C H A P T E R 1

Introductory Background

1.1 Introduction

Although pavement design has gradually evolved from art to science, empirical methodologies still play an important role up to date. Prior to early 1920s, determination of pavement thickness was based purely on experience. Generally, the same thickness would be used for different sections of varying pavement soil conditions. As experience was gained and following pavement research throughout the years, various methods were developed by different agencies for determination of the required pavement thickness. It is not feasible to document all design methods that have evolved and applied. However, in this study, only a few typical methods will be cited and discussed to indicate the trend. [6]

Rigid (or concrete) pavements (RPs) are constructed of Portland cement concrete (PCC). The first concrete pavement was built in Bellefontaine, Ohio in 1893 (Fitch, 1996), 15 years earlier than the one constructed in Detroit, Michigan, in 1908. As of 2001, there were about 59,000 miles (95,000 km) of rigid pavements in the United States. The development of design methods for rigid pavements is not as dramatic as that of flexible pavements, because the flexural stress in concrete has long been considered as a major design factor.

Concrete pavements can be classified into four types: jointed plain concrete pavement (JPCP), jointed reinforced concrete pavement (JRCP), continuous reinforced concrete pavement (CRCP), and pre-stressed concrete pavement (PCP). Except for PCP with lateral pre-stressing, a longitudinal joint should be installed between two traffic lanes to prevent longitudinal cracking. The JPCP, requiring no steel reinforcements and thus the least expensive to construct, is a popular form of construction. Depending on the thickness of the slab, typical joint spacings for plain concrete pavements are between 10 and 20 ft (3 and 6 m). For slabs with joint spacing greater than 6 m, steel reinforcements have to be provided for crack control, giving rise to the use of JRCP and CRCP. [6]

Structural design of rigid pavements includes thickness and reinforcement designs. Two major approaches for RP thickness design methods are applied in this study: The first approach relies on empirical relationships derived from performance of full-scale experimental pavements and in-service pavements performed by the American Association of State Highway and Transportation Officials (AASHTO, 1993). The second one develops relationships in terms of the properties of pavement materials as well as load-induced and thermal stresses, calibrating these relationships with pavement performance data. The Portland Cement Association (PCA) method of design adopted this approach (1984).

In this research investigation, thickness design for JPCP by AASHTO and PCA methods was determined for the case study pavement. However, reinforced design by AASHTO procedure was performed for the JRCP suggested to be used in the construction of the proposed road. A computer program with Visual Basic software was developed, entitled GalalM-RP program, to determine the rigid pavement design thickness in accordance with PCA method. Comparison was then made for the rigid pavement design thickness between the manual method and GalalM-RP program.

Sparseness and increasing cost of construction materials along with heavy axle loads, environmental conditions and inadequate design and construction lead to premature failure of roads and force engineers to consider more economical and long life pavement design methods to build roads using indigenous pavement materials and advanced construction techniques. The situation becomes even more critical in underdeveloped countries (Ali, 2003). Additionally, in order to achieve minimum production costs, it is considered necessary to have cost-effective construction of roads with optimum performance and low maintenance costs. (Ali et al., 2012) and Ali and Gasim (2014) conducted comparative studies for JPCP versus flexible pavement for Sudan highways and urban roads under different soil strength and traffic conditions. It was found that:

1. Using rigid pavement reduces construction costs by 10 to 35 % depending on subgrade strength and ESAL compared to flexible.
2. Considering the fact that the natural ground in most residential areas targeted with road project is black cotton soil with high plasticity, it was shown that using rigid pavement would reduce the overall construction cost.
3. The availability of natural gravel and sand in many areas in the country will further reduce the cost of rigid pavement compared to flexible pavement due to their suitability for use in rigid pavement compared to costly crushed aggregate foe asphalt pavement. [5]

Similar studies were carried out in India (Prasad, 2007) and Turkey (Ukar et al., 2007) [10] and [12]. In the present investigation, comparison was made between two types of rigid pavements: Jointed plain concrete pavement (JPCP) versus jointed reinforced concrete pavement (JRCP). [4]

1.2 Problem Statement and Significance

Selecting a pavement type is an important decision. Similar to other aspects of pavement design, such as traffic loading and materials, the 1993 *AASHTO Guide* indicates that the selection of pavement type is based on many varying factors, material selection representing along with design traffic the main factors related to desired pavement performance. Proper selection of materials and understanding of how they perform in the field within the composite pavement structure must be based on careful consideration of expected traffic loads with all related variables, the environment, construction practices and evaluation. Other considerations, such as availability of materials and economics, will often influence which materials are ultimately selected. While it is preferred to use the highest quality of materials for all road projects, materials must be of sufficient uniformity and quality to provide the following performance indicators under expected traffic loading and environmental conditions:

- Adequate serviceability at minimum cost;
- Best serviceability according to available funds; and
- Maximum mobility at minimum cost.

Pavement distresses and their causes, in addition to long-term performance, are other important factors for selection and adoption of pavement type. For instance, the pavement type selected should in general provide the following required improvements: reduced life-cycle cost, shorter construction periods, less disruption to traffic, residents and business, and safe and manageable field activities (Ali et al., 2012). Additionally, utility cuts, a major concern, should be minimized, knowing that poor performance is getting difficult to manage.

As a result of the several steps involved in the PCA design method with trial thicknesses, development of a computer program was considered necessary to assist an contribute in reducing the time for iterations.

1.3 The Design Methods and Procedures

Two methods of design methods were selected for both pavement types. A case study and applications included an urban-rural highway (Al-Ilaifoun Road).

Thickness design for JPCP by AASHTO and PCA methods was determined for the case study pavement. However, reinforced design by AASHTO procedure was performed for the JRCP suggested to be used in the construction of the proposed road. A computer program with Visual Basic software was developed, entitled GalalM-RP program, to determine the rigid pavement design thickness in accordance with PCA method.

1.4 Objectives and Scope of Research

The general objectives of the study are:

1- Design thickness of rigid pavement obtained through manual and Galal R.P. which uses Visual Basic software with PCA procedure.

2- Application of popular structural design methods for the purpose of comparing costs was another objective of undertaking this study

1.5 Out Line of Thesis

This thesis has six chapters (in addition to this one) and two appendices. Chapter 2 describes rigid pavement types and design methodology for Joint Plan and Jointed Reinforce Concrete Pavements .Chapter 3 describes Location and Characteristics, ESAL of the case study road project. Chapter 4 designs of Joint Plan and Jointed Reinforce Concrete Pavement for case study road project. Chapter 5 application of software program and introduces the computer software tool developed under the project. Chapter 6 summarizes the results. Chapter 7 summarizes the thesis conclusions and recommendations. Appendix A contains the PCA design method tables and Charts. Appendix B contains the AASHTO design method tables and Charts.

C H A P T E R 2

RIGID PAVEMENT TYPES AND DESIGN METHODOLOGIES FOR JOINTED PLAIN AND JOINTED REINFORCED CONCRETE PAVEMENTS

2.1 Introduction

A rigid pavement, by virtue of its rigidity, and according to the novel approach and pioneer work in Westergaard's theory (1925) is able to cause slab or bending action to spread the wheel load over the entire slab area with the subgrade responding through the modulus of subgrade reaction k. Figure 2.1 illustrates this phenomenon as well as typical cross-sections of rigid highway and airport pavements. The structural capacity of the rigid pavement known as modulus of rigidity, D defined in Eq. (2-1), is largely provided by the slab itself.

$$D = \frac{Eh^3}{12(1-\mu^2)} \qquad (2\text{-}1)$$

Where E is the modulus of elasticity of concrete h is the slab thickness, and μ is Poisson's ratio of concrete. Equation (2-1) divided by the modulus of subgrade reaction, k leads to the famous Westergaard's "radius of relative stiffness, l" Eq. (2-2):

$$l^4 = D/k = \left[\frac{Eh^3}{12k(1-\mu^2)}\right] \qquad (2\text{-}2)$$

For the common range of subgrade soil strength, the required rigidity for a Portland cement concrete slab can be achieved during construction without much variation in slab thickness. The effect of the subgrade soil properties on rigidity and the thickness of rigid pavement is reflected by Eq. (2-2).It is claimed that in this regard the subgrade effect is less important than in the case of flexible pavement.

Regarding the base course for rigid pavement, sometimes subbase might suffice, is often provided to prevent pumping resulting from ejection of foundation material through cracks or joints due to vertical movement of slabs under traffic. The base course is generally required to provide good drainage and resistance to the erosive action of water. When dowel bars are not provided in short jointed pavements, it is common practice to construct cement-treated base (CTB) to assist in load transfer across the joints. [7]

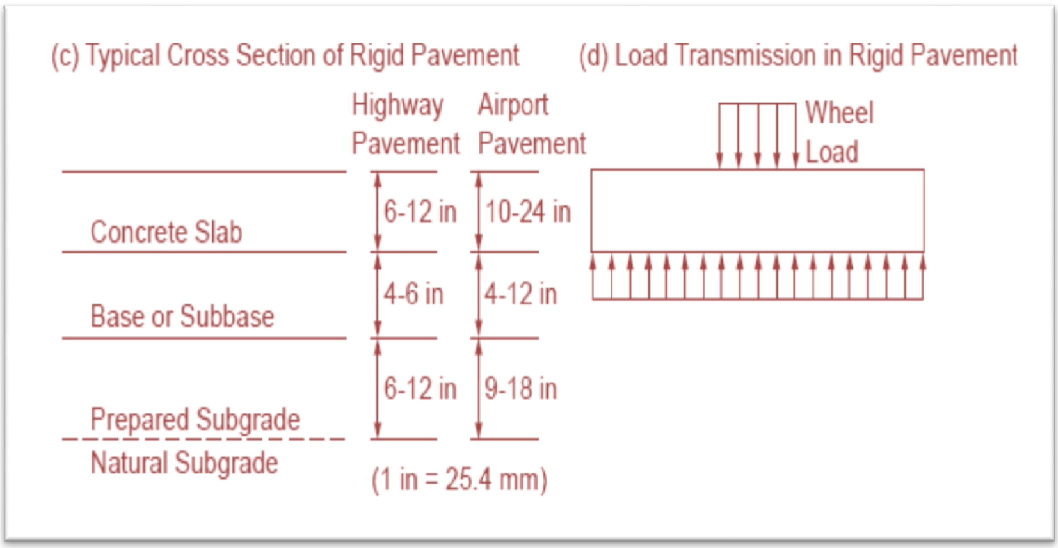


FIGURE 2.1 Rigid Pavement rigidity and Typical Thickness

Concrete is material which is strong in compression, but relatively weak when placed in tension. Tensile stresses may build up in concrete pavements because of shrinkage during the hydration process, temperature and moisture changes, and/or traffic loadings. When the tensile stresses are great enough, cracks occur. Joints are often used as a means of relieving stresses to control cracking. Joints can also serve to protect adjacent structures, or to accommodate paving operations. [11]

2.2 Types of Concrete Pavements

As detailed in Chapter one above, concrete pavements can be classified into four types: JPCP, JRCP, CRCP and PCP. Except for PCP with lateral prestressing, a longitudinal joint should be installed between two traffic lanes to prevent longitudinal cracking. Figure 2.2 illustrates the main characteristics of the four types of rigid pavements as presented below:

1. Jointed Plain Concrete Pavements (JPCPs):

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. The practice of using or not using dowels, and whether dowels or aggregate interlocks are used, varies among various agencies. Some practitioners use both types of load transfer across joints. [6]

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 efficiency of aggregate interlock decreases with 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. [7]

2. Jointed Reinforced Concrete Pavements (JRCPs):

The purpose using steel reinforcements in the form of wire mesh or deformed bars is to allow the use of longer joint spacing rather than increasing the structural capacity of rigid pavements. In this type of rigid pavements joint spacing varies from 30 to 100 ft (9.1 to 30 m), and thus 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. However, the number of joints and dowel costs decrease with the increase in joint spacing. Based on the unit costs of sawing, mesh, dowels, and joint sealants, Nussbaum and Lokken (1978) found that the most economical joint spacing was about 40 ft (12.2 m). Maintenance costs generally increase with the increase in joint spacing, and hence the selection of 40 ft (12 .2 m) as the maximum joint spacing appears to be warranted.[6]

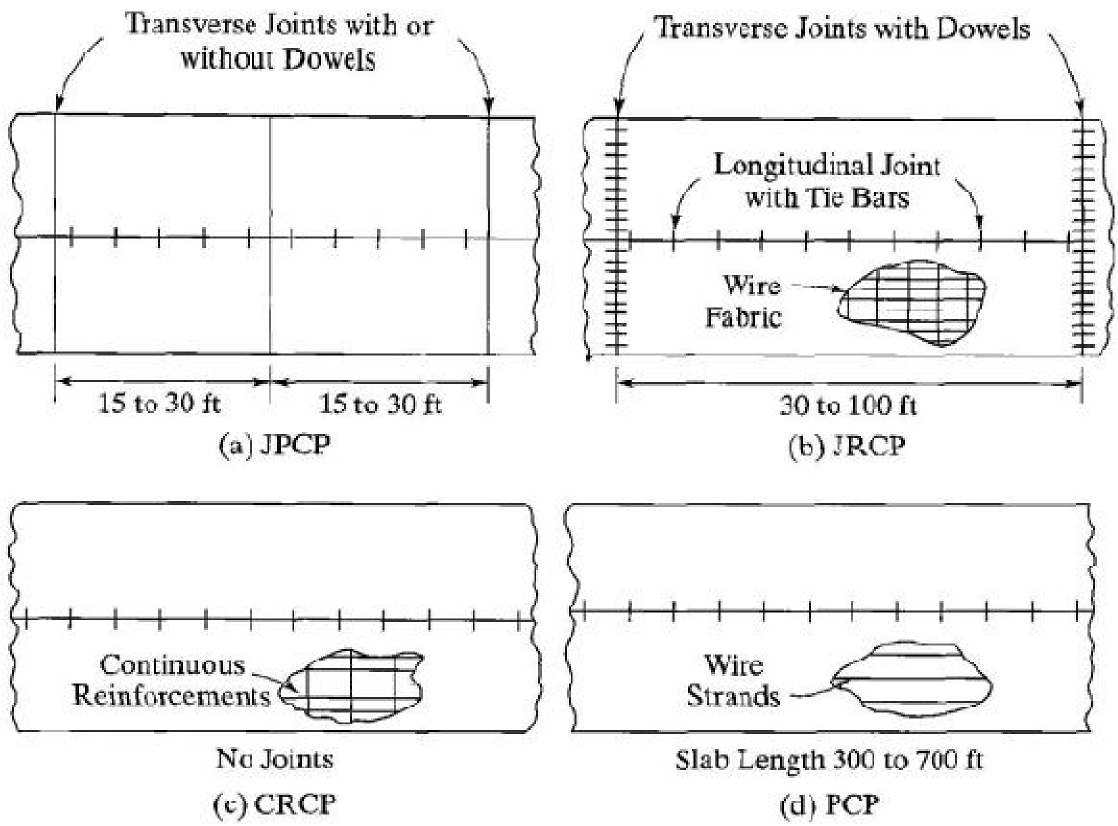


FIGURE 2.2 Four types of concrete pavements (1 ft = 0 .305 m)

3. Continuous Reinforced Concrete Pavements (CRCPs)**:**

Elimination of joints prompted the first experimental use of CRCP in 1921 on Columbia Pike near Washington, D.C. The advantages of the joint-free design were widely accepted by many agencies. In the United States of America (USA) more than two dozen States have used CRCP with a two-lane mileage totaling over 20,000 miles (32,000 km). It was originally reasoned that joints were the weak spots in rigid pavements and that the elimination of joints would decrease the required thickness of pavement. 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.

Formation of transverse cracks at relatively close intervals is a distinct 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. Occurrence of failure at the pavement edge rather than at the joint, does not necessarily justify using thinner CRCP. The 1986 AASHTO design guide suggests using the same equation or nomograph for determining the thickness of JRCP and CRCP. The amount of longitudinal reinforcing steel should be designed to control the spacing and width of cracks and the maximum stress in the steel.[6]

4. Prestressed Concrete Pavements (PCPs):

The thickness of concrete pavement required is governed by its modulus of rupture, MR which varies with the tensile strength of concrete. The pre-application of compressive stress to the concrete, greatly reduces the tensile stress caused by the traffic loads and thus decreases the required thickness of concrete. Prestressed concrete pavements have less probability of cracking and fewer transverse joints and therefore result in less maintenance and longer pavement life. They have been used more frequently in airport pavements than in highway pavements because the saving in thickness for airport pavements is much greater than that for highway pavements. Prestressed concrete pavements are still at the experimental stage, and their design arises primarily from the application of experience and engineering judgment (Huang, 2004)[6]. In this thesis investigation, jointed plain concrete pavement (JPCP) and jointed reinforced concrete pavement (JRCP) have been selected for comparative study.

