rupture S_c

The modulus of rupture required by the design procedure is the mean value determined after 28 days by using third-point loading, as specified in AASHTO T97 or ASTM C78.

The values 650 and 600 psi (4.5 and 4.1 MPa) for good concrete with normal aggregates and are recommended for general design use; the value 550 psi (3.8 MPa) is for a special case where high quality aggregates are not available.

1.1.4. Load Transfer Coefficient J

The load transfer coefficient J is a factor used in rigid pavement design to account for the ability of a concrete pavement structure to transfer a load across joints and cracks.

The use of load transfer devices and tied concrete shoulders increases the amount of load transfer and decreases the load-transfer coefficient.

Table B-2 of Appendix B, shows the recommended load transfer coefficients for various pavement types and design conditions. The AASHO Road Test conditions represent a J value of 3.2, because all joints were doweled and there were no tied concrete shoulders.

1.1.5. Drainage Coefficient C_d

The drainage coefficient C_d has the same effect as the load transfer coefficient J. Table B-3 of Appendix B, provides the recommended C_d values based on the quality of drainage and the percentage of time during which the pavement structure would normally be exposed to moisture levels approaching saturation.

2. PCA Method

The thickness design procedure published by PCA was developed by relating theoretically computed values of stress, deflection, and pressure to pavement performance criteria derived from data of:

(1) Major road test programs
(2) Model and full-scale tests
(3) Performance of normally constructed pavements subject to normal mixed traffic.

Traffic-loading data in terms of axle load distribution are obtained in the usual way. Each axle load is further multiplied by a load safety factor (LSF) according to the following
Recommendations:

1. LSF = 1.2 for interstate highways and other multilane projects with uninterrupted traffic flow and high volumes of truck traffic
2. LSF = 1.1 for highways and arterial streets with moderate volumes of truck traffic
3. LSF = 1.0 for roads, residential streets, and other streets with small volumes of truck traffic.

The design procedure consists of a fatigue analysis and an erosion analysis, which are considered separately using different sets of tables and design charts presented in Appendix B. The final thickness selected must satisfy both analyses.

The design method based on the following two criteria:

2.1 Fatigue Design

Fatigue design is performed with the aim to control fatigue cracking. The slab thickness based on fatigue design is the same for JRCP, for JPCP with doweled and un doweled joints, and for CRCP. This is because the most critical loading position is near mid slab and the effect of joints is negligible.

The steps in the design procedure which adopted for our two study cases are:

1. Multiply the load of each design axle load group by the appropriate LSF.
2. Assume a trial slab thickness.
3. Obtain from Table B-4, Appendix B the equivalent stress for the input slab thickness and *k*, and calculate the stress ratio factor as

Stress ratio factor = Equivalent stress /Concrete flexural strength

(2.10)

4. For each axle load *i*, obtain from Figure B-2 the allowable load repetitions N_i.
5. Make the summation of damage percents D; If D exceeds 1, select a greater trial thickness and repeat steps 3 through 5. The trial thickness is adequate if D is less than or equal to 1.

2.2 Erosion Design

PCA requires erosion analysis in pavement thickness design to control foundation and shoulder erosion, pumping, and faulting. The most critical deflection occurs at the corner. The design steps shall be applied are:

1. Multiply the load of each design axle load group by the LSF.
2. Assume a trial slab thickness.
3. Obtain from Table B-4 and Table B-5 Appendix B, the erosion factor for the input slab thickness and k.
4. For each axle load *i*, obtain from Figure B-3, Appendix B, the allowable load repetitions Ni.

5. Make the summation of the damage percents D. If D exceeds 1, select a greater trial thickness and repeat steps 3 through 5. The trial thickness is adequate if D is less than or equal to 1.

D. Jointed Plain Concrete Pavement Construction

To prepare for paving, the sub-grade, the native soil on which the pavement is built, must be graded and compacted.

Preparation of the sub-grade is often followed by the placing of sub-base.

The essential function of the sub-base is to prevent displacement of soil from underneath the pavement. Once the sub-base has hardened sufficiently to resist marring or distortion by construction traffic, dowels tie bars are placed and properly aligned in preparation for paving.

There are two methods for paving with concrete, slip form and fixed form. In slip form paving, a machine ride on treads over the area to be paved. Fresh concrete is deposited in front of paving machine which then spreads, shapes, consolidates, screeds, and float finishes concrete in one continuous operation.

In fixed form paving, stationary metal forms are set and aligned on a solid foundation and staked rigidly. Final preparation of sub-grade and sub-base is completing after the forms are set. Forms are cleaned and oiled first to ensure that they release from concrete after the concrete hardens. Once the concrete is deposited near its final position on the sub-grade, spreading is completed by a mechanical spreader riding on top of the preset forms and the concrete. The spreading machine is followed by one or more machines that shape, consolidate, and float finish the concrete. After the concrete has reached a required strength, the forms are removed and curing of edges begins immediately.

Joints are created to control cracking and to provide relief for concrete expansion caused by temperature and moisture change, joints are normally created by sawing.

Once joints have been inserted, the surface must be textured. To obtain the desired amount of skid resistance, texturing should be done just after water is disappeared and just before the concrete become non plastic.

Texturing is done using burlap drag, artificial turf drag, wire brooming, grooving the plastic concrete with roller or comp equipped with steel tines, or combination of these methods.

Curing begins immediately after finishing operations as soon as the surface will not be marred by the curing medium. Common curing methods include using white pigmented liquid membrane curing compounds. Occasionally, curing is accomplished by water proof paper or plastic covers such as polyethylene sheets, or wet cotton mats or burlap.

As the concrete pavement hardens, the joints will open providing room for concrete to expand in hot weather and in moist conditions. The joints then are cleaned and sealed to exclude the foreign materials that would be damaging to the concrete when it extends.

The pavement is opened to traffic after the specific curing period and when the tests indicate that the concrete has reached the required strength.

E. Jointed Plane Concrete Maintenance

Pavement maintenance can be categorized under two main groups, preventive maintenance and rehabilitation.

1. Preventive Maintenance

The objective of preventive maintenance is to keep the pavement condition above the level that would require corrective, maintenance or other strategies.

1.1 Joints and Cracks Sealing

Resealing of joints and cracks shall be done to prevent intrusion of water and incompressible materials. The water infiltration may cause pumping, faulting, joint spalling or deterioration, voids under slab and corrosion of dowels and tie bars. Joints resealing can be done by removing the old sealant either manually or by sawing, plowing or cutting. After removing the old sealant, shaping of reservoir shall be done by widening the joint and dislodging the old materials and cleaning. Then installing the backer rod and finally installing the sealant. Cracks resealing is more difficult in shaping, cleaning and sealant due to its un-uniform reservoir. Tape may be used instead of backer rod.

1.2 Retrofitting of Dowels

Retrofitting of dowels can be done for faulting transverse joints and cracks by cutting required slots on them parallel to center line. Then cleaning and preparing the slots for dowels placing and backfill the slots as per specifications.

1.3 Sub-sealing (Under-sealing)

Can be done through filling voids under the slabs, stabilize the slabs and grind to restore riding quality by removing faulting at joints, slab warping and surface deformation, reestablish skid resistance and correct cross slopes.

2. Rehabilitation

Rehabilitation improves the structural and functional pavement condition and can be categorized into three types depending on existing condition:

2.1 Restoration

The main restoration techniques are the full depth repair and partial depth repair, full depth repair, repairs distress greater than 1/3 of slab depth and consist of removing and replacing at least a portion of existing slab to the bottom of slab. The partial depth repair, repairs deterioration occurs on the top 1/3 of the slab. Generally located at joints but can be done anywhere the surface defects occur. Restoration is used to repair isolated areas of deterioration.

2.2 Resurfacing

Resurfacing is used to repairs medium to high severity levels of distress. Concrete overlays for concrete pavement are the bonded concrete overlay and unbounded concrete overlays. Bonded overlay consist of a thin concrete layer (100 mm or less) on top of an existing concrete surface. Un-bonded overlays consist of a thick concrete layer (125 mm or greater) on top of an existing concrete surface. This type of overlays uses a separation interlayer (> 50 mm) to separate the overlay from the existing concrete. The separation interlayer allows the layers to act independently, and prevents the distress from reflecting into the overlay.

2.3 Reconstruction

Final stage of rehabilitation, which involving in complete or partial removing and replacing of the existing pavement with a new pavement. Reconstruction is used at the end of pavement life when it has very high severity level of distress.

2.4 Pavement Design Parameters

A. General

The thickness design of pavements is a complex engineering problem involving a large number of variables. The main variables which controlling the design are the design traffic calculated in term of ESAL during the pavement life for flexible and rigid pavement, and the sub-grade soil strength determined in term of resilient modulus M_R for flexible pavement and sub-grade reaction modulus k for rigid pavement.

1. Design Traffic

The ultimate aim of conducting traffic loading analysis is to determine the magnitude of different wheel loads and their repetitions that will be applied among the pavement design life.

The computation of the design traffic involves the following steps:

1. Estimation of initial year traffic volume.
2. Estimation of annual traffic growth rate.
3. Estimation of traffic stream composition.
4. Computation of traffic loads.
5. Estimation of directional split of design traffic loads.

Information concerning the first two steps can be obtained from traffic surveys and forecasts based on historical trends or prediction using transportation models. The analyses required for the remaining Steps are explained in the discussions that follow.

1.1 Computation of Design Traffic (ESAL)

The pavement design is based on the total number of passes of the standard axle load during the design period, defined as the equivalent single-axle load (ESAL) usually the 18-kip (80-kN) single-axle load .and computed by the following equation:

$$\mathbf{ESAL} = \sum_{i=1}^{m} ft\; nt \qquad \mathbf{(2.11)}$$

m = Number of axle load groups.
ft = Equivalent axle load factor (EALF) for each axle load group.
nt = Number of repetitions of each axle load group during the design period

1.1.1 EALF for Flexible & Rigid Pavements

AASHTO equivalent factors with pt = 2.5 and SN = 5 are used by the Asphalt Institute for flexible pavement, as shown in table C-1of Appendix C for the most design cases unless the design thickness is significantly different.

The EALF can also be computed for rigid pavement by using AASHTO equivalent factors for pt = 2.5, D = 9 in. (229 mm) as shown in Table C-2 of Appendix C. If the thickness is not known in the design stage, the value *D* = 9 shall be used.

1.1.2 Number of Repetition of Each Axle Load Group (n_t)

The prediction of n_t based on the Information of the initial traffic n_0, which can be obtained from field measurements. The initial daily traffic is in two directions over all traffic lanes and must be multiplied by the directional and lane distribution factors to obtain the initial traffic on the design lane.

The traffic to be used for design is the average traffic during the design period, so n_0 must be multiplied by a growth factor. If n_t is the total number of load repetitions to be used in design for each axle load group, then

$$n_t = (no)(G)(D)(L)(365)(Y) \quad (2.12)$$

Where:
G = The growth factor.
D = Directional distribution factor.
L = Lane distribution factor.
Y = Design period in years.

The initial traffic n_0 can be computed from:

$$n_o = (p_i F)(ADT)_o(T)(A) \quad (2.13)$$

Where:
P_i = Number of repetitions of each axle load group.
F = Equivalent axle load factor (EALF).
ADT_0 = Average daily traffic at the beginning of project.
T = Truck factor.
A = Average number of axle per truck.

a. Growth Factor (G)

The Asphalt Institute (AI, 1981a) and the AASHTO design guide (AASHTO, 1986) recommend the use of traffic over the entire design period to determine the total growth factor, as indicated by:

$$\text{Total growth factor} = G\times Y = ((1+r)^Y - 1) / r \quad (2.14)$$

Where:

r = Growth ratio.

b. Directional distribution factor (D)

To determine the design traffic loading on the design lane, one must split the traffic by direction and distribute the directional traffic by lanes. An even split assigning 50% of the traffic to each direction appears to be the normal. In circumstances where an uneven split occurs, pavements are designed based on the heavier directional traffic loading.

c. Lane Distribution Factor (L)

The design lane for pavement structural design is usually the slow lane next to the shoulder, in which a large proportion of the directional heavy vehicle traffic is expected to travel. Studies have shown that depending upon road geometry, traffic volume, and composition as much as 50% of the directional heavy vehicles may not travel on the design lane.

The factors in AASHTO recommendations are for lane distributions of ESAL are presented in Table C-3 of Appendix C. The latter tends to provide a better estimate of traffic loading in cases involving a higher concentration of heavily loaded vehicles in the slow lane.

d. Truck Factor (T_f)

A single truck factor can be applied to all trucks, or separate truck factors can be used for different classes of trucks. The sum of ESALs for all trucks weighed divided by the number of trucks weighed gives the truck factor.

1.1.3 Traffic Analysis for Individual Axle Loads Group

In this method both traffic and vehicle are considered individually , the loads can be divided into a number of groups, and the stresses, strains, and deflection under each load group can be determined separately for design purpose.

PCA employed this method for rigid pavement design.
The following formula had been used to estimate the expected repetitions (ni) for each i-th load group:

$$\mathbf{N_i = N_a/trucksurveyed \times 365 \times G \times ADT \times D/100 \times P_T/100 \times L/100} \quad \textbf{(2.15)}$$

Where:

N_a = Number of axles per trucks surveyed.
P_T = Truck percentage %.
D = Directional Split.
G = Growth factor.
L = Lane distribution factor.

2. Resilient Modulus M_R

The resilient modulus is the elastic modulus to be used with the elastic theory. It is well known that most paving materials are not elastic, but experience some permanent deformation after each load application.
The resilient modulus can be defined as the recoverable strain under repeated loads

$$\mathbf{M_R = \sigma_d / \varepsilon_r} \quad \textbf{(2.16)}$$

Where:
σ_d : repeated stress.
ε_r : recoverable strain.
The resilient modulus test is a nondestructive test, and the same sample can be used for many tests under different loading and environmental conditions. Since many laboratories are not performing the resilient modulus test for soils, due to its long time consumption, and expensive equipments, it is common practice to estimate M_R through empirical correlation with other soil properties.

2.1 Sub-grade Soils M_R

The Asphalt Institute method for determination of sub-grade resilient modulus required at least six to eight test values are usually used to determine the design sub-grade resilient modulus.

The design sub-grade resilient modulus is defined as the modulus value that is smaller than 60, 75, or 87.5% of all the test values.
These percentages are known as percentile values and have relation to traffic levels, as shown in Table C-4 of Appendix C.

After selecting the design CBR corresponding to the design percentile the correlation is used to obtain the resilient modulus. The correlation suggested by AASHTO for fine-grained soils with soaked CBR of 10 or less and for soil with CBR more than 10>

$$\mathbf{M_R\ (psi) = 1500 \times CBR} \quad \textbf{(2.17)}$$

$$\mathbf{M_R\ (MPa) = 10.342 \times CBR^{0.64}} \quad \textbf{(2.18)}$$

2.2 Sub-bases, E_{SB}

Figure C-1of Appendix C shows the correlation chart for estimating the resilient modulus of granular sub-bases from CBR, R values, and Texas tri-axial classification.

2.3 Bases, E_{BS}

Figure C-2 of Appendix C shows the correlation charts for untreated granular base, bituminous treated base, and cement-treated base. The resilient modulus of untreated bases, bituminous- treated bases and cement treated bases are correlated with CBR, R value, Texas tri-axial classification, Marshall Stability and the unconfined compressive strength. Eq. (2.18) also gives reasonable resilient modulus correlation for both granular base and sub-base near to that obtained through the related charts.

2.4 Asphalt Concrete Surface Course, E_P

Figure C-1 of Appendix C is a chart relating the layer coefficient of a dense graded HMA to its resilient modulus at 70°F (21°C). Caution should be used in selecting layer coefficients with modulus values greater than 450,000 psi because the use of this larger modulus is accompanied by increased susceptibility to thermal and fatigue cracking.

The layer coefficient a_l for the dense-graded HMA used in the AASHO Road Tests is 0.44, which corresponds to a resilient modulus of 450,000

3. Modulus of Sub-grade Reaction, k

The sub-grade reaction modulus is very important design parameter required for the rigid pavement design

$$\mathbf{k = P/\Delta} \qquad \mathbf{(2.19)}$$

Where:
P = The applied load.
Δ = The caused deflection

3.1 Composite Modulus of Sub-grade, k_∞

Figure C-4 of Appendix C is first applied to account for the presence of sub-base course between the sub-grade and the slab to obtain the composite modulus of sub-grade reaction $\mathbf{k}_\infty$ based on sub-grade of infinite depth.

3.2 Design Modulus of Sub-grade Reaction due to Rigid Foundation near Surface k

If a rigid foundation lies below the sub-grade and the sub-grade depth to rigid foundation Ds_G is smaller than 10 ft (3 m), then the modulus of sub-grade reaction must be modified by applying Figure C-5 of appendix C to include adjustment for the depth of rigid foundation.

CHAPTER THREE
CASE STUDY

3.1 Introduction

The study was conducted for El Moneera – El Saffaya highway the Road A which represents the national road network. The other road is Road B the section of Omdurman Ring Road, representing a state road.

A. Road A Characteristics

El Moneera - El Saffaya road is one of projects associated to the re-residence of the effected peoples from construction of Seteit & Atbara dams.
The alignment about 60 km starts from El Moneera passing through Wad Jabir and El Drabi cities towards El Saffaya at the northern east of country. Roughly between latitude 15^0 and longitude 36^0 in Butana area (see figure 3.1).

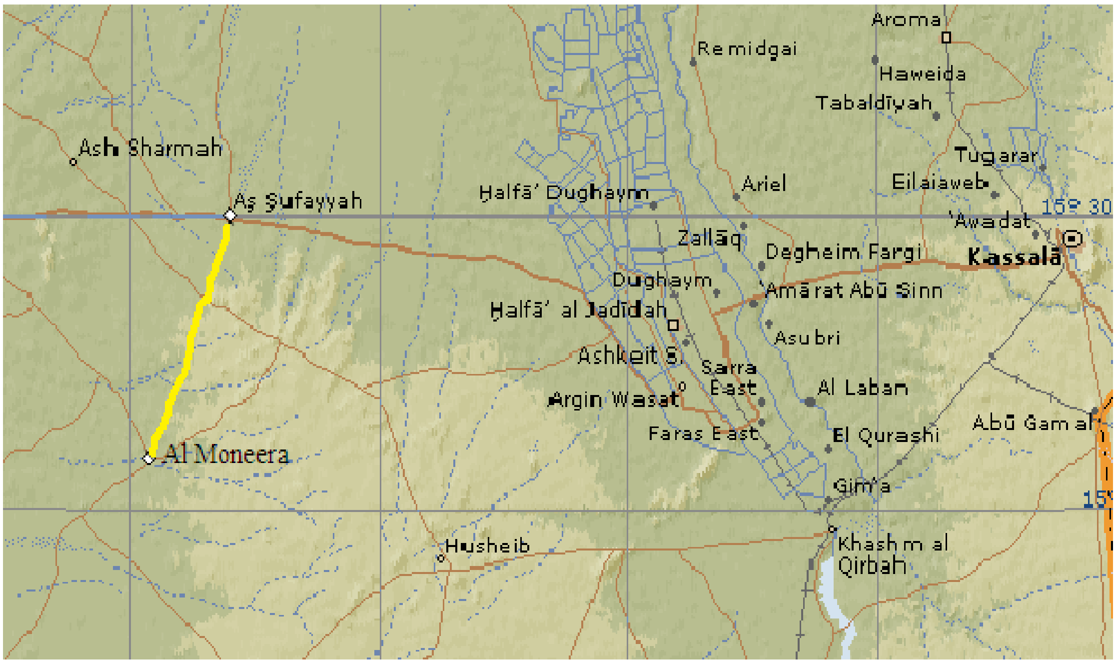

Figure 3.1: Road A Project Area Plan

Figure 3.2: Road A Project Site

The whole road corridor passes through semi flat terrain with some hills up to El Saffaya. The type of soils along the alignment is clayey soils (black cotton soil - see figure 3.2).

The geometric design of the road consists of single carriageway with 3.65m single lane in each direction.

B. Road B Characteristics

About seven km of Omdurman ring road the section of Khartoum ring road project, which connecting the three parts of the capital area together (Khartoum, Omdurman and Bahry) was selected to perform the study. This section is now under construction and reaches to the crushed stone base works.

The alignment starts from Sheryan El Shmal Road, and ends in the intersection of Dar Elsalam Road with Qandahar. The corridor is crossing three small valleys (khours) discharging water to Abu Annja the main khour. There is a hill at the end of the alignment which had been crushed to maintain the required road level and to supply the base layer material.

Figure 3.3: Road B Project Area Plan

The soil along the alignment varies from sandy, silty and gravely soils.

Four culverts were constructed over the small valleys, two box culverts with four cells, two with two cells and two pipe culverts with two cells located along the road alignment.

The geometric design of the road consists of double carriage way with three lanes on each direction; the lane width is 3.65 m.

3.2 Determination of Road A and B, ESAL Factors

The cumulative equivalent standard axle load (ESAL) for road A and road B, for a design life of 20 years can be computed from the following equation mentioned earlier in this study

ESAL = (ADT)o(T)(Tf)(G)(D)(L)(365)(Y)

A. Initial Traffic Volume & Annual Growth Rate

1. Road A

The forecasting of traffic growth rate based on present and future transportation demand for the project region was assumed to be average of **7%** for the whole project design life, considering the generated and diverted traffic after road construction.

2. Road B

The growth rate had been estimated by 3% depending on previous studies for similar roads constructed at Khartoum state, Khartoum structural map and Khartoum traffic master plan.

Data had been collected to predict the initial traffic volume using the proposed routes and tabulated as follows:

Table 3.1: Road A Initial Year Traffic Volume (AADT0):

Passenger Car	Mini Bus	Light Truck	Heavy Truck	Total
12940	18429	5879	381	37628

Source: Traffic counting and transportation demand report for the year 2013

- Passenger car = 83%   Trucks & Buses = 17%

Table 3.2: Road B, Sheryan El Shamal Segment ADT Statistic

Vehicle Classification	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
ADT	28	561	484	4	138	34	2	9	9	4	0	1	10	0	36

Source: Dar consult traffic analysis and transportation demand report

The output summary presented in the traffic analysis and transportation demand report, prepared by DAR CONSULT CIVIL ENGINEERING for Sheryan Al Shamal segment has direct impact on determining traffic volume that uses road B

- Sheryan Al Shamal traffic survey station Total / Day = 1365 V.p.d
- Passenger car = 79% Trucks & Buses = 21%

B. Traffic Loading Composition:

The traffic counting survey data shown in Table 3.1, 3.2 classifies vehicles by vehicle type which enables to compute the number of repetitions by axle type, (single axle, tandem axle, and tridem axle).

Paragraph 1.1 and 1.2 of Appendix D shows the assumption made to convert traffic data to axle load repetitions by applying standard axle loads for the traffic configuration to obtain the initial equivalent axle load via the two roads in Table 3.3, Table 3.4.

Table 3.3: Road A, Axle-Load Data Presentation

Single Axle		Tandem Axle		Trible Axle	
Axle Load (kips)	**No. Axles per Year**	**Axle Load (kips)**	**No. Axles per Year**	**Axle Load (kips)**	**No. Axles per Year**
2	25,880	32	5876	52	381
4	18,429	36	381	58	381
6	18,429	38	381		
12	5876				
18	6260				

Source: made from traffic counting and transportation demand report

TABLE 3.4: Road B Trucks Axle-Load Repetition per Day

Single Axle		Tandem Axle		Trible Axle	
Axle Load (kips)	**No. Axles per Day**	**Axle Load (kips)**	**No. Axles per Day**	**Axle Load (kips)**	**No. Axles per Day**
12	183	32	90	52	42
16	58	36	68	56	36
18	239				
22	12				
24	12				

Source: made from traffic counting and transportation demand report -Dar consult.

C. Road A and Road B, Growth Factor (G)

To determine the total growth factor eq. (2.14) was used for both roads A and B for 20 years design life.

Total growth factor = $G \times Y = ((1+r)^{Y} - 1) / r$

Table 3.5: Road A and Road B Growth Factors

Description	Growth Factor
Road A	41
Road B	26.87

D. Road A and Road B, Directional Distribution Factor (D)

The directional distribution factor = 0.5 due to two direction traffic for both roads.

Table 3.6: Road A and Road B Directional Distribution Factors

Description	Directional Distribution Factor
Road A	0.5
Road B	0.5

E. Road A and Road B, Lane Distribution Factor (L)

Table C-3 of Appendix C was used to determine the roads A and B lane distribution factors as follows:

Table 3.7: Road A and Road B Lane Distribution Factors

Description	No. of Lanes in Each Direction	Lane Distribution Factor
Road A	1	1
Road B	3	0.8

F. Road A and Road B, Truck Factor (T_f)

The EALF values for flexible pavement were obtained from Table C-1 of Appendix C to calculate the truck factor. Results of Road A and B are tabulated in Table 3.8 and 3.9

Table 3.8: Computation of Road A Truck Factor for Flexible Pavement

Axle Load (Kips)	EALF	No. of Axles (Per Year)	ESAL
Single Axle			
12	0.189	5879	1,099.4
18	1.00	6260	6,260
Tandem Axle			
32	0.857	5879	5,038.3
36	1.38	381	525.78
38	1.70	381	647.7
Tridem Axle			
52	1.43	381	544.83
58	2.20	381	838.2
ESAL for all trucks weighed			**14,954.21**

Truck Factor = ESAL for all trucks weighed /No. of trucks weighed
= 14,954.21/ (5879 + 381) = **2.39**

Table 3.9: Computation of Road B Truck Factor for Flexible Pavement

Axle Load (Kips)	EALF	No. of Axles	ESAL
Single Axle			
12	0.189	183	34.60
16	0.623	58	36.13
18	1.00	239	239
22	2.18	12	26.16
24	3.03	12	36.36
Tandem Axle			
32	0.857	90	77.13
36	1.38	68	93.84
Tridem Axle			
52	1.43	42	60.06
56	1.91	36	68.76
ESAL for all trucks weighed			672.04

Truck Factor = ESAL for all trucks weighed /No. of trucks weighed
= 672.04/ 288 = 2.33

3.3 Determination of ESAL for the Design Lane Traffic

A. Flexible Pavement ESAL for Road A

$$ESAL = 37628 \times 2 \times 2.39 \times 0.17 \times 41 \times 0.5$$
$$= 6.2 \times 10^5$$

B. Flexible pavement ESAL for Road B

$$ESAL = 1365 \times 2 \times 2.33 \times 0.21 \times 0.8 \times 0.5 \times 26.87 \times 365$$
$$= 5.24 \times 10^6$$

3.4 Material Properties

A. Road A Sub-grade and Construction Materials Laboratory Testing Results

Along the proposed alignment, seventeen pits were dug, and samples had been taken for approximately 60 cm depth to carry out the sub-grade soil laboratory testing. Thirteen samples had been taken from the proposed aggregate quarries which had been located at twelve villages within the project area near to beginning of the proposed road.

