A COMPARATIVE STUDY ON THE DIFFERENT RIGID PAVEMENT DESIGN METHODOLOGIES USED FOR PLAIN AND FIBER REINFORCED CEMENT-BASED MATERIALS

by

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To my family,

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ABSTRACT

A COMPARATIVE STUDY ON THE DIFFERENT RIGID PAVEMENT DESIGN METHODOLOGIES USED FOR PLAIN AND FIBER REINFORCED CEMENT-BASED MATERIALS

The application of concrete pavements that have been practiced for many years in developed countries is relatively new in Turkey. As well as choosing the most appropriate type of pavement, it is important to determine the optimum pavement thickness for the given traffic level, subgrade condition, and environmental factor to minimize the life cycle cost of the structure and protect the pavements. However, in most methodologies, the lack of consideration of the various main factors (such as vehicle loads, loss of support, thermal gradient, or environmental conditions) that produce stress on the concrete pavement prevents accurate calculation of the required thickness. Contributions of structural fibers to the mechanical and durability properties of concrete have been known for a very long time. However, the number of design methods that cover the use of fiber reinforcement in concrete pavements is very limited and all the methods use different approaches. The main objectives of this study are to compare the differences between the thickness design methodologies of AASHTO (1993), IRC SP 46 (2013), and StreetPave v12 (2014) and to emphasize the lack of the methodologies. The thickness design based on three methodologies was carried out for a sample road using material parameters retrieved from the literature. The contribution of fibers with different types (synthetic, steel and carbon) and amounts (0.2-1.0%) to thickness requirement was evaluated by means of different approaches. To see the effects of the concrete matrix on the effectiveness of fibers two types of concrete (RCC and conventional PCC) were considered. The effects of the presence of edge support and dowel bars on the required thickness were also examined. Finally, Turkey's policy on concrete roads were evaluated briefly. The results of the study showed that usage of the appropriate type and amounts of fibers in the well-designed concrete mixture (whether RCC or PCC for this study) provides to use of lower pavement thickness. Also, using concrete shoulder or dowel bar reduces the required thickness.

ÖZET

LİFLİ VE LİFSİZ ÇİMENTO BAZLI MALZEMELER İÇİN KULLANILAN FARKLI RİJİT KAPLAMA TASARIM METODOLOJİLERİ ÜZERİNE KARŞILAŞTIRMALI BİR ÇALIŞMA

Gelişmiş ülkelerde uzun yıllardır uygulanmakta olan beton kaplamaların uygulanması Türkiye'de nispeten yenidir. En uygun kaplama tipini seçmenin yanı sıra, yapının yaşam döngüsü maliyetini en aza indirmek ve kaldırımları korumak için, mevcut trafik seviyesi, zemin durumu ve çevresel faktör için optimum kaplama kalınlığının belirlenmesi önemlidir. Bununla birlikte, çoğu metodolojide, beton kaplama üzerinde gerilmeye sebep olan çeşitli ana faktörlerin (araç yükleri, destek kaybı, termal gradyan veya çevresel koşullar) dikkate alınmaması, gerekli kalınlığın doğru hesaplanmasını önler. Yapısal liflerin betonun mekanik ve dayanıklılık özelliklerine katkısı çok uzun zamandır bilinmektedir. Bununla birlikte, beton kaplamalarda lif kullanımını kapsayan tasarım yöntemlerinin sayısı oldukça sınırlıdır ve tüm yöntemler farklı yaklaşımları kullanmaktadır. Bu çalışmanın temel amaçları AASHTO (1993), IRC SP 46 (2013) ve StreetPave v12 (2014) 'un kalınlık tasarım metodolojileri arasındaki farkları karşılaştırmak ve metodolojilerin eksikliğini vurgulamaktır. Kalınlık tasarımı literatürden alınan malzeme parametreleri kullanılarak üç (3) metodolojiye göre örnek bir yol için gerçekleştirilmiştir. Farklı tiplerde (sentetik, çelik ve karbon) ve miktarlarda (% 0.2-1.0) liflerin kalınlık gereksinimine katkısı farklı yöntemler açısından değerlendirilmiştir. Beton matrisinin liflerin etkinliği üzerindeki etkilerini görmek için iki tip beton (SSB ve geleneksel beton) dikkate alınmıştır. Kenar desteği ve kayma donatısı varlığının gerekli kalınlık üzerindeki etkileri de incelenmiştir. Her üç metodoloji için hesaplanan gerekli kalınlık değerleri, yukarıda bahsedilen konular için karşılaştırılmıştır. Son olarak, Türkiye'nin beton yollara ilişkin politikası ve yakın zamanda yayınlanan tasarım prosedürleri kısaca değerlendirilmiştir. Çalışmanın sonuçları, iyi tasarlanmış beton karışımında (bu çalışma için SSB veya geleneksel beton) uygun tip ve miktarda lif kullanımının daha düşük kaplama kalınlığının kullanılmasını sağladığını göstermiştir. Ayrıca, kenar desteği veya kayma donatısı kullanmak gerekli kalınlığı azaltır.