2.3 Joints and Dowel Bars

Pavement joints are vital to control pavement cracking and pavement movement. Without joints, most concrete pavements would be riddled with cracks within one or two years after placement. Water, ice, salt and loads would eventually cause differential settlement and premature pavement failures. These same effects may be caused by incorrectly placed or poorly designed pavement joints. Joint spacing in feet for plain concrete pavements should not greatly exceed twice the slab thickness in inches, and the ratio of slab width to length should not be greater than 1.25. There are four types of joints in common use: contraction, expansion, construction, and longitudinal joints. Contraction joints are usually placed at regular intervals perpendicular to the center line of pavements. Expansion joints are used only at the connection between pavement sections and structures adjacent to the road. Longitudinal joints are used to relieve curling and warping stresses. Details of these joints, design and dimensions may be found elsewhere (AASHTO, 2003; Huang, 2004). [1]

Dowel bars (figure2-3) are used at joints on long slabs or where load transfer by interlock is suspect. Interlock depends upon many factors including the distance a joint will open as a result of shrinkage and/or temperature contraction. Joints without dowels are generally satisfactory if the joint opening is 0.04 inch or less. For doweled joints, the opening should be 0.25 inch or less. Hence, short slab pavements generally do not use dowels. However, it has become the practice of many engineers to use dowels regardless of joint spacing. It is to be recalled that the short slabs on the AASHO Road Test contained dowels. [13]

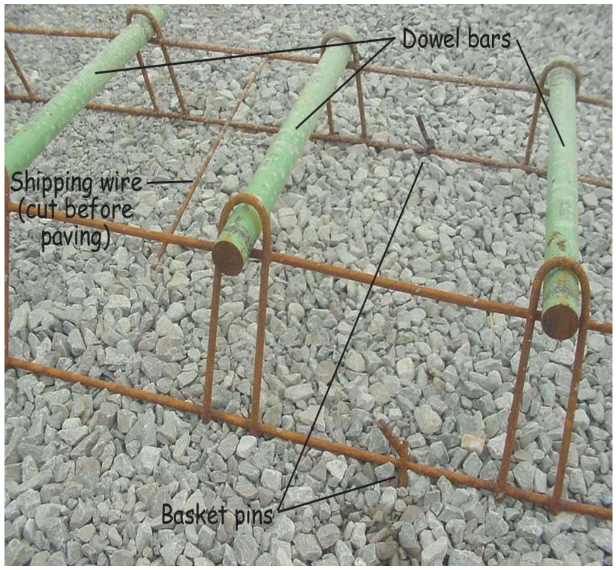


Figure 2-3 Dowel Bars

2.4 Design Parameters and Methodologies for JPCP and JRCP

The design methods adopted for rigid pavements is according to the Portland Cement Association's (PCA,1984) and American Association of State Highway and Transportation Officials (AASHO, 1993).

2.4.1 Portland Cement Association (PCA) Method

The Portland Cement Association (PCA) thickness-design procedure for concrete highways and streets can be applied to JPCP, JRCP, and CRCP. [8]

The design method is based on the following two criteria:

1. Fatigue: The method keeps pavement stresses due to repeated loads within allowable limits to prevent failure from fatigue cracking. The effects of wheel loads which produce stresses less than 51% of the modulus or rupture are ignored, indicating concrete will withstand unlimited stress repetition without failure.

2. Erosion: The method attempts to limit the effects of pavement deflections at joints and slab corners in order to control subgrade erosion, thus minimize joint faulting. [2]

2.4.1.1 Design factors

Based on the selection of doweled joints and concrete shoulders, the thickness design is governed by five design factors, namely concrete modulus of rupture (MR), modulus of subgrade reaction (*k*), subbase elastic modulus (E_{SB}), design period (n) and design traffic (Cumulative ESAL). These factors are discussed below.

1. Concrete Modulus of Rupture (MR)

The flexural strength of concrete represents the modulus of rupture determined at 28 days using ASTM C78-84 Standard Test Method specified for Flexural Strength of Concrete which applies simple-beam, third-point loading. In view of the fact that variations in MR have greater effect on design thickness than those in other material properties, the procedure recommends reduction of design MR by one coefficient of variation (CV). A CV of 15 % was incorporated into the design charts and tables, along with the effect of 28-day strength gain. [6]

2. Subgrade *k* and Subbase E_{SB}

If granular subbase or cement-treated base / subbase are used; the subgrade *k* is modified (increased) to obtain design *k* using Table A-1 or Fig A-1 of Appendix A.

3. Design Period

Design period is typically represented by the traffic analysis period. Because of variation in reliability of traffic prediction for longer periods, 20 years are generally a common pavement design period. However, shorter or longer design periods may be considered if economically justified.

4. Design Traffic

It is necessary to predict the number of repetitions of each axle load group during the design period. Information on initial traffic can be obtained from field measurements or other procedures. The initial daily traffic in two directions is multiplied by the directional (D) and lane (L) distribution factors to obtain the initial traffic in the design lane and projected for n years using a growth factor for a growth rate of r %. If n_i is the total number of load repetitions to be used in the design for the *i-th* load group, then

$$n_i = (N_A/100)*365/r*[(1+r)^n-1]*(ADT*D/100*P_T/100)*L \qquad (2.1)$$

Where

N_A = number of axles per trucks surveyed, say 100

ADT = Average Daily Traffic, veh. /day in both directions

D = direction split (the larger value is used in the design)

P_T = Percentage trucks in the traffic mix (% trucks)

r = annual traffic growth factor for design period n, and

L = the lane distribution factor which varies with the volume of traffic and the number of lanes.

Axle load distribution of truck traffic is required to compute the number of single, tandem and tridem axles of various weights expected during the design period. [6]

2.4.1.2 Load Safety Factors

In the PCA design procedure, the axle load is multiplied by a load-safety factor (LSF) 1.0, 1.1 or 1.2 depending on the volume of truck traffic.

2.4.1.3 Design Methodology

For the details and application of the design procedure, refer to the work sheet illustrated in Figure 6.5 of Chapter 6 on result and discussion. The design steps which are in tabular form are summarized hereunder:

1. Enter all design parameters and data
2. Assume a Trial thickness
3. Multiply Axle Loads by Load Safety Factor.
4. Compute the estimated projected (expected) repetition (n_i) for i-th load group using Eqn. (2.1).
5. If granular subbase or cement-treated base / subbase is used; modify the subgrade k to obtain the design k using Table A-1 or Figure A-1.
6. Determine the equivalent load stress for single / tandem axles from Table A-3. Use Table A-4 for erosion..
7. Divide stresses by M_R to get stress ratio factors.
8. from Figure A.2 obtain allowable repetition (N_i) for i-th load group corresponding to the load stress ratios (column 4); use Figure A.3 for erosion stress ratios.
9. Divide n_i by N_i to obtain fatigue ratio, and erosion damage. Report the sum for each and identify the larger value of the two as the design control criteria, normally fatigue:
10. If the total damage ratio (D_r) accumulated over the design period resulting from all load groups (Eqn. 2.2) is much greater than 1, the thickness is increased by successive 0.5 in. (127 mm) until the ratio is less than 1, and vice versa if the total damage ratio (D_r) is much less than 1 until the ratio is close to 1.[2]

$$D_r = \sum_i^m n_i / N_i \leq 1 \tag{2.2}$$

2.4.2 AASHTO Method

The design guide for rigid and flexible pavements was concurrently developed and published in the same manual. The design is based on empirical equations obtained from the AASHO Road Test, with further modifications based on theory, calibration and experience. [1]

2.4.2.1 Design Variables

a. Time Constraints: To achieve the best use of available funds, AASHTO design guide encourages using longer analysis period for high-volume facilities

b. Design Traffic: The design procedures are based on cumulative expected 18-kip (80-kN) equivalent single-axle load (ESAL) as in Table A.13

c. Reliability: Reliability is a means of incorporating degree of certainty into the design process to ensure that the various design alternatives will last the analysis 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.

Application of the reliability concept requires the selection of a representative standard deviation, S_0. Recommended values of S_0 range between 0.25 and 0.35. [6]

d. Serviceability: Initial and terminal serviceability indices must be established to compute the change in serviceability, ΔPSI used in the design equations.

The initial serviceability index PSI_i is a function of pavement type and construction quality. A typical value from the AASHO Road Test was 4.5 for rigid pavements. The terminal serviceability index PSI_t is the lowest value tolerated before rehabilitation, resurfacing and reconstruction are required. An index of 2.5 is suggested for design of major highways and 2.0 for highways with lower traffic.[6]

2.4.2.2 Design Equations

If an equivalent 18-kip (80-kN) single axle load is used, the design equation for rigid pavement is:

$$\log(w_{18}) = (Z_R \cdot S_O) + 7.35 \cdot \log(D+1) - 0.06 + \frac{\log\left(\frac{\Delta PSI}{4.5-1.5}\right)}{1+\frac{1.624 \cdot 10^7}{(D+1)^{8.46}}} + (4.22 - 0.32 \cdot P_t) \cdot \log\left[\frac{S_c \cdot C_d \cdot (D^{0.75} - 1.132)}{215.63 \cdot J \cdot \left[D^{0.75} - \frac{18.42}{\left(\frac{E_c}{k}\right)^{0.25}}\right]}\right] \quad (2.3)$$

w_{18} = the number of 18-kip (80-kN) single-axle load applications

Z_R = Normal deviate for a given reliability R

S_O = Overall Standard Deviation

D = Slab Thickness in Inches

ΔPSI = present serviceability index

P_t = the serviceability at time t

S_c = Modulus of Rupture of Concrete

C_d = Drainage coefficient

J = Load Transfer Coefficient

E_c = Elastic Modulus

k= Modulus of subgrade Reaction

Figure B.1 is a Nomograph for solving the design Eq. 2.3.

2.4.2.3 Design Chart

In order to apply the design Nomograph of figure 4.1 for determining design slab thickness, it is necessary to estimate the following input values required in the chart:

a. Modulus of Subgrade Reaction

The property of roadbed soil to be used for rigid pavement design is the modulus of subgrade reaction *k.* Figures B.2 and B.3 are used to estimate the appropriate design *k* values for various conditions. If a rigid foundation is near the surface [≤ 10 ft], Figure B.2 is used. In Figure B.3 the starting point is the subbase thickness, D_{SB}. If the slab is placed directly on the subgrade without subbase, the design *k* is obtained from Eq. 2.4, which relates *k*-value from a plate-load test to the resilient modulus of the roadbed soil, M_R.

$$k = \frac{M_R}{18.8} \tag{2.4}$$

The *k*-value is further modified to obtain effective modulus of subgrade reaction, k_{eff} using seasonal damage factor. [6]

b. Elastic Modulus of Concrete

The elastic modulus of concrete, E_c can be determined according to the procedure described in ASTM C469 or correlated with compressive strength. The following is a correlation recommended by the American Concrete Institute:

$$E_c = 57{,}000\,(f_c)^{0.5} \quad \text{psi} \tag{2.5}$$

Where

f_c is compressive strength of concrete.

The value of f_c usually used for concrete structures = 7,690 psi, giving elastic modulus of concrete, $E_c = 5\times10^6$ psi

c. Load Transfer Coefficient

The load transfer coefficient, J is a factor used in rigid pavement design to account for the ability of concrete pavement structure to transfer 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.[6]

Table B.1 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]

f. Drainage Coefficient

The drainage coefficient, C_d has similar effect as the coefficient J. As Eq. 2.3 indicates, increase in C_d is equivalent to increase in J, both causing increase in W_{18}. Table B.2 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.

2.5 Other Design Features

The performance of rigid pavements is affected by a variety of design features, including slab thickness, base type, joint spacing, reinforcement, load transfer, dowel bar, longitudinal joint design, tied concrete shoulders, and sub-drainage.

2.5.1 Joint spacing

The JPCP and JRCP design concept is to provide a sufficient slab thickness and joint spacing to minimize the development of transverse cracking.[9]

2.5.1.1 Jointed Plain Concrete Pavement (JPCP)

The spacing of joints in JPCPs depends more on the shrinkage characteristics of the concrete rather than on the stress in the concrete. Longer joint spacing causes the joint to open wider and decrease the efficiency of load transfer. Allowable joint spacing or slab length, *L* can be computed approximately by Eq. 2.6 (Darter and Barenberg, 1977). For JPCP, typical length of slabs range from 7.75 to 30 ft (2.4 to 9.1 m). In general, reducing the slab length decreases both the magnitude of the joint faulting and the amount of transverse cracking (Huang, 2004).[6]

$$L = \frac{\Delta L}{c(\alpha_t \times \Delta T + \epsilon)} \tag{2.6}$$

Where

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

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

α_t = The coefficient of thermal expansion of concrete, generally 5 t o 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, and

ϵ = The drying shrinkage coefficient of concrete, approximately 0.5 to 2.5x10^{-4}

If $\Delta L > 0.05''$ dowels are used.

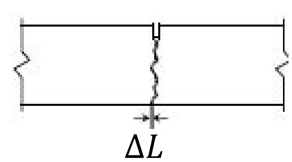


Figure 2.4 Joint Opening

2.5.1.2 Jointed Reinforced Concrete Pavement (JRCP)

For JRCP, typical length of slabs range between 21 and 78 ft (6.4 - 23.9 m). Generally, shorter joint spacing performs better, as measured by the deteriorated transverse cracks, joint faulting, and joint spalling (Huang, 2004). Eq. 2.7 is also applicable for JRCP.[6]

2.5.2 JRCP Reinforcement

Wire fabric or bar mats may be used in concrete slabs for control of temperature cracking. These reinforcements do not increase the structural capacity of the slab but are used for two purposes: to increase the joint spacing and to tie the cracked concrete together and maintain load transfers through aggregate interlock. When steel reinforcements are used, it is assumed that all tensile stresses are taken by the steel alone,

$$A_s = \frac{\gamma_c h L f_a}{2 f_s} \tag{2.8}$$

Where

A_s = is the area of steel required per unit width

γ_c= is the unit weight of the concrete

h = is the thickness of the slab

L = is the joint spacing or slab length

$f_a =$ Average friction coefficient between slab and foundation usually taken as 1.5, and

f_s = is the allowable stress in steel.

The steel is usually placed at the mid depth of the slab and discontinued at the joint. The amount of steel obtained from Eq. 2.8 is at the center of the slab and can be reduced toward the end. However, in actual practice the same amount of steel is used throughout the length of the slab. Pavement sections with less than 0.1% reinforcing steel often display significant deteriorated transverse cracking. Thus, a minimum of 0.1% reinforcing steel is recommended.[6]

C H A P T E R 3

CASE STUDY ROAD PROJECT

3.1 Location and Characteristics of the Case-Study Road Project

Al-Ilaifoun highway segment starts from km 1 at the junction of Al-Ilaifoun road with the Ring road to 22 km southwards in Al-Ilaifoun region (Figure 3.1). Data on the project include details of traffic volumes at 3 stations together with vehicle classifications and speed distribution. The 3 survey stations were located along the road from East-Nile region to Al-Ilaifun area.

Fig.3.1: Al-Ilaifoun highway segment showing the survey Stations 1, 2 and 3

The project offers a convenient option for the public transport within the State for domestic use. Most of the inhabitant areas and districts are not far from the proposed road location. During the traffic surveys, it was found that minibuses did not constitute high percentage in the traffic mix. On the other hand, the share of trucks and buses was significantly high. This was attributed to the fact that use of this part of the road is mandatory for Interstate buses and trucks to and from Al-Jazeera and River Nile States.

The region for the study was defined to encompass the area of the expected policy impact. The study area is bound by the parts influenced by the transportation system. It is anticipated that Al-Ilaifoun road will impose impact on the domestic transport system in future, as well as having immediate effect on freight transport.

Interactions with the area outside the cordon are defined via external stations which effectively serve as doorways to trips. This includes trips from and to other States by buses, trucks or passenger cars), and traffic through the study area.

Once the study area was defined, it was then divided into a number of small traffic-analysis zones (TAZs) represented by Stations 1, 2 and 3 (Figure 3.1). The external zones were defined by the catchment area of the major transport links from other States to and from Khartoum State in terms of trucks and interstate buses. Three stations were used for traffic surveys in this study, the proposed triple carriageway highway road is represented by existing single carriageway road type by now, the study were chose the most typical points in existed road to represent as close as much circumstances and features of the proposed highway. Summary of the traffic data from the 3 stations of the study area were as follows:

Station 1: Daily volume: 12716 vpd Passenger cars, 61% Trucks and buses 39%.

Station 2: Daily volume: 11288 vpd Passenger cars, 69% Trucks and buses 31%.

Station 3: Daily volume: 8266 vpd Passenger cars, 58% Trucks and buses 42%.

3.2 ESAL for the Case Study

For ESAL computations, recently, the Ministry of Interior converted the operation of Khartoum-Medani Highway west side of the Blue Nile one-way to Al-Jazeera State for trucks and buses. North-bound commercial traffic from Al-Jazeera was directed to use east side of the Blue Nile. As such, the percentage of trucks and buses in this direction was taken as 53 %. Since the proposed design is providing 3 lanes for each direction, so percentage of trucks in design lane was taken as 80%. Table 3.1 summarizes the results of traffic analyses at the 3 stations for pavement design purposes.