Results of sieve analysis, liquid limits, plasticity index, soil classification accordance to AASHTO M145 and the CBR values had been shown in Table D-1of Appendix D

It's noticeable that the sub-grade soil all along the alignment falls under the group of A-7-6 which categorized as plastic clayey soil with high plasticity indexes ranging between 26.5% - 32.5% this type of soil is subject to extremely high volume change between wet and dry seasons.

The CBR values were taking range from 1% - to 3% at the seven samples which selected randomly. This type of soil is not suitable for pavement construction.

Improved sub-grade shall be required to be replaced after excavation and removing of 12 in (30 cm) of the existing sub-grade.

There are many quarries near the project area have got considerable amounts of soil with CBR greater than 10%, suitable for improved sub-grade construction.

The aggregate falls in group A-1-a including gravel and A-1-b including coarse sand. The plasticity indexes are ranging from non-plastic to PI = 11%. The obtained CBR values fall in the rang of 40% - 100%.

Aggregate investigation clearly shows appropriate materials for construction of sub base and base layers tabulated in Table D-2 of Appendix D.

B. Road B Sub-grade Soil and Construction Material Laboratory Testing Results

Through the samples which had been taken for soil analysis, the sub-grade soil encountered on the proposed alignment vary from non to medium plastic soils(SM,SC, and CL) and classified as A-4,A-2-6,A-2-4 and A-6 according to AASHTO classification system.

The percentage of fine soil varies between 17.0 – 61.2%
The liquid limits have rang between Non – 37.2
The plasticity index varies between Non -13.2
CBR values were measured for selected sub-grade and varies between 2.5 – 25 with

Swell varies between 0 – 30%. Table D-3 of Appendix D is shown the mentioned above results.

The exposed sub-grade soil generally is suitable to be used as embankment materials. The sub-grade shall be excavated to depth of 12 in (30 cm), and placed in layers with good compaction to perform the embankment.

The samples were taken from the nearby borrow pits proposed to supply the construction materials generally consist of Sandy Gravel with Silt, Gravely Sand with Silt, Sandy Silt with Gravel, Sand with Silty Gravel. They are classified according to ASTM system (SC, SM & CL) and according to AASHTO system they belong to A-2-4 & A-2-6.

The liquid limits have range of 19 – 26.
The plasticity index varies between 6.1 -10.7.

CBR values were measured for selected construction and varies between18 – 43% with a swell % between 0 - 0.02%.

The construction material tests results were shown in Table D-3 of Appendix D. The construction materials meet the requirements of the design standard of sub base layers in one borrow pit only which shows CBR value of 43% coordinates of E= 4412911 , N= 1742193.

For the construction of base there are many quarries for supplying rock materials which can be processed and crushed to prepare the required material.

C. Sub-grade Resilient Modulus M_R

The design CBR had been determined by using Asphalt Institute percentile value, as shown in table D-4 of Appendix D.

1. The percentile value opposite to Road A flexible pavement design traffic ESAL = 6.2×10^5 is 75%.

2. The percentile value opposite to Road B flexible pavement design traffic ESAL = 5.24×10^6 is 87.5%.

1. Road A, Existing and Improved Sub-grade Resilient Modulus M_R.

The CBR values of the seven samples had been arranged in ascending order in Table 3.10 and analyzed to get the cumulative curve obtained in Figure 3.5. By entering figure 3.5 with the design percentile value of 75% the design CBR value was found to be equal to 1.9%**.**

The sub-grade soil is not suitable for pavement construction. It is recommended to be cut and removed from site and replaced with selected material to work as improved sub-grade (capping) with CBR = 10%, this soil is available near the site.

Table 3.10: Determination of Road A Existing Sub-grade Design CBR

CBR values	No. of observations	No. of CBR values ≥ value shown	% CBR values ≥ value shown
1	2	7	100
2	1	5	71.43
2.2	2	4	57.14
2.8	1	2	28.57
3	1	1	14.29
Total	**7**		

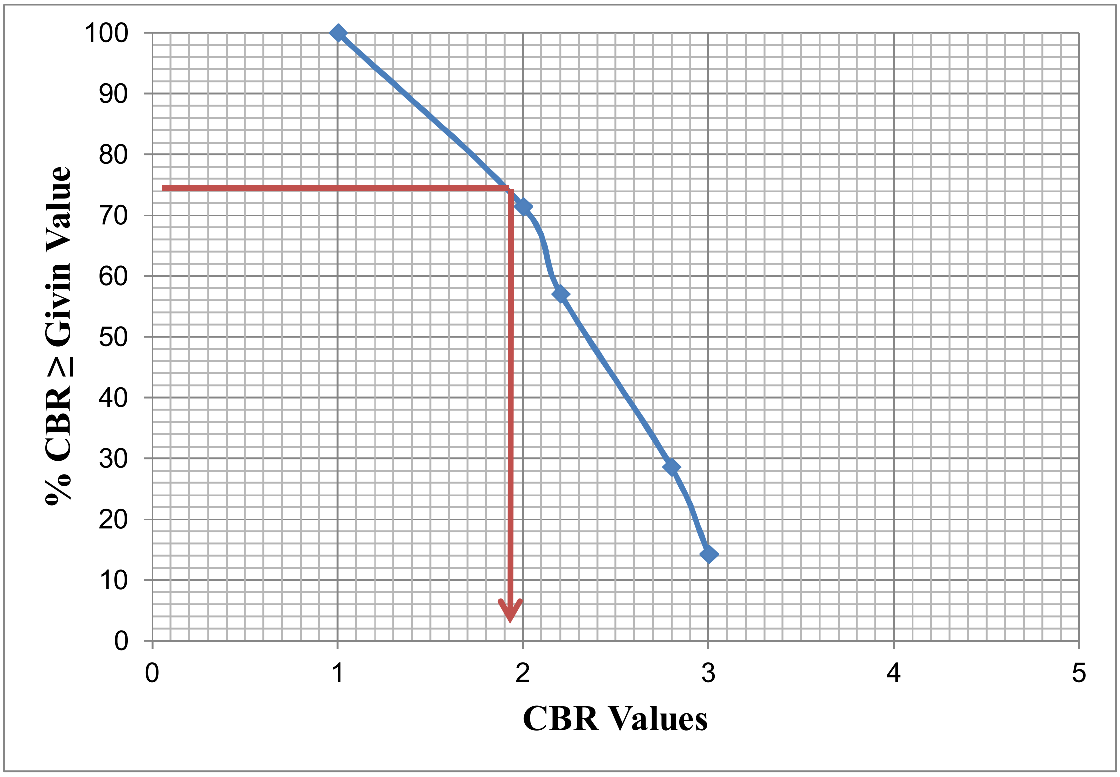

Figure 3.4: Road A Existing Sub-grade Design CBR

The materials shall be selected for the improved sub-grade construction shall have a CBR value of 10%.

CBR = 10%
$M_R = 1500 \times 10 = 15{,}000$ psi

- The design will be performed based on the improved sub-grade resilient modulus.

2. Road B Existing and Improved Sub-grade Resilient Modulus M_R

Fifteen CBR values had been analyzed adopting the above procedure in Table 3.11. The 87.5 percentile had been used to determine the design CBR value which is equal to 5 **%** from the cumulative curve in Figure 3.6

$$M_R \text{ (psi)} = 1500 \times CBR$$
$$= 1500 \times 5$$
$$= 7{,}500 \text{ psi}$$

The sub-grade soil shall be processed through good compaction to achieve desirable strength, and perform embankment layer with CBR greater than the actual. For the improved sub-grade the design CBR = 8 is assumed.

M_R (psi) = 1500×CBR
= 1500 × 8
= 12,000 psi

Table 3.11: Determination of Road B Existing Sub-grade Design CBR

CBR values	**No. of observations**	**No. of CBR values ≥ value shown**	**% CBR values ≥ value shown**
4.5	1	11	100.00
5	2	10	90.91
6	3	8	72.73
7	1	5	45.45
8	1	4	36.36
10	1	3	27.27
15	1	2	18.18
17	1	1	9.09
Total	**11**		

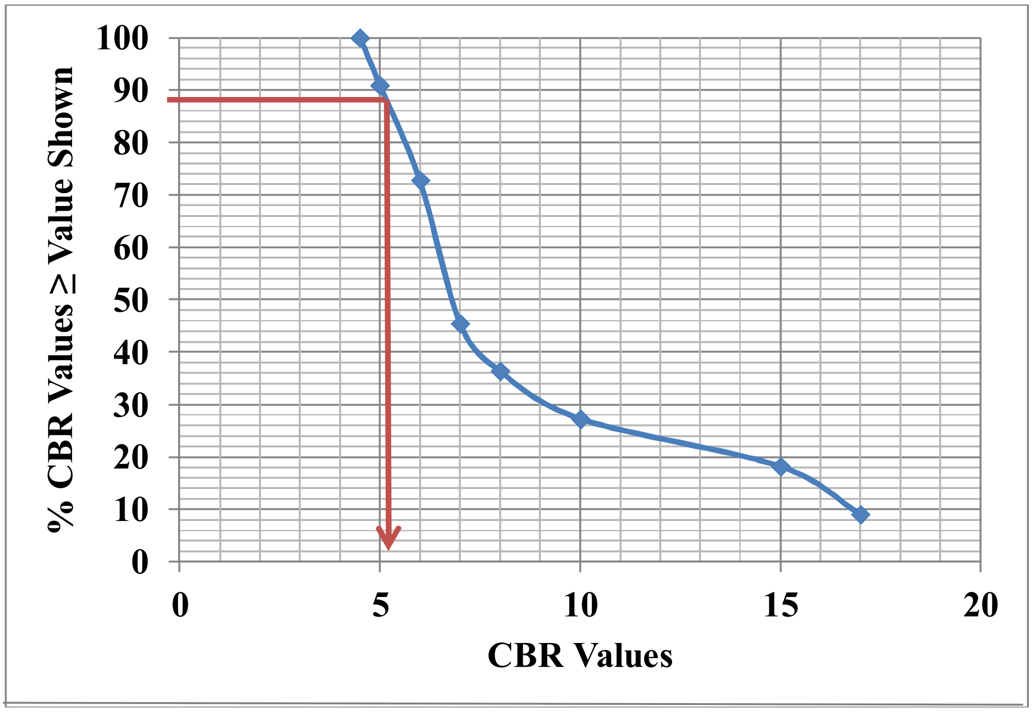

Figure 3.5: Road B Existing Sub-grade Design CBR

3. Pavement Structure Resilient Modulus and Moduli

3.1 Road A Sub-Base Resilient Modulus E_{SB}

Four CBR data were analyzed using the same procedure as indicated in Table 3.12. Figure 3.7 shows the cumulative curve made by the analysis. Dropping line from the design percentile value of 75% to intersect the cumulative curve .then from the intersection point another line had been made to intersect the horizontal axis to obtain the design CBR which found to be equal to 45%.

The suggested correlation to obtain the resilient modulus E_{SB} from CBR value:

$$
\begin{aligned}
E_{SB}\ (\text{MPa}) &= 10.342 \times \text{CBR}^{0.64} \\
&= 10.342 \times 45^{0.64} \\
&= 118\ \text{MPa} \times 145 \\
&= 17{,}110\ \text{psi} \mathrel{\supseteq} 17{,}000\ \text{psi}
\end{aligned}
$$

Table 3.12: Determination of Road A Sub-Base Design CBR

CBR values	No. of observations	No. of CBR values ≥ value shown	% CBR values ≥ value shown
40	1	4	100
45	1	3	75.00
49	1	2	50.00
63	1	1	25.00
Total	**4**		

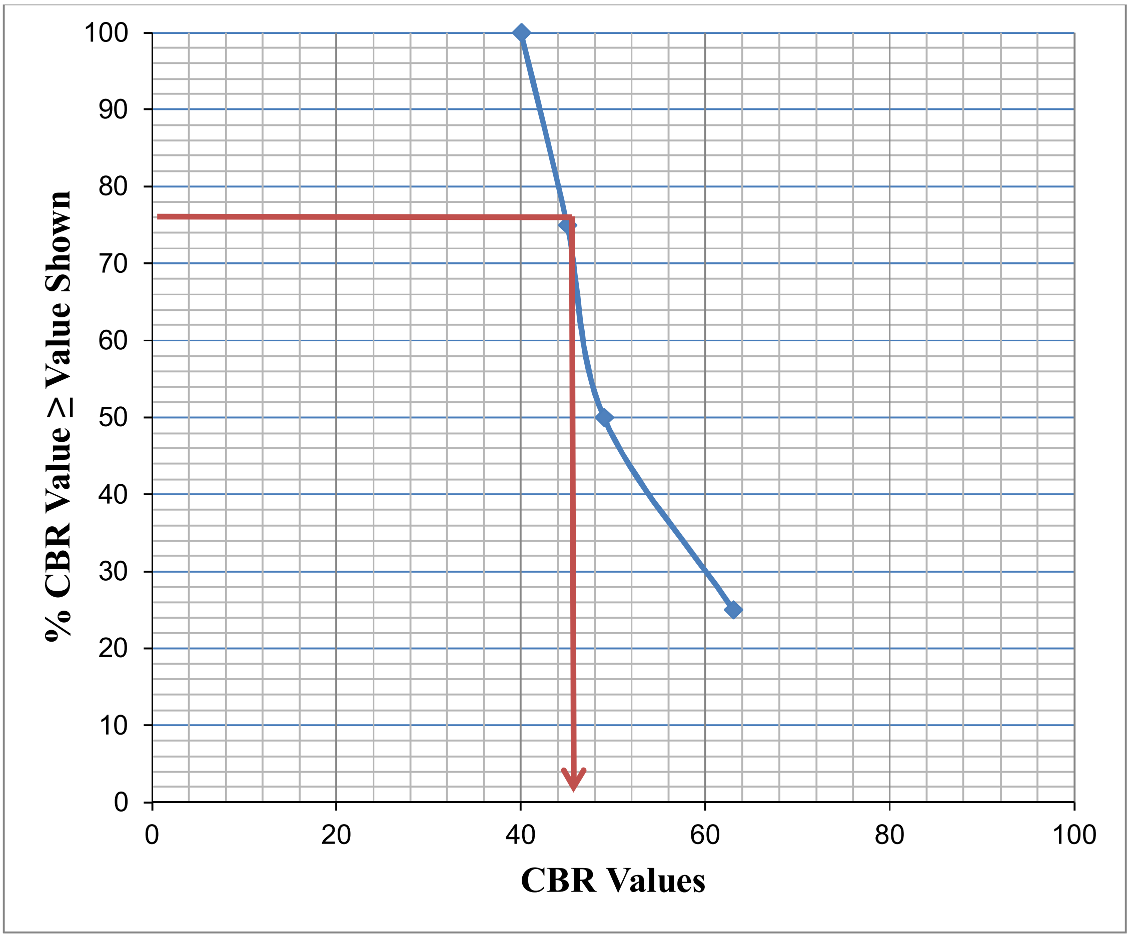

Figure 3.6: Road A Sub-base Design CBR

3.2 Road B Sub-Base Resilient Modulus E_{SB}

Hence there are one borrow bit meet the required design specification as mentioned above with CBR value equal to 43% the correlated resilient modulus for this value can be determined as follows:

$$
\begin{aligned}
E_{SB}\ (\text{MPa}) &= 10.342 \times CBR^{\,0.64} \\
&= 10.342 \times 43^{\,0.64} \\
&= 114.8 \times 145 \\
&= 16{,}949 \text{ psi} \approx 17{,}000 \text{ psi}
\end{aligned}
$$

3.3 Road A Base Resilient Modulus E_{BS}

Seven out of nine samples of construction materials laboratory tests results show a CBR value of 100%. It is suggested to make this value the design value.

The suggested correlation to obtain the resilient modulus E_{BS} from CBR value:

$$
\begin{aligned}
E_{BS}\ (\text{MPa}) &= 10.342 \times CBR^{\,0.64} \\
&= 10.342 \times 100^{\,0.64} \\
&= 197 \text{ MPa} \times 145 \\
&= 28{,}574 \text{ psi} \approx 28{,}600 \text{ psi}
\end{aligned}
$$

3.4 Road B Base Resilient Modulus E_{BS}

There are no suitable quarries found near the project area to supply the granular base material according to the soil investigation. There a hill cross the alignment will be excavated to maintain the road level and shall be crushed and graded for supplying of base material.CBR value of this material will exceed the 80**%.** For design purpose it will be taken as 80% the minimum design specification required for base material

$$
\begin{aligned}
E_{BS}\ (\text{MPa}) &= 10.342 \times CBR^{\,0.64} \\
&= 10.342 \times 80^{\,0.64} \\
&= 171 \times 145 \\
&= 24{,}795 \text{ psi} \approx 25{,}000 \text{ psi}
\end{aligned}
$$

3.5 Road A Asphalt Concrete Resilient Modulus E_P

By using Figure C-3 in Appendix C which represents the relation of the HMA layer coefficient a_1, and HMA resilient modulus E_P with assumed a_1 = 0.42. The HMA resilient modulus E_P = 400,000 psi.

3.6 Road B Asphalt Concrete Resilient Modulus E_P

By using Figure C-3 in Appendix C which represents the relation of the HMA layer coefficient a_1, and HMA resilient modulus E_P with assumed a_1 = 0.42. The HMA resilient modulus E_P = 400,000 psi.

3.5 Geometrical Properties:

The geometrical properties of the road for each pavement type were made as follows:

1. Road A

1. Length of road: 60 Km.
2. Width of driving lanes: 24 ft (7.3 m) - 2 lanes.
3. Design Speed: 80 Km/h.
4. Traffic Volume: AADT = 37,628

2. Road B

1. Length of road: 7 Km.
2. Width of driving lanes: 72 ft. (22m) - 6 lanes.
3. Design Speed: 100 Km/h.
4. Traffic Volume: ADT = 1365 Vpd

3.6 Flexible Pavement Structural Design

The flexible pavement design was adopted for this study was limited to two design methods that are, Asphalt Institute design method and American Association of State Highway and Transportation Officials (AASHTO) 1993 design.

A. Asphalt Institute Design Method

1. Road A Structural Pavement Design

1. The ESAL for flexible pavement is equal to 6.2×10^{5}.
2. The improved sub-grade suggested to strengthen the existing one had been selected for thickness of 12-in (306 mm) and M_R = 15000 psi.
3. 2 in minimum HMA surface thickness.
4. Assume emulsifies asphalt type Π base.
5. Assume 8 in untreated sub-base thickness.

Appling the above parameters to the design charts in Figures A-1, A-3, A-6 presented in Appendix A to determine the pavement structure thickness.

The flexible pavement design of Road A consist of 2in (51mm) HMA asphalt surface, 3in (76mm) emulsified asphalt base, 8 in (203mm) untreated sub-base and 12 in (306mm) improved sub-grade.

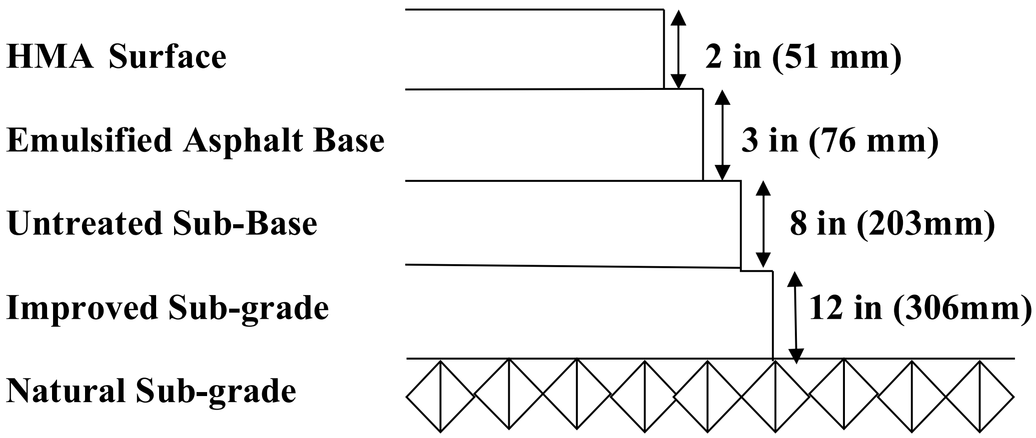

Figure 3.7: Pavement Layers Thickness According to AI Design Method For Road A

2. Road B Structural Pavement Design

1. The ESAL for flexible pavement is equal to 5.24×10^{6}
2. The embankment suggested to be processed and replaced from existing sub-grade with M_R = 12,000 psi had been selected for thickness of 12in (305 mm).
3. 2in minimum HMA surface Thickness.

4. Assume emulsified asphalt base.
5. Assume 6 in untreated sub-base thickness.

Appling the above parameters to the design charts in Figures A-1, A-3, A-6 presented in Appendix A to determine the pavement structure thickness.

The flexible pavement design of Road B consist of 2in (51mm) HMA asphalt surface, 10.5 in (267mm) emulsified asphalt base, 6 in (152mm) untreated sub-base and 12 in (306mm) embankment.

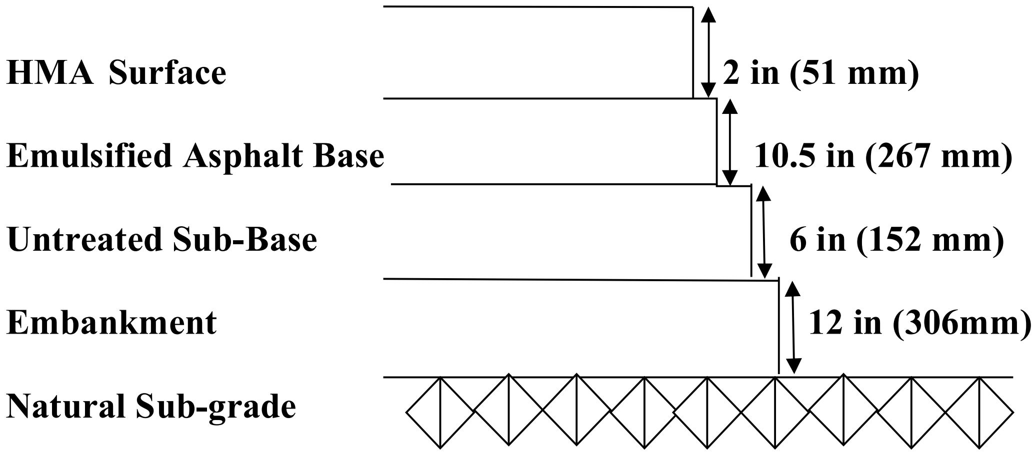

Figure 3.8: Pavement Layers Thickness According to AI Design Method for Road B

B. AASHTO Design Method

The design variables that had been mentioned in chapter two had been determined for the two roads A and B to derive the final pavement design via AASHTO method as follow:

1. Road A Structural Pavement Design

1. Design Traffic ESAL = 6.2×10^5
2. Reliability R = 90 % from Table A-2 of Appendix A.
3. Standard Deviation S_0 = 0.45
4. Serviceability ΔPSI = 2.2 Assumed.
5. Improved Sub-grade Resilient Modulus (M_R) = 15,000 psi, calculated
6. Sub-base Course E_{SB} = 17,000 psi, calculated
7. Base Course E_{BS} = 28,600 psi, calculated

8. HMA E_P = 400,000 psi Assumed.
9. Concrete Surface Course Layer Coefficient a_1= 0.42
   From Figure A-12 of Appendix A
10. Granular Base Course Layer Coefficient a_2 = 0.13
    From Eq. (2.3)
11. Untreated Sub-base Course Layer Coefficient a_3 = 0.12
    From Eq. (2.4)
12. Base Course Drainage Coefficient m_2 = 1
    From Table A-3 of Appendix A

13. Sub-base Course Drainage Coefficient m_3 = 1
    From Table A-3 of Appendix A

Appling the above parameter to the design nomograph of Figure A-10 presented in Appendix A, were values of structure numbers and thickness computed for the pavement structure.

The flexible pavement design of Road A consist of 3in (76mm) HMA surface, 9 in(229mm) untreated base, 6 in (152mm) untreated sub-base and 12in (305 mm) improved sub-grade.