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b	Width of the prismatic specimen
C_1	An adjustment factor
C_d	Drainage coefficient
d	Depth of the prismatic specimen
D	The thickness of concrete pavement slab
D_D	Directional distribution factor
D_L	Lane distribution factor for trucks
D_{SB}	Subbase thickness
E_c	Elastic modulus of PCC
E_{SB}	Elastic modulus of subbase
f_1	Characteristic residual flexural strength at CMOD = 0.5 mm
f_4	Characteristic residual flexural strength at CMOD = 3.5 mm
f_{ctk}	Characteristic flexural strength of concrete
f_{ctm}	Mean equivalent flexural strength of concrete
f_{e150k}	Characteristic equivalent flexural strength
f_{e150m}	Mean equivalent flexural strength
f_c'	Compressive strength of PCC
F_R	Reliability design factor
G	Growth factor
h	Pavement thickness
i	Truck class
J	Load transfer coefficient
j	Total load category number
k	Modulus of subgrade reaction
k_{eff}	Effective modulus of subgrade reaction
L	Span length
m	Total number of loads
M_R	Elastic modulus of the roadbed soil
n_i	Expected number of load repetitions

N	Allowable number of load repetitions to failure
N_e	The allowable load repetitions for erosion damage
N_f	The maximum number of allowable load repetitions to failure
N_{ei}	Allowable number of repetitions for erosion at i^{th} load group
N_i	Number of allowable load repetitions over the stated six-hour period for the i^{th} load category
n_i	Number of expected load repetitions over the stated six-hour period for the i^{th} load category
P	The power (rate of work)
PS	The percent of the strength distribution
P_t	Terminal serviceability index of the pavement
$R_{e,3}$	Equivalent flexural strength ratio
$R_{T,150}^{D}$	Equivalent flexural strength ratio
S_0	Total standard deviation
SD_s	Estimated standard deviation of concrete modulus of rupture
S_c	Construction specification on the concrete modulus of rupture
S_c'	Estimated mean value for PCC modulus of rupture
SR	Maximum stress ratio
T_{150}^{D}	Flexural toughness of the specimen
TF_i	Truck factor for truck class i
u_r	Relative damage parameter
w_{18}	Traffic in the design lane in 18-kip ESAL
$\acute{w}_{18}$	Design ESAL (the cumulative two-directional 18-kip ESAL)
W_{18}	Predicted number of 18-kip equivalent single axle load applications
z	Standard normal variate
Z_R	Standard normal deviate
σ_{eq}	The flexural stress under traffic loading
δ_{eq}	Corner deflection
ΔPSI	Design serviceability loss
γ_m	Material safety factor
τ	Tyre pressure
μ	Poisson’s ratio

LIST OF ACRONYMS/ABBREVIATONS

ASSHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ACPA	American Concrete Pavement Association
ADTT	Average Daily Truck Traffic
BUC	Bottom-up cracking
CBR	California bearing ratio
CFD	Cumulative fatigue damage
CRCP	Continuously reinforced concrete pavement
DLC	Dry lean concrete
EN	Euro Norm
ESAL	Equivalent Single Axle Load
FRC	Fiber reinforced concrete
IRC	Indian Road Congress
JPCP	Jointed plain concrete pavement
JRCP	Jointed reinforced concrete pavement
LEF	Load Equivalency Factors
LS	Loss of support
MOR	Modulus of Rupture
PCA	Portland Cement Association
PCC	Portland cement concrete
PQC	Pavement Quality Concrete
PSI	Present serviceability index
RCC	Roller compacted concrete
SR	Stress Ratio
TR	Technical Report
TDC	Top-down cracking
UW	Unit Weight

1. INTRODUCTION

1.1. General

A pavement structure consists of one or more structural layers that take place between the subgrade course and the base courses. To distribute applied vehicle loads to the subgrade is the primary function of this structure. Two types of pavements are generally considered to serve this purpose. These are flexible pavements with bitumen as binders and rigid pavements with cement as binders (Delatte, 2008). The advantages of rigid pavements can be listed as higher flexural strength, higher service life and lower maintenance costs compared to flexible pavements; advantages of flexible pavements can be listed as lower initial construction cost and lower opening time compared to rigid pavements (Mohod and Kadam, 2016). As stated here, the various advantages and disadvantages of both types of pavements indicate the importance of pavement type selection.