TABLE 3.1 Traffic Analyses for Pavement Design

	Station 1	Station 2	Station 3
Traffic Composition and Parameters:			
Analysis Period (years)	20	20	20
AADT (vpd)	12716	11288	8266
Percentage of heavy trucks (above class 4)	39	31	42
Directional split of truck traffic, %	53	53	53
Percentage of trucks in the design lane	70	70	70
Truck equivalency factor	1.35	1.35	1
Annual truck-volume growth rate, %	3	3	3
Annual truck weight growth rate, %	0.6	0.6	0.6
Traffic Analysis for Pavement Design			
Traffic volume growth factor	1.75	1.75	1.75
Truck growth factor	1.12	1.12	1.12
Design year AADT	22, 298	19, 794	14, 494
Average AADT	17, 507	15, 541	11, 380
Design year truck factor	1.12	1.51	1.12
Average truck factor	1.06	1.43	1.06
AADT in one direction	9, 279	8, 237	6, 032
Truck AADT in one direction	3, 619	2, 553	2, 533
Number of Daily 80-kN (18-kip) ESALs	3836	3654	2686
Design 80-kN (18 kip) ESALs	19.6E+06	18.6E+06	13.7E+06

3.3 Upgrading of Al-Ilaifoun Road to Dual Highway

Later the Project Administration upgraded Al-Ilaifoun highway to two-way divided facility in order to accommodate the increasing traffic from neighboring States as well as reducing traffic accidents (Figure 3.2).

Figure 3.2: Al-Ilaifoun Highway Upgrading Under Construction

3.4 Pavement Structural Design Methodology

The present study included independent structural design of JPCP and JRCP for the road project and compare design thickness of rigid pavement obtained through manual and GalalM-RP software program.

In general, the main pavement design factor is the design traffic in term of cumulative equivalent standard axle load (ESAL). Data were collected for the road project including study of traffic reports. Traffic analysis was carried out to determine the design-life ESAL for rigid pavement design. The procedure is detailed in Chapter four.

The recommendations in the Material reports for the road project are presented in chapter 4. The strength parameters of the various pavement layer materials were measured in term of California Bearing Ratio (CBR). The design CBR was carried out in accordance with AASHTO. Established correlations were applied to obtain the resilient modulus, M_R values and reported in Chapter Four and Appendix B. Furthermore, Chapter Four also includes AASHTO modification of the modulus of sub-grade reaction k to determine the combined k for rigid-pavement design. AASHTO and PCA structural design methods were then selected for jointed plain concrete pavement (JPCP) and jointed reinforced concrete pavement (JRCP).

GalalM-RP software program for the design thickness of JPCP and JRCP were prepared for PCA design method.

CHAPTER 4

DESIGN OF JPCP AND JRCP FOR CASE STUDY ROAD

4.1 Introduction

In this chapter the structural design of JPC and JRC road pavements are presented. The 22-km Al-Ilaifoun Highway segment introduced in Chapter 3 will be able to accommodate all traffic generated in future. In this proposed road the percentage trucks in design lane is taken as 80 %. Al-Ilaifoun highway location, characteristics and all relevant data were detailed in chapter 3. For design consideration, the following design aspects are discussed:

- Load Stresses, subgrade resilient modulus, M_R and modulus of subgrade reaction, *k*
- Thickness design
- Joint spacing, reinforcement design, longitudinal joint design, ties bars and transverse joints and design of dowels.

4.2 Load Stresses

A rigid pavement for highways consists of relatively thin concrete slab placed on the sub-grade or a base course/subbase. The load-carrying capacity of the pavement base-subgrade structure is brought about largely by the beam action of the pavement. Since the concrete slab is the major component of the structure, stresses in concrete pavements have been given detailed consideration by various investigators. Stresses in rigid pavements can result from several causes in addition to wheel loads. These include volumetric changes in the subgrade and/or subbase, changes in moisture and restrained temperature variations introducing curling stresses.

The anticipated traffic carried by a highway pavement related to equivalent 18,000-lb single-axle loads (ESALs), average daily traffic (ADT), or average daily truck traffic (ADTT). Since truck traffic is the major stress-inducing load to pavements compared to passenger cars, the estimate of trucks using the pavement is critical to the structural design for the pavement life.

4.3 Subgrade Resilient Modulus M_R and Reaction Modulus *k*

The resilient modulus, M_R represents the elastic modulus of subgrade in conjunction with the elastic theory, although most paving materials are not elastic as they experience some permanent deformation as well after each load application. However, for small repeated loads compared to the material strength, the deformation under each load repetition is mostly recoverable and proportional to the load and as such may be assumed elastic.

Determination of a specific subgrade strength parameter required for design, whether M_R, *k* or California Bearing Ratio (CBR), depends on available test equipment. In the event of non-availability of the particular device to directly determine M_R (Level 1), this study resorted to correlations with other parameters that can be determined (Level 2). Typical relationships include the following:

Asphalt Institute (AI) equation (Heukelom and Klomp, 1962)

$$M_R \text{ (psi)} = 1500*(CBR) \tag{4-1a}$$

$$M_R \text{ (MPa)} = 10.342*(CBR) \tag{4-1b}$$

These correlations have the limitation that they were developed for fine-grained, non-expansive soils with soaked CBR ≤ 10. To account for materials with CBR greater than 10, the Mechanistic Design Guide (NCHRP 1-37A, 2002) recommended (4-2).

$$M_R \text{ (psi)} = 2555*(CBR)^{0.64} \tag{4-2a}$$

$$M_R \text{ (MPa)} = 17*(CBR)^{0.64} \tag{4-2b}$$

The soils encountered along the alignment of the proposed road are not suitable for embankment construction and should be removal and replaced with fill material for a depth of at least 6 in. (300 mm). As the existing subgrade material had an average CBR of 4 only, the value of 12 was selected as satisfactory for design.

Hence, design $M_R = 2555\,(CBR)^{0.64} = 2555\,(12)^{0.64}$ =12, 500 psi (86.2 MPa)

According to AASHTO, the modulus subgrade reaction, k is then obtained from equation (2.4):

$$k_{subgrade} = \frac{12500}{18.8} = 83 \text{ lb/in.}^3 \text{ (MPa/mm}^3\text{)}$$

4.4 Design of Slab thickness for JPCP and JRCP

The two design methods applied in this study included the Portland cement Association Method (PCA, 1984) and AASHTO Method (1993)

4.4.1 Portland Cement Association Design Method

The design parameters and factors for PCA Design Method depend on the selected design category. In the present case the design uses doweled joints without concrete shoulders and thus the main four design factors are:

1. Concrete modulus of rupture (MR): From Section 2.4.1.1the MR = 650 psi
2. Subgrade and Subbase combined support (*k*): With an 8-in. (203.2-mm) untreated subbase placed on the subgrade of *k* value = 83 Ib/in.^{3}, from Figure A-1 (a) the design *k* was found to be 125 Ib/in.^{3}
3. Design period = 20 years
4. Design Traffic: Annual traffic growth rate was typically assumed to be 5 % for the project design life. The data gathered from the selected three stations was analyzed to determine the average daily traffic volumes for the different types of vehicles using the proposed route and as tabulated in Table 4.1:

Table 4.1 Average Daily Traffic Volumes (Current $ADT_{current}$)

Passenger Cars	Buses	Trucks	Total
10,757	3,977	6,780	21,514

Source: Traffic counting and transportation demand report for the year 2012

2-way Design ADT = $ADT_{current} \times G$, where G = growth factor (Figure A.2)

2-way Design ADT = $21514 \times 1.6 = 34422$

Design ADT in one direction = $34422 \times 0.6 = 20653$ veh / day

Average daily truck traffic (ADTT) = Design ADT $\times P_t$, where P_t = % trucks = 20% $= 34422 \times 0.20 = 6884$, or 4130 Trucks / day in one direction. Therefore, the total number of trucks on the design lane during the design period

$= 4130 \times 365 \times 20 \times 0.80 = 24.12 \times 10^6$ trucks, which was the basis for obtaining the axle-load distribution in Table 4.2. Column 3 in Table 4.2 is the number of load repetitions to be used for Al-Ilaifoun Road and can be obtained by multiplying Column 2 by (Trucks on the design lane during the design period) / 1000.

The Design Procedure for PCA Design Method is conducted in a tabular form as in Figure 6.5:

1. Assume a thickness = 9.5 in. (241.3 mm).

2. Multiply Axle Loads of column 1 by Load Safety Factor and enter in column 2

3. Calculate the estimated projected (expected) repetition (n_i) Table 4.2

4. k =125 Ib/in.^{3} from Section 3.1.1

TABLE 4.2 Axle Load Distributions for Al-Ilaifoun Road

Axle Load (kips)	Axles per 1000 trucks	Axles in the design period
Single Axles		
30	0.45	10854
28	0.85	20501
26	1.78	42932
24	5.21	125661
22	7.85	189336
20	16.33	393867
18	25.15	606598
16	31.82	767473
14	47.73	1151209
12	182.02	4390177
Tandem Axles		
52	1.19	28702
48	2.91	70187
44	8.01	193195
40	21.31	513980
36	56.25	1356705
32	103.63	2499473
28	121.22	2923729
24	72.54	1749607
20	85.94	2072804
16	99.34	2396001

5. The equivalent stress 207.5/(194 axles) from Table 4.3 (items 9 and 12, respectively). Use Table 4.4 for erosion 2.595/2.793 (items 11 and 12, respectively).

6. Divide stresses by M_R to get stress ratio factors 0.319/0.298 (items 10 and 13, respectively).

7. From Figure A.3 obtain allowable repetition (N_i) for *i*-th load group corresponding to the stress ratios (column 5); use figure A.4 for erosion (column 7).

8. Divide n_i by N_i and to get fatigue ratio (column 6), and erosion damage (col. 8).

9. the damage ratio (D_r) accumulated over the design period resulting all m load groups (sum of column 6 = 98.5 %) is for fatigue and (sum of column 8 = 64 %) is for erosion damage, Both are less than 100%, with fatigue criteria being critical.

However, 98.5 % is much less than 100 % indicating the slab thickness of 9.5 in. (241.3 mm) is over design. Thus, the design was repeated using 9-in (229 mm) thickness resulting in fatigue damage of 193.9%, much higher than 100 % (under design). Therefore, a slab thickness of 9.49 in. (241 mm) would be adequate. In general, fatigue criteria will normally control the design of pavements subjected to light to medium traffic. Erosion criteria will usually control the design of pavements subjected to heavy traffic with doweled joints.

TABLE 4.3 Equivalent Stresses for Slabs without Concrete Shoulders

Slab thickness (in.)	k of Subgrade–subbase (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

Note. Number at left is for single axle and number at right is for tandem axle (single/tandem); 1 in. = 25 .4 mm, 1 pci = 271.3 kN/m^3.

Source. After PCA (1984).

TABLE 4.4 Erosion Factors for Slabs with Doweled Joints and no Concrete Shoulders

Slab thickness (in.)	k of Subgrade-subbase (pci)					
	50	100	200	300	500	700
4	3.74/3.83	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

Note. Number at left is for single axle and number at right is for tandem axle (single/tandem); 1 in. = 25 .4 mm, 1 pci = 271.3 kN/m^3.

Source. After PCA (1984).

4.4.2 AASHTO (1993) design method

The design parameters, factors and input variables for AASHTO design method are much more than for PCA. For design with subbase thickness, D_{SB} = 8 in., these design elements are as follows:

- Subbase Elastic Modulus, $E_{SB} = 17.6 \times (30)^{0.64} = 22503$ psi
- Roadbed Soil Resilient Modulus M_R (*k*) =12500 psi
- Composite Modulus of Subgrade Reaction, k_{∞}=500 pci (Figure B.3)
- Effective Modulus of subgrade Reaction k = 550 pci (Figure B.2 with Subgrade depth to rigid foundation D_{SG} = 5 ft.)
- Traffic, From Table 3.1 say W18 =20 million
- Design Reliability, R =95% (Table B.3)
- Overall Standard Deviation $[0.25 - 0.35]$, assume S_0 =0.29
- ΔPSI = 4.5 - 2.5=2
- Elastic Modulus, E_c =5,000,000 psi (equation 2.5)
- Modulus of Rupture, S_c (MR) = 650 psi
- Load Transfer Coefficient, J =3.1 (Table B.1)
- Drainage Coefficient, C_d =1.0 (Table B.2)
- Normal Deviate for a given Reliability R, Z_R = -1.645 (Table B.4)

The required thickness *D* can be determined by using the two-part nomograph of Figures 4.1 (a) and 4.1 (b) following the steps below:

1. Starting from Figure 4.1 (a) with *k* = 550pci (149 MN/m ^{3}), a series of lines, as indicated by the arrows, are drawn through E_c = 5 x 10^6 psi (34.5 GPa), S_c = 650 psi (4. 5MPa), J = 3.1, and Cd = 1.0 until a scale of 60 is obtained at the match line.

2. Starting at 60 on the match line in Figure 4.1 (b), a line is drawn through ΔPSI = 2 until it intersects the vertical axis.

3. From the scale with R = 95%, a line is drawn through S_o = 0.29 and then through W18 = 20 x 10^6 until it intersects the horizontal axis.

4. A horizontal line is drawn from the last point in Step 2, a vertical line from that in Step 3. The intersection of these two lines gives a D of 9.85 in. (250 mm), which is rounded to 10 in. (254 mm).

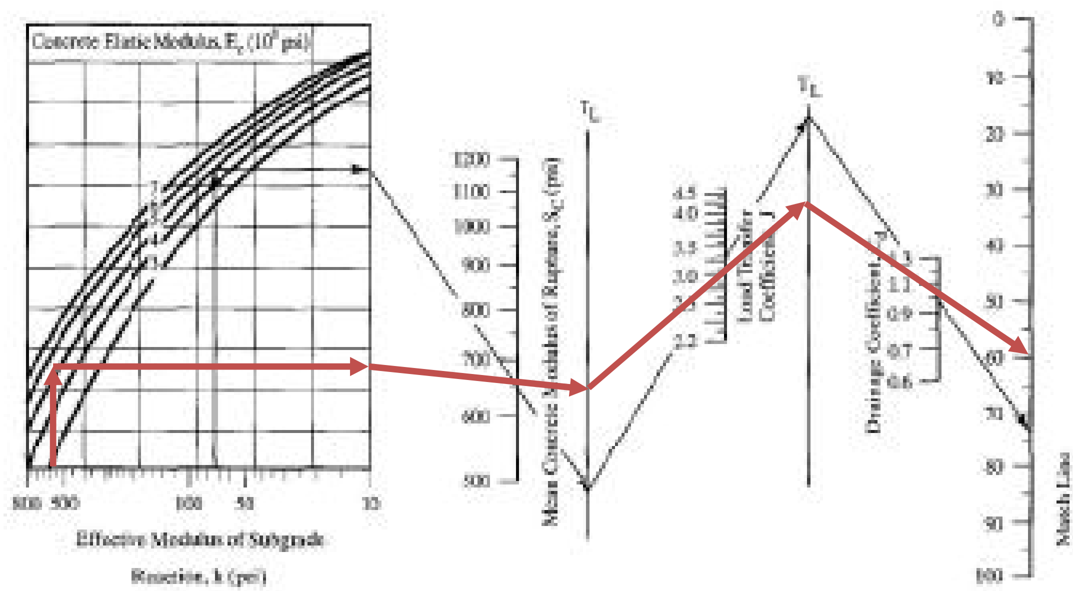


FIGURE 4.1 (a)

Design chart for rigid pavements based on mean values (1 in. = 25 .4 mm, 1 psi = 6.9 kPa, 1 pci = 271 .3 kNIm^3). (From the *AASHTO Guide for Design of Pavement Structures.* Copyright 1986 . American Association of State Highway and Transportation Officials, Washington, DC. Used by permission).

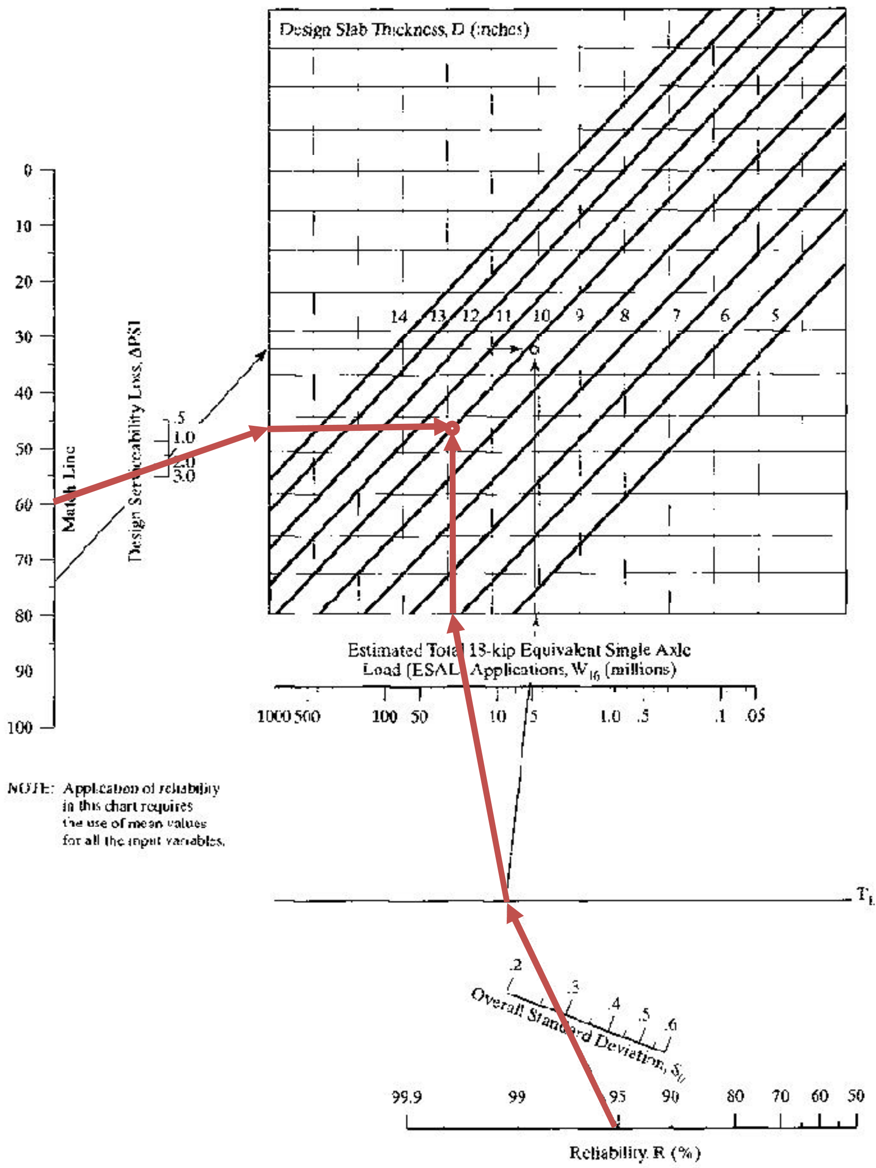


FIGURE 4.1 (b) (Continued)

4.4.3 Other design features:

1. **Joint spacing for JPCP:**

ΔT = 145°F (63°C)

α_t =6 x 10^{-6}/°F (9.9 x 10^{-6}/°C)

ΔL = 0.25 in (doweled joint)

C = 0.8 for granular sub-base.