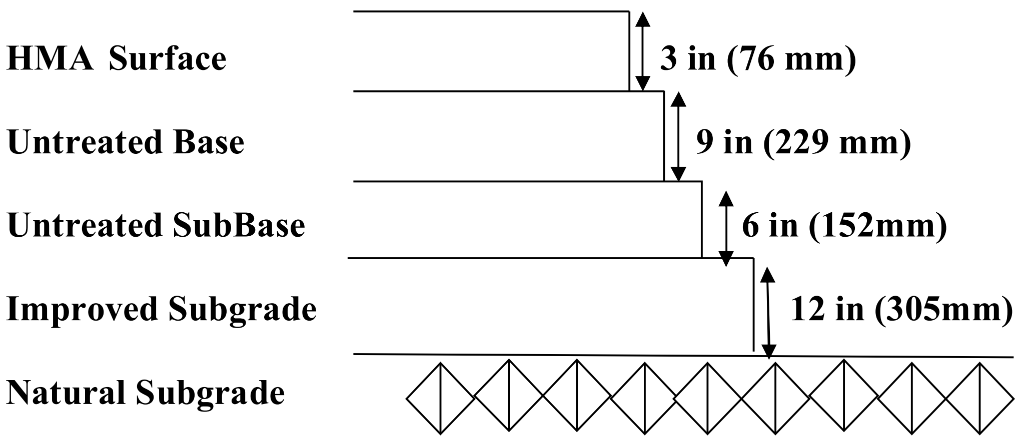

Figure 3.9: Pavement Layers Thickness According to AASHTO Design Method for Road A

2. Road B Structural Pavement Design

1. Design Traffic ESAL = 5.24×10^6
2. Reliability R = 95 % from Table A-2 of Appendix A
3. Standard Deviation S_0 = 0.45
   Recommended by AASHTO for flexible pavements
4. Serviceability ΔPSI = 2.2 Assumed
5. Improved Sub-grade Resilient Modulus (M_R) = 12,000 psi, calculated
6. Sub-base Course E_{SB} = 17,000 psi, calculated.
7. Base Course E_{BS} = 25,000 psi, calculated.
8. HMA E_P = 400,000 psi, assumed.
9. HMA Surface Course Layer Coefficient a_1 = 0.42From Figure A-12 of Appendix A
10. Granular Base Course Layer Coefficient a_2 = 0.15
    From Eq. (2.3)
11. Untreated Sub-base Course Layer Coefficient a_3 = 0.12
    From Eq. (2.4)
12. Base Course Drainage Coefficient m_2 = 1
    From Table A-3 of Appendix A
13. Sub-base Course Drainage Coefficient m_3 = 1
    From Table A-3 of Appendix A

Appling the above parameter to the design nomograph of Figure A-10 presented in Appendix A, were values of structure numbers and thickness computed for the pavement structure.

The flexible pavement design of proposed road (B) consist of 3in (76mm) HMA surface, 8 in (203mm) untreated base, 12 in (305mm) untreated sub-base and 10 in (254mm) improved sub-grade.

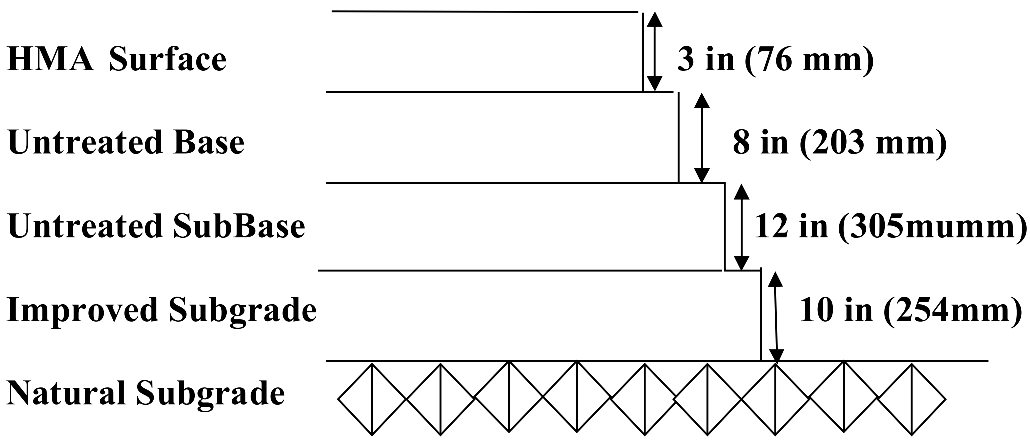

Figure 3.10: Pavement Layers Thickness According to AASHTO Design Method for Proposed Road (B)

3.7 Comparison of Flexible Pavement Thickness between AI and AASHTO Design Method

Table 3.13 and 3.14 shows the pavement thicknesses obtained for road A and road B according to AI and AASHTO method.

Table 3.13: Comparison of Pavement Thickness between AI and AASHTO Design Method for Road A

Road A	AI Design Method	AASHTO Design Method	Difference (%)
HMA Thickness (in).	2	3	33
Base Layer Thickness (in).	3	9	66
Sub Base Layer Thickness (in).	8	6	-25
Embankment (in).	12	12	0
Total Pavement Thickness (in).	**25**	**30**	**17**

Total 3.14: Comparison of Pavement Thickness between AI and AASHTO Design Method for Road B

Road B	AI Design Method	AASHTO Design Method	Difference (%)
HMA Thickness (in).	2	3	33
Base Layer Thickness (in).	10.5	8	-31
Sub Base Layer Thickness (in).	6	12	50
Embankment (in).	12	10	-20
Total Pavement Thickness (in).	**30.5**	**33**	**7.6**

Table 3.13 shows the comparison in Road A pavement layer thickness and the difference in each layer thickness of each standard accordingly. Between two methods that have been used, it is clear that AI Standard gave the thinner pavement layer compared to the AASHTO.

From the table, AI Design procedure produced 2 in (490mm) of HMA layer thickness followed by 3 in (735 mm) in base course and 8 in (196 mm) for the sub base course. This is different with AASHTO layer thickness value that gave 3 in (735 mm) for asphalt layer, 9 in (221 mm) for base course layer and followed by 6 in (147 mm) for the sub base layer.

AASHTO gave thicker Aggregate Base layer because too much variables needed in the standard in order to reduce rehabilitation costs so that the road will be long lasting. AI gave thicker aggregate sub-base layer not far from the thickness produced by AASHTO.

From Table 3.14 the same comparison was made for Road B. It is clear that AI Standard gave the thinner pavement layer compared to the AASHTO.

From the table, AI Design procedure produced 2 in (490mm) of HMA layer thickness followed by 10.5 in (245 mm) in base course and 6 in (147 mm) for the sub base course. This is different with AASHTO layer thickness value that gave 3 in (735 mm) for asphalt layer, 8 in (196 mm) for base course layer and followed by 12 in (294 mm) for the sub base layer.

AI gave thicker emulsified base layer in order to sustain under the heavy load was determined for this road. AASHTO gave thicker aggregate sub-base layer double the thickness produced by AI.

For both Road A and Road B, AASHTO design method was adopted for the availability of aggregate sources for base construction which remain cheaper then processing the emulsified asphalt base.

3.8 Determination of Rigid Pavement ESAL Factors

The same ESAL factors were calculated for flexible pavement, will be used for road A and B rigid pavement design traffic ESAL. The only difference is calculation of truck factor T_f. This difference occurs according to the slight changes in EALF values given for rigid pavement.

A. Road A and Road B, Truck Factor T_f

Rigid pavement EALF values were used for calculation of Road A, and B truck factor is presented in Table C-2 of Appendix C

Table 3.15: Computation of Road A Truck Factor for Rigid Pavement

Axle Load (Kips)	EALF	No. of Axles	ESAL
Single Axle			
12	0.176	5879	1,034.7
18	1.00	6260	6,260
Tandem Axle			
32	1.49	5879	8,759.7
36	2.43	381	925.8
38	3.03	381	1,154.4
Tridem Axle			
52	3.44	381	1,310.6
58	5.32	381	2,026.92
			21,472.12

Truck Factor = ESAL for all trucks weighed /No. of trucks weighed
= 21,472.12/ (5879 + 381) = 3.43

Table 3.16: Computation of Road B Truck Factor for Rigid Pavement

Axle Load (Kips)	EALF	No. of Axles	ESAL
Single Axle			
12	0.176	183	32.21
16	0.604	58	35.03
18	1.00	239	239
22	2.34	12	28.08
24	3.36	12	40.32
Tandem Axle			
32	1.49	90	134.1
36	2.43	68	165.24
Tridem Axle			
52	3.44	42	144.48
56	4.63	36	166.68
ESAL for all trucks weighed			985.14

Truck Factor = ESAL for all trucks weighed /No. of trucks weighed
= 985.14/ 288 = 3.42

3.9 Determination of Rigid ESAL for the Design Lane Traffic

1. Rigid Pavement Design Traffic for Road A

ESAL = $37628 \times 2 \times 3.43 \times o.17 \times 41 \times 0.5$
$= 9 \times 10^{5}$

2. Rigid Pavement Design Traffic for Road B

ESAL $_{T}$ = $1365 \times 2 \times 3.42 \times 0.21 \times 0.8 \times 0.5 \times 26.87 \times 365$
$= 7.7 \times 10^{6}$

3.10 Traffic Analysis for Individual Axle Load Groups

The following formula was used to estimate the expected repetition N_i for each i-th load group

$$N_i = N_a / \text{truck surveyed} \times 356 \times G \times ADT \times D/100 \times P_t/100 \times L/100 \quad \textbf{(2.15)}$$

Where:

N_a = Number of axles per truck surveyed.

P_t = Truck percentage %

A. Expected Repetition (N_i) for Road A and Road B

Estimation of the expected repetition for the axle load group applied for the Road A and Road B according to the above equation leads to the results are tabulated in Table 3.17, 3.18.

Table 3.17: Computation of Expected Road A Repetition (N_i) for the Applied Axle Load Group

Axle Load (Kips)	No. of Axles	Expected Repetition (N_i)
Single Axle		
12	5879	246,305
18	6260	262,267
Tandem Axle		
32	5879	246,305
36	381	15,962
38	381	15,962
Tridem Axle		
52	381	15,962
58	381	15,962

Source: made from traffic counting and transportation demand report - TECHNO-CON Co.

Table 3.18: Computation of Road B Expected Repetition (N_i) for the Applied Axle Load Group

Axle Load (Kips)	No. of Axles	Expected Repetitions (N_i)
Single Axle		
12	183	1,429,095
16	58	452,937
18	239	1,866,413
22	12	93,711
24	12	93,711
Tandem Axle		
32	90	702,833
36	68	531,030
Tridem Axle		
52	42	327,989
56	36	281,133

Source: made from traffic counting and transportation demand report -Dar consult

3.11 Modulus of Sub- grade Reaction k

A. Road A Design Modulus of Sub-grade Reaction

1. Composed Modulus of Sub-grade Reaction k_∞:

- Assume 8 in sub-base thickness D_{SB}.
- Actual Sub-grade resilient modulus M_R = 2,850 psi.
- Sub-Base course E_{SB} = 17,000 psi.

By using Figure C-4 of Appendix C the composed modulus of sub-grade reaction k_∞ = 200 pci

2. Design Modulus of Sub-grade Reaction k

- Assume 7.5ft sub-grade depth to rigid foundation $D_{SG.}$
- K_∞ = 200 pci
- Actual Sub-grade resilient modulus M_R = 2,850 psi.

By using Figure C-5 of Appendix C the modified modulus of sub-grade reaction k = 250 pci

B. Road B Modulus of Sub-grade Reaction

1. Composed Modulus of Sub-grade Reaction k_∞

- Assume 6 in sub-base thickness D_{SB}.
- Actual Sub-grade resilient modulus M_R = 4,500 psi
- Sub-Base course E_{SB} = 17,000 psi

Using Figure C-4 of Appendix C the composed modulus of sub-grade reaction k_∞ = 250 pci

2. Design Modulus of Sub-grade Reaction k

- Assume 5ft sub-grade depth to rigid foundation D_{SG}
- K_∞ = 250 pci
- Actual Sub-grade resilient modulus M_R= 4,500 psi.

By using Figure C-5 of Appendix C the modified modulus of sub-grade reaction k = 300 pci.

3.12 Jointed Plain Concrete Pavement Thickness Design Methods

A. AASHTO Method

1. Determination of Road A Layers Thicknesses

1. Reliability R = 90% - Table A-2 of Appendix A
2. Standard deviation S_0 = 0.35
   Value is recommended by AASHTO for rigid pavements.
3. Serviceability loss ΔPSI = 2.0 - Assumed.
4. Design traffic W_{18} = 9×10^5 - Calculated.
5. Drainage coefficient C_d = 1.0 - Table B-3of Appendix B
6. Load transfer coefficient J = 3.2 without shoulders
   Table B-2 of Appendix B
7. Elastic modulus of concrete E_C = 5×10^6 psi- From Eq. (2.9).

8. Concrete modulus of rupture S, = 600 psi.
   Recommended by AASHTO and ASTM
9. Design modulus of sub-grade reaction k = 250 pci - Calculated.
10. Sub-base course thickness = 8 in - Assumed.

Appling the above parameter to the design nomograph of Figure B-1 presented in Appendix B, to determine concrete pavement slab thickness.

The slab thickness for Road A, D_A = 7 in(178mm)
The sub-base thickness D_{sb} = 8 in (203mm)

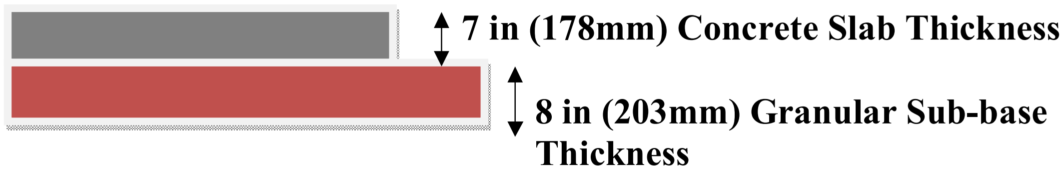

Figure 3.11: Road A Layers Pavement Thickness According to AASHTOO Method

2. Determination of Road B Layers Thicknesses

AASHTO nomograph for rigid pavement design in figure B-1, Appendix B had been used with the following parameter to obtain concrete slab thickness:

1. Reliability R% = 92% -Table A-2 of Appendix A
2. Standard deviation S_0 = 0.25 - Assumed.
3. Serviceability loss ΔPSI = 2.0 - Assumed
4. Design traffic W_{18} = 7.7×10^6 - Calculated
5. Drainage coefficient C_d = 1.0 - Table B-3 of Appendix B
6. Load transfer coefficient J = 3.2 - without shoulders.
   Table B-2 of Appendix B
7. Elastic modulus of concrete E_C = 5×10^6 psi - Eq. (2.9)
8. Concrete modulus of rupture S = 600 psi.
   Recommended by AASHTO and ASTM
9. Modulus of sub-grade reaction k = 300 pci. - Calculated.
10. Sub-base course thickness = 6 in - Assumed

The slab thickness for Road (B), D_B = 9 in (230mm).
The sub-base thickness D_{sb} = 6 in (150mm).

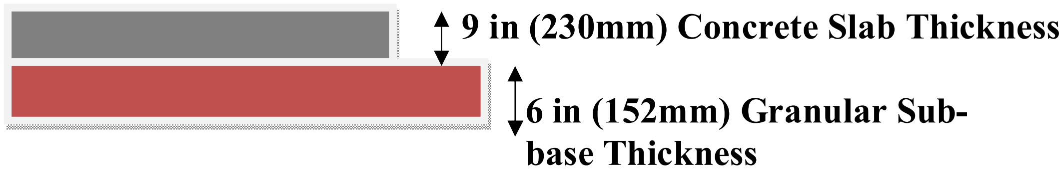

Figure 3.12: Road B Pavement Layers Thickness According to AASHTOO Method

B. PCA Method

1. Determination of Road A Pavement Thicknesses

Table 3.19 presents the calculation that had been made according to the steps mentioned in Chapter Two to determine the slab thickness. The calculation shows that damages caused by fatigue and erosion are 82 and 13, respectively. Both are less than 100%, so the use of a 7 -in. (178-mm) slab is quite adequate. Final structural pavement design was adopted as follows:

The slab thickness D_A = 7 in (178mm)
The sub-base thickness D_{sb} = 8 in (203mm)

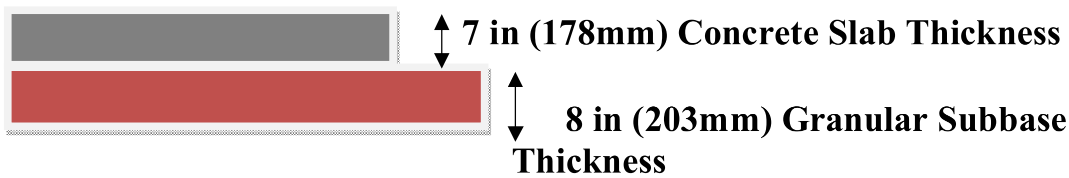

Figure 3.13: Road A Pavement Layers Thickness According to PCA Method

Table 3.19: Calculation of Road A Jointed Plain Concrete Pavement Slab Thickness

- Project: Design two lane interstate rural road
- Trail Thickness: 7 in
- Sub-grade k: 250 psi
- Doweled joints
- No concrete shoulder

- Modulus of rupture MR: 600 psi
- Load safety factor: 1.1
- Design Period: 20 yrs
- Untreated Sub-base course thickness: 8 in.

- Single axle and tridem axle:
1. Equivalent stress = 281.5
(From Table B-4 of Appendix B)
2. Stress ratio factor = 0.4
3. Erosion factor = 2.965
(From table B-5, B-6 of Appendix B)

-Tandem axle:
1. Equivalent stress = 233
(From Table B-4 of Appendix B)
2. Stress ratio factor = 0.39
3. Erosion factor = 3.065
(From Table B-5 of Appendix B)

axle load Kips	Multiplied by LSF factor	Expected Repetitions	Fatigue analysis		Erosion analysis	
			Allowable repetitions	Fatigue percent	Allowable repetitions	Damage percent
1	2	3	4	5	6	7
Single axle						
18	19.8	262,267	400,000	0.65567	4,500,000	0.06
12	13.2	246,305	Unlimited	-	unlimited	-
Tandem axle						
38	41.8	15,962	Unlimited	-	2,000,000	0.01
36	39.6	15,962	Unlimited	-	2,400,000	0.007
32	35.2	246,305	Unlimited	-	9,000,000	0.027
Tridem axle						
58	21.3	15,962	130,000	0.12278	880,000	0.0181
52	19.1	15,962	380,000	0.04201	1,700,000	0.0094
			Total	**0.82**	**Total**	**0.13**

2. Determination of Road B Pavement Thicknesses

The same calculation had been conducted to determine the slab thickness for this road is presented in Table 3.20. The calculation shows that damages caused by fatigue and erosion are 65 and 50, respectively. Both are less than 100%, so the use of a 8 -in. (203-mm) slab is quite adequate. Final structural pavement design was adopted as follows:

The slab thickness D_B = 8 in (203mm)
The subbase thickness D_{sb} = 6 in (152mm)

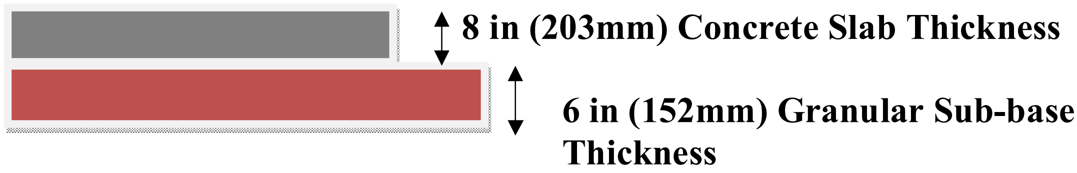

Figure 3.17: Road B Pavement Layers Thickness According to PCA Method

Table 3.20: Calculation of Road B Jointed Plain Concrete Pavement Slab Thickness

- Project: Design six lane interstate urban road
- Trail Thickness: 8 in
- Sub-grade k: 300 psi
- Modulus of rupture MR: 600 psi
- Load safety factor: 1.1
- Doweled joints
- No concrete shoulder
- Design Period: 20 yrs
- Untreated Sub-base course thickness: 6 in.

- Single axle and tridem axle:
1. Equivalent stress = 225
(From Table B-4 of Appendix B)
2. Stress ratio factor = 0.375
3. Erosion factor = 2.79
(From table B-5, B-6 of Appendix B)

-Tandem axle:
1. Equivalent stress = 188
(From Table B-4 of Appendix B)
2. Stress ratio factor = 0.31
3. Erosion factor = 2.89
(From Table B-5 of Appendix B)

axle load Kips	Multiplied by LSF factor	Expected Repetitions	Fatigue analysis		Erosion analysis	
			Allowable repetitions	Fatigue percent	Allowable repetitions	Damage percent
1	2	3	4	5	6	7
Single axle						
24	26.4	93,711	180,000	0.52	2,020,000	0.046
22	24.2	93,711	700,000	0.13	5,000,000	0.019
18	19.8	1,866,413	Unlimited	-	16,000,000	0.117
16	17.6	452,937	Unlimited	-	unlimited	-
12	13.2	1,429,095	Unlimited	-	unlimited	-
Tandem axle						
36	39.6	531,030	Unlimited	-	7,000,000	0.076
32	35.2	702,833	Unlimited	-	15,000,000	0.047
Tridem axle						
56	20.5	281,133	Unlimited	-	2,800,000	0.10
52	19.1	327,989	Unlimited	-	3,300,000	0.10
			Total	**0.65**	**Total**	**0.50**

3.13 Jointed Plain Concrete Pavement Components Design

A. Road A JPCP Components Design

1. Slab Dimensions

Slab width = the lane width = 12 ft (3.65m).
Slap length (joint opening): = 53ft (16m) from:

$$\mathbf{L = \Delta L / C \times (\alpha_t \, \Delta T + \varepsilon)} \qquad \textbf{(2.5)}$$

ΔL = 0.25 in (doweled joint)
C = 0 .8 for granular sub base.
$\boldsymbol{\alpha_t}$ = 5.5 x 10-6 /°F (9 to 10 .8 x 10^{-6}/°C).
ΔT = 35 F^0
ε = $3\text{x}10^{-4}$

L = 0.25/ 0.8(5.5 x 10-6 x 35 + $3\text{x}10^{-4}$)
= 634.5 in = 53ft = 16m

2. Tie Bars

$$\mathbf{A_S = \gamma_c \, h \, L \, f_a/f_s} \qquad \textbf{(2.6)}$$

γ_c = 0.0868 pci (23.6 KN/m^3).
H = 7 in.
L' = 12 ft (144 in).
F_a = 1.5
F_s = assume f_s = 27,000 psi for billet steel

A_S = 0.0868 x 7 x 144 x 1.5 / 27,000
= 0.00486 in^2/in.

Assume bar size of No. 4 (0.5 in, 12mm) to be used as tie bar with cross sectional area of one bar = 0.2 in^2

The spacing of the bars = 0.2/0.00486 = 40 in

The bar length determined from:

$$t = \frac{1}{2}\left(\frac{fs\, d}{\mu}\right) \qquad \textbf{(2.6)}$$

μ = allowable bond stress for deformed bars = 350 psi.
fs = assume f_s = 27,000 psi for billet steel (table 4.15).
d = bar size 0.5 in (12mm).
t = ½ (27,000 x 0.5/350).

= 19.3 + 3 in for misalignment
= 22.3 in, use 24in for bar length.

The final tie bar design is No.4 (0.5 in, 12mm), 24 in length at 40 in at centers.

3. Dowel Bars

The last edition of PCA 1991 recommended 1.25 in, (32mm) dowel bar size for the pavements less than 10 in, and length of 16 in at 12 in at centers.

4. Contraction Joint Design:

The spacing between contraction joints in ft shall not exceed twice the slab thickness in inches.

The slab thickness = 7 in, the space between contraction joints shall not exceed 14ft , we can use 13ft length between contraction joints so that for one slab length of 53ft shall have two dummy joints and two contraction joints with dowel bars.

The width of the joint shall be 1 in, and shall be fill with bituminous mixture consist of bitumen, sand and cement.

B. Road B JPCP Components Design

1. Slab dimensions

Slab width = the lane width = 12 ft (3.65m).
Slap length (joint opening): = 47.5ft (14.5m) from:

$$\mathbf{L = \Delta L / C \times (\alpha_t \, \Delta T + \varepsilon)} \qquad \mathbf{(2.5)}$$

ΔL = 0.25 in (doweled joint)
C = 0 .8 for granular sub base.
$\boldsymbol{\alpha_t}$ = 5.5 x 10-6 /°F (9 to 10 .8 x 10^{-6}/°C).
ΔT = 45 F^0
ε = $3x10^{-4}$

L = 0.25/ 0.8(5.5 x 10-6 x 45 + $3x10^{-4}$)

=570.77 in = 47.5ft ⊐ 48 ft = 14.6m

2. Tie Bars

$$\mathbf{A_S = \gamma_c \, h \, L \, f_a/f_s} \qquad \mathbf{(2.6)}$$

γ_c = 0.0868 pci (23.6 KN/m^3).
H = 10 in.
L' = 12 ft (144 in).
f_a = 1.5

f_s = assume fs = 27,000 psi for billet steel.

A_S = 0.0868 x 10 x 144 x 1.5 / 27,000
= 0.006944 in^2/in.

Assume bar size of No. 4 (0.5 in, 12mm) to be used as tie bar with cross sectional area of one bar = 0.2 in^2

The spacing of the bars = 0.2/0.006944 = 28.8 ≓ 29 in

The bar length determined from:

$$t = \frac{1}{2}\left(\frac{fs\ d}{\mu}\right) \quad \textbf{(2.6)}$$

μ = allowable bond stress for deformed bars = 350 psi.
fs =assume f s = 27,000 psi for billet steel.
d = bar size 0.5 in (12mm).
t = ½ (27,000 x 0.5/350).

= 19.3 + 3 in for misalignment
= 22.3 in, use 24in for bar length.

The final tie bar design is No.4 (0.5 in, 12mm), 24in length at 30 in at centers

3. Dowel bars

The last edition of PCA 1991 recommended 1.5 in, (32mm) dowel bar size for the pavements less than 10 in, and length of 16 in at 12 in at centers.

4. Contraction Joints Design

The spacing between contraction joints in ft shall not exceed twice the slab thickness in inches.

The slab thickness = 10 in, the space between contraction joints shall not exceed 20 ft , we can use 16 ft length between contraction joints so that for one slab length of 48ft shall have one dummy joints and two contraction joints with dowel bars.

The width of the joint shall be 1 in, and shall be filled with bituminous mixture consist of bitumen, sand and cement.

3.14 Comparison of JPCP Thickness between AASHTO and PCA Design Method

Table 3.21 and 3.22 shows the pavement thicknesses obtained for road A and road B according to AASHTO and PCA method.

Table 3.21: Comparison of Thickness between AASHTO and PCA Design Method for Road A

Road A	AASHTO Design Method	PCA Design Method
Concrete Slab Thickness (in).	7	7
Sub Base Layer Thickness (in).	8	8
Total Pavement Thickness (in).	**15**	**15**

Table 3.22: Comparison of Thickness between AASHTO and PCA Design Method for Road B

Road B	AASHTO Design Method	PCA Design Method
Concrete Slab Thickness (in).	9	8
Sub Base Layer Thickness (in).	6	6
Total Pavement Thickness (in).	**15**	**14**

From Table 3.21 the comparison of the concrete pavement thickness between AASHTO and PCA procedure for Road A produced same slab thickness which is about 7 in (172mm) followed by same sub-base course thickness about 8 in (196 mm).
From table 3.22 the concrete slab thickness for Road B shows a value produced by AASHTO method is 1 in higher than PCA method which is

about 8 in (196 mm). Followed by the same sub-base course thickness produced by the two standards, which is about 6 in (147 mm).

The AASHTO design method was adopted for Jointed Plain Concrete Pavement (JPCP) for Road A and Road B.

CHAPTER FOUR
MATERIALS AND PAVEMENTS COMPARATIVE ANALYSIS

4.1 Background

Since the main components of flexible and rigid pavements are asphalt and cement, it was necessary to spot light on the two materials availability, price status and their future expected scenarios. This can assist a lot through providing better judgment for taking the appropriate decision regarding selecting one of the two pavement types.

A. Petroleum Oil

In the last few years a dramatic escalation caused in asphalt prices reflected in a 250% increase during 2005-2008. It is likely that once the economic recovery gains traction, large shortages may reappear, oil prices will rise and asphalt prices will resume their upward climb. From 2003 to 2008 oil prices increased nearly 300%. During the same period, liquid asphalt increased 250%.

The increases in asphalt prices during this period were not only a result of rising oil prices, but also by changes in oil refining practices which has led to a reduction in heavy crude production and reduced supply.

The global economic weakness has resulted in a 50% decline in oil prices during the past year. Despite reduced paving demand, asphalt prices have declined about 12% from record high levels during the same period.

Oil price given in Figure 4.1, 4.2, and 4.3 reflects significant changes in global energy demand. Emerging markets are increasingly becoming a major driver. Longer term world economic growth is expected to be characterized by developing and transitional economies adding new demand pressures on oil prices.

It is expected by 2015 that the oil prices are conservatively will exceed $133 per barrel, reestablishing past peaks. Longer term projections made by the

United States government suggest that oil prices may exceed $180 per barrel by 2030

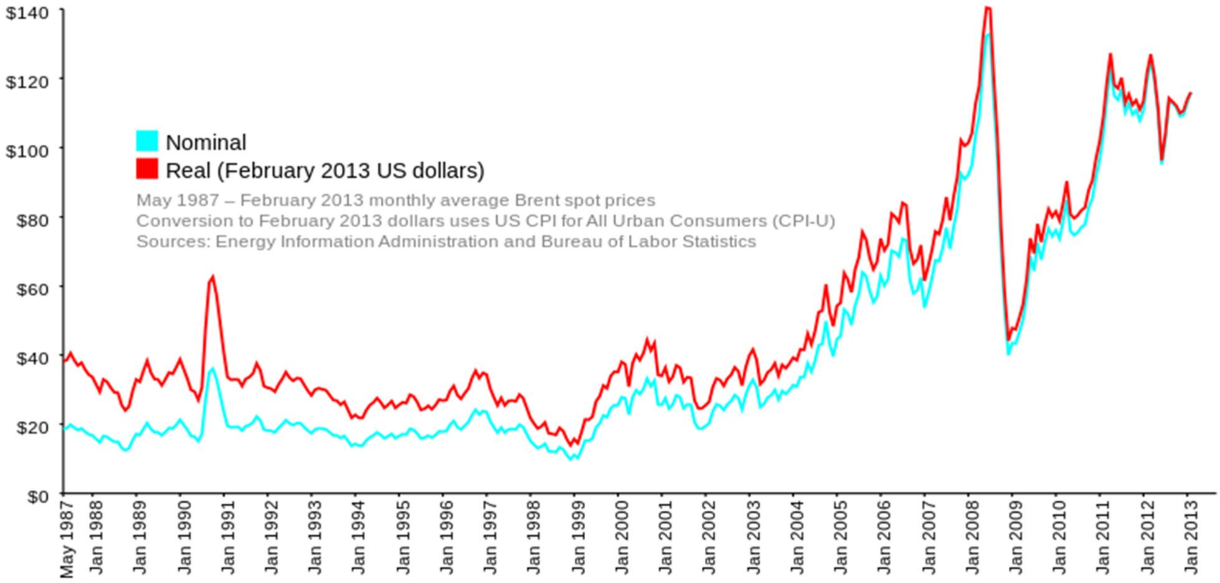

Figure 4.1: Medium term crude Petroleum oil prices since May 1987

Figure 4.2: Monthly and Daily West Texas Intermediate Oil Prices since 2000.

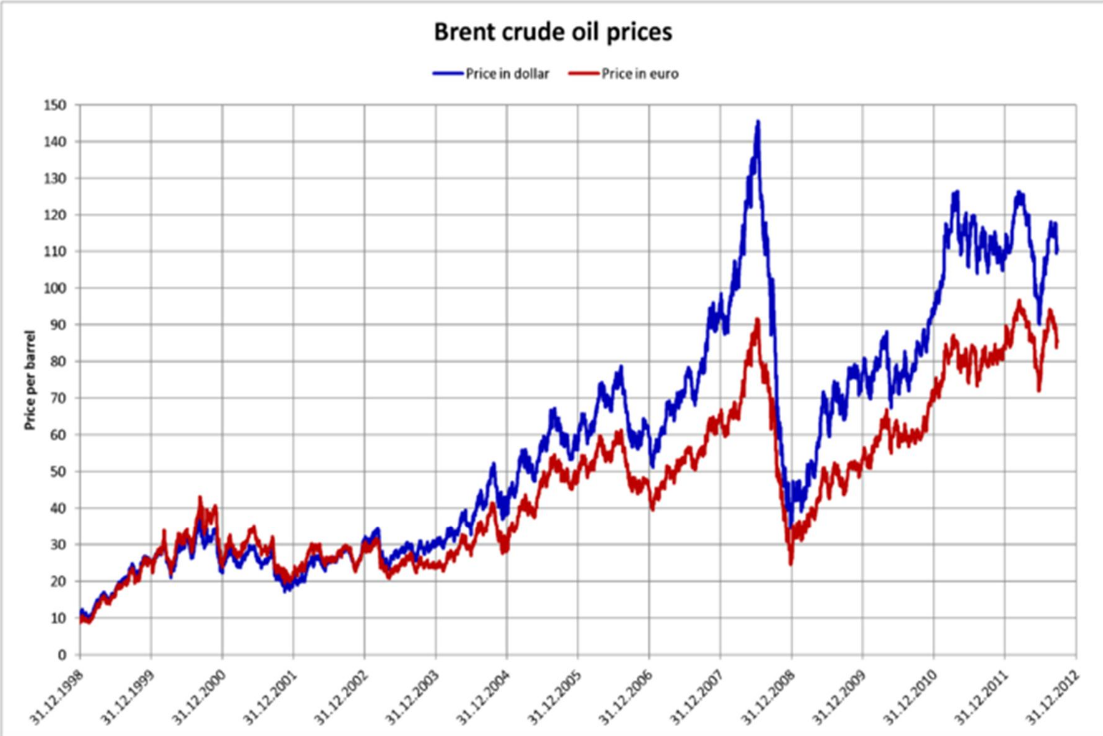

Figure 4.3: Petroleum Oil Prices for Brent in US$ (blue) and Euro (red)

B. Cement

Figure 4.4 shows the national cement production and consumption from 1999 to 2008.It is clearly shown that shortage in the cement production which remain constant up to 2007 in a range less than 500,000 tons per year in opposite of the strong demand that grow higher due to building development to reach 4 million ton per year of consumed cement in 2008.

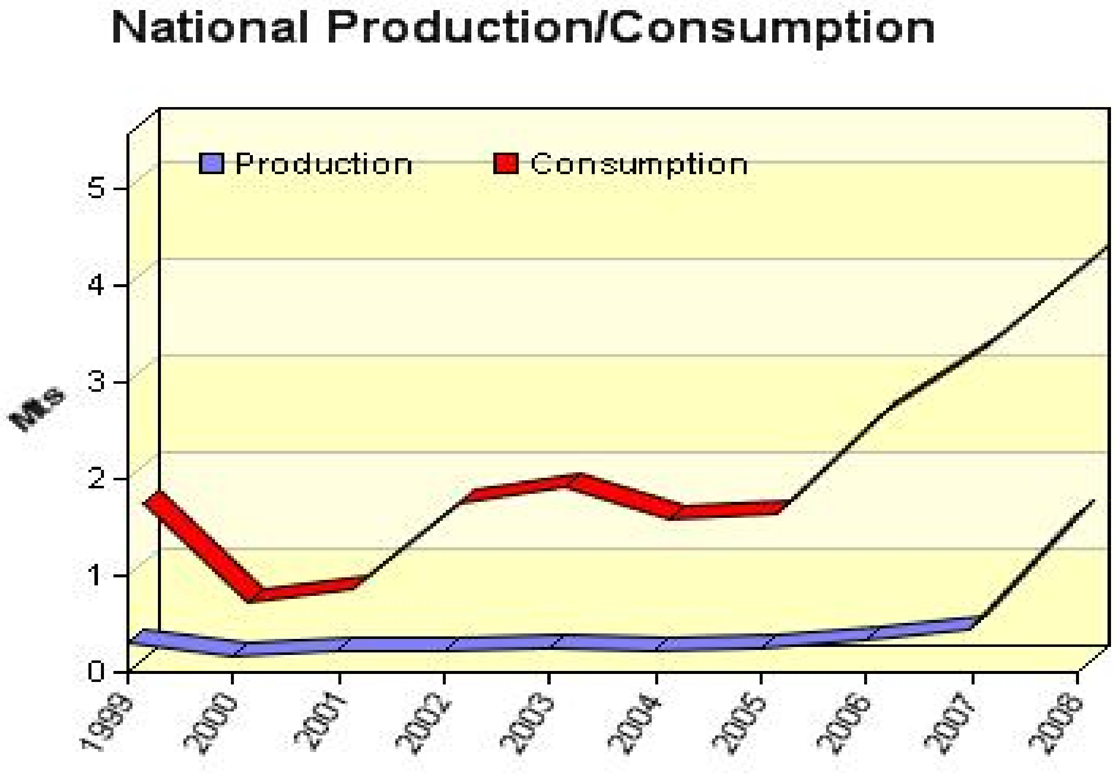

Figure 4.4: National Cement Production and Consumption from 1999 to 2008

Sudan achieved the self sufficiency of Cement commodity. The cement consumption reached to 3.0 Million ton per year while the production exceeded the 4 million tones. In addition the first half of the year 2012 achieved an income of about 7 million Us dollar as revenue from exporting about 79 .000 tons. (Report issued from the ministry of industry - 2013).

In spite the hike of the dollar and the energy prices, one ton of cement is about of 500 SDG comparing with last prices of 1000 SDG for a ton of Cement.

Now the investments in sector of cement reached 1.389 billion SDG while the total production of the operating factories is 7.4 million tons annually.

There are 7 factories in field of cement industry, 5 of them are in the state of Nile River, one factory in Rabak at the state of Blue Nile and one factory in Gezira state.

Table 4.1: Annual Cement Production and Prices 2007 – 2011

No	Year	Price in SDG	Production
1	2007	1000	328.779
2	2008	850	282.188
3	2009	570	624.506
4	2010	515	1646.37
5	2011	430	2987.22

4.2 Pavements Life Cycle Cost Analysis LCCA

The U.S. Federal Highway Administration [Walls, et al, 1998] describes LCCA as “an analysis technique that builds on the well-founded principles of economic analysis to evaluate the over-all-long-term economic efficiency between competing alternative investment options”. Comparing life-cycle costs has become standard for selecting among different pavement types, but also to evaluate different, feasible rehabilitation plans over the service life of pavements alternatives.

Life-cycle costing quantifies pavements initial construction and activity costs such as maintenance and rehabilitation over an analysis period could be extended to cover the design life.

Future costs are discounted to today’s rates by selecting a discount rate. The discount rate is a key factor in determining the net present value of future costs. Lower discount rates tend to favor pavements with long service lives and higher initial costs such as jointed concrete pavement.

Life cycle cost analysis is particularly important in answering the question, can the related authorities afford to continue to replace deteriorated asphalt pavements with more asphalt pavements given that its future costs will inevitably rise?

A. Life Cycle Cost Analysis Components

To evaluate the life-cycle cost, it is important to evaluate the initial costs, maintenance and rehabilitation costs in terms of several key parameters. These include the overall life-cycle costing assumptions such as the initial design, analysis period, discount rate, types and timing of maintenance and rehabilitation activities. All of these factors should be considered when comparing flexible and rigid pavements.

1. Road A and Road B, Initial Cost (Construction Cost).

Initial design and construction costs are typically the largest expenses over the life cycle.

The AASHTO structural design for Road A and Road B flexible and rigid pavements was selected to perform the comparative analysis. Estimation of the initial cost was made for the entire roads after preparing their quantities and activities.

1.1 Road A Flexible Pavement Structural Design and Quantities

The flexible pavement structural design was determined earlier in chapter three of this study as follows:

Design Traffic = 6.2×10^5 ESALs.

Improved Sub-grade M_R = 15,000 psi.

Asphalt Concrete Surface = 3in (76mm).
Untreated Base = 9in (229mm).
Untreated Sub-base = 6in (152mm).
Improved Sub-grade = 12in (305mm).

The other parameters necessary for calculation of Road A flexible pavement quantities:

Length of pavement = 60 Km.
Width of pavement = 7.3 m (two lanes).

Table 4.2 represent Road A structural layers quantities according to the above parameters.

Table 4.2: Road A Flexible Pavement Quantities

Description	Unit	Quantity
Improved sub-grade (Capping layer)	M^3	131,400
Untreated sub-base	M^3	65,700
Untreated base	M^3	100,740
Asphalt concrete surface	M^3	33,288

The costs were calculated according to current rates of Sudanese pound for the projects activities and the results had been presented in Table 4.3

Table 4.3: Road A Flexible Pavement Construction Cost

Item	Description	Unit	Quantity	Unit Price	Amount in SDG
1.	Provide and lying of selected materials for improved sub grade and reach the required thickness through adequate compaction.	Cu. M	131400	35	4,599,000
2.	Provide and lay of granular sub-base materials in uniform layers, to the required thickness with adequate compaction to achieve the desired density, as per technical specification.	Cu. M	65,700	60	3,942,000
3.	Provide and lay of granular base materials in uniform layers, to the required thickness with adequate compaction to achieve the desired density, as per technical specification.	Cu. M	100,740	90	9,066,600
4.	Provide and spraying single coat at low viscosity				

	bituminous prime coat over granular base with bitumen grade 80/100 @ 8 to 9 Kg/10 Sq. m	Sq. M	438,000	12	5,256,000
5.	Provide and laying HMA with bitumen grade 60/70 @ 5% of the weight of total mix. Laying to the required level and alignment.	Cu. M	33,288	1,700	56,589,600
Total					**79,453,200**

1.2 Road A JPCP Structural Design and Quantities

The jointed plain concrete pavement structural and components design according to AASHTO is summarized as follow:

Design Traffic = 9 x 10^5 ESALs.
Design k = 250 pci.
JPCP Slab Thickness = 7in (17.8mm).
JPCP Slab Length = 53ft (16m).
JPCP Slab Width = 12ft (3.65m).
Untreated Sub-base Thickness = 8in (203mm).

Table 4.4: Road A Jointed Plain Concrete Pavement Quantities

Description	Unit	Quantity
Untreated sub-base	M^3	88,914
Tie Bar	Ton	20.4
Dowel Bar	Ton	203
Cement Concrete surface	M^3	78,840

The costs were calculated according to current rates of Sudanese pound for the projects activities and the results had been presented in Table 4.5

Table 4.5: Road A JPCP Construction Cost

Item	Description	Unit	Quantity	Unit Price	Amount in SDG
1.	Provide and lay of granular sub-base materials in uniform layers, to the required thickness with adequate compaction to achieve the desired density, as per technical specification.	Cu. M	88,914	60	5,334,840
2.	mix, spread, level and float concrete mix 1:2:4 to the thickness of 23 cm. the rate include replacement of dowel bars (32mm size, 0.4m length) at 30 cm c/c in transverse joints and tie bars (12mm size, 0.6m length at 1m c/c in longitudinal joints, also include surface texturing with one of the known methods and concrete curing with plastic cover or chemical liquid membrane as per specifications.	Cu.M	78,840	700	55,188,000
3.	Saw, insert joints and fill it with recommended filler after concrete harden as per specifications.	L. M	109,500	30	3,285,000
Total					**63,807,840**

1.3 Road B Flexible Pavement Structural Design and Quantities

The flexible pavement structural design was also determined in chapter three of this study as follows:

Design Traffic = 5.24 x 10^6 ESALs.
Improved Sub-grade M_R = 12,000 psi
Asphalt Concrete Surface = 3in (76mm).
Untreated Base = 8in (203mm).
Untreated Sub-base = 12in (305mm).
Improved Sub-grade = 8in (205mm).

The other parameters necessary for calculation of Road B flexible pavement quantities:

Length of pavement = 7 Km.
Width of pavement = 21.9 m (Six lanes).

Table 4.6: Road B Flexible Pavement Quantities

Description	Unit	Quantity
Improved sub-grade	M^3	38,500
Untreated sub-base	M^3	46,970
Untreated base	M^3	31,262
Asphalt concrete surface	M^3	11,704

The costs were calculated according to current rates of Sudanese pound for the projects activities and the results had been presented in Table 4.5

Table 4.7: Road B Flexible Pavement Construction Cost

Item	Description	Unit	Quantity	Unit Price	Amount in SDG
1.	Cut the existing sub-grade soil to depth of 25 cm and process the excavated soil to perform embankment layer with adequate compaction as per specification.	Cu. M	38,500	25	962,500
2.	Provide and lay of granular sub-base materials in uniform layers, to the required thickness with adequate compaction to achieve the desired density, as per technical specification.	Cu. M	46,970	56	2,630,320
3.	Provide and lay of crushed stone base materials in uniform layers, to the required thickness with adequate compaction to achieve the desired density, as per technical specification.	Cu. M	31,262	80	2,500,480
4.	Provide and spraying single coat at low viscosity bituminous prime coat over granular base with bitumen grade 80/100 @ 8 to 9 Kg/10 Sq. m	Sq. M	154,000	10	1,450,000
5.	Provide and laying				

	HMA with bitumen grade 60/70 @ 5% of the weight of total mix. Laying to the required level and alignment.	Cu. M	11,704	1,375	16,093,000
Total					**23,726,300**

1.4 Road B JPCP Structural Design and Quantities

The jointed plain concrete pavement structural and components design according to AASHTO is summarized as follow:

Design Traffic = 7.7 x 10^6 ESALs.
Design k = 300 pci.
JPCP Slab Thickness = 9in (230mm).
JPCP Slab Length = 47.5ft (14.5m).
JPCP Slab Width = 12ft (3.65m).
Untreated Sub-base Thickness = 6in (152mm).

Table 4.8: Road B Jointed Plain Concrete Pavement Quantities

Description	Unit	Quantity
Untreated sub-base	M^3	23,408
Tie Bar	Ton	12
Dowel Bar	Ton	78.4
Cement Concrete surface	M^3	35,420

The costs were calculated according to current rates of Sudanese pound for the projects activities and the results had been presented in Table 4.5

Table 4.9: Road B JPCP Construction Cost

Item	Description	Unit	Quantity	Unit Price	Amount in SDG
1.	Provide and lay of granular sub-base materials in uniform layers, to the required thickness with adequate compaction to achieve the desired density, as per technical specification.	Cu. M	23,408	56	1,310,848
2.	mix, spread, level and float concrete mix 1:2:4 to the thickness of 23 cm. the rate include replacement of dowel bars (32mm size, 0.4m length) at 30 cm c/c in transverse joints and tie bars (12mm size, 0.6m length at 1m c/c in longitudinal joints, also include surface texturing with one of the known methods and concrete curing with plastic cover or chemical liquid membrane as per specifications.	Cu. M	35,420	650	23,023,000
3.	Saw, insert joints and fill it with recommended filler after concrete harden as per specifications.	L. M	31580	20	631600
Total					**24,965,448**

2. Road A and Road B Maintenance and Rehabilitation Cost

One of the key components for the evaluation of total costs over the pavement life-cycle is in estimating maintenance and rehabilitation costs. This is typically accomplished by reviewing the potential activities that will occur throughout the service life of a pavement, their frequency, and costs.

2.1 Maintenance and Rehabilitation Plan

The maintenance and rehabilitation (M&R) plan is established as a typical scenario to maintain the pavement in a cost-effective and serviceable manner. It reflects the maintenance and rehabilitation activities as well as the timing and quantity for each activity. These activities typically include mill and overlay for flexible pavement, concrete pavement restoration (e.g. slab repairs, crack sealing, etc.) for rigid pavement.

A typical maintenance and rehabilitation plans for flexible and rigid pavement structure are given in Table 4.10 and 4.11

Table 4.10: Flexible Pavement Maintenance and Rehabilitation Plan.

YEAR	ACTIVITY	QUANTITY (%)
5	Rout and Seal Cracks	10
10	Machine Patching	15
15	Mill and Overlay (50 mm)	100

Table 4.11: Rigid Pavement Maintenance and Rehabilitation Plan.

YEAR	ACTIVITY	QUANTITY (%)
5	Reseal Joints	10
15	Minor Concrete Pavement Repair	10
20	Reseal Joints	15

B. Estimating Total Life Cycle Cost

Total life-cycle cost combines estimated initial costs and the future maintenance and rehabilitation costs for each alternative. The costs are expressed in today Sudanese pound as a net present value.

The required inputs include:

1. General inputs:
   - Analysis period.
   - Discount rate.
   - Site description/dimensions.

2. All pavement types.
   - Unit costs.
   - Initial pavement layer thickness.
   - Maintenance and rehabilitation plan and quantities.

1. Calculation of Net Present Value (NPV)

The NPV represents the total cost today that would be required, accounting for the interest and inflation expressed as the discount rate. The NPV of all activities are summed up to estimate the total maintenance and rehabilitation cost.

$$\mathbf{NPV} \sum_{\mathrm{i}}^{n} \frac{\mathbf{M}\&R\ Cost}{(\mathbf{1+Discount\ Rate})\mathbf{n}} \tag{4.1}$$

Where:

Discount Rate (%) = Interest Rate (%) – Inflation Rate (%) (4.2)

2. Pavement Residual Value

The residual value is estimated by linear depreciation of the last capital activity cost. The prorated life method is used in the LCCA procedure to estimate the residual value. The recoverable cost is estimated by dividing the remaining life of the last rehabilitation treatment, by the expected life of the treatment.

Residual Value = Last Rehab Cost x [(Service Life - Activity Age) / Service Life] (4.3)

C. Road A and Road B Life Cycle Cost (LCC)

The total cost to construct and maintain each design option is the key focus of a LCCA. To accomplish this, the total sum of all costs, in equivalent NPV is required. The total cost is thus calculated as:

LCC = (Initial Cost + Total Discounted M&R Cost) – Residual Value

(4.4)

This value can then be used to benchmark other potential options and determine which is the most cost effective.

1. Road A Flexible Pavement Maintenance and Rehabilitation Cost

Table 4.12 shows the maintenance and rehabilitation costs were calculated according to the plan presented in Table 4.10

Table 4.12: Road A Flexible Pavement Maintenance and Rehabilitation Cost

Item	Activity	Unit	Quantity	Unit Rate	Total Amount in SDG
1	Rout and Seal Cracks	M^2	43,800	10	438,000
2	Machine Patching	M^2	65,700	120	7,884,000
3	Mill and Overlay	M^3	21,900	1,720	37,668,000

All the activities over the analysis period which was selected to be 20 years the design life of the two roads pavements, are summed up by using Equation 4.1, to obtain the net present value of maintenance and rehabilitation cost.

The discount rate was used to conduct the analysis is 5%.

Total M&R Cost = 438,000 / $(1.05)^5$ + 7,884,000 / $(1.05)^{10}$ + 37,668,000 / $(1.05)^{15}$

= 23,302,228 SDG.

Last major rehabilitation cost = 37,668,000 / $(1.05)^{15}$ = 18,118,952 SDG

Service life = 15 years
Activity age = 5
Residual Value = 18,118,952 x (15-5) /15
= 12,079,301 SDG

2. Road A JPCP Maintenance and Rehabilitation Cost

Maintenance and rehabilitation activities are typically scheduled to maintain and improve the serviceability of the concrete pavement as presented in Table 4.11.
The maintenance and rehabilitation costs via the above plan were calculated and tabulated in Table 4.13

Table 4.13: Road A JPCP Maintenance and Rehabilitation Cost

Item	Activity	Unit	Quantity	Unit Rate	Total Amount in SDG
1	Reseal Joints	L.M	1,095	15	16,425
2	Minor Concrete Pavement Repair	M^2	43,800	100	4,380,000
3	Reseal Joints	L.M	2,190	15	32,850

The same procedure mentioned above was followed to calculate total maintenance and rehabilitation costs for road A JPCP.