ϵ = 2.5 x 10^{-4}

From Eqn. (2.7)

$$L = \frac{0.25}{0.8(6\times10^{-6}\times145+2.5\times10^{-4})} = 279 \text{ in.} = 23 \text{ ft} = 5.8 \text{ m}$$

2. **Joint spacing for JRCP:**

ΔT = 120°F (49°C)

α_t =5 x 10^{-6}/°F (9.9 x 10^{-6}/°C)

ΔL = 0.25 in (doweled joint)

C = 0.8 for granular sub-base.

ϵ = 0.5 x 10^{-4}

From Equation (2.7)

$$L = \frac{0.25}{0.8(5\times10^{-6}\times120+0.5\times10^{-4})} = 480.8 \text{in.} = 40 \text{ft} = 12.2 \text{ m}$$

3. **JRCP Reinforcement**

It is intended to determine the required wire fabric for a three-lane concrete pavement, 10-in. (254-mm) thick, 40-ft (12.2-m) long and 36-ft (10.98-m) wide, with a longitudinal joint at the center as illustrated in Figure 4.2.

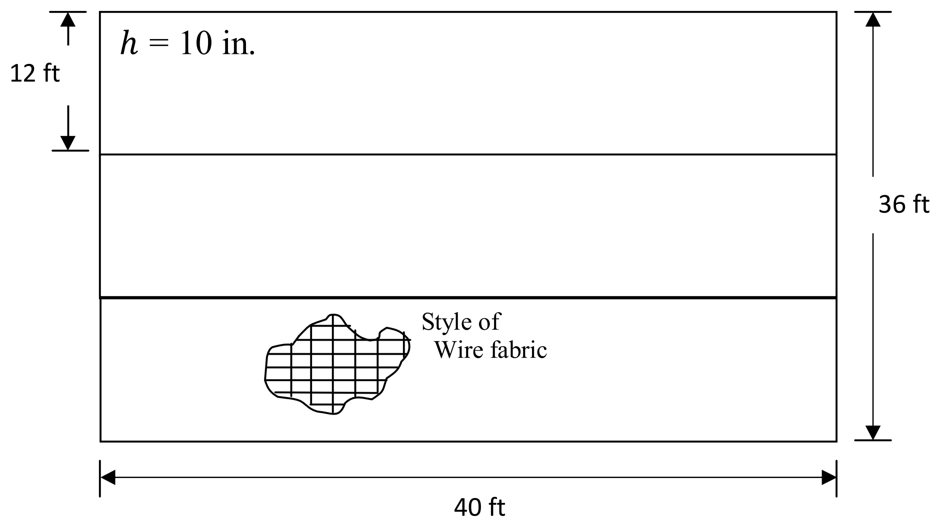


Figure 4.2: Schematic illustration of wire reinforcement

Pavement thickness, h = 10 in.

γ_c = 150 pcf = 0.0868 pci (23.6 kN/m^3); f_a = 1.5

f_s = 43, 000 psi, smooth, cold-drawn wire (Table B.4)

Computation of the required longitudinal steel is as follows:

L = 40ft = 480 in.

From equation (2.8):

$$A_s = \frac{0.0868 \times 10 \times 480 \times 1.5}{2 \times 43000} = 0.00727 \frac{\text{in.}^2}{\text{in.}} = 0.08724 \frac{\text{in.}^2}{\text{ft}}$$

The required transverse steel: 12' (lane) +12' (lane) + 12' (lane)

L = 36ft = 432 in.

From equation (2.8):

$$A_s = \frac{0.0868 \times 10 \times 432 \times 1.5}{2 \times 43000} = 0.006540 \frac{\text{in.}^2}{\text{in.}} = 0.07848 \frac{\text{in.}^2}{\text{ft}}$$

From Table (B.5), use 6 × 12 – W4.5 × W8 with cross sectional areas of 0.09 in.^{2} (58 mm^2) for longitudinal wires and 0 .08 in .^{2} (52 mm^2) for transverse wires.

4. Longitudinal Joint Design for JPCP and JRCP

The longitudinal joint design was found to be a critical design element. Both inadequate forming techniques and insufficient depths of joint can contribute to the development of longitudinal cracking. There was evidence of the advantage of sawing the joints over the use of inserts. The depth of longitudinal joints is generally recommended to be one-third of the actual, not designed, slab thickness, but might have to be greater when stabilized bases are used.

Longitudinal Joints run parallel to the pavement length (along the lane) and serve to control longitudinal cracking. These joints are produced by either sawing the slab early in the curing process, or by placing an insert in the plastic concrete at the desired joint location. Longitudinal joints are normally placed at the edges of traffic lanes.

5. 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.

The length of tie bars is governed by the allowable bond stress μ. For deformed bars, an allowable bond stress of 350 psi (2.4 MPa) may be assumed. The length of bar should be based on the full strength of the bar, namely,

$$t = \frac{1}{2}\left(\frac{f_s d}{\mu}\right) \tag{4.3}$$

The length t should be increased by 3 in. (76 mm) for misalignment. 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.

To determine the diameter, spacing, and length of the tie bars required for JPCP, as shown in Figure 3.2 Assume f_s = 27,000 psi (186 MPa) for billet steel (Table B.4) . With L = 12 ft = 144 in. (3.66 m), from equation 2.8:

$$A_s = \frac{0.0868 \times 10 \times 144 \times 1.5}{2 \times 27000} = 0.00347\ \frac{\text{in.}^2}{\text{in.}}$$

If No.4 (0.5 in. or 1.2 mm) bars are used, from Table B.6, the cross-sectional area of one bar is 0.2 in.^{2} =129 mm^2. The spacing of the bar = 0.2/0.00347 = 58 in. (1464 mm).

Assume that μ = 350 psi (24 MPa), from equation 4.3:

$t = \frac{1}{2}\left(\frac{27000 \times 0.5}{350}\right)$ = 19.3 in. (353 mm). After adding 3 in. (76 mm), t = 19.3 + 3 = 22.3 in. (use 24 in. or 610 mm).

The design selected is No.4 deformed bars, 24 in. (610 mm) long and 3 ft (0 .9 m) on centers.

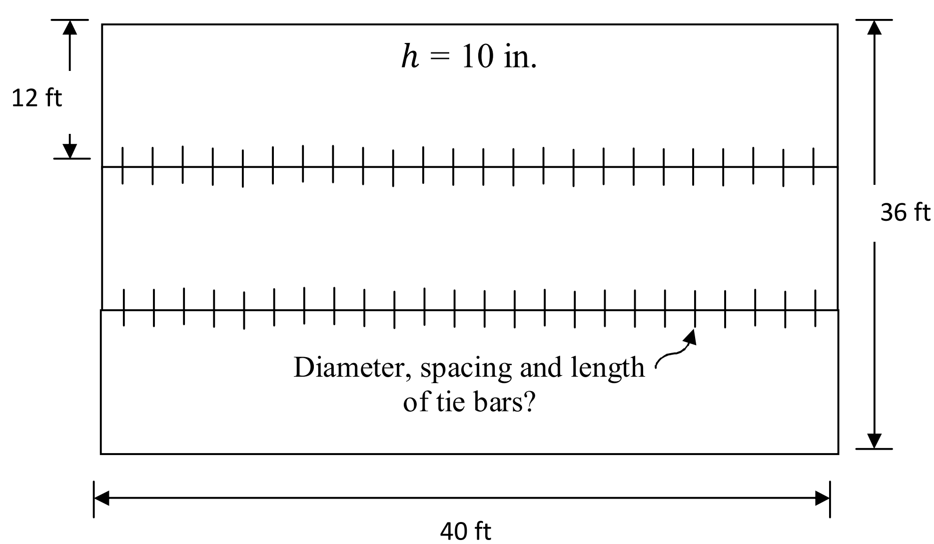


Figure 4.3: Diameter, spacing, and length of the tie bars for JPCP

6. Transverse Joints for JPCP and JRCP

Transverse Joints run perpendicular to the pavement length (across the lane) and serve different functions depending on the pavement type. Expansion Joints allow for expansion of the pavement due to temperature changes. Expansion joints are typically 2 inches wide, although widths up to 4 inches are sometimes used. Due to the width of the joint, load transfer devices are necessary. These are usually dowel bars with caps that allow the pavement and bar to move independently in the longitudinal direction. Expansion joints are costly to construct and maintain.

7. Design of Dowels

Dowel bars are usually used across a transverse joint to transfer the loads to the adjoining slab. The stress and deflection at the joint are much smaller when the loads are carried by two slabs, instead of by one slab alone. The use of dowels can minimize faulting and pumping which has been considered by the Portland Cement Association (PCA, 1984) as a factor for thickness design.

The design of dowels and joints is mostly based on experience, although some theoretical methods on the design of dowels are available. The size of dowels to be used depends on the thickness of slab. Table A.2 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. In a recent edition of joint design, PCA (1991) recommended the use of 1.25 in. (32 mm) diameter dowels for highway pavements less than 10 in. (254 mm) thick and 1.5 in. (38 mm) diameter dowels for pavements 10 in. (254 mm) thick or greater .

A minimum dowel diameter of 1.25 to 1.5 in. (32 to 38 mm) is needed to control faulting by reducing the bearing stress in concrete.

Dowel bars were found to be effective in reducing the amount of joint faulting when compared with non doweled sections of comparable designs. The diameter of dowels had an effect on performance, because larger diameter bars provided better load transfer and control of faulting under heavy traffic than did smaller dowels. It appeared that a minimum dowel diameter of 1.25 in. (32 mm) was necessary to provide good performance.

CHPTER 5

APPLICATION OF SOFTWARE PROGRAM

5.1 Introduction

GalalM-RP computer program was written in Visual Basic and can be run on computers with visual studio 2008 Windows 95 or higher. Details on the use of the software can be found in this chapter.

The GalalM-RP computer program applies only to rigid pavements with doweled joints and without Concrete shoulder. It can be applied only to layered systems under single, dual, dual-tandem with each layer behaving differently.

Galal-R.P has been developed for design thickness by Portland Cement Association's (PCA) thickness-design procedure for concrete highways and streets 1984 as same as in chapter 4, and it is educational and training tool as well as a design tool. The user is assisted in selecting design inputs by using the recommended values are shown on the screen along with a brief explanation during the design process.

To facilitate entering and editing data, some tables and charts can be used. The program uses menus and data entry forms to create and edit the data file.

Although the large number of input parameters appears overwhelming, default values are provided to many of them, so only a limited number of inputs will be required.

Rigid pavement computer screens divided into three screens as follows:

Screen one: main Screens

Screens two: PCA traffic Analysis and it is divided into two data input (user input) and data output (program calculates).

Screen three: calculations of pavement thickness

The program described in this chapter has been applied to several examples to test its accuracy and suitability for the proposed applications. Case study was prepared to include all software applications in the fild of pavements design.

5.2 MAIN SCREEN:

Galal-R.P has been designed screens and windows of the program by clicks and mouse movement light.

Figure 5.1 main screen

To open new file click File ⟶ open ⟶ PCA (Figure 5.1)

5.3 TRAFFIC ANALYSIS SCREEN:

Window of traffic Analysis appears immediately after the screen one, this window allows user to enter all input about traffic like current ADT (Average Daily Traffic), annual growth rate (%), growth Factor (G), percent of trucks, lane distribution factor L and Design period. When the user presses a button total Truck (figure 5.2) the program calculation: design ADT, ADTT (Average of Daily trucks traffic), truck traffic each way and total trucks in design period on design lane.

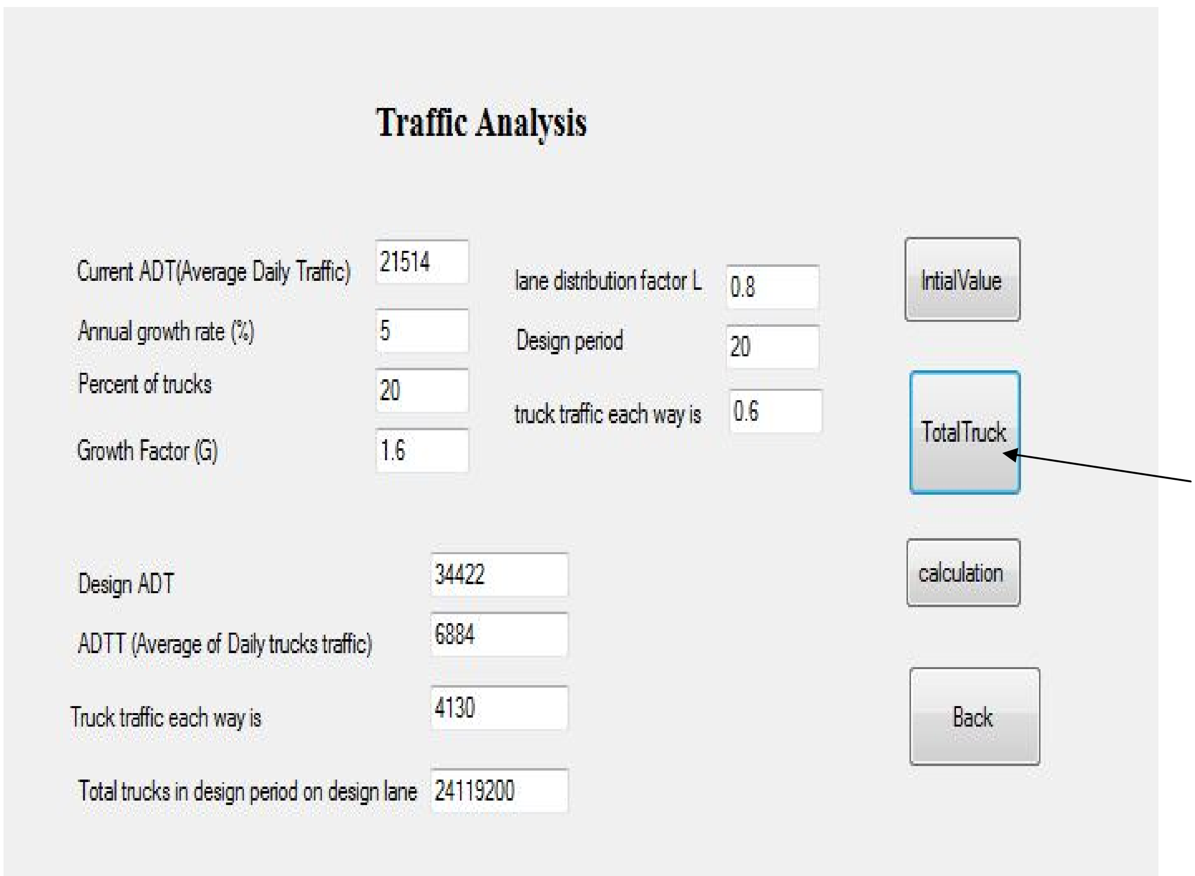


Figure 5.2 Traffic Analysis screen

5.4 CALCULATIONS OF PAVEMENT THICKNESS SCREEN:

When we click a button calculation in screen two the program open calculations of pavement thickness screen figure 5.3 show the screen.

User input in this window is: Trial thickness, subbase-subgrade *k*, modulus of rupture MR, Load safety factor LSF, however the default values are recommended and represent the values assumed Then the program does all the other calculation by clicks and mouse movement.

This screen is provided for the user's information and may be skipped as it is repeated by new parameter. This screen is particularly useful when analyzing the impact of one design variable. Suppose the user wants to see the impact of increasing subbase-subgrade *k* while keeping modulus of rupture MR, Load safety factor LSF constant. By entering the same design period, the user can immediately see the change in thickness required for each change in weight. Likewise, any variable can be changed while holding other variables constant.