Total M&R Cost = 16,425 / $(1.05)^5$ + 4,380,000 / $(1.05)^{15}$ + 32,850 / $(1.05)^{20}$
= 2,132,105 SDG

Last major rehabilitation cost = 4,380,000 / $(1.05)^{15}$ = 2,106,855 SDG
Service life = 10 years
Activity age = 5
Residual Value = 2,106,855 x (10-5) /10
= 1,053,427 SDG

3. Road B Flexible Pavement Maintenance and Rehabilitation Cost

Maintenance and rehabilitation costs were calculated according to the same plan presented in Table 4.10

Table 4.14 Road B Flexible Pavement Maintenance and Rehabilitation Cost

Item	Activity	Unit	Quantity	Unit Rate	Total Amount in SDG
1	Rout and Seal Cracks	M^2	15,400	8	123,200
2	Machine Patching	M^2	23,100	100	2,310,000
3	Mill and Overlay	M^3	7,700	1,400	10,780,000

The same analysis was done to calculate total net present value of maintenance and rehabilitation cost

Total M&R Costs = 123,200 / $(1.05)^5$ + 2,310,000 / $(1.05)^{10}$ + 10,780,000 / $(1.05)^{15}$ = 6,700,034 SDG

Last major rehabilitation cost = 10,780,000/ $(1.05)^{15}$ = 5,185,364 SDG
Service life = 15 years
Activity age = 5
Residual Value = 5,185,364 x (15-5) /15
= 3,456,909 SDG

4. Road B JPCP Maintenance and Rehabilitation Cost

Maintenance and rehabilitation costs were calculated according to the same plan presented in Table 4.11

Table 4.15: Road B JPCP Maintenance and Rehabilitation Cost

Item	Activity	Unit	Quantity	Unit Rate	Total Amount in SDG
1	Reseal Joints	L.M	3,158	12	37,896
2	Minor Concrete Pavement Repair	M^2	15,400	60	924,000
3	Reseal Joints	L.M	6,316	12	75,792

The same procedure mentioned above was followed to calculate total maintenance and rehabilitation costs for Road B JPCP.

Total M&R Cost = 37,896 / $(1.05)^5$ + 924,000 / $(1.05)^{15}$ + 75,792 / $(1.05)^{20}$

= 502,718 SDG

Last major rehabilitation cost = 924,000 / $(1.05)^{15}$ = 444,460 SDG

Service life = 10 years

Activity age = 5

Residual Value = 444,460 x (10-5) /10

= 222,230 SDG

CHAPTER FIVE
RESULTS AND DISCUSSION

5.1 Results

The life cycle cost for each pavement type of the two road projects were determined by using the present worth of the initial cost and maintenance and rehabilitation cost. The residual value for each pavement type was calculated to ensure fair comparison between alternatives.

The summary of overall results for the two roads pavements were tabulated in Table 5.1and Table 5.2 and figure 5.1.

Table 5.1: Summary of Road A Present worth Life Cycle Cost Analysis

Pavement Type	Initial Cost in SDG	Maintenance and Rehabilitation Cost in SDG	Residual Value	Total Present Worth of Costs
Flexible Pavement	79,453,200	23,302,228	12,079,301	90,676,127
JPCP	63,807,840	2,132,105	1,053,427	64,886,518

Table 5.2: Summary of Road B Present worth Life Cycle Cost Analysis

Pavement Type	Initial Cost in SDG	Maintenance and Rehabilitation Cost in SDG	Residual Value	Total Present Worth of Costs
Flexible Pavement	23,726,300	6,700,034	3,456,909	26,964,925
JPCP	24,965,448	502,718	222,230	25,245,936

From the life cycle cost analysis the total present worth of costs for Road A two pavement types shows advantage of using JPCP in favor of flexible pavement with saving of 25,789,609 SDG which demonstrate roughly 28% savings.

For Road B, the JPCP total present worth of costs is lower by 1,718,989 SDG from flexible pavement total present worth of costs with advantage of 6% saving.

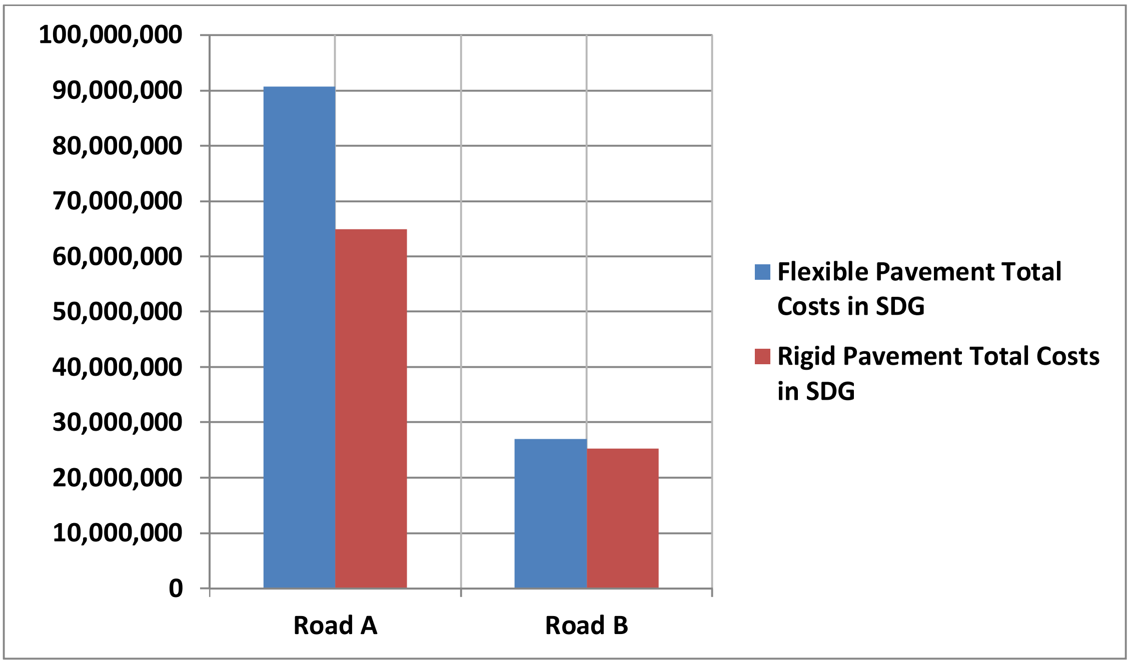

Figure 5.1: Comparison between Flexible and Rigid Pavements Total Costs for Road A and Road B

5.2 Discussion

It's noticeable that the advantage of concrete paving over asphalt is much greater in case of Road A which produced 28 % saving in cost. The main reason for this great reduction goes to the type of sub-grade soil, which has been classified as black cotton soil as mentioned earlier. This type of soil needs either stabilization with appropriate stabilizer material such as lime or to be excavated and removed and replaced with capping layer in case of using flexible pavement.

The structural capacity of the rigid pavement is largely provided by the slab itself. For the common range of sub-grade soil strength, the required rigidity for a concrete slab can be achieved without much variation in slab thickness.

The effect of sub-grade soil properties on the thickness of rigid pavement is therefore much less important than in the case of flexible pavement.

The concrete slab is applied over one layer of sub base or base course which may be constructed of granular materials, cement treated materials, lean concrete or open graded over native sub-grade to prevent pumping of sub-grade soil and to reduce the concrete slab critical stress. Increment in the slab thickness associated with the increasing of traffic loading the road experienced.

Flexible pavement required strong sub-grade to make a good performance with combination of the above layers.

Globally, the price change of asphalt and concrete pavement given in Figure 5.2 is calculated by PCA for a one mile "standard" two lane roadway. Asphalt had a $280,000 cost advantage over a concrete paved road in 2003 – roughly a 48% advantage. With 2009 prices concrete paved roads, has an $82,000 cost advantage over asphalt paved roads. And with 2010 prices concrete roads are $58,500 cheaper per one mile "standard" two lane roadway.

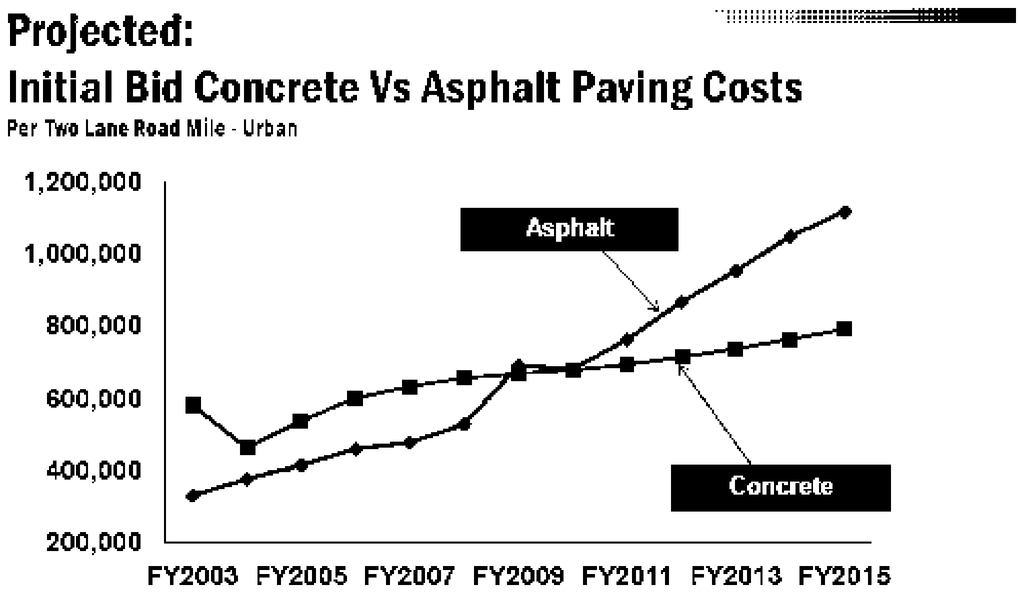

Figure 5.2: Initial cost of asphalt and concrete pavements (PCA)

In the longer term oil prices are expected to reach more than $180 per barrel by 2030 according to the Energy Information Agency (EIA). So comparative initial bid costs will increasingly favor concrete paved roads. PCA estimates by 2015 concrete paved roads will enjoy a $500,000 initial bid cost advantage over asphalt for a one mile “standard” two lane roadway – roughly a 41% savings.

CHAPTER SIX
SUMMARY, CONCLUSIONS AND RECOMMANDATIONS

6.1 Summary

This study was conducted to characterize and compare currently available used pavement types, flexible and rigid pavement design methods without ignoring that the pavement total cost is generally the major factor in deciding the type of pavement to be constructed.

Two roads were selected as a case study for this research. Road A illustrate the national road and named as El Moneera- El Saffaya road. Road B is section of Omdurman Ring Road represent the state road.

Field data of traffic surveying and materials tests of the two roads were gathered to determine the traffic loading and sub-grade soil strength. Cumulative equivalent axle load (ESAL), resilient modulus (M_R) obtained through correlation with CBR and modulus of sub-grade reaction (k) modified by AASHTO were calculated to be used for flexible and jointed plain concrete pavements.

The AASHTO and Asphalt Institute design methods were implemented for flexible pavement and AASHTO & PCA design methods were applied to determine JPCP slab thickness. The slab dimensions, tie and dowel bars quantities and joints design were calculated according to AASHTO.

A comparison was conducted for pavement thickness and materials of the two roads flexible pavement designed according to AASHTO and AI and concrete pavement designed according to AASHTO and PCA. The comparison encouraged to adopt flexible and concrete pavement cross section designed with AASHTO.

The life cycle cost analysis (LCCA) was done for Road A and Road B flexible and concrete pavement. The two components of LCCA are construction and maintenance costs.

For calculating of construction costs, the two roads activities, quantities and rates worksheets were prepared. The road project life Maintenance plan was assumed in aim to derive the total present maintenance costs for Road A and Road B adopted flexible and concrete pavement cross sections.

The comparison was conducted between flexible pavement and jointed plain concrete pavement (JPCP) of Road A and B in total present costs. The results show advantage of applying JPCP in favor of flexible pavement for Road A with saving of 28%. For Road B the saving in cost is not far only 6% obtained through implementing of JPCP instead of flexible pavement.

6.2 Conclusions

The life cost analysis outlined in this study and applied for the two study cases, Road A and Road B flexible pavement and JPCP clearly shows the whole life cost advantage of using JPCP as a pavement surface for national highway and state road in Sudan.

The availability of natural aggregate in many areas in the country will further reduce the pavement cost compared to flexible pavements due to their suitability for use in rigid pavement. On the other hand, for asphalt pavement which uses crushed aggregate, quarry must be sought for sources suitable for crushed stone production.

Recent studies show some advantages of concrete paving. It can be summarized as providing smoother, durable and safer riding surface. Concrete pavement can be designed for 40 years and more. It has an average life of 30 to 35 years according to FHWA. Also required less annual maintenance comparing with flexible pavements and when repairs are necessary they are in small scope. Concrete pavement provides less fuel consumption and maintenance costs for car owners. Due to its reflectivity, roads required less lighting this saves electricity energy and reduction in light poles installation and maintenance cost. Many other benefits make concrete pavement the best choice.

6.3 Recommendations

The following recommendations pertain within the scope of this study:

1. It is recommended to make more traffic studies periodically in most parts of country to develop mathematical models relating axle load distribution which gives economical way of determined ESAL without going for axle load survey. Also this way gives more certainty for growth rates of different vehicles classes which can be used in determining of design traffic.

2. It is recommended to use JPCP in areas with poor sub-grade consisting of black cotton soil, due to its advantage over the flexible pavement as what has been investigated through the Road A with roughly 28% saving. . This kind of soil existing in the following country states (Kasala State, El Gadarif State, Blue Nile State, Sinar State, El Jazeera State, South Kurdufan State, South Darfour State, and South White Nile (the Rich Savanna areas).

3. It is recommended to make more comparative studies for Khartoum State for flexible and JPCP using different volume of traffic loading and soil strength. These studies may increase the cost saving of 6 % which was determined through Road B cost analysis and make clear view regarding using JPCP as cost effective option.

4. The government shall find some economical solutions for fuel rising rates especially for the industrial sector and prevent constrains that might decelerate investment in cement production industry the very important material in development process. This will encourage more investments in this field reflects in more rates reduction which makes rigid pavement the best choice.

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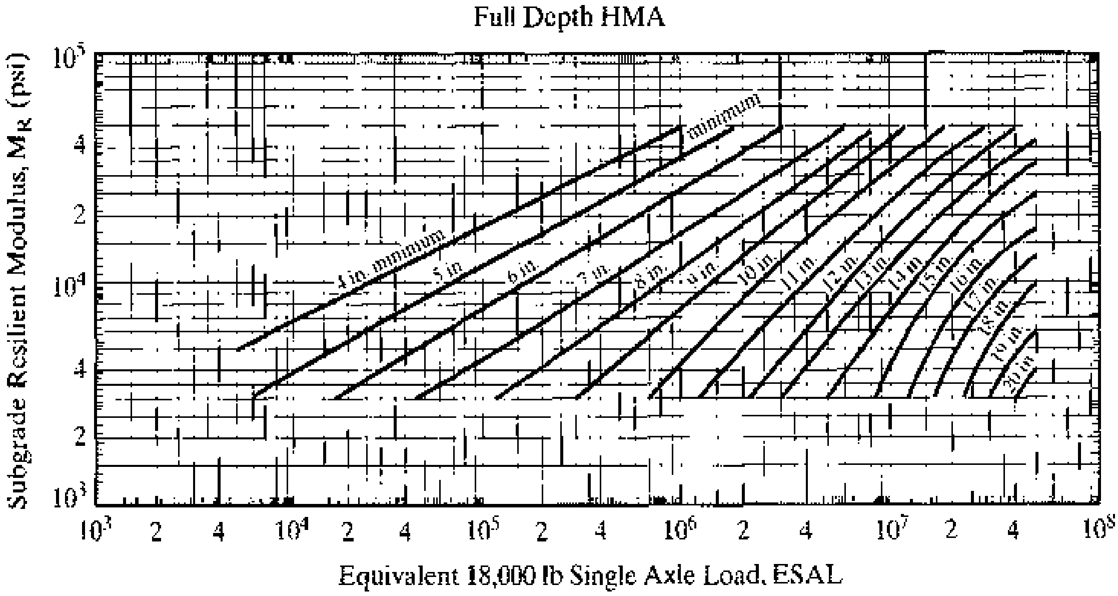
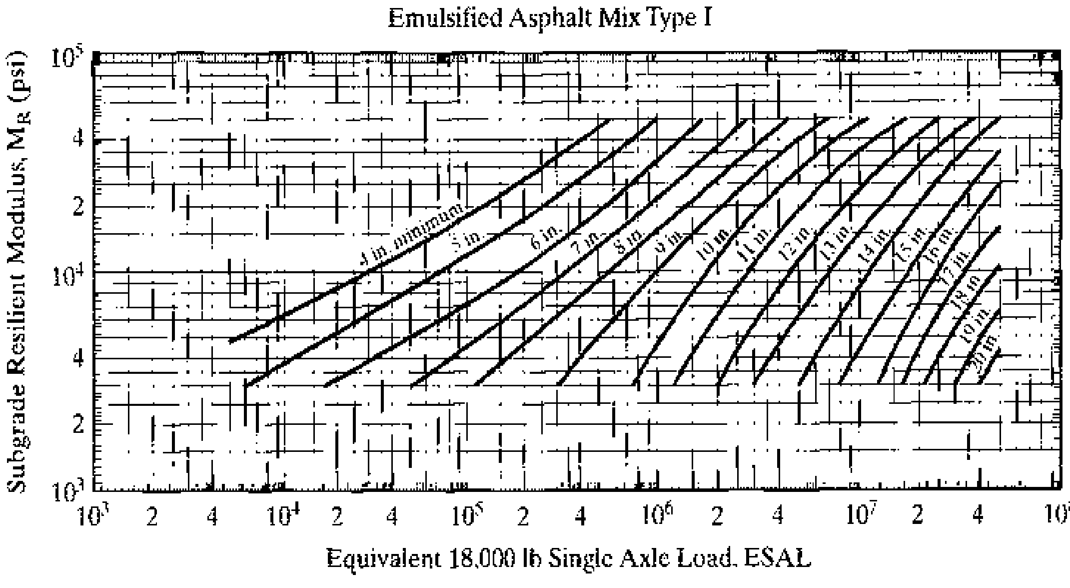
APPENDIX A
PAVEMENT DESIGN

Figures:



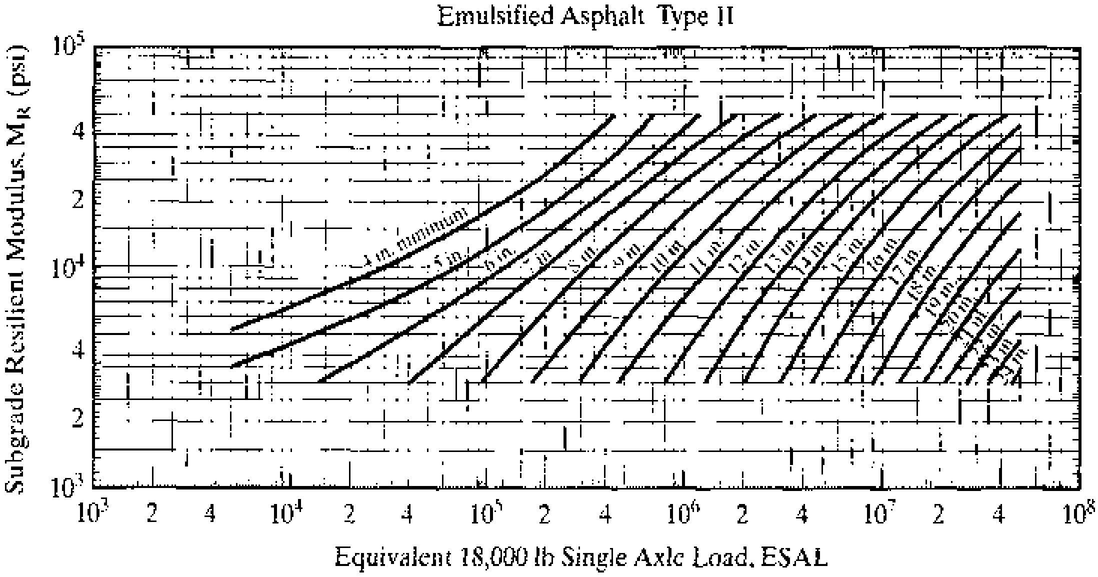
Source: After AI 1981a

Figure A-1: Design Chart for Full Depth HMA



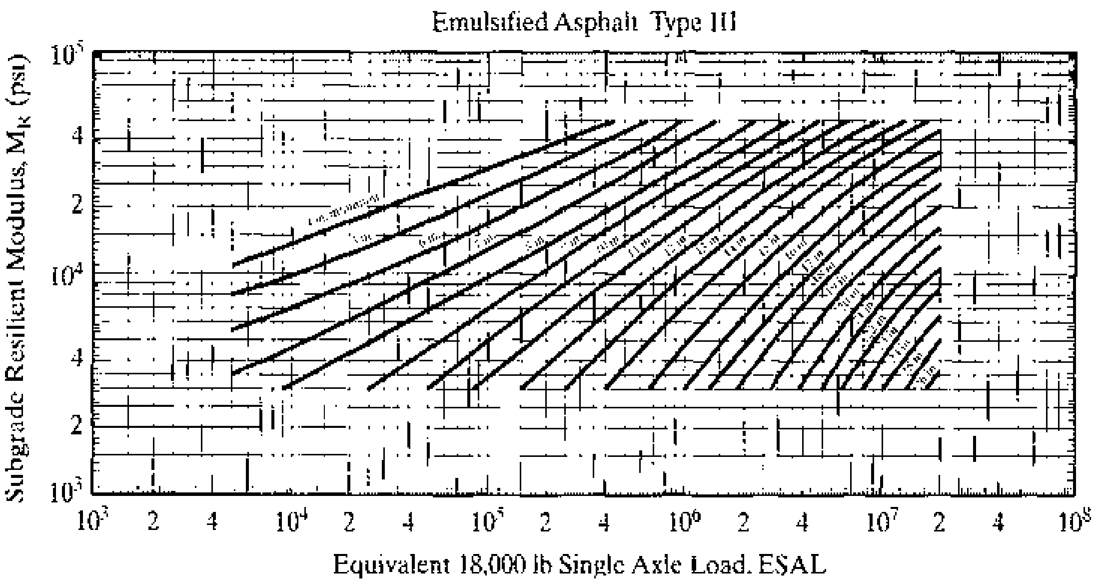
Source: After AI 1981a

Figure A-2: Design Chart for Emulsified Asphalt Mix Type I



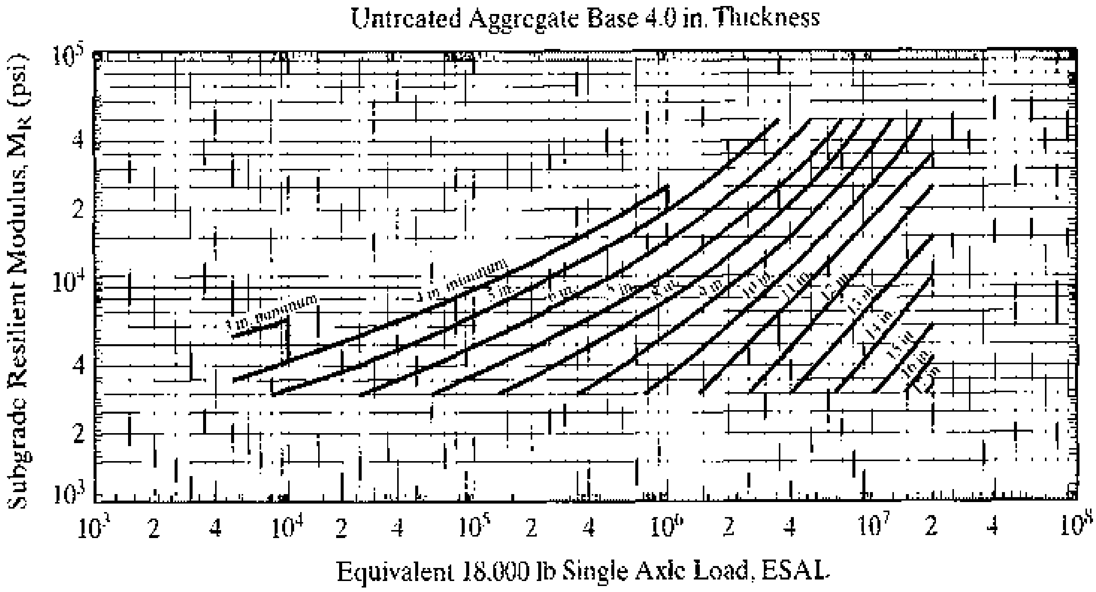
Source: After AI 1981a

Figure A-3: Design Chart for Emulsified Asphalt Type II



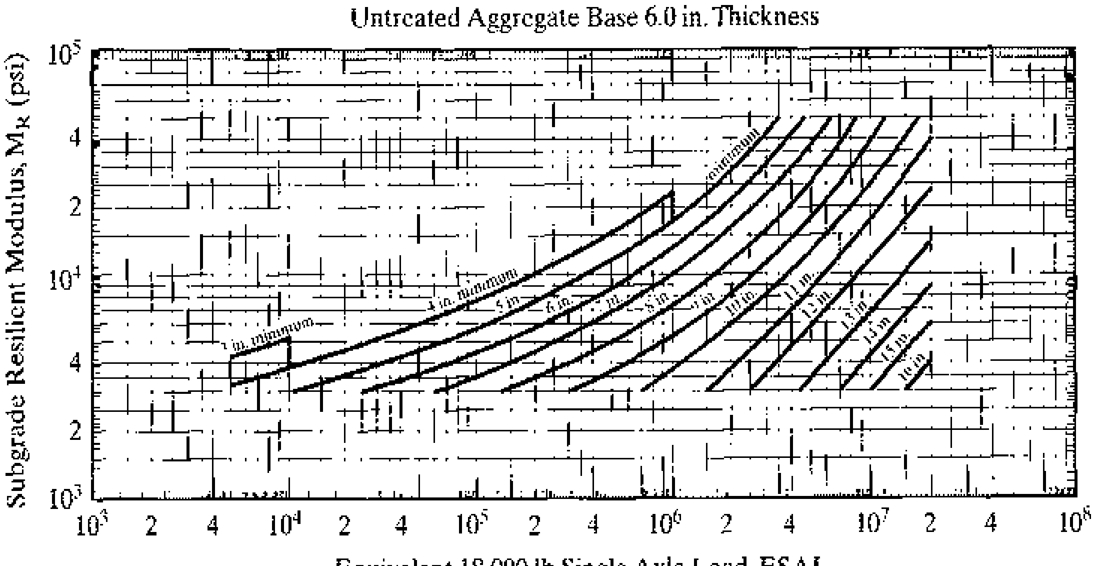
Source: After AI 1981a

Figure A-4: Design Chart for Emulsified Asphalt Type III



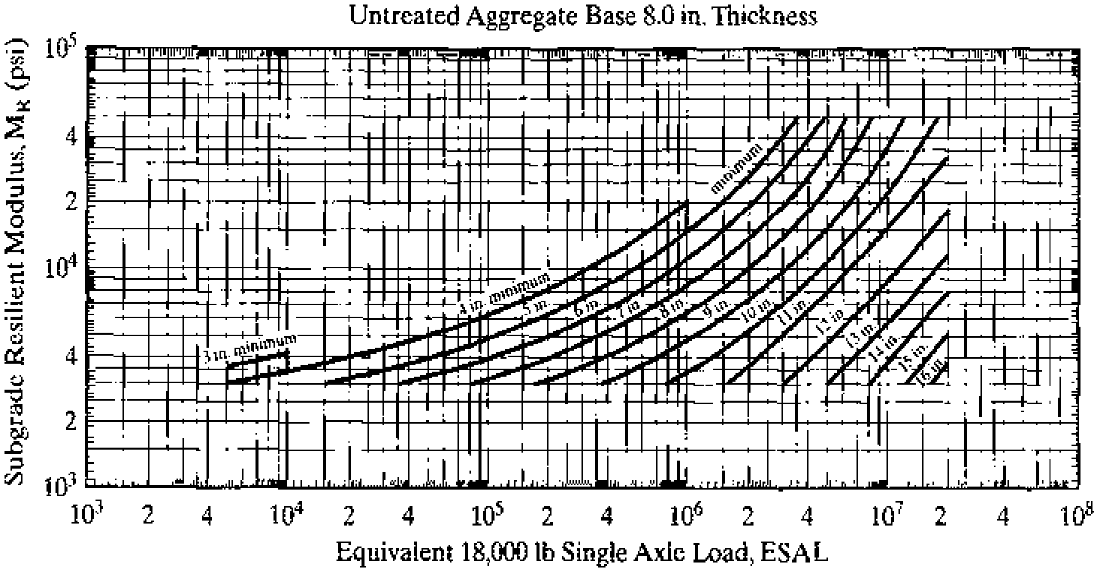
Source: After AI 1981a

Figure A-5: Design Chart for HMA Over 4in Untreated Base



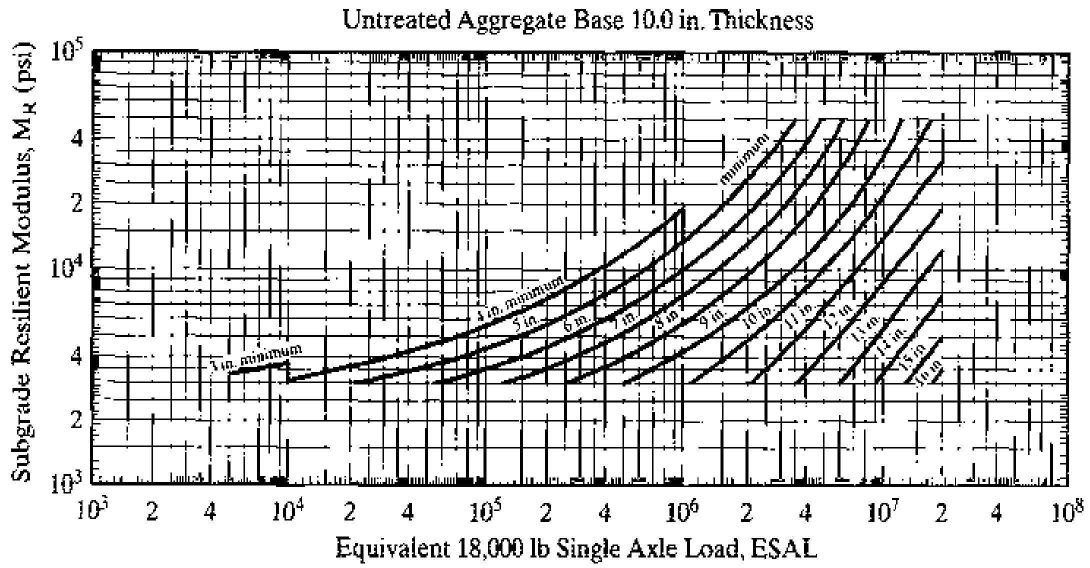
Source: After AI 1981a

Figure A-6: Design Chart for HMA Over 6in Untreated Base



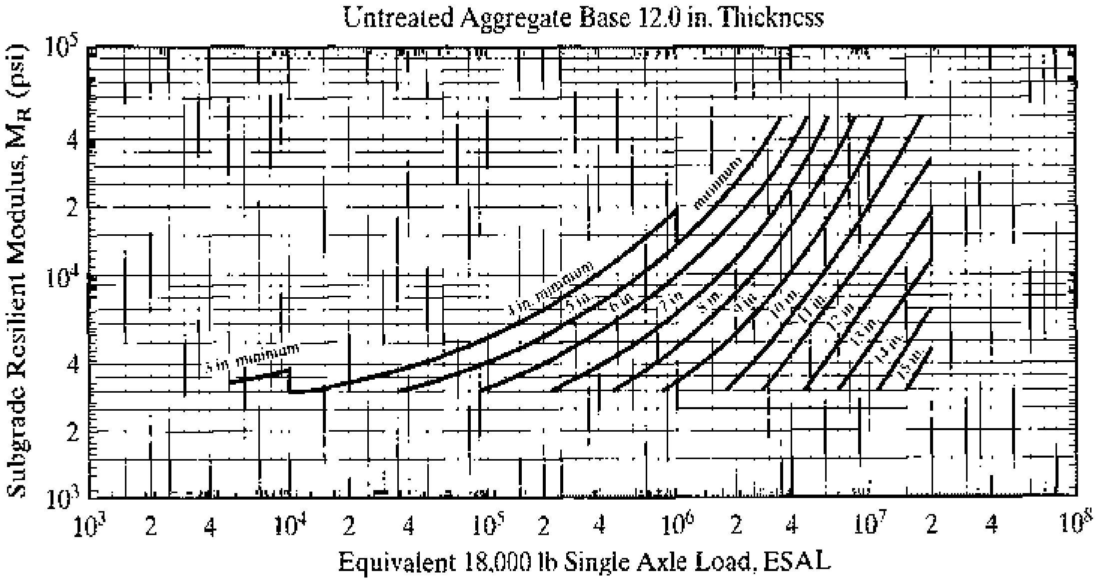
Source: After AI 1981a

Figure A-7: Design Chart for HMA Over 8in Untreated Base



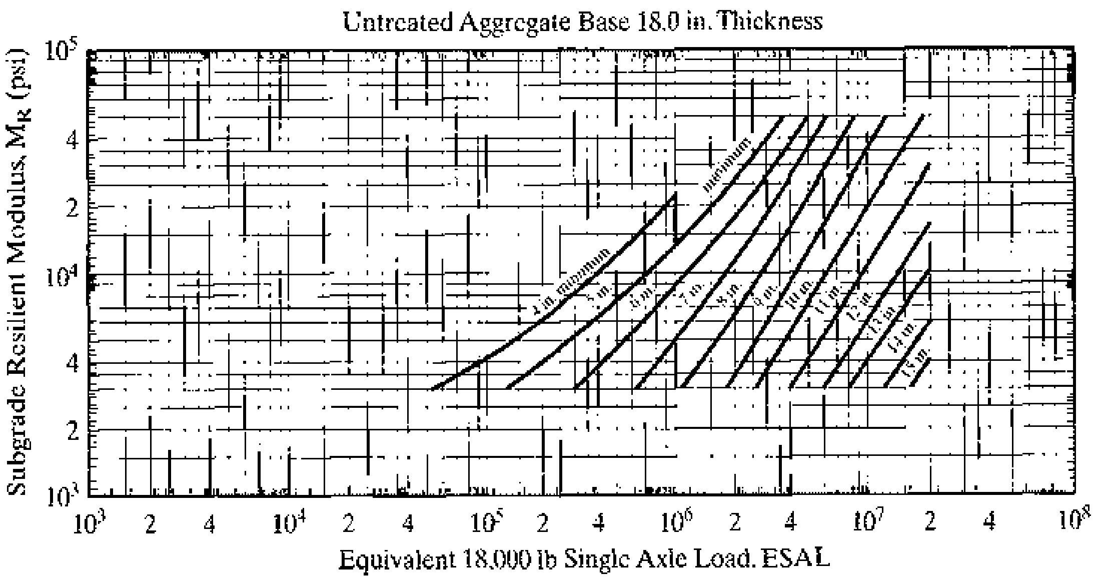
Source: After AI 1981a

Figure A-8: Design Chart for HMA Over 10in Untreated Base



Source: After AI 1981a

Figure A-9: Design Chart for HMA Over 12in Untreated Base



Source: After AI 1981a

Figure A-10: Design Chart for HMA Over 18in Untreated Base

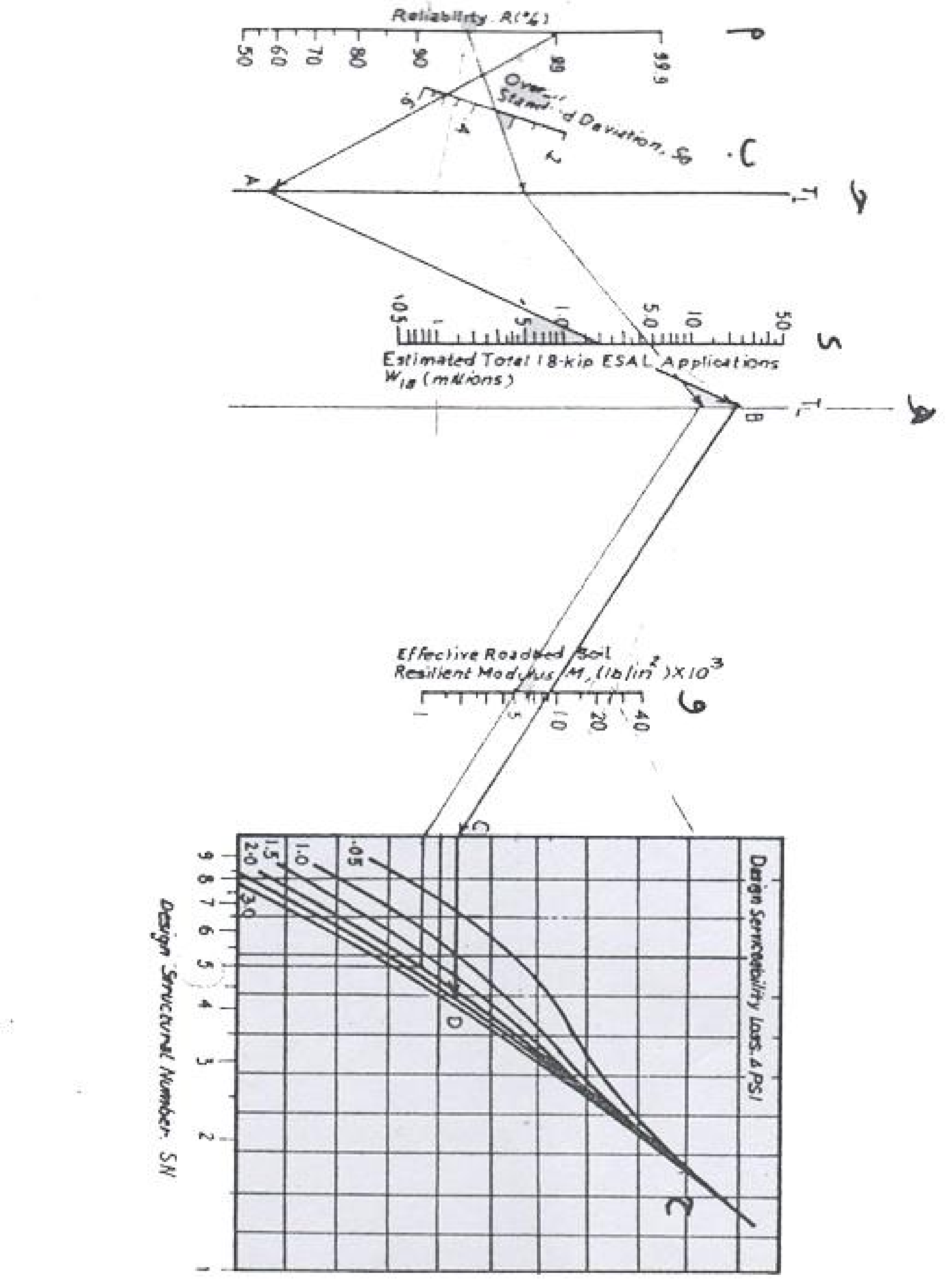


Figure A-11: Design Chart for Flexible Pavement Based on the Mean Values of Each Input.
(from AASHTO design guide of pavement structure 1986)

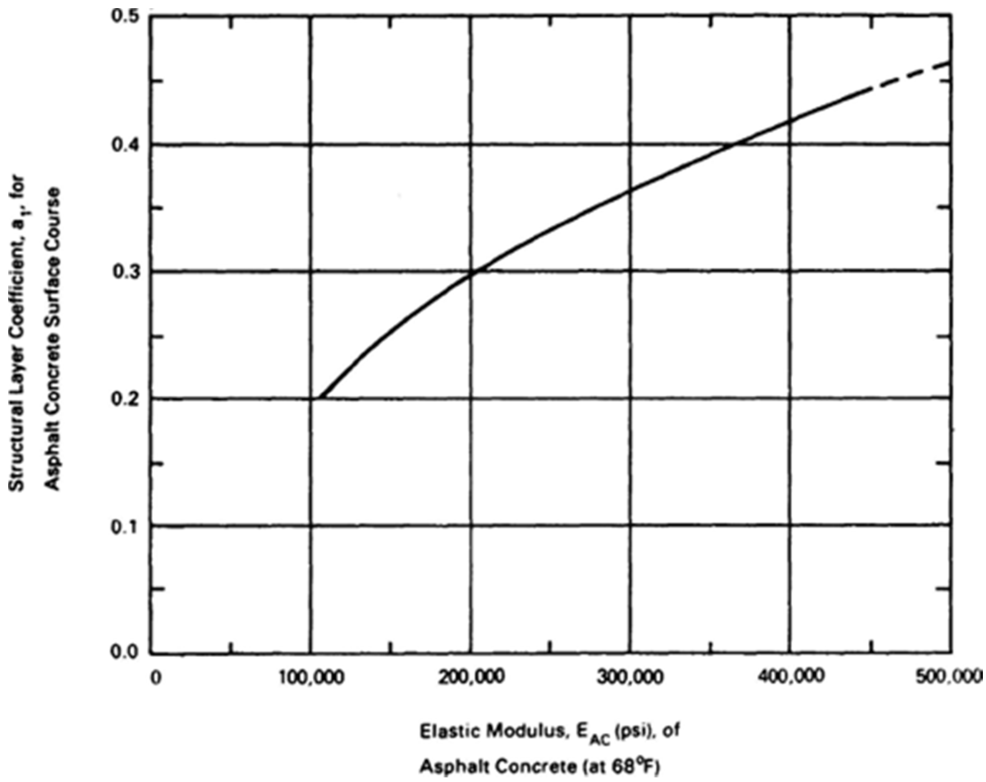


Figure A-12: Chart for Estimating Layer Coefficient of Dense-Graded Asphalt Concrete based on Elastic Modulus. (After Van Til et al. (1972))

Tables:

Table A-1: Minimum Thickness of HMA over Emulsified Asphalt Bases.

Traffic level ESAL	HMA thickness for Type I mix (in.)	HMA thickness for type II and type III mixes (in.)
10^4	1	2
10^5	1.5	2
10^6	2	3
10^7	2	4
$>10^7$	2	5

Source: After AI(1981a)

Table A-2: Suggested Levels of Reliability for Various Functional Classifications

Functional Classification	Recommended level of reliability	
	Urban	Rural
Interstate & other freeways	85 – 99.9	80-99.9
Principle arterials	80 – 99	75 – 95
Collectors	80 – 95	75 – 95
Locals	50 – 80	50 – 80

Note. Results based on a survey of AASHTO Pavement Design Task Force.
Source: After AASHTO (1986).

Table A-3: Standard Normal Deviates for Various Levels o f Reliability

Reliability (%)	Standard normal Z_R	Reliability (%)	Standard normal Z_R
50	0.000	93	-1.476
60	-0.253	94	-1.555
70	-0.524	95	-1.645
75	-0.674	96	-1.751
80	-0.841	97	-1.881
85	-1.037	98	-2.054
90	-1.282	99	-2.327
91	-1.340	99 .9	-3.090
92	-1.405	99 .99	-3.750

Table A-4: Recommended Drainage Coefficients for Untreated Bases and Sub-bases in Flexible Pavements

Quality of drainage		Percentage of time pavement structure is exposed to moisture levels approaching saturation			
Rating	Water removed within	Less than 1%	1—5%	5—25%	Greater than 25%
Excellent	2 hours	1 .40-1 .35	1 .35-1 .30	1 .30-1 .20	1 .20
Good	1 day	1 .35-1 .25	1 .25-1 .15	1 .15-1 .00	1 .00
Fair	1 week	1 .25-1 .15	1 .15-1 .05	1 .00-0 .80	0 .80
Poor	1 month	1 .15-1 .05	1 .05-0.80	0 .80-0 .60	0 .60
Very poor	Never drain	1 .05-0 .95	0 .95-0 .75	0 .75-0 .40	0 .40

Source: After AASHTO (1986)

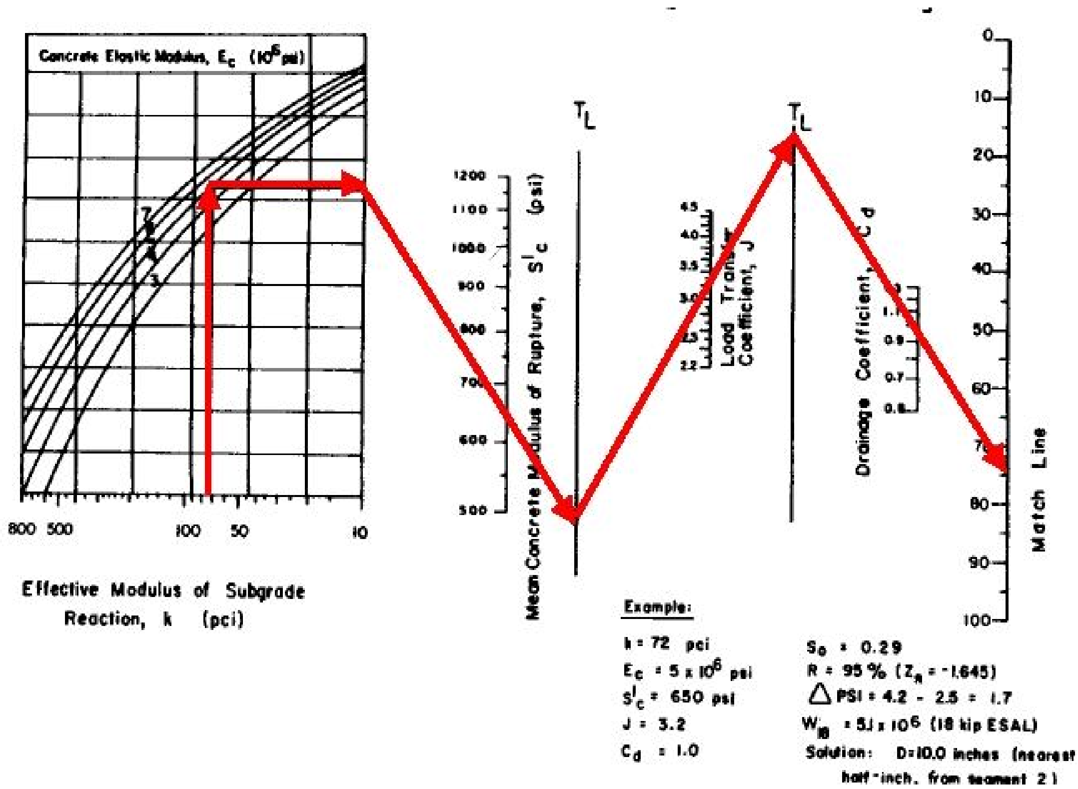
Table A-5: Minimum Thicknesses for Asphalt Surface and Aggregate Base

Traffic (ESAL)	Asphalt concrete	Aggregate base
Less than 50,000	1.0	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: After AASHTO (1986)

APPENDIX B
RIGID PAVEMENT DESIGN

Figures:



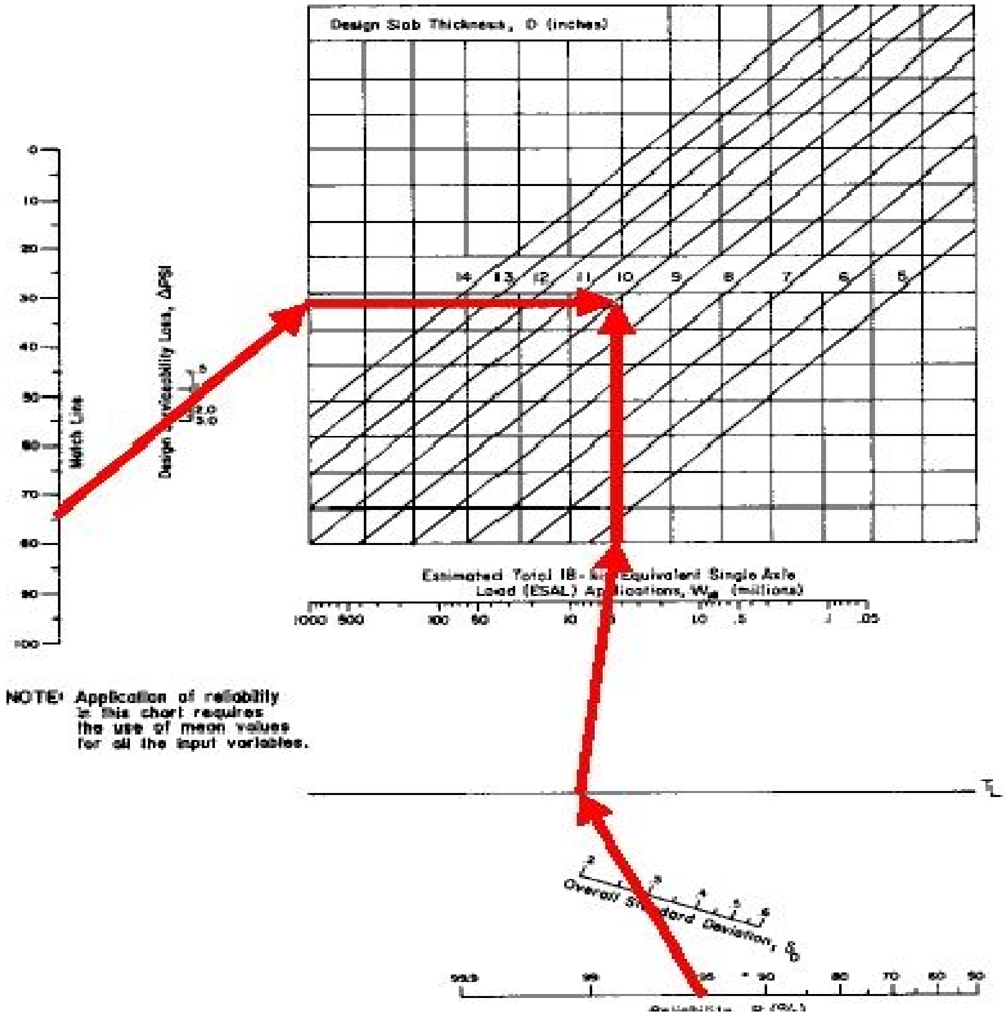


Figure B-1: Design chart for rigid pavement based on the mean values of each input
(from AASHTO design guide of pavement structure 1986)