When the user presses a button display Result, total fatigue percent and total damage percent via trail thickness design regarding the design is displayed on the summary screen. The summary display is dynamic and will change depending upon design features.

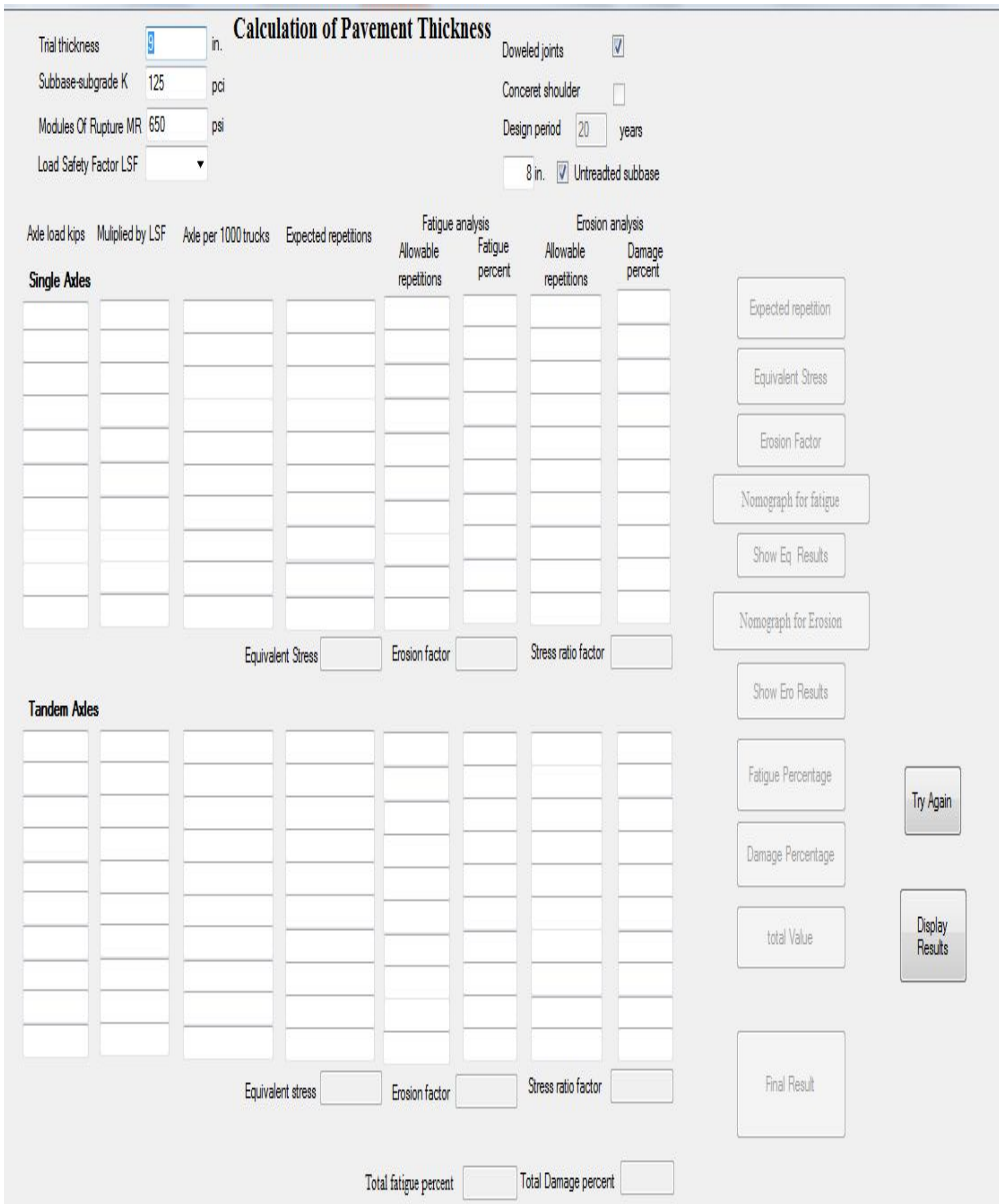
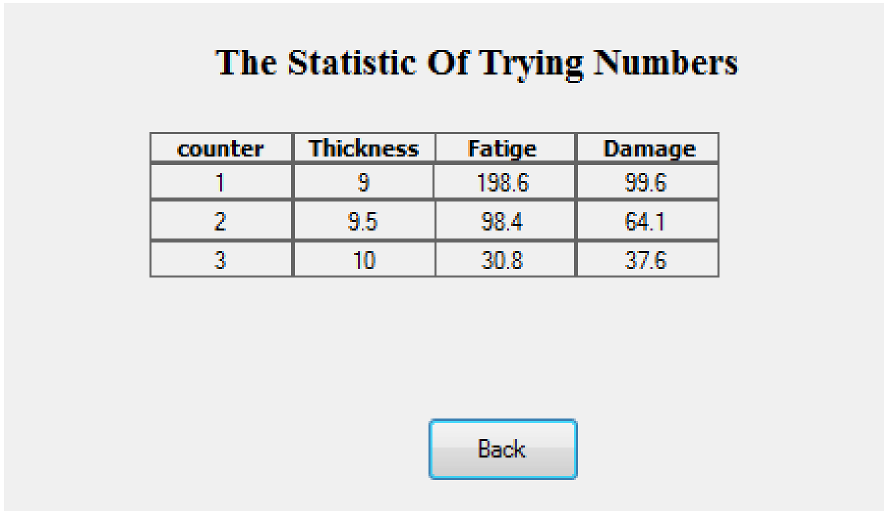


Figure 5.3 calculations of pavement thickness screen



counter	Thickness	Fatige	Damage
1	9	198.6	99.6
2	9.5	98.4	64.1
3	10	30.8	37.6

Figure 5.4 Display Result Screen

The objective of processing controls in GalalM-RP is to provide reasonable assurance that data processing has been performed accurately, without any omission or duplication of transactions. Examples of processing controls include:

- Run-to-run total: total such as expected repetition, equivalent Stress, erosion factor obtained at the end of one processing run are distributed to the next run and corresponding totals produced at the end of the second run.
- Control total reports: Control totals, such as equivalent Stress, erosion factor, can be calculated during processing and reconciled to input totals or totals from earlier processing runs.
- File and operator controls: External and internal table and figure ensure that the proper files are used in applications.

The objective of output controls is to ensure that only authorized persons receive output or have access to files produced by the system. Some common output controls include:

- Control total reports: Compare controls totals to input and run-to-run control totals produced during transaction processing.
- Master file changes: Any changes to master file information should be properly authorized by the entity and reported in detail to the user department from which the request for change originated.
- Output distribution: Systems output should only be distributed to persons authorized to receive the output.

CHAPTER 6

RESULTS AND DISCUSSION

6.1 Results

As presented in Chapter 2 PCA design method, the follow up specifies design input layer strength parameters design traffic thereafter, its assumes a trial thickness and evaluates total fatigue and erosion doesn't exceed 100%. The GalalM-RP develop converts the above manual procedure into systematic computer program analysis. The results obtained from GalalM-RP are presented and discussed below.

Results of PCA Traffic Analysis when using the program GalalM-RP give:

- Design ADT = 34422
- ADTT (Average of Daily trucks traffic) = 6884 Trucks / day
- Truck traffic each way is = 4130
- Total number of trucks on the design lane during the design period = 24119200 trucks

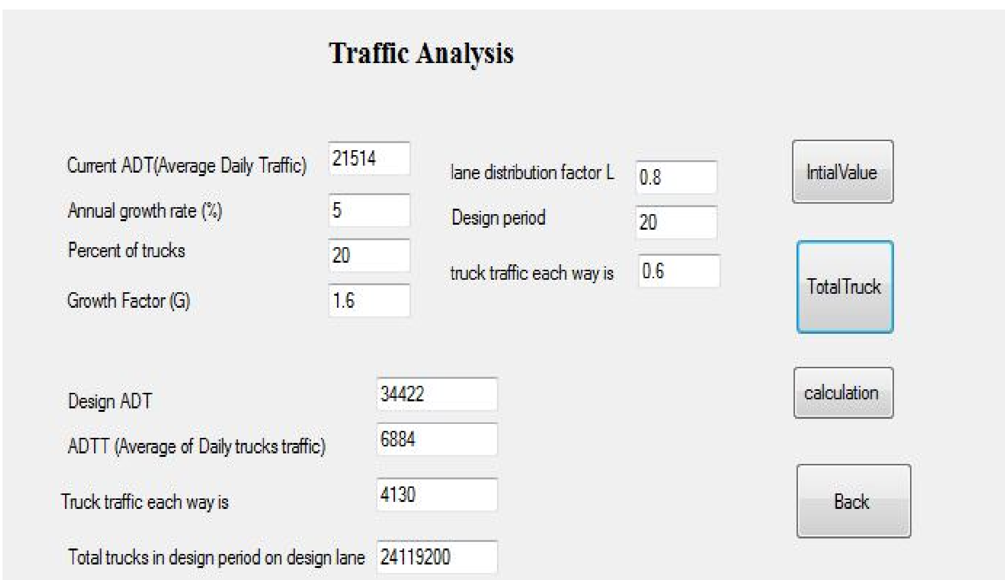


Figure 6.1 GalalM-RP Results of PCA Traffic Analysis

Computed the trials thickness by GalalM-RP and manual solution are exhibited in Figs 6.2 through Fig 6.7.

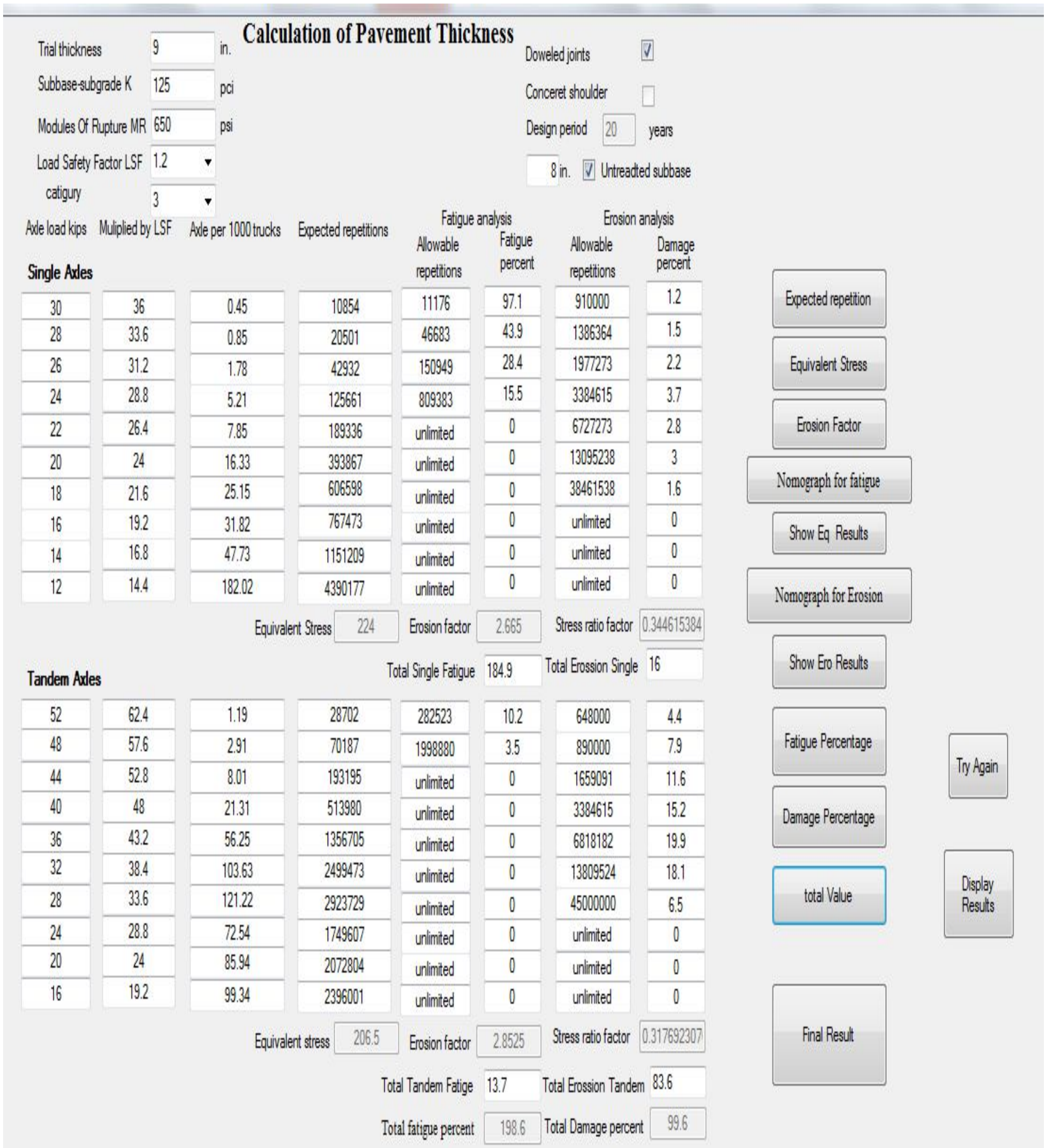


Figure 6.2 GalalM-RP calculation of pavement thickness 9in.

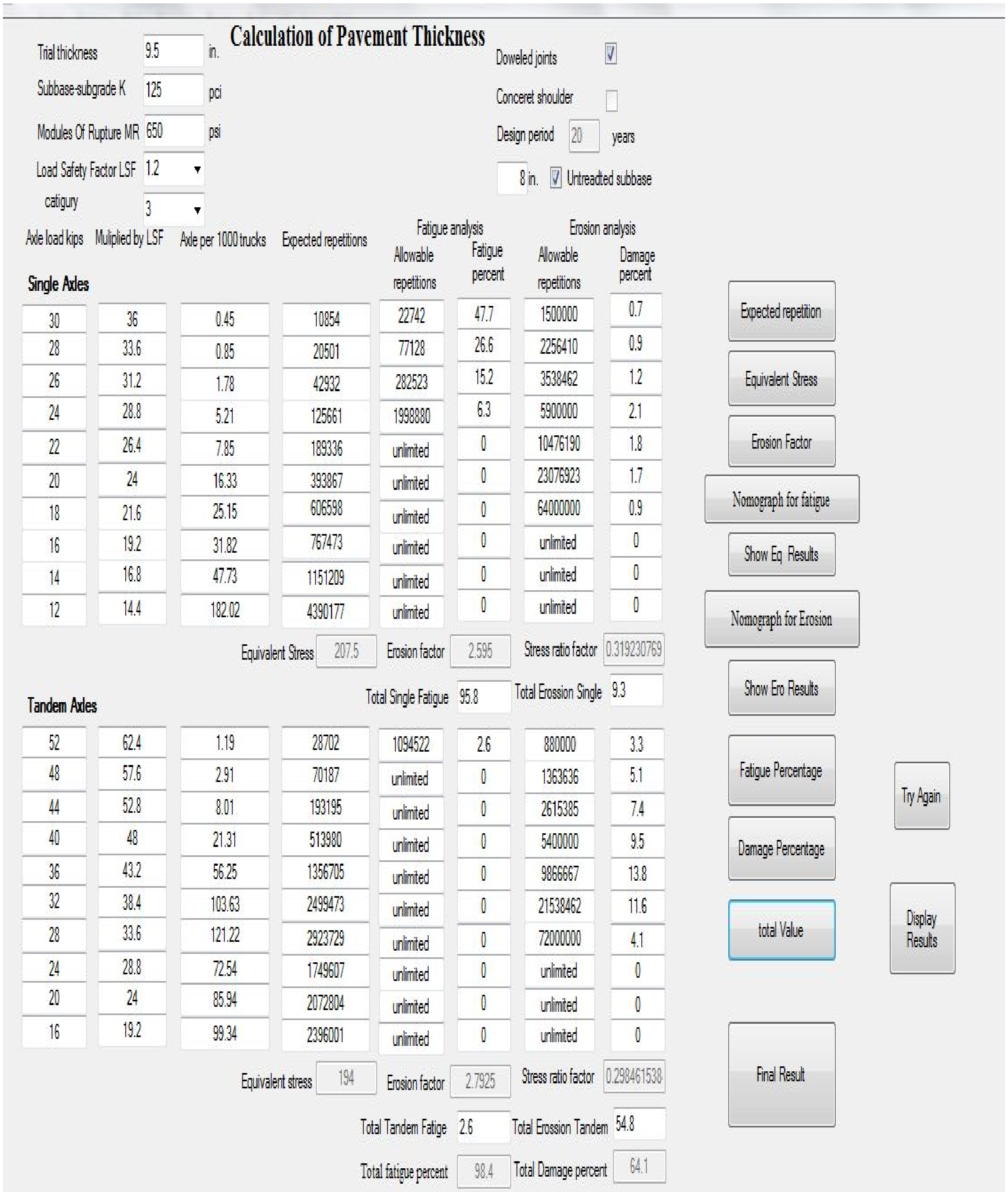


Figure 6.3 GalalM-RP calculation of pavement thickness 9.5in.