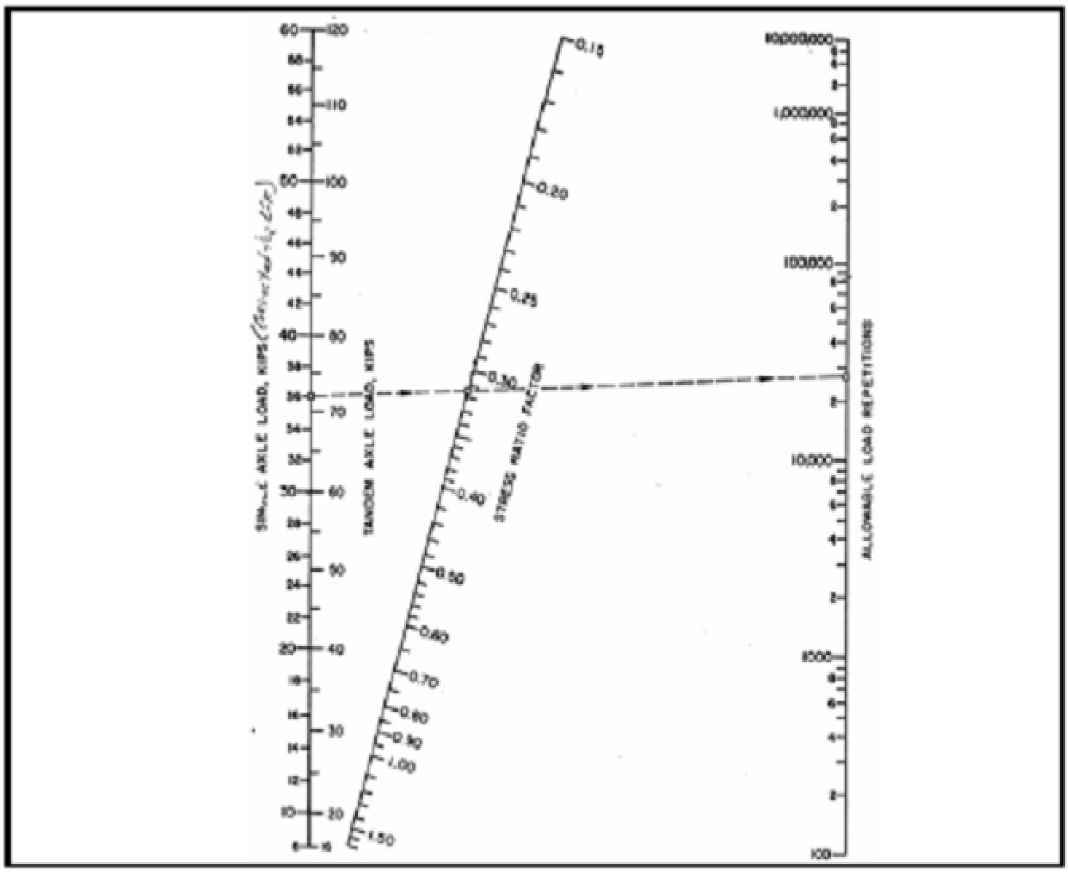


Figure B-2: Stress Ratio Factors versus Allowable Load Repetitions Both With and Without Concrete Shoulder (1 kip = 4 .45 KN). (After PCA (1984))

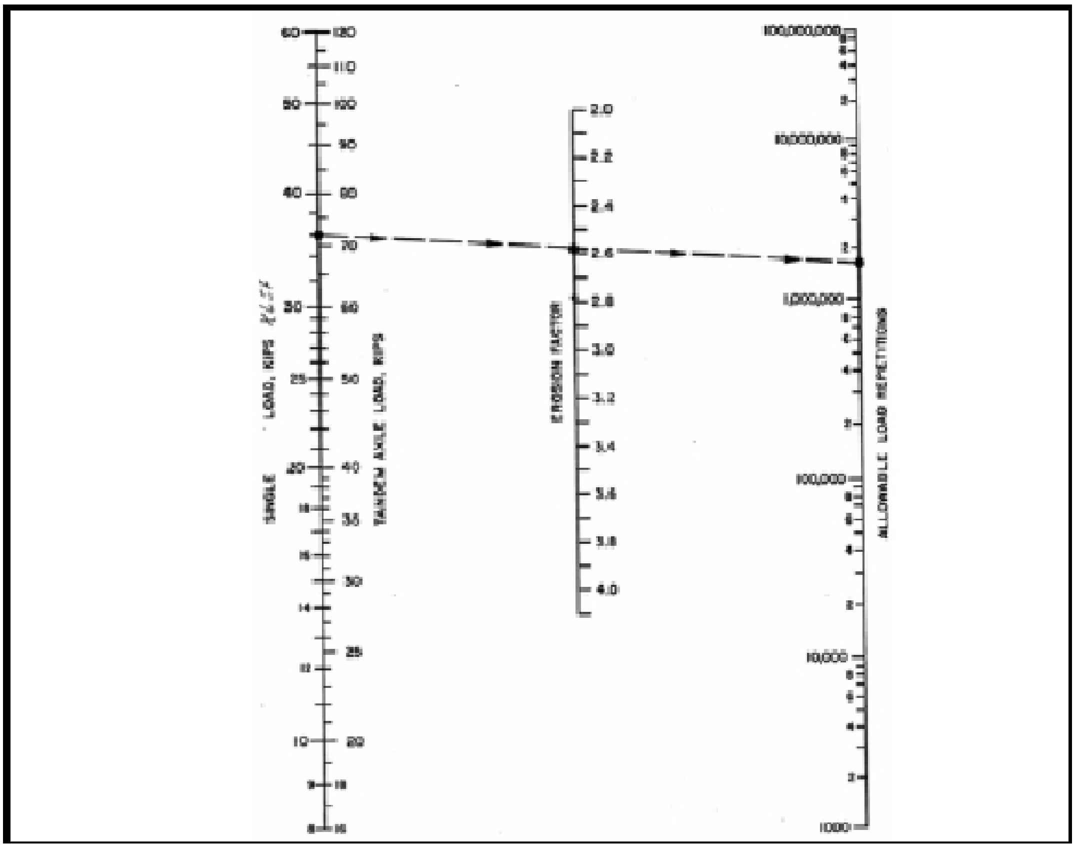


Figure B-3: Erosion Factors Versus Allowable Load Repetitions Both With and Without Concrete Shoulders (1 kip = 4 .45 KN). (After PCA (1984))

Table B-1: Recommended Dowel Size and Length

Slab thickness (in.)	Dowel diameter (in.)	Dowel length (in.)
5	5/8	12
6	¾	14
7	7/8	14
8	1	14
9	1 1/8	16
10	1 ¼	18
11	1 3/8	18
12	1 ½	20

Source: After PCA (1975)

Table B-2: Recommended Load Transfer Coefficients for Various Pavement Types and Design Conditions

Type of shoulder	Asphalt		Tied PCC	
Load transfer devices	Yes	No	Yes	No
JPCP and JRCP	3.2	3 .8-4.4	2 .5-3 .1	3 .6-4 .2
CRCP	2 .9-3 .2	N/A	2 .3-2 .9	N/A

Source: After AASHTO (1986)

Table B-3: Recommended Drainage Coefficients Values C_d for Rigid Pavements

Quality of drainage		**Percentage of time pavement structure is exposed to moisture levels approaching saturation**			
Rating	**Water removed within**	**Less than 1%**	**1—5%**	**5—25%**	**Greater than 25%**
Excellent	2 hours	1 .25-1 .20	1 .20-1 .15	1 .15-1 .10	1 .10
Good	1 day	1 .20-1 .15	1 .15-1 .10	1 .10-1 .00	1 .00
Fair	1 week	1 .15-1 .10	1 .10-1 .00	1 .00-0 .90	0 .90
Poor	1 month	1 .10-1.00	1 .00-0. 90	0 .90-0 .80	0 .80
Very poor	Never drain	1. 00-0 .90	0 .90-0 .80	0 .80-0 .70	0 .70

Source: After AASHTO (1986).

Table B-4: Equivalent Stresses for Slabs without Concrete Shoulders

Slab Thickness (in.)	k of Sub-grade—sub-base (pci) 50	100	150	200	300	500	700
4	825/679	726/585	671/542	634/516	584/486	523/457	484/443
4.5	699/586	616/500	571/460	540/435	498/406	448/378	417/363
5	602/516	531/436	493/399	467/376	432/349	390/321	363/307
5 .5	526/461	464/387	431/353	409/331	379/305	343/278	320/264
6	465/416	411/348	382/316	362/296	336/271	304/246	285/232
6 .5	417/380	367/317	341/286	324/267	300/244	273/220	256/207
7	375/349	331/290	307/262	292/244	271/222	246/199	231/186
7 .5	340/323	300/268	279/241	265/224	246/203	224/181	210/169
8	311/300	274/249	255/223	242/208	225/188	205/167	192/155
8 .5	285/281	252/232	234/208	222/193	206/174	188/154	177/143
9	264/264	232/218	216/195	205/181	190/163	174/144	163/133
9 .5	245/248	215/205	200/183	190/170	176/153	161/134	151/124
10	228/235	200/193	186/173	177/160	164/144	150/126	141/117
10 .5	213/222	187/183	174/164	165/151	153/136	140/119	132/110
11	200/211	175/174	163/155	154/143	144/129	131/113	123/104
11 .5	188/201	165/165	153/148	145/136	135/122	123/107	116/98
12	177/192	155/158	144/141	137/130	127/116	116/102	109/93
12.5	168/183	147/151	136/135	129/124	120/111	109/97	103/89
13	159/176	139/144	129/129	122/119	113/106	103/93	97/85
13 .5	152/168	132/138	122/123	116/114	107/102	98/89	92/81
14	144/162	125/133	116/118	110/109	102/98	93/85	88/78

Source: After PCA (1984)

Table B-5: Erosion Factors for Slabs with Doweled Joints and no Concrete Shoulders

Slab thickness (in .)	k of Sub-grade—sub-base (pci)					
	50	100	200	300	500	700
43.7	4/3.83	3.73/3.79	3.72/3.75	3.71/3.73	3.70/3.70	3.68/3.67
4 .5	3.59/3.70	3.57/3.65	3.56/3.61	3.55/3.58	3.54/3.55	3.52/3.53
5	3.45/3.58	3.43/3.52	3.42/3.48	3.41/3.45	3.40/3.42	3.38/3.40
5 .5	3.33/3.47	3.31/3.41	3.29/3.36	3.28/3.33	3.27/3.30	3.26/3.28
6	3.22/3.38	3.19/3.31	3.18/3.26	3.17/3.23	3.15/3.20	3.14/3.17
6 .5	3.11/3.29	3.09/3.22	3.07/3.16	3.06/3.13	3.05/3.10	3.03/3.07
7	3.02/3.21	2.99/3.14	2.97/3.08	2.96/3.05	2.95/3.01	2.94/2.98
7 .5	2.93/3.14	2.91/3.06	2.88/3.00	2.87/2.97	2.86/2.93	2.84/2.90
8	2.85/3.07	2.82/2.99	2.80/2.93	2.79/2.89	2.77/2.85	2.76/2.82
8 .5	2.77/3.01	2.74/2.93	2.72/2 .86	2.71/2.82	2.69/2.78	2.68/2.75
9	2.70/2.96	2.67/2.87	2.65/2.80	2.63/2.76	2.62/2.71	2.61/2.68
9 .5	2.63/2.90	2.60/2.81	2.58/2.74	2.56/2.70	2.55/2.65	2.54/2.62
10	2.56/2.85	2.54/2.76	2.51/2.68	2.50/2.64	2.48/2.59	2.47/2.56
10.5	2.50/2.81	2.47/2.71	2.45/2.63	2.44/2.59	2.42/2.54	2.41/2.51
11	2.44/2.76	2.42/2.67	2.39/2.58	2.38/2.54	2.36/2.49	2.35/2.45
11 .5	2.38/2.72	2.36/2.62	2.33/2.54	2.32/2.49	2.30/2.44	2.29/2.40
12	2.33/2.68	2.30/2.58	2.28/2.49	2.26/2.44	2.25/2.39	2.23/2.36
12.5	2.28/2.64	2.25/2.54	2.23/2.45	2.21/2.40	2.19/2.35	2.18/2.31
13	2.23/2.61	2.20/2.50	2.18/2.41	2.16/2.36	2.14/2.30	2.13/2.27
13 .5	2.18/2.57	2.15/2.47	2.13/2.37	2.11/2.32	2.09/2.26	2.08/2.23
14	2.13/2.54	2.11/2.43	2.08/2.34	2.07/2 .29	2.05/2.23	2.03/2.19

Source .After PCA (1984)

Table B-6: Erosion Factors for Slabs with Doweled Joints under Tridem Axles

Slab Thickness (in.)	k of Sub-grade—sub-base (pci)					
	50	100	200	300	500	700
4	3.89/3.33	3.82/3.20	3.75/3.13	3.70/3.10	3.61/3.05	3.53/3.00
4 .5	3.78/3.24	3.69/3.10	3.62/2.99	3.57/2.95	3.50/2.91	3.44/2.87
5	3.68/3.16	3.58/3.01	3.50/2.89	3.46/2.83	3.40/2.79	3.34/2.75
5 .5	3.59/3.09	3.49/2.94	3.40/2 .80	3.36/2.74	3.30/2.67	3.25/2.64
6	3.51/3.03	3.40/2.87	3.31/2 .73	3.26/2.66	3.21/2.58	3.16/2.54
6 .5	3.44/2.97	3.33/2.82	3.23/2.67	3.18/2.59	3.12/2.50	3.08/2.45
7	3.37/2.92	3.26/2.76	3.16/2.61	3.10/2.53	3.04/2.43	3.00/2.37
7 .5	3.31/2.87	3.20/2.72	3.09/2.56	3.03/2.47	2.97/2.37	2.93/2.31
8	3.26/2.83	3.14/2.67	3.03/2.51	2.97/2.42	2.90/2.32	2.86/2.25
8 .5	3.20/2.79	3.09/2.63	2.97/2.47	2.91/2.38	2.84/2.27	2.79/2.20
9	3.15/2.75	3.04/2.59	2.92/2.43	2.86/2.34	2.78/2.23	2.73/2.15
9 .5	3.11/2.71	2.99/2.55	2.87/2.39	2.81/2.30	2.73/2.18	2.68/2.11
10	3.06/2.67	2.94/2.51	2.83/2.35	2.76/2.26	2.68/2.15	2.63/2.07
10 .5	3.02/2.64	2.90/2.48	2.78/2.32	2.72/2.23	2.64/2.11	2.58/2.04
11	2.98/2.60	2.86/2.45	2.74/2.29	2.68/2.20	2.59/2.06	2.54/2.00
11 .5	2.94/2.57	2.82/2.42	2.70/2.26	2.64/2.16	2.55/2.05	2.50/1.97
12	2.91/2.54	2.79/2.39	2.67/2.23	2.60/2.13	2.51/2.02	2.46/1.94
12 .5	2.87/2.51	2.75/2.36	2.63/2.20	2.56/2.11	2.48/1.99	2.42/1.91
13	2.84/2.48	2/2.33	2.60/2.17	2.53/2.08	2.44/1.96	2.39/1.88
13 .5	2.81/2.46	2.68/2.30	2.56/2.14	2.49/2.05	2.41/1.93	2.35/1.86
14	2.78/2.43	2.65/2.28	2.53/2.12	2.46/2.03	2.38/1.91	2.32/1.83

Source .After PCA (1984)

APPENDIX C
PAVEMENT DESIGN PARAMETERS AND PROPERTIES

Tables:

Table C-1: Asphalt Institute's Equivalent Axle Load Factors for Flexible Pavement

Axle Load (lb)	Equivalent axle load factor: Single axles	Tandem axles	Tridem axles	Axle Load (lb)	Equivalent axle load factor: Single axles	Tandem axles	Trid axles
1000	0 .00002			41,000	23 .27	2.29	0 .540
2000	0 .00018			42,000	25 .64	2.51	0.597
3000	0 .00072			43,000	28.22	2.76	0.658
4000	0 .00209			44,000	31 .00	3.00	0.723
5000	0 .00500			45,000	34.00	3.27	0.793
6000	0 .01043			46,000	37.24	3.55	0.868
7000	0 .0196			47,000	40 .74	3.85	0.948
8000	0 .0343			48,000	44.50	4.17	1.033
9000	0 .0562			49,000	48 .54	4.51	1.12
10,000	0 .0877	0.00688	0 .002	50,000	52 .88	4.86	1.22
11,000	0 .1311	0 .01008	0 .002	51,000		5.23	1.32
12,000	0 .189	0.0144	0 .003	52,000		5.63	1.43
13,000	0 .264	0 .0199	0 .005	53,000		6.04	1.54
14,000	0 .360	0.0270	0 .006	54,000		6.47	1.66
15,000	0 .478	0 .0360	0 .008	55,000		6.93	1.78
16,000	0 .623	0 .0472	0 .011	56,000		7.41	1.91
17,000	0 .796	0.0608	0 .014	57,000		7.92	2.05
18,000	1 .000	0 .0773	0 .017	58,000		8.45	2.20
19,000	1 .24	0 .0971	0 .022	59,000		9.01	2.35
20,000	1 .51	0 .1206	0 .027	60,000		9.59	2.51
21,000	1 .83	0.148	0 .033	61,000		10.20	2.67
22,000	2 .18	0 .180	0 .040	62,000		10.84	2.85
23,000	2 .58	0.217	0 .048	63,000		11.52	3.03

24,000	3 .03	0 .260	0 .057	64,000		12 .22	3.22
25,000	3 .53	0 .308	0 .067	65,000		12 .96	3.41
26,000	4 .09	0 .364	0 .080	66,000		13 .73	3.62
27,000	4 .71	0 .426	0 .093	67,000		14 .54	3.83
28,000	5 .39	0 .495	0 .109	68,000		15 .38	4.05
29,000	6 .14	0 .572	0 .126	69,000		16 .26	4.28
30,000	6 .97	0 .658	0 .145	70,000		17 .19	4.52
31,000	7 .88	0 .753	0 .167	71,000		18 .15	4.77
32,000	8 .88	0 .857	0 .191	72,000		19 .16	5.03
33,000	9 .98	0 .971	0 .217	73,000		20 .22	5.29
34,000	11 .18	1 .095	0 .246	74,000		21 .32	5.57
35,000	12 .50	1 .23	0 .278	75,000		22 .47	5.86
36,000	13 .93	1 .38	0 .313	76,000		23 .66	6.15
37,000	15 .50	1 .53	0 .352	77,000		24 .91	6.46
38,000	17 .20	1 .70	0 .393	78,000		26 .22	6.78
39,000	19 .06	1 .89	0 .438	79,000		27 .58	7.11
40,000	21 .08	2 .08	0 .487	80,000		28 .99	7.45

Note . 1 kip = 4.45 kN, 1 in . = 25 .4 mm

Table C-2: Equivalent Axle Load Factors for Rigid Pavement, D = 9in, P_t = 2

	Equivalent axle load factor				Equivalent axle load factor		
Axle Load (kips)	**Single axles**	**Tandem axles**	**Tridem axles**	**Axle load (kips)**	**Single axles**	**Tandem axles**	**Tridem axles**
2	0 .0002	0 .0001	0.0001	48	56 .8	7 .73	2 .49
4	0 .002	0.0005	0.0003	50	67 .8	9 .07	2 .94
6	0 .01	0 .002	0 .001	52		10.6	3 .44
8	0 .032	0 .005	0.002	54		12.3	4 .00
10	0 .082	0.013	0 .005	56		14 .2	4 .6 3
12	0 .176	0.026	0 .009	58		16 .3	5 .32
14	0 .341	0.048	0 .017	60		18.7	6 .08
16	0 .604	0.082	0 .028	62		21 .4	6 .91
18	1 .00	0.133	0 .044	64		24 .4	7 .82
20	1 .57	0.206	0 .067	66		27 .6	8 .83
22	2.34	0.308	0 .099	68		31 .3	9 . 9

24	3.36	0.444	0.141	70	35 .3	11 .1
26	4 .67	0 .622	0 .195	72	39.8	12.4
28	6 .29	0 .850	0 .265	74	44.7	13.8
30	8 .28	1 .14	0 .354	76	50.1	15.4
32	10.7	1 .49	0 .463	78	56.1	17.1
34	13 .6	1 .92	0 .596	80	62.5	18 9
36	17 .1	2 .43	0 .757	82	69.6	20.9
38	21 .3	3 .03	0.948	84	77.3	23.1
40	26.3	3 .74	1 .17	86	86.0	25.4
42	32.2	4 .55	1 .44	88	95.0	27.9
44	39 .2	5 .48	1 .74	90	105.0	30.7
46	47.3	6 .53	2 .09			

Note. 1 kip = 4.45 kN, 1 in . = 25 .4 mm.
Source: After AASHTO (1986).

Table C-3: Lane Distribution Factor

No. of lanes in each direction	**Percentage of 18-kip ESAL in design lane**
1	100
2	80 – 100
3	60 – 80
4	50-75

Source: After AASHTO 1986.

Table C-4: Design Sub-grade Resilient Modulus

Traffic level ESAL	**Design resilient modulus Percentile value (%)**
10^4 or less	60
10^4 - 10^6	75
10^6 or more	87.5

Source: After AI (1981a).

Figures:

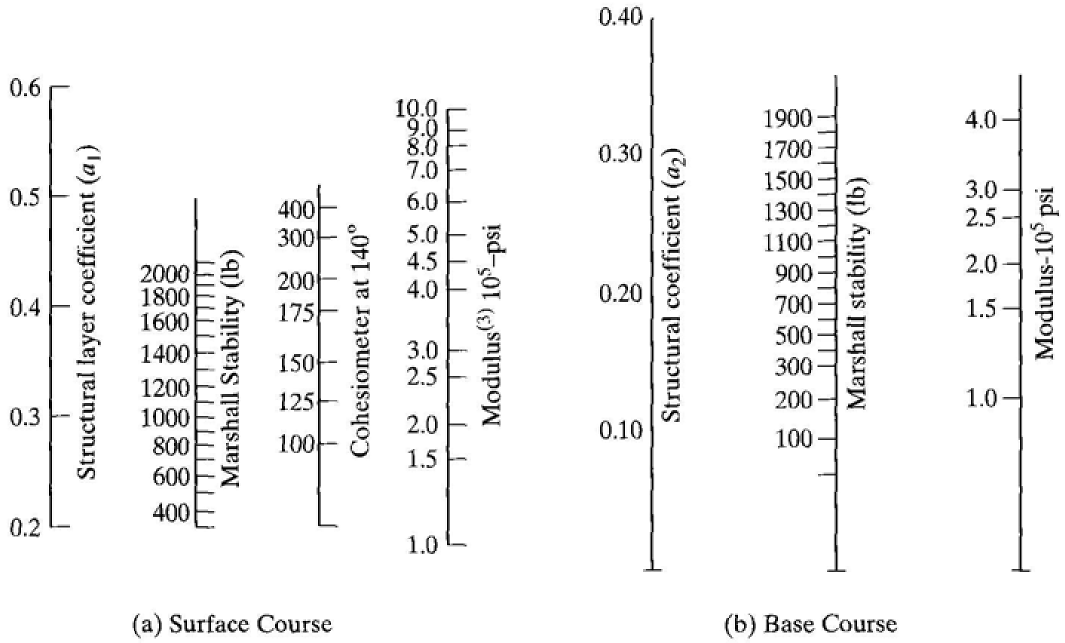


Figure C-1: Correlation for Estimating Resilient Modulus of HMA (After Van Til et al 1972)

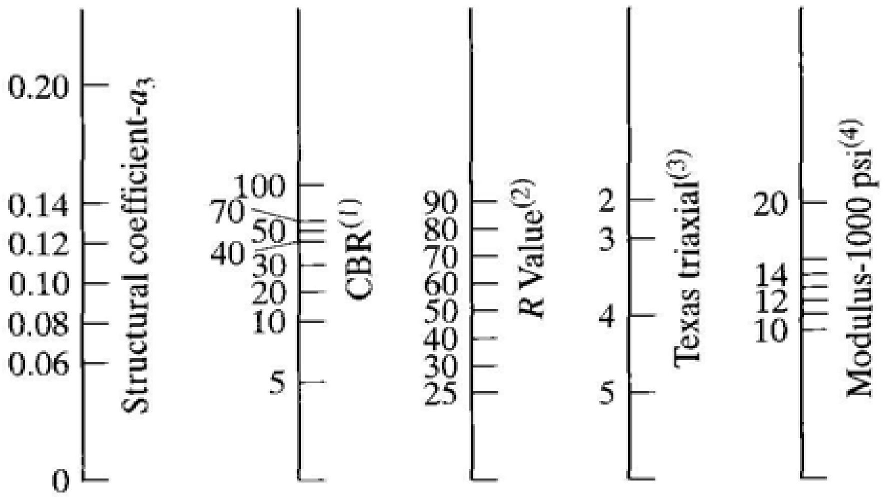


Figure C-2: Correlation for Estimating Resilient Modulus of Sub-base (After Van Til et al 1972)

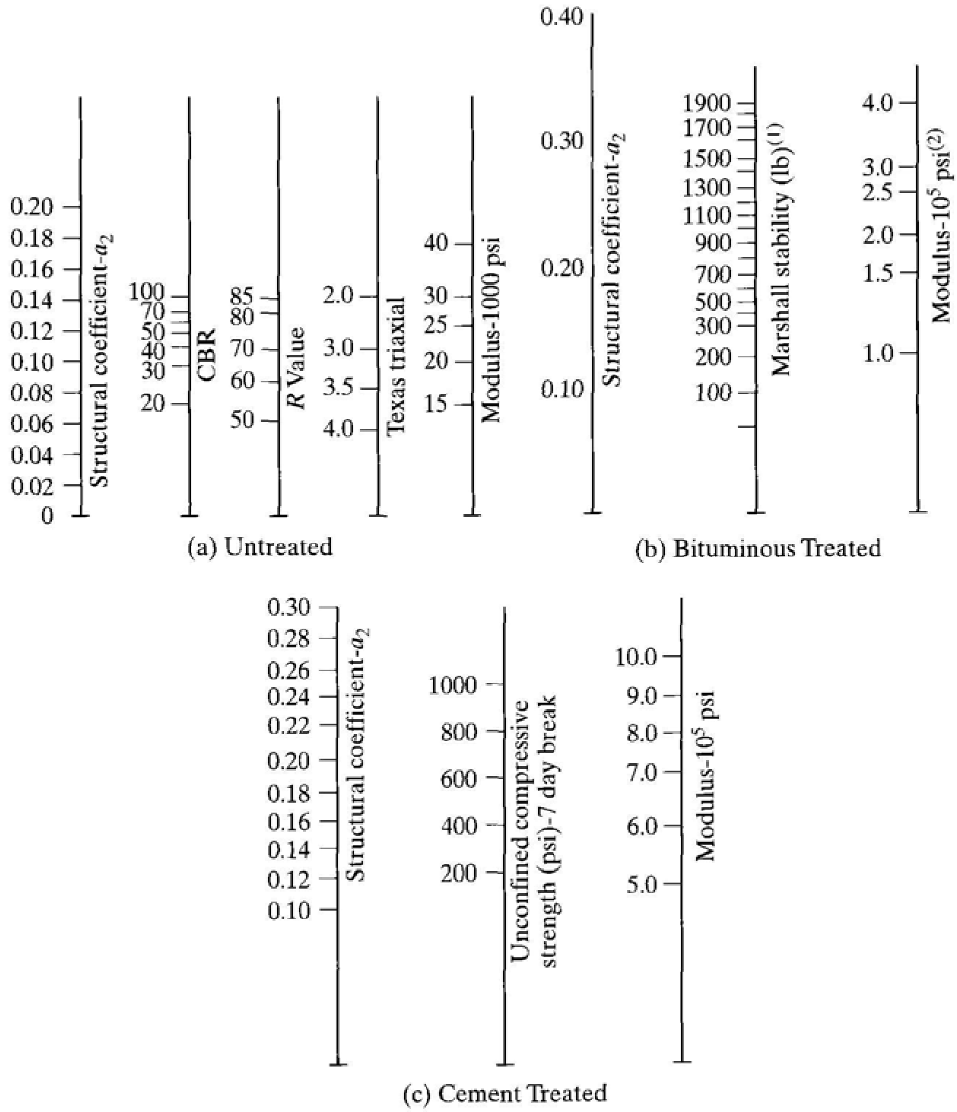


Figure C-3: Correlation for Estimating Resilient Modulus of bases (After Van Til et al 1972)