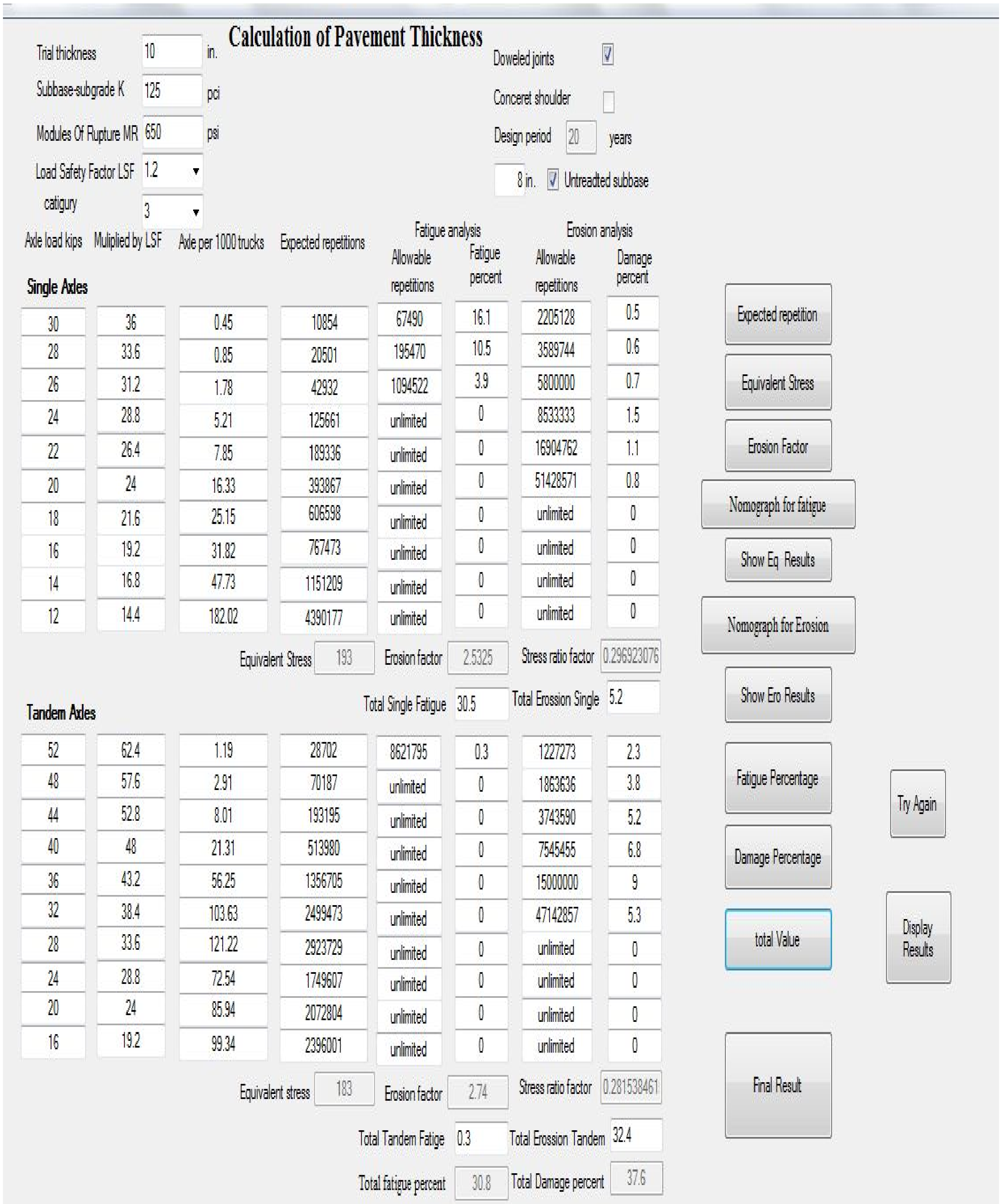


Figure 6.4 GalalM-RP calculation of pavement thickness 10in.

Calculation of pavement thickness

Trial thickness 9 in — Doweled joint yes ✓ no

Subbase-subgrade *k* 125 pci — Concrete shoulder yes no ✓

Modulus of rupture MR 650 psi — Design period 20 years

Load safety factor. LSF 1.2

Axle load kips	Multiplied by LSF	Axle per 1000 Trucks	Expected repetitions	Fatigue analysis		Erosion analysis	
				Allowable repetitions	Fatigue percent	Allowable repetitions	Damage percent
1	2	3	4	5	6	7	8

9. Equivalent stress 224 — 11.Erosion factor 2.665

10. Stress ratio factor 0.345

Single Axles

30	36	0.45	10854	12000	90.45	910000	1.2
28	33.6	0.85	20501	46000	44.6	1400000	1.5
26	31.2	1.78	42932	150000	28.6	2000000	2.1
24	28.8	5.21	125661	800000	15.7	3600000	3.5
22	26.4	7.85	189336	Unlimited	0.0	6800000	2.8
20	24	16.33	393867	Unlimited	0.0	13000000	3
18	21.6	25.15	606598	Unlimited	0.0	36000000	1.7
16	19.2	31.82	767473	Unlimited	0.0	Unlimited	0.0
14	16.8	47.73	1151209	Unlimited	0.0	Unlimited	0.0
12	14.4	182.02	4390177	Unlimited	0.0	Unlimited	0.0

12. Equivalent stress 206.5 — 14.Erosion factor 2.853

13. Stress ratio factor 0.318

Tandem Axles

52	62.4	1.19	28702	270000	10.6	640000	4.5
48	57.6	2.91	70187	1800000	3.9	890000	7.9
44	52.8	8.01	193195	Unlimited	0.0	1700000	11.3
40	48	21.31	513980	Unlimited	0.0	3300000	15.6
36	43.2	56.25	1356705	Unlimited	0.0	6800000	20
32	38.4	103.63	2499473	Unlimited	0.0	14000000	17.9
28	33.6	121.22	2923729	Unlimited	0.0	46000000	6.4
24	28.8	72.54	1749607	Unlimited	0.0	unlimited	0.0
20	24	85.94	2072804	Unlimited	0.0	unlimited	0.0
16	19.2	99.34	2396001	Unlimited	0.0	unlimited	0.0
				Total	193.9	Total	99.4

FIGURE6.5 Manual Calculation by Design Worksheet PCA: 9in.

Calculation of pavement thickness

Trial thickness 9.5 in Doweled joint yes ✓ no

Subbase-subgrade *k* 125 pci Concrete shoulder yes no ✓

Modulus of rupture MR 650 psi Design period 20 years

Load safety factor. LSF 1.2

Axle load kips	Multiplied by LSF	Axle per 1000 Trucks	Expected repetitions	Fatigue analysis		Erosion analysis	
				Allowable repetitions	Fatigue percent	Allowable repetitions	Damage percent
1	2	3	4	5	6	7	8

9. Equivalent stress 207.5 11.Erosion factor 2.595

10. Stress ratio factor 0.319

Single Axles

30	36	0.45	10854	25000	43.4	1500000	0.7
28	33.6	0.85	20501	77000	26.6	2200000	0.9
26	31.2	1.78	42932	280000	15.3	3500000	1.2
24	28.8	5.21	125661	1200000	10.5	5900000	2.1
22	26.4	7.85	189336	Unlimited	0.0	11000000	1.7
20	24	16.33	393867	Unlimited	0.0	23000000	1.7
18	21.6	25.15	606598	Unlimited	0.0	64000000	0.9
16	19.2	31.82	767473	Unlimited	0.0	Unlimited	0.0
14	16.8	47.73	1151209	Unlimited	0.0	Unlimited	0.0
12	14.4	182.02	4390177	Unlimited	0.0	Unlimited	0.0

12. Equivalent stress 194 14.Erosion factor

2.793

13. Stress ratio factor 0.298

Tandem Axles

52	62.4	1.19	28702	1100000	2.6	920000	3.1
48	57.6	2.91	70187	Unlimited	0.0	1500000	4.7
44	52.8	8.01	193195	Unlimited	0.0	2500000	7.7
40	48	21.31	513980	Unlimited	0.0	4600000	11.2
36	43.2	56.25	1356705	Unlimited	0.0	9500000	14.3
32	38.4	103.63	2499473	Unlimited	0.0	24000000	10.4
28	33.6	121.22	2923729	Unlimited	0.0	92000000	3.2
24	28.8	72.54	1749607	Unlimited	0.0	Unlimited	0.0
20	24	85.94	2072804	Unlimited	0.0	Unlimited	0.0
16	19.2	99.34	2396001	Unlimited	0.0	Unlimited	0.0
					98.5		64.0

FIGURE6.6 Manual Calculation by Design Worksheet PCA: 9.5in.

Calculation of pavement thickness

Trial thickness 10 in Doweled joint yes ✓ no

Subbase-subgrade *k* 125 pci Concrete shoulder yes no ✓

Modulus of rupture MR 650 psi Design period 20 years

Load safety factor. LSF 1.2

Axle load kips	Multiplied by LSF	Axle per 1000 Trucks	Expected repetitions	Fatigue analysis		Erosion analysis	
				Allowable repetitions	Fatigue percent	Allowable repetitions	Damage percent
1	2	3	4	5	6	7	8

9. Equivalent stress 193 11.Erosion factor 2.532

10. Stress ratio factor 0.297

Single Axles

30	36	0.45	10854	67000	16.2	2200000	0.5
28	33.6	0.85	20501	200000	10.3	3500000	0.6
26	31.2	1.78	42932	1100000	3.9	5800000	0.7
24	28.8	5.21	125661	Unlimited	0.0	9000000	1.4
22	26.4	7.85	189336	Unlimited	0.0	10000000	1.9
20	24	16.33	393867	Unlimited	0.0	50000000	0.8
18	21.6	25.15	606598	Unlimited	0.0	Unlimited	0.0
16	19.2	31.82	767473	Unlimited	0.0	Unlimited	0.0
14	16.8	47.73	1151209	Unlimited	0.0	Unlimited	0.0
12	14.4	182.02	4390177	Unlimited	0.0	Unlimited	0.0

12. Equivalent stress 183 14.Erosion factor

2.740

13. Stress ratio factor 0.282

Tandem Axles

52	62.4	1.19	28702	4000000	0.7	1227273	2.3
48	57.6	2.91	70187	Unlimited	0.0	1863636	3.8
44	52.8	8.01	193195	Unlimited	0.0	3743590	5.2
40	48	21.31	513980	Unlimited	0.0	7545455	6.8
36	43.2	56.25	1356705	Unlimited	0.0	15000000	9.0
32	38.4	103.63	2499473	Unlimited	0.0	47142857	5.3
28	33.6	121.22	2923729	Unlimited	0.0	Unlimited	0.0
24	28.8	72.54	1749607	Unlimited	0.0	Unlimited	0.0
20	24	85.94	2072804	Unlimited	0.0	Unlimited	0.0
16	19.2	99.34	2396001	Unlimited	0.0	Unlimited	0.0
				Total	31.1	Total	38.3

FIGURE6.7 Manual Calculation by Design Worksheet PCA: 10in.

The result of the numerical case study in chapter four above, for the three trails thickness of pavement are recorded and tabulated against the output of the program in Table 6.1. The fatigue and damage of pavement expressed in percent were plotted in histograms form in Figures 6.8 and 6.9 for the manual solutions and GalalM-RP respectively.

Figure 6.10 and 6.11 shows the relationship between fatigue and thickness pavement using manual solution and GalalM-RP solution respectively, which are merged in Figure 6.12.

Comparison of thickness design between JPCP and JRCP for AASHTO and PCA design methods is represented in Table 6.2.

Table 6.1 Manual solution Against GalalM-RP program

Trial	Thickness in.	Manual solution		GalalM-RP solution		Difference%	
		fatigue percent	damage percent	fatigue percent	damage percent	fatigue percent	damage percent
1	9	193.9	99.4	198.6	99.6	-4.7	-0.2
2	9.5	98.5	64.0	98.4	64.1	+0.1	-0.1
3	10	31.1	38.3	30.8	37.6	+0.3	+0.7

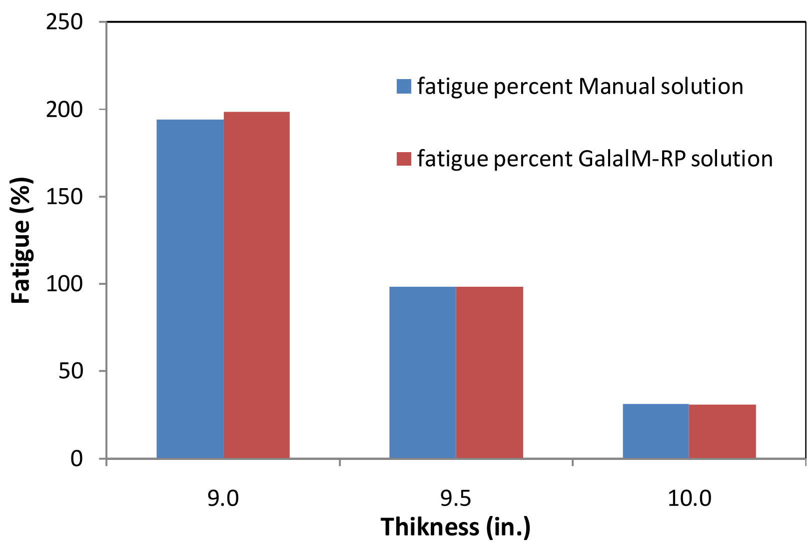


Figure 6.8 Fatigue Percent Manual Solutions against Fatigue Percent GalalM-RP

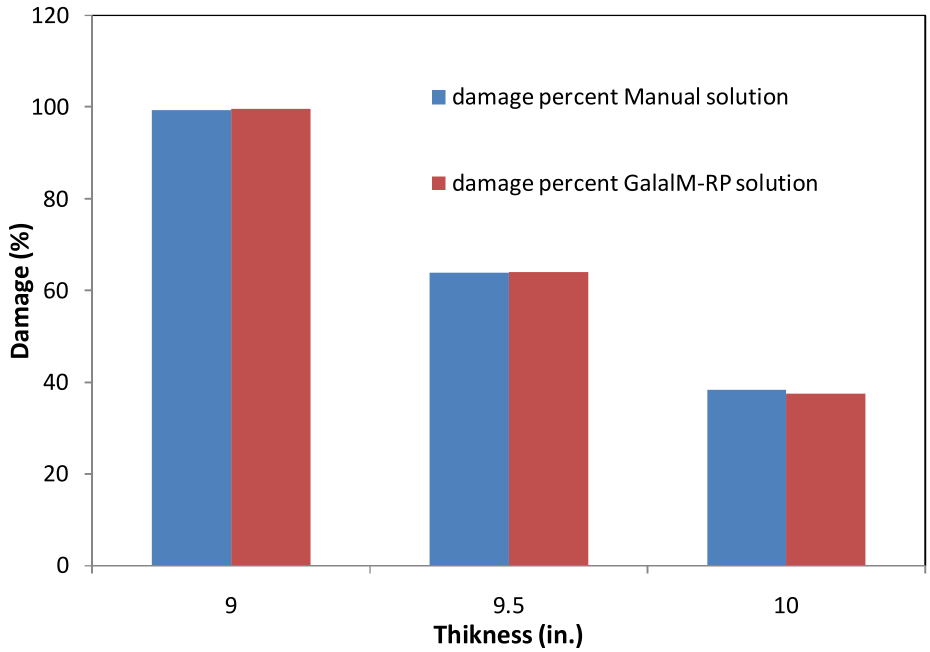


Figure 6.9 damage percent manual solution against damage percent GalalM-RP

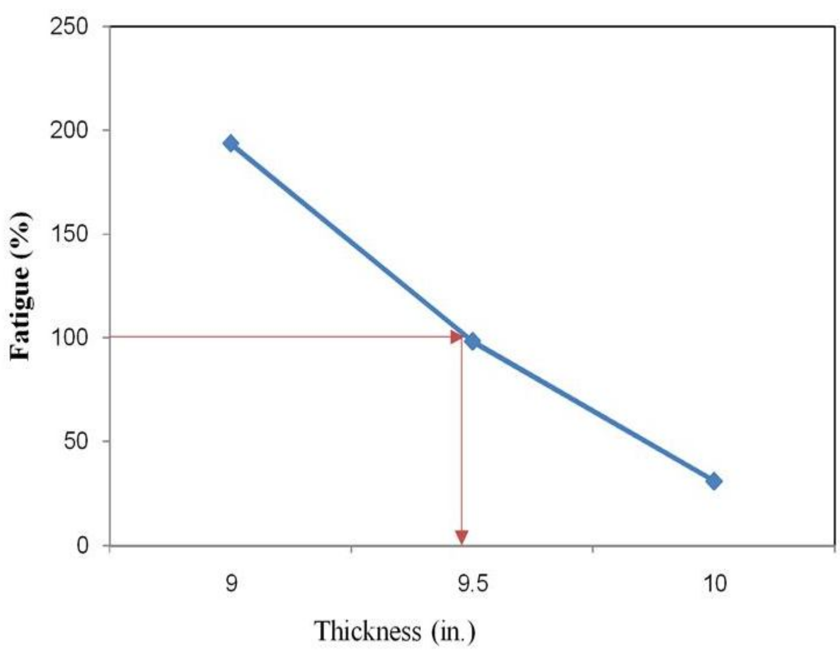


Figure6.10 graphical presentation for thickness pavement design using manual solution