Example:

D_{SB} = 6 inches

E_{SB} = 20,000 psi

M_R = 7,000 psi

Solution: k_∞ = 400 pci

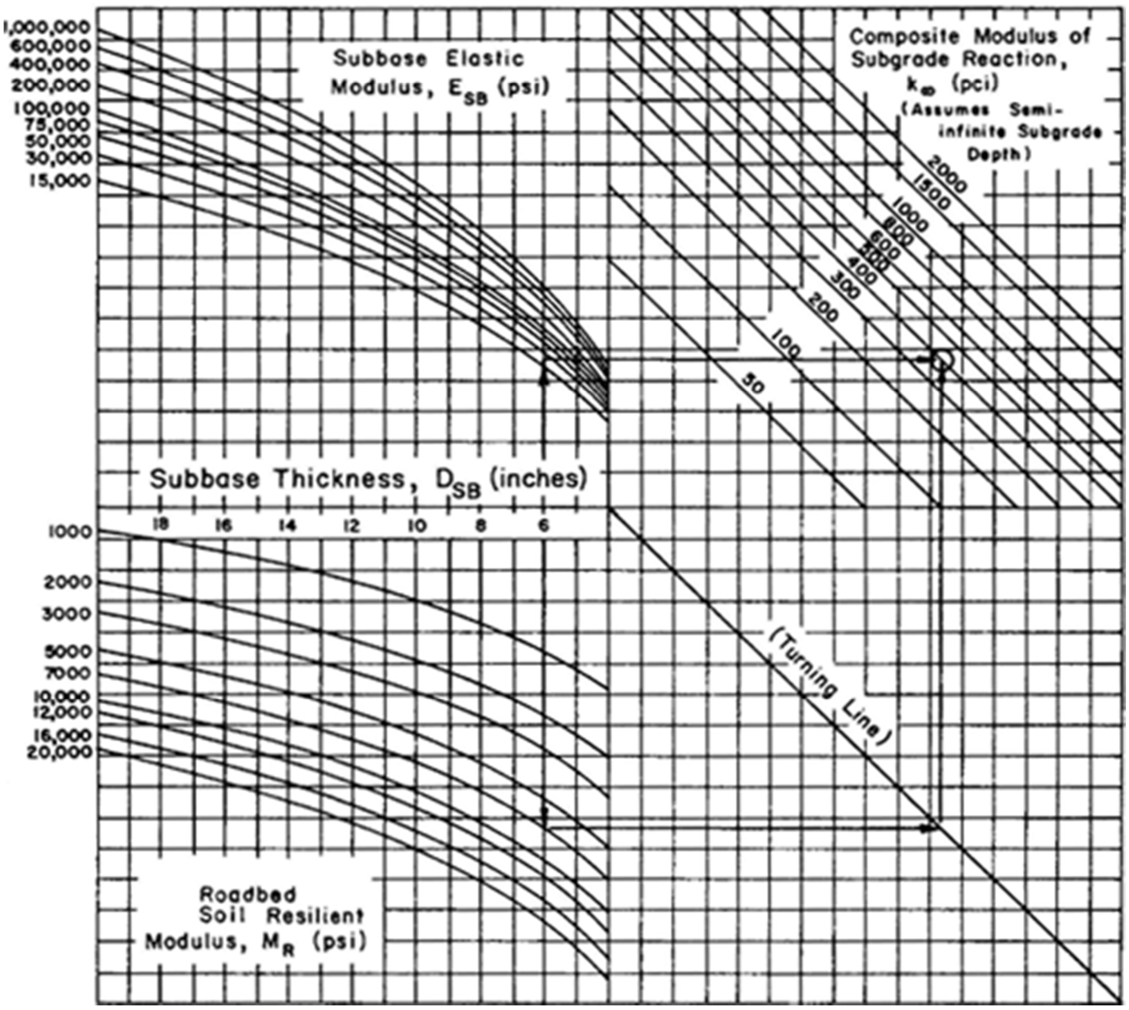


Figure C-4: Chart for Estimating Modulus of Sub-grade Reaction.
(From: the AASHTO Guide for Design of Pavement Structures. 1986.)

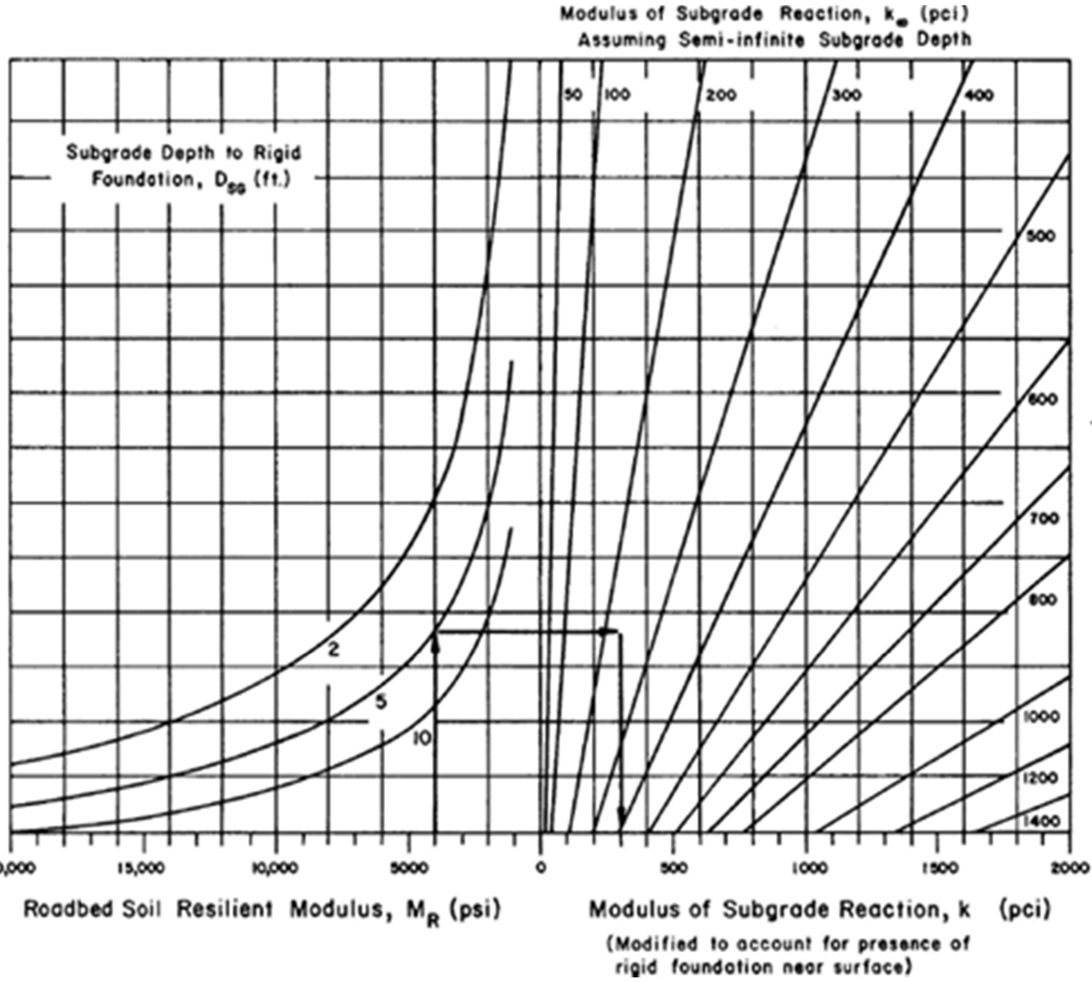


Figure C-5: Chart for Modifying Modulus of Subgrade Reaction due to Rigid Foundation Near Surface
(From: the AASHTO Guide for Design of Pavement Structures, 1986.)

APPENDIX D
STRUCTURAL DESIGN OF FLEXIBLE PAVEMENT

1.1 Road A Traffic Loading Composition:

The following assumption was made to convert traffic counting data of Road A to axle load repetitions:

Suggested axle loads for the applied vehicles.

1. Passenger car: Front single axle = 2 Kips.
   Rear single axle = 2 Kips.

2. Mini Bus: Front single axle = 6 Kips.
   Rear single axle = 4 Kips.

3. Bus: Front single axle = 10 Kips.
   Rear single axle = 8 Kips.

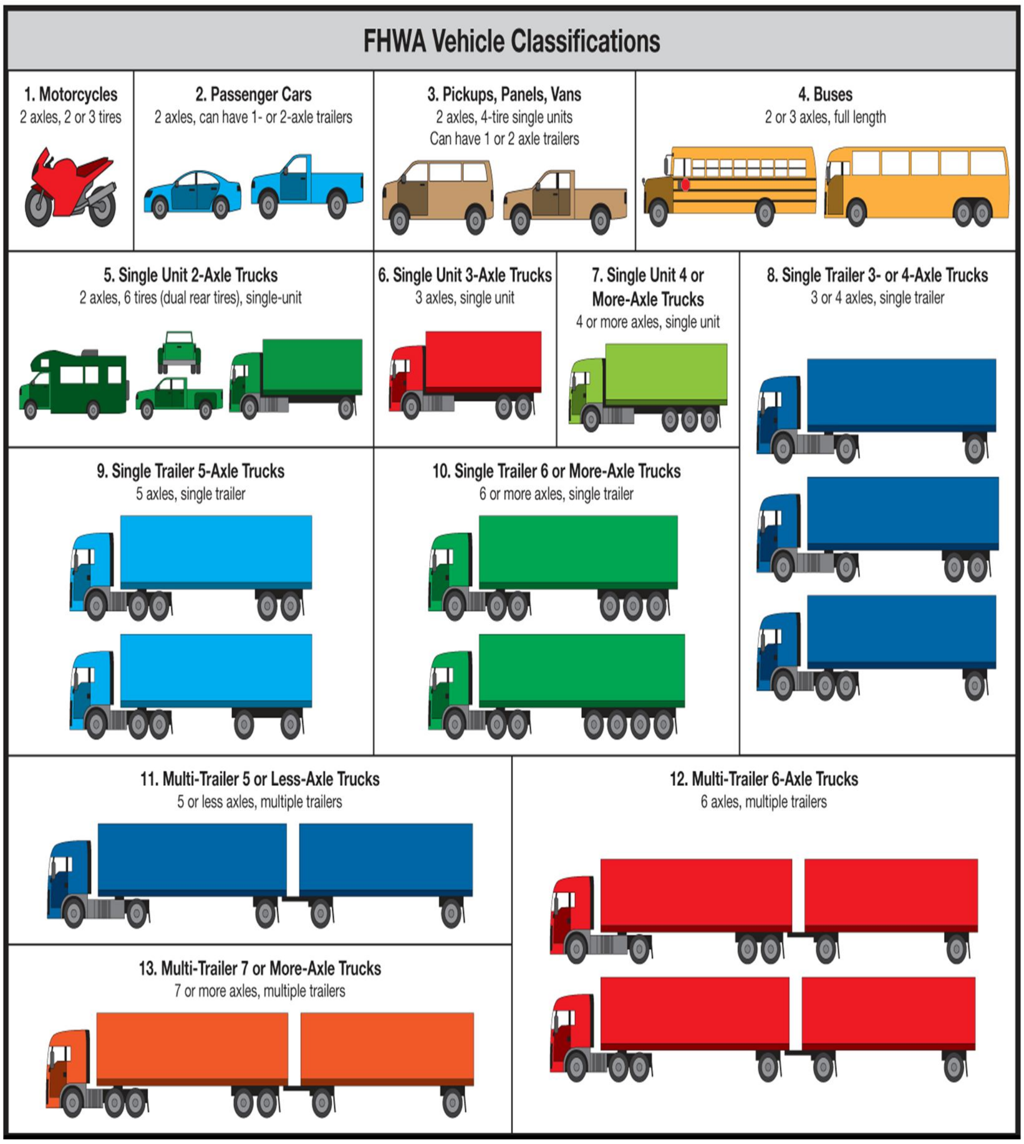
4. Light Truck Front single axle = 12 Kips.
   Middle single axle = 18 Kips.
   Rear Tandem axle = 32 Kips.

5. Heavy Truck Front single axle = 18 Kips.
   First trailer front tandem axle = 36 Kips
   First trailer rear trible axle = 52 Kips
   Second trailer front tandem axle = 38 Kips
   Second trailer rear trible axle = 54 Kips

Note: (1Kip = 1000 lb = 4.45 N = 450 Kg).

1.2 Road B Traffic Loading Composition:

The classification of Road B vehicles with definitions used in this study follow FHWA (Federal High Ways Administration) classification. The axle configurations which have wide spectrum of operation in Sudan are presented in the following figure and described below:



1. Motorcycles.
2. Passenger cars.
3. Other two axle- four tire single unit vehicles.
4. Buses.
5. Two axles- six tire, single unit trucks.
6. Three axle single unit trucks.
7. Four or more axle single unit trucks.
8. Four or fewer axle single trailer trucks.
9. Five axle single trailer trucks.

10. Six or more axle single trailer trucks.
11. Five or fewer axle multi trailer trucks.
12. Six axle multi trailer trucks.
13. Seven or more axle multi trailer trucks.
14. Eleven axle multi trailer trucks.

The traffic survey data were collected by using Canadian temporary tube counter device. The equipment was programmed for FHWA vehicle classes' definitions to estimate the required AADT.

The following assumed axle's loads were applied for the truck configurations

Single unit two axle trucks (5):

Front single axle load = 12 kips
Rear single axle load = 18 kips

1. Single unit three axle trucks (6):
   Front single axle load = 16 kips
   Rear tandem axle load = 32 kips

2. Single unit four axle trucks (7):
   Front single axle load = 16 kips
   Rear tandem axle load = 52 kips

3. Single trailer four axle trucks (8):
   Prime mover
   Front single axle load = 16 kips
   Rear single axle load = 18 kips
   Trailer
   Rear tandem axle load = 36 kips

4. Single trailer five axle trucks (9):
   Prime mover
   Front single axle load = 16 kips
   Rear tandem axle load = 32 kips
   Trailer
   Rear tandem axle load = 36 kips
5. Single trailer six axle trucks (10):
   Prime mover

Front single axle load = 16 kips
Rear tandem axle load = 36 kips
Trailer
Rear tridem axle load = 52 kips

6. Multi trailer five axles trucks (11):
   Prime mover
   Front single axle load = 18 kips
   Rear single axle load = 22 kips
   First trailer
   Rear single axle load = 22 kips
   Second trailer
   Front single axle load = 22 kips
   Rear single axle load = 24 kips

7. Multi trailer six axles truck (12):
   Prime mover
   Front single axle load = 18 kips
   Rear single axle load = 22 kips
   First trailer
   Rear tandem axle load = 32 kips
   Second trailer
   Front single axle load = 22 kips
   Rear single axle load = 24 kips

8. Multi trailer seven axles trucks (13):
   Prime mover
   Front single axle load = 18 kips
   Rear tandem axle load = 32 kips
   First trailer
   Rear tandem axle load = 36 kips
   Second trailer
   Front single axle load = 22 kips
   Rear single axle load = 24 kips

9. Multi trailer nine axles trucks (14):
   Prime mover
   Front single axle load = 18 kips
   Rear tandem axle load = 32 kips
   First trailer

Rear tandem axle load = 36 kips
Second trailer
Front tandem axle load = 32 kips
Rear tandem axle load = 36 kips

10. Eleven axle multi trailer trucks- (15):
    Prime mover
    Front single axle load = 18 kips
    Rear tandem axle load = 32 kips
    First trailer
    Rear tridem axle load = 52 kips
    Second trailer
    Front tandem axle load = 36 kips
    Rear tridem axle load = 56 kips

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Table D-1: Road A Construction Materials Laboratory Tests Results

chainage (Km)	Offset (Km)	Sample No.	Material Description	L.L %	PI %	Percent passing sieve								AASHTO	MDD	OMC (%)	CBR at 100% of MDD (%)	CBR at 95% of MDD (%)
						1.5"	1"	3/4"	3/8"	#4	#10	#40	#20	Class	(Kg/M3)			
Km 11 of Katorgi	42	1	Gliaa - Balsam	N.P	N.P	100	77.7	69.8	49..3	39.4	31	13	5	A-1-a	2.31	13.30	100	80.00
Km 11 of Katorgi	4.5	2	El Hashaba- Seteet 1	N.P	N.P	100	73.2	59.2	42	31.4	25	19	9	A-1-a	2.35	18.00	100	38.0
Km 11 of Katorgi	4.5	3	El Hashaba- Seteet 2	N.P	N.P	100	74.2	69.9	54	48.9	33	24	10	A-1-a	2.27	18.00	45	
Km 0 of Katorgi	12	4	Gonza	N.P	N.P	100	85.8	71.9	50	38.5	29	17	5	A-1-a	2.26	18.00	98	
Km 8 of Katorgi	5	5	El Tomat	N.P	N.P	100	94.9	90	66	47.1	33	16	6	A-1-a	2.23	16.70	100	
Km 11 of Katorgi	4	6	El Morabaa	N.P	N.P	88.6	84	72.8	53	42.6	33	17	6	A-1-a	2.29	18.00	88	
Km 0 of Katorgi	12	7	Gonza 2	N.P	N.P	95.7	81.8	71.6	50	37.3	26	17	7	A-1-a	2.32	19.00	100	
Km 0 of Katorgi	12	8	Gonza 3	N.P	N.P	88.1	83.4	71.9	53	42.5	33	17	6	A-1-a	2.29	16.00	100	
Km 0 of El moneera	7	9	Kogor	N.P	N.P	100	93.8	71	42	30.4	23	15	4	A-1-a	2.35	18.00	100	
Km 13 of Zakaria	3	10	Wad elhelew	23.9	7.8	92.5	74.3	64.1	48	39.3	32	22	10	A-1-b	2.26	16.40	63	51.0
Km 0 of Zakaria	2	11	Zakaria	28.0	9.8	89.8	76.8	69.1	49	34.6	27	22	12	A-1-b	2.26		40	30.0
Km 17 of El moneera	6	12	El Kreeda (1)	36.7	11.0	86.8	61.1	41	28	25.8	24	22	14	A-1-a	2.23	16.60	49	32.0
		13	El Dehab	N.P	N.P	88.9	74.6	60	34	26.8	23	16	5	A-1-a	2.29		100	67.0
Km 17 of El moneera	6	14	El Kreeda(2)	N.P	N.P	100	88.2	82	64	46.4	31	14	5	A-1-a	2.31	16.90	100	65.0

Table D-2: Road A Sub-grade Laboratory Tests Results

chainage (Km)	Depth (M)	Sample No.	Material Description	L.L %	PI %	Percent passing sieve						AASHTO Class	MDD Kg/M	OMC (%)	Swell (%)	CBR at
						3/4'	3/8"	#4	#10	#40	#20					
0+000	0.6	1	light brown silty clay	N.P	N.P		100	99.7	98.8	98	40		1.86	13.30		
2+500	0.6	2	Black cotton soil	56.0	30.6		100	100	99.6	97	88	A-7-6	1.71	18.00		2.0
5+000	0.6	3	Black cotton soil	55.0	30.0		100	100	99.8	97	88	A-7-6	1.47	18.00		
7+500	0.6	4	Black cotton soil	56.5	31.3		100	100	99.7	97	89	A-7-6	1.47	18.00		1.0
10+000	0.6	5	Black cotton soil	58.3	29.9		100	100	98.7	97	90	A-7-6	1.53	16.70		2.2
12+500	0.6	6	Black cotton soil	54.0	30.0		100	100	99.8	97	89	A-7-6	1.43	18.00		
17+500	0.6	7	Black cotton soil	55.0	31.1		100	100	99.7	97	88	A-7-6	1.55	19.00		
20+000	0.6	8	Black cotton soil	57.0	31.5		100	100	99.8	98	88	A-7-6	1.46	16.00		1.0
24+000	0.6	9	Black cotton soil	55.5	27.9		100	100	99.7	97	88	A-7-6	1.49	18.00		
28+000	0.6	10	Black cotton soil	66.8	30.4		100	100	99.2	97	90	A-7-6	1.58	16.40		3.0
32+000	0.6	11	Black cotton soil	60.5	27.2		100	100	99.2	97	87	A-7-6	1.54	16.60		2.8
35+000	0.6	12	Black cotton soil	59.0	26.5		100	100	96.7	97	90	A-7-6	1.56	16.90		
39+000	0.6	13	Black cotton soil	56.0	32.5		100	100	99.7	97	89	A-7-6	1.53	17.00		
44+000	0.6	14	Black cotton soil	56.5	30.3		100	100	99.8	98	89	A-7-6	1.40	16.00		
49+000	0.6	15	Black cotton soil	56.5	30.9		100	100	99.6	97	89	A-7-6	1.40	16.00		
55+500	0.6	16	Black cotton soil	61.5	31.8		100	100	99	97	86	A-7-6	1.49	18.00		2.10
59+500	0.6	17	Black cotton soil	58.0	31.3		100	100	99.8	98	88	A-7-6				

Table D-3: Road B Construction Materials Laboratory Tests

Sample No.	coordinates		Material Description	L.L %	PI %	N.M.C	% Passing			AASHTO Class	MDD Kg/M2	OMC (%)	Swell (%)	CBR at 90% of MDD (%)
	E	N					#10	#40	#200					
1	4412911	1742193	Sandy soil with silty gravel	19	6.4	1.3	36.8	27.4	13.2	A-2-4	2.25	6.50	0.00	43.000
2	439408	1737542	Sandy soil with gravely silt	23.0	6.5	2.0	42.7	29.8	17.5	A-2-4	2.20	6.80	0.00	24.000
3	437046	1734941	Sandy soil with silt	26.0	10.7	0.9	44.2	28.8	23.9	A-2-6	2.18	7.80	0.02	18.000
4	شمال المرخيات		Silty sand soils	19.0	6.5	1.1	48.5	11.9	6.9	A-2-4	2.11	7.00	0.00	26.000
5	شمال المرخيات		Sandy silt soil	21.8	6.1	1.6	42..3	26.8	9.4	A-2-4	2.14	7.40	0.00	27.000

Table D-4: Road B Sub-grade Soil Laboratory Tests

Sample No.	coordinates E	coordinates N	Material Description	L.L %	PI %	N.M.C	% Passing #10	% Passing #40	% Passing #200	AASHTO Class	MDD Kg/M2	OMC (%)	CBR at 95%	Swell (%)
1	436270	1732798	Sandy soil with silty gravel	37.2	13.2	4.2	38.3	26.4	17	A-2-6	2.22	7.00	15.00	0.120
2	436158	1732743	Sandy soil with gravely silt	22.5	9.0	2.0	67	55.2	37.1	A-4	2.15	8.20	6.20	0.021
3	435690	1735575	Sandy soil with silt	Non	Non	0.6	88.3	67.1	29.7	A-2-4	2.20	6.20	6.20	0.000
4	4352210	1732436	Silty sand soils	27.0	10.0	1.1	90.1	75.8	47	A-4	2.01	12.00	7.90	0.030
5	434753	1732240	Sandy silt soil	21.0	6.5	0.9	99.2	85.4	61.2	A-4	2.00	8.20	5.10	0.001
6	434402	1731765	Sandy silty calyey soil	36.7	13.0	3.1	95.7	82.7	51.4	A-6	1.80	16.00	2.50	0.300
7	434344	1731271	Silty sand soils	20.2	5.6	1.3	82.7	63.1	37.5	A-4	2.05	10.80	4.50	0.001
8	434339	1730871	Silty sand soil with gravel	20.5	6.0	0.8	77.4	55.7	30.5	A-2-4	2.08	9.20	5.00	0.011
9	434334	1730372	Gravety sand with silt	25.6	8.8	1.8	49	35.7	24.3	A-2-4	2.12	9.60	10.00	0.110
10	434329	1729872	Gravely sand soil with silt	23.4	10.0	2.3	43.1	37.1	17.2	A-2-4	2.08	10.20	7.00	0.021
11	434321	1729372	Silty gravely sand	24.0	7.7	2.0	50.8	45.3	31.1	A-2-4	2.15	8.00	6.30	0.030
12	434314	1728872	silty sand soil	24.0	9.6	1.8	92.9	79.2	46.6	A-4	2.06	11.60	3.50	0.001
13	434308	1728372	silty clayey sand soil	30.0	11.4	3.5	98.4	83.1	55.3	A-6	2.00	12.40	3.00	0.290
14	434301	1727872	Silty sand soil	22.9	7.1	1.3	98.6	83.5	41.2	A-4	2.02	10.00	6.10	0.011
15	434294	1727372	Silty sand soil	23.5	6.8	0.6	32.1	29	25.8	A-2-4	2.18	8.80	17.00	0.001