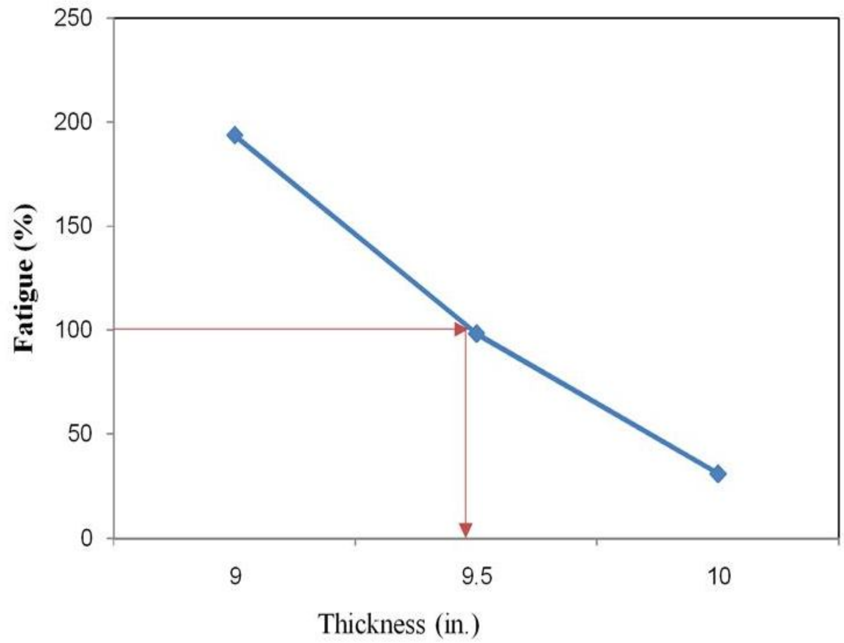


Figure6.11 graphical presentation for thickness pavement design using GalalM-RP

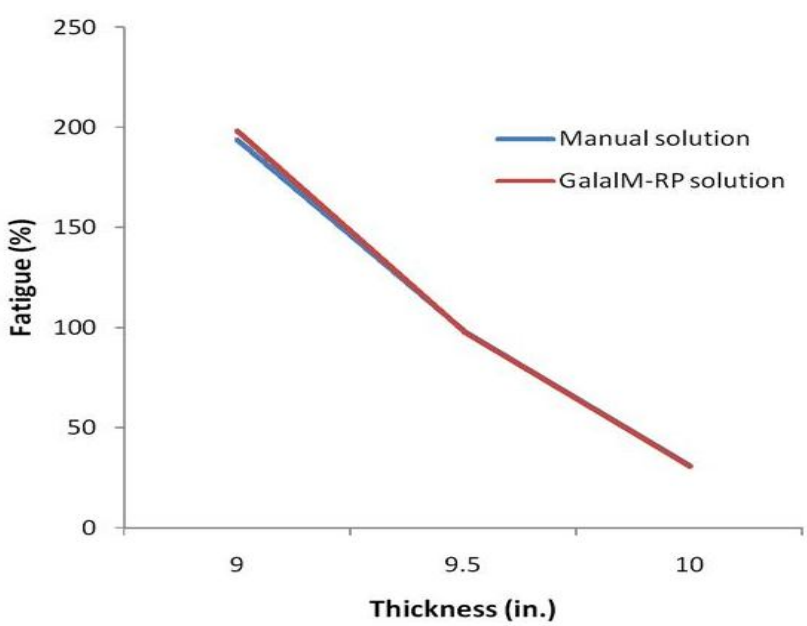


Figure6.12 Manual solution VS GalalM-RP Software Program

Table 6.2 Comparison of thickness design between AASHTO and PCA design methods

Item	JPCP		JRCP	
Design Method	PCA	AASHTO	PCA	AASHTO
Typical total pavement thickness, in.	9.49	9.85	9.49	9.85

6.2 Discussions

From a general comparison of the thickness design between manual solution and GalalM-RP solution illustrated in Figures 6.8, 6.9, 6.10, 6.11 and 6.12 and Table 6.2 it may be noted that:

(i) As represented in histograms Figure 6.8; manual solution and GalalM-RP solution gave average fatigue values of 107.8% and 109.3% respectively with corresponding discrepancies of -4.7 to 0.3%. That seen the difference percentage was acceptable.

(ii) As represented in Figure 6.9; manual solution and GalalM-RP solution gave average damage values of 67.2% and 67.1% respectively with corresponding discrepancies of -0.2to +0.7%. Thus GalalM-RP compared very well with manual solution

(iii) However, 98.5 % is much less than 100 % indicating the slab thickness of 9.5 in. (241.3 mm) is over design. Thus, the design was repeated using 9-in (229 mm) thickness resulting in fatigue damage of 193.9%, much higher than 100 % (under design). Therefore, a slab thickness of 9.49 in. (241 mm) would be adequate can be inferred from the patterns of the shown in Figure 6.10 and 6.11.

(iv) From Fig 6.12 It seen that the results of both manual solution and the program for the thickness are identical.

(v) As represented in Table 6.2 the difference in slab thickness was only 3.8 % with AASHTO design thicker. Since both methods do not differentiate between JPCP and JRCP.

(vi) Finally, the typical thickness design can be clearly shown in Figure 6.13 below.

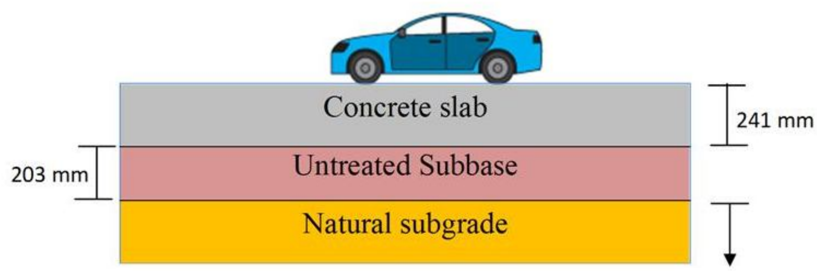


Figure 6.13 Typical Thickness Design

CHAPTER SEVEN

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

7.1 Summary

In this research investigation, thickness design for JPCP by AASHTO and PCA methods was determined for the case study road. However, reinforced design by AASHTO procedure was performed for the JRCP suggested to be used in the construction of the proposed road. A computer program with Visual Basic software was developed, entitled GalalM-RP program, to determine the rigid pavement design thickness in accordance with PCA method. Comparison was then made for the rigid pavement design thickness between the manual method and GalalM-RP program.

Comparison of the results for the design thickness between manual solution and the program was very favorable.

7.2 Conclusions

Within the scope of this study and the design conditions applied for the case study, the following conclusions are warranted:

1. A computer program (GalalM-RP) in Visual Basic was developed which was not easy that took a lot of effort and time. GalalM-RP program proved to possess simplicity with comprehensiveness in treating and translating design PCA procedure to computer application.

2. The case-study rigid pavement design was determined by AASHTO and PCA methods for both JPCP and JRCP. The difference in slab thickness was only 3.8 % with AASHTO design thicker. Since both methods do not differentiate between JPCP and JRCP, there was difference in the basic design thickness.

3. Comparison of the program results with the manual-computational design were very favorable varying within 5 %

4. According to AASHTO and PCA design methods used in this study and the results achieved with favorable comparison with manual design, it is justifiable to conclude that the GalalM-RP program can be used reliably as design thickness program for rigid pavements with doweled joints without concrete shoulders.

7.3 Recommendations

The following are several recommendations that can be considered for future work:

1. Computer programmers have access to computer room. Modify access procedures to restrict access to the computer room to computer operators only.
2. Deficient documentation, Documentation of program changes, systems software, and testing should be required.
3. No computer Subbase-subgrade *k*. For manual entry process, Subbase-subgrade *k* should not need to manually enter. This information should be accessed from a computer file.
4. Program cannot design in the case of Slabs without doweled joints and with concrete shoulders. The Galal R.P. system should be programmed to design in the case of slabs without doweled joints and with concrete shoulders.
5. Control totals determined by the trail thickness do not appear to be used appropriately. The economical of thickness should be used by the computer.
6. No range checks or limit or reasonableness tests in nomograph for fatigue if the Stress ratio factors were not in range of (0.3-0.4).

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APPENDIX A

PCA DESIGN METHOD: TABLES AND CHARTS

A.1 PCA Design Method: Tables

TABLE A.1 Effect of Untreated Subbase on *k* Values

Subgrade *k* value (pci)	Subbase *k* values (pci)			
	4 in.	6 in.	9 in.	12 in.
50	65	75	85	110
100	130	140	160	190
200	220	230	270	320
300	320	330	370	430

Note. 1 *in.* = 25.4 mm, 1 pci = 271.3 kN/m^3.

Source. After PCA (1984).

TABLE A.2 Recommended Dowel Size and Length

Slab thickness (in.)	Dowel diameter (in.)	Dowel length (in.)
5	$\frac{5}{8}$	12
6	$\frac{3}{4}$	14
7	$\frac{7}{8}$	14
8	1	14
9	$1\frac{1}{8}$	16
10	$1\frac{1}{4}$	18
11	$1\frac{3}{8}$	18
12	$1\frac{1}{2}$	20

Note. All dowels spaced at 12 in. on centers, 1 in. = 25.4 mm.

Source. After PCA (1975).

A.2 PCA Design Method: Charts

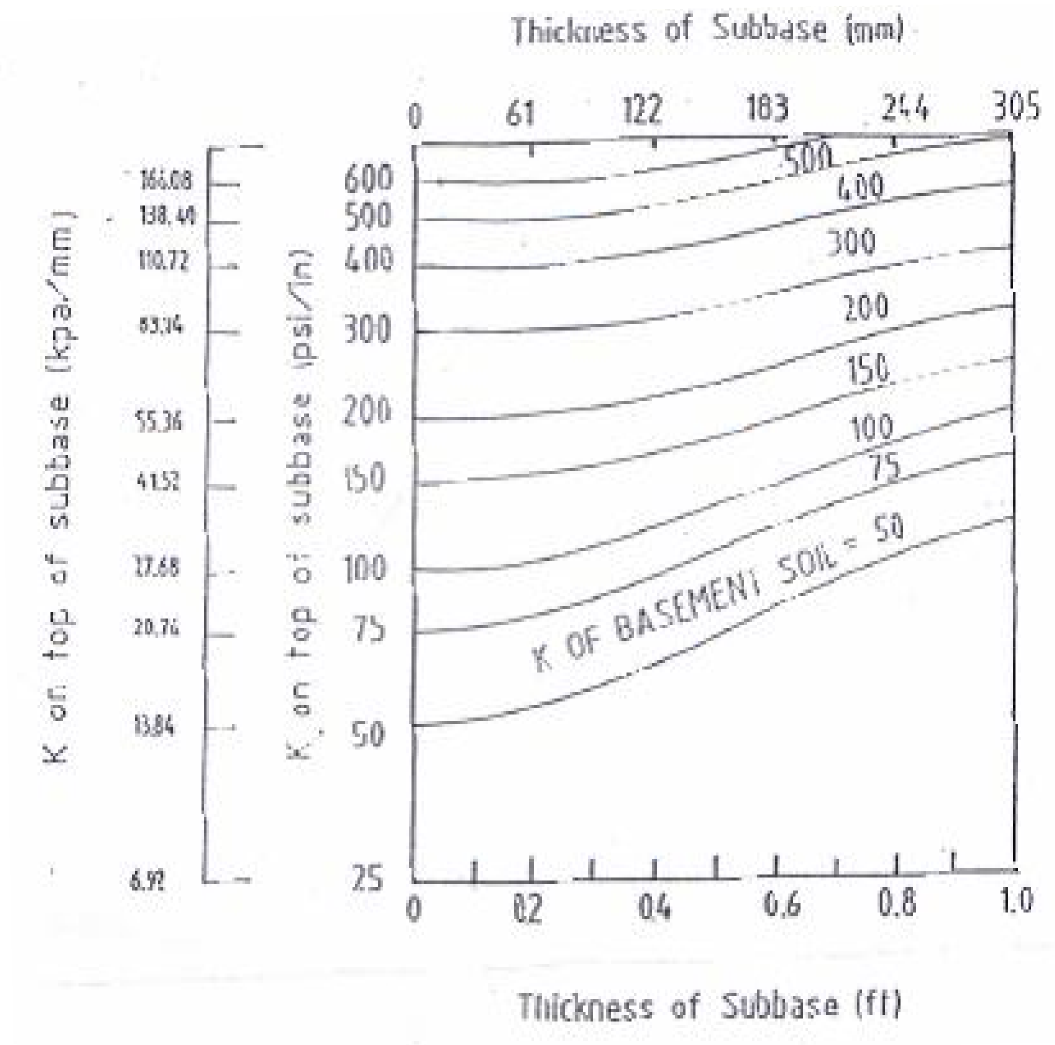

Figure A-1 Effect of various thicknesses of granular subbase on K values

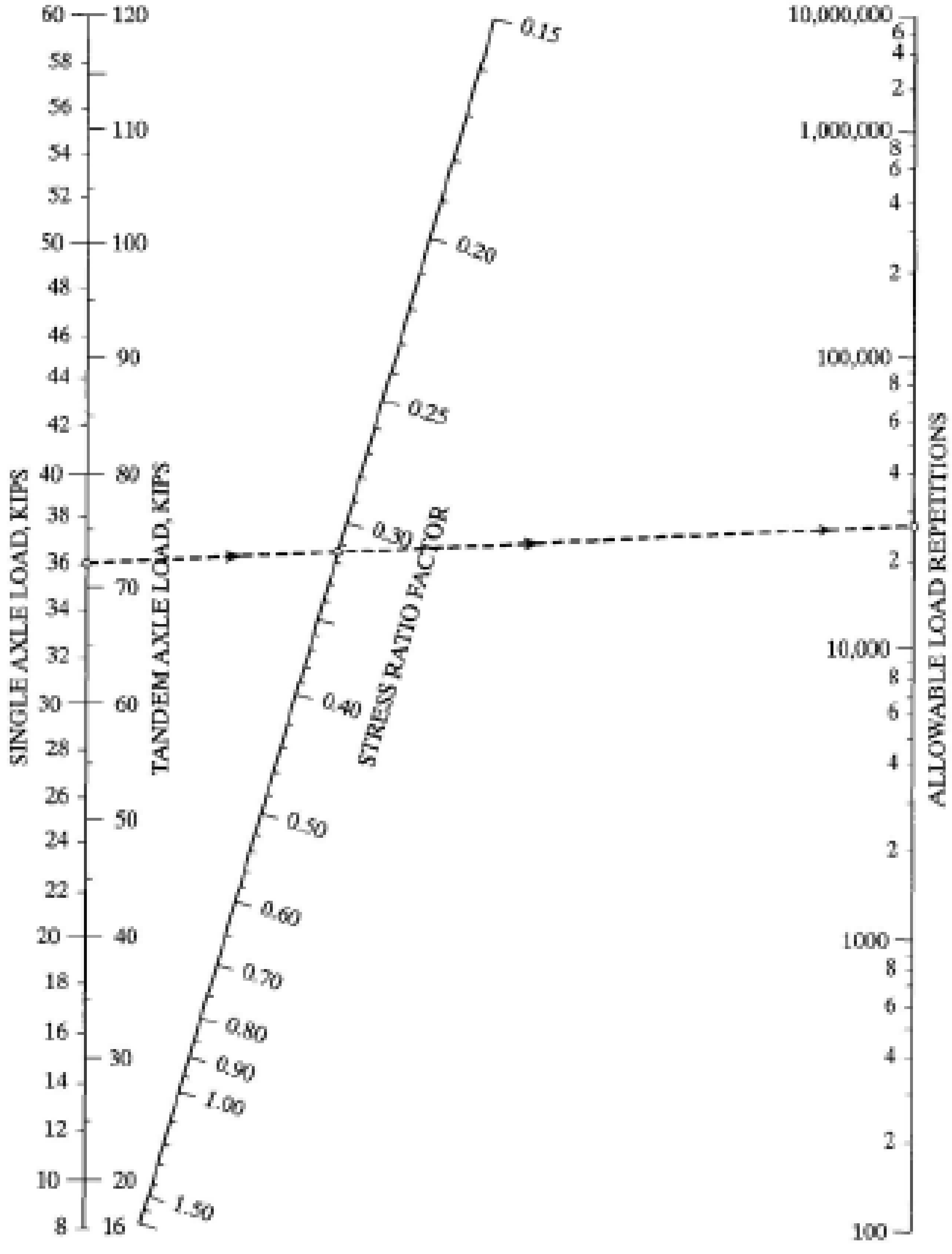

FIGURE A-2

Stress ratio factors versus allowable load repetitions both with and without concrete shoulders

(1 kip = 4.45KN). (After PCA (1984))

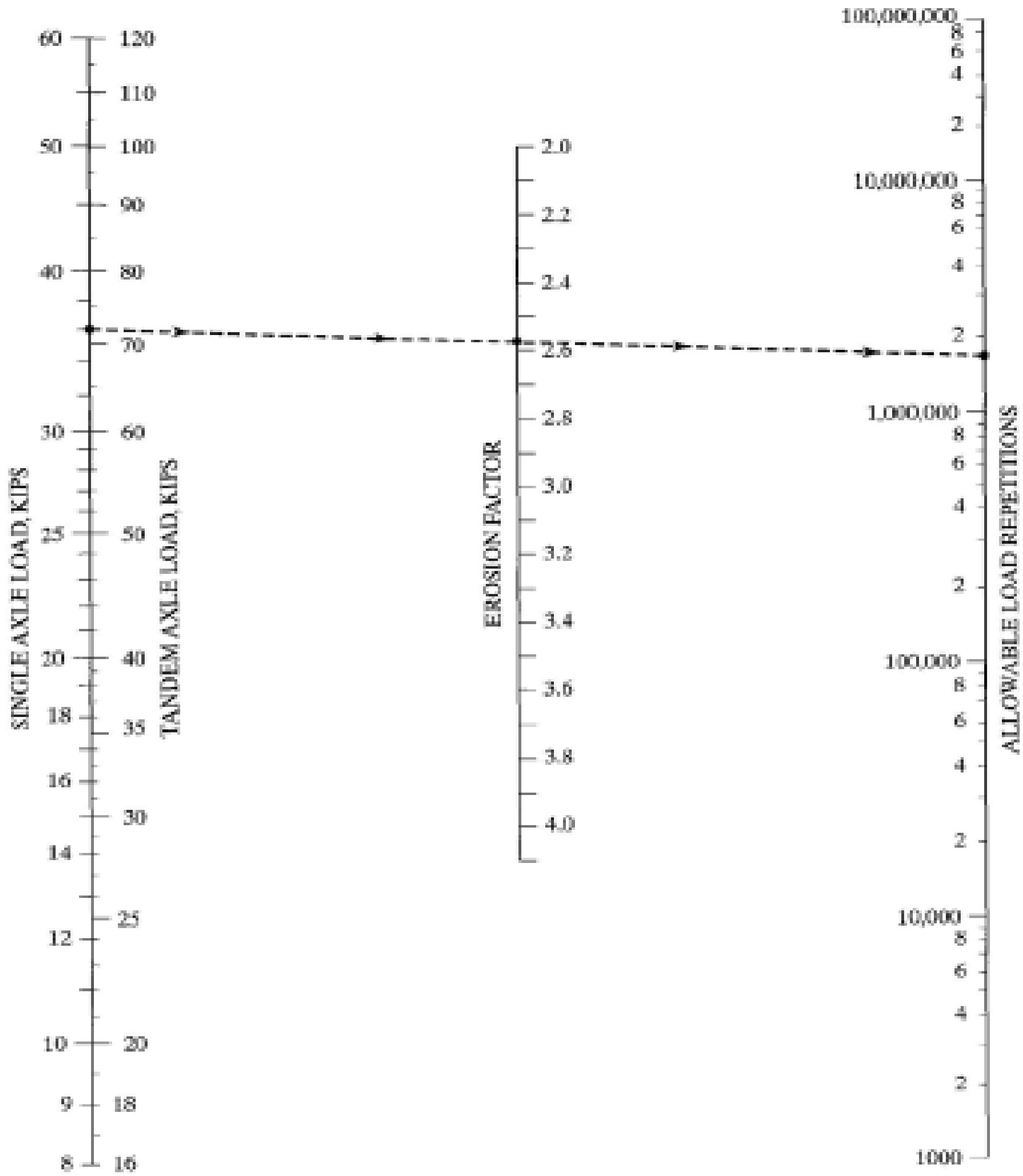

FIGURE A-3

Erosion factors versus allowable load repetitions without concrete shoulders (1 kip = 4.45KN).

(After PCA (1984)).

APPENDIX B

AASHTO DESIGN METHOD: TABLES AND CHARTS

B.1 AASHTO Design Method: Tables

TABLE B.1 Recommended Load Transfer Coefficient J for Various Pavement Types and Design Conditions

Shoulder	Asphalt		Tied P.C.C.	
Load Transfer Device	Yes	No	Yes	No
Pavement Type				
Plain Jointed and Jointed reinforced	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.2 Recommended Value of Drainage Coefficient, *Cd*, for Rigid Pavement Design

	Percent of Time Pavement Structure Is Exposed to Moisture Levels Approaching Saturation			
Quality of Drainage	Less than 1%	1–5%	5–25%	Greater than 25%
Excellent	1.25–1.20	1.20–1.15	1.15–1.10	1.10
Good	1.20–1.15	1.15–1.10	1.10–1.00	1.00
Fair	1.15–1.10	1.10–1.00	1.00–0.90	0.90
Poor	1.10–1.00	1.00–0.90	0.90–0.80	0.80
Very Poor	1.00–0.90	0.90–0.80	0.80–0.70	0.70

Source: AASHTO. 1993. *AASHTO Guides for Design of Pavement Structures*. Copyright 1993 by the American Association of State Highway and Transportation Officials, Washington, D.C. Used by permission.

TABLE B.3 Suggested Levels of Reliability for Various Functional Classifications

Functional classification	Recommended level of reliability: Urban	Rural
Interstate and other freeways	85–99.9	80–99.9
Principal arterials	80–99	75–95
Collectors	80–95	75–95
Local	50–80	50–80

Note. Results based on a survey of AASHTO Pavement Design Task Force.

Source. After AASHTO (1986)

TABLE B.4 Yield Strength and Allowable Stress for Steel

Type and grade of steel	Yield strength (psi)	Allowable stress (psi)
Billet steel, intermediate grade	40,000	27,000
Rail steel or hard grade of billet steel	50,000	33,000
Rail steel, special grade	60,000	40,000
Billet steel, 60,000 psi minimum yield	60,000	40,000
Cold drawn wire (smooth)	65,000	43,000
Cold drawn wire (deformed)	70,000	46,000

Note. 1 *psi* = 6.9 kPa.

TABLE B-5 Weights and Dimensions of Welded Wire Fabric

Wire size no.		Diameter	Weight	Cross-sectional area (in.^{2}/ft) center-to-center spacing (in.)						
Smooth	Deformed	(in.)	lb/ft	2	3	4	6	8	10	12
W31	D31	0.628	1.054	1.86	1.24	.93	.62	.465	372	.31
W30	D30	0.618	1.020	1.80	1.20	.90	.60	.45	36	.30
W28	D28	0.597	.952	1.68	1.12	.84	.56	.42	336	.28
W26	D26	0.575	.934	1.56	1.04	.78	.52	.39	312	.26
W24	D24	0.553	.816	1.44	.96	.72	.48	.36	288	.24
W22	D22	0.529	.748	1.32	.88	.66	.44	.33	264	.22
W20	D20	0.504	.680	1.20	.80	.60	.40	.30	24	.20
W18	D18	0.478	.612	1.08	.72	.54	.36	.27	216	.18
W16	D16	0.451	.544	.96	.64	.48	.32	.24	192	.16
W14	D14	0.422	.476	.84	.56	.42	.28	.21	168	.14
W12	D12	0.390	.408	.72	.48	.36	.24	.18	144	.12
W11	D11	0.374	.374	.66	.44	.33	.22	.165	132	.11
W10.5		0.366	.357	.63	.42	.315	.21	.157	126	.105
W10	D10	0.356	.340	.60	.40	.30	.20	.15	12	.10
W9.5		0.348	.323	.57	.38	.285	.19	.142	114	.095
W9	D9	0.338	.306	.54	.36	.27	.18	.135	108	.09
W8.5		0.329	.289	.51	.34	.255	.17	.127	102	.085
W8	D8	0.319	.272	.48	.32	.24	.16	.12	096	.08
W7.5		0.309	.255	.45	.30	.225	.15	.112	09	.075
W7	D7	0.298	.238	.42	.28	.21	.14	.105	084	.07
W6.5		0.288	.221	.39	.26	.195	.13	.097	078	.065
W6	D6	0.276	.204	.36	.24	.18	.12	.09	072	.06
W5.5		0.264	.187	.33	.22	.165	.11	.082	066	.055
W5	D5	0.252	.170	.30	.20	.15	.10	.075	06	.05
W4.5		0.240	.153	.27	.18	.135	.09	.067	054	.045
W4	D4	0.225	.136	.24	.16	.12	.08	.06	048	.04

Note. Wire sizes other than those listed above may be produced provided the quantity required is sufficient to justify manufacture.

1 in. = 25.4 mm, 1 lb = 4.45 N, 1 ft = 0.305 m.

TABLE B-6 Weights and Dimensions of Standard Reinforcing Bars

Bar size designation	Weight (lb/ft)	Nominal dimensions, round sections: Diameter (in.)	Cross-sectional area (in.^{2})	Perimeter (in.)
No. 3	0.376	0.375	0.11	1.178
No. 4	0.668	0.500	0.20	1.571
No. 5	1.043	0.625	0.31	1.963
No. 6	1.502	0.750	0.44	2.356
No. 7	2.044	0.875	0.60	2.749
No. 8	2.670	1.000	0.79	3.142
No. 9	3.400	1.128	1.00	3.544
No. 10	4.303	1.270	1.27	3.990
No. 11	5.313	1.410	1.56	4.430

Note. 1 *in.* = 25.4 *mm,* 1 lb = 4.45 N, 1 ft = 0.305 *m*.

B.2 AASHTO Design Method: Charts

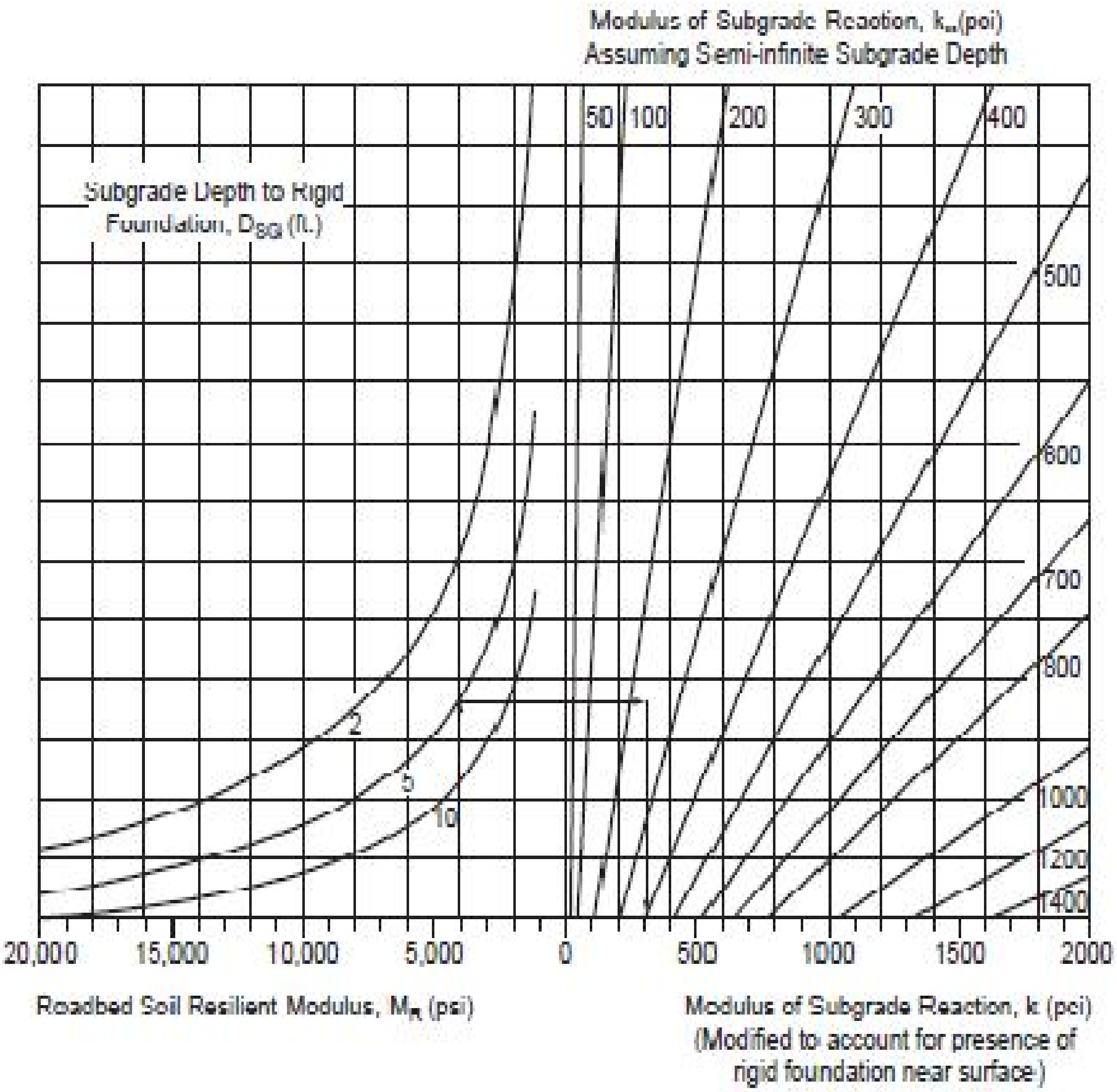

FIGURE B-1 Chart for *k* as a function of bedrock depth. (*Source:* AASHTO. 1993. *AASHTO Guides for Design of Pavement Structures*. Copyright 1993 by the American Association of State Highway and Transportation Officials, Washington, D.C. Used by permission).

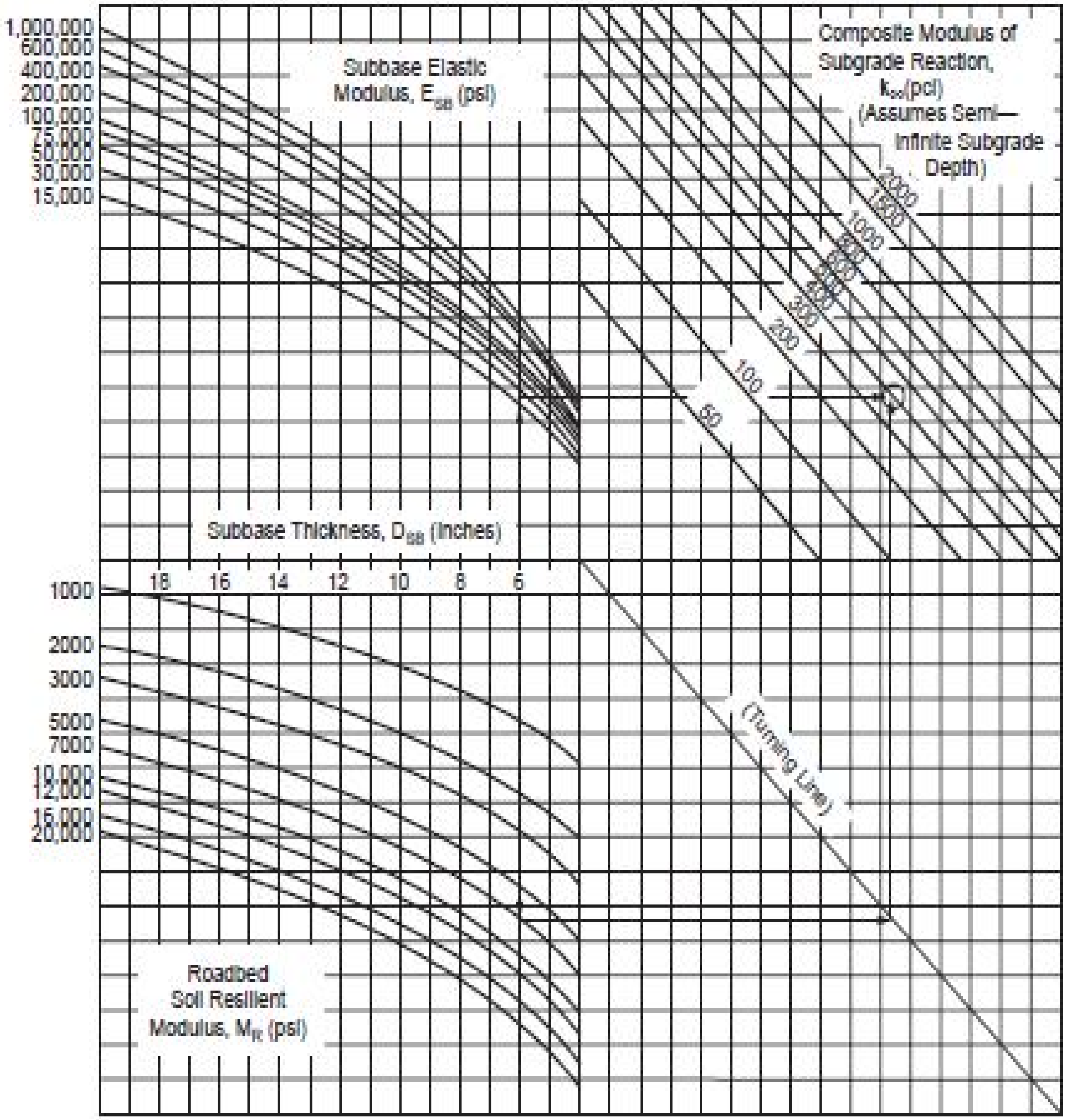

FIGURE B-2 Chart for estimating composite k_{∞}. (*Source:* AASHTO. 1993. *AASHTO Guides for Design of Pavement Structures*. Copyright 1993 by the American Association of State Highway and Transportation Officials, Washington, D.C. Used by permission).