Due to their various advantages, the number of applications of rigid pavements constitutes an important part of the total road network in many developed countries while it is quite low in our country because of the limited knowledge of designers and contractors about the concrete pavements. Although the number of concrete road applications has increased, the number of new roads is still limited in Turkey (Çelik et al., 2019).

Roller compacted concrete (RCC) is a type of concrete that has been used for pavement construction projects for many years. RCC has similar strength properties as conventional concrete and consists of the same materials but has different mixing ratios. Also, RCC has a lower cement content than conventional concrete with the same strength. Basically, RCC consists of a mixture of well-graded aggregates, cement, and water. As a result of this composition, RCC mixtures have zero slump and are drier than conventional concrete mixtures. Typically, dowel bars and conventional steel reinforcement are not used in RCC pavements. As a result, when a suitable mix design is considered, more load transfer can be achieved with the aggregate interlock mechanism (Delatte, 2008). Due to the similar structural behavior of concretes, the thickness design process of RCC pavements is similar to conventional concrete pavements. The thickness design of RCC and conventional concrete pavements relies on keeping the fatigue damage and flexural stresses on the pavement due to wheel loads within the permissible limits (ACI Committee 325, 1995).

Also, the mechanical properties (such as flexural strength, tensile strength, compressive strength, creep behavior, impact resistance and toughness) of both types of pavements can be enhanced by using fiber reinforcement (ACI Committee 544, 1996). Nanni and Johari (1989) stated that fiber addition may be the best alternative to the conventional dowel bars, as fibers are the only reinforcements that can be placed into the RCC, easily.

The most important part of concrete pavement design is calculating an accurate thickness of the pavement. Whereas unnecessary pavement thickness increases the cost, inadequate thickness design may cause loss of service before design life. For this reason, the required importance should be given to pavement thickness design (Delatte, 2008; Öztürk, 2018).

In countries where rigid pavements are frequently used, various design methods have been developed for slab thickness design. The most important and widely used methods are AASHTO (1993) and PCA (1984) which are empirical and mechanistic-empirical design methods, respectively. With increasing knowledge of design methods, new approaches have been proposed based on these design methodologies such as StreetPave software based on PCA (1984), IRC 58 (2011) for plain concrete pavement design and IRC SP 46 (2013) for fiber-reinforced concrete pavement design. Since each design methodology calculates pavement thickness using different parameters and approaches, calculated pavement thicknesses differ. Also, a significant number of these design methods do not consider the contribution of fiber reinforcement to the thickness requirement (Öztürk, 2018).

Within the scope of this study, thickness designs performed for the study were done by using AASHTO (1993), IRC SP 46 (2013) and StreetPave v12 (2014) methodologies. Also, three pavement design methodologies were compared about their differences in determining design input parameters and their influences on the pavement thickness. The thickness design results of both plain and fiber reinforced RCC and conventional PCC pavements are examined under the following situations. First, the results of thickness design for RCC and conventional PCC pavements were first evaluated in terms of plain and fiber-reinforced within each methodology. The thickness values obtained for the RCC and conventional PCC pavements were then evaluated separately for design methodologies to observe the causes of thickness similarities or differences according to methodologies. In addition, the effects of concrete matrix on the contribution of fibers in terms of RCC and conventional PCC was examined. Finally, the effects of the presence of edge support and load transfer devices such as dowel bars on the thicknesses calculated for RCC and conventional PCC were evaluated in terms of design methodologies. It was also examined which failure mode governed the design.

1.2. Objectives

The main objectives of this thesis are:

- To observe the difference between AASHTO (1993), IRC SP 46 (2013), and StreetPave v12 (2014) pavement design methodologies in terms of the concept and used parameters,
- To highlight the missing aspects of the design methodologies,
- To evaluate the contribution of fibers with different amounts and types on the thickness requirement,
- To evaluate the different approaches of the methodologies by means of fiber contribution,
- To evaluate the effects of concrete matrix on the contribution of fibers in terms of RCC and conventional PCC,
- To evaluate the effects of the presence of edge support and load transfer device such as dowel bars on the thicknesses calculated for RCC and conventional PCC, and
- To assess the Turkey's policy on concrete roads and recently published design procedures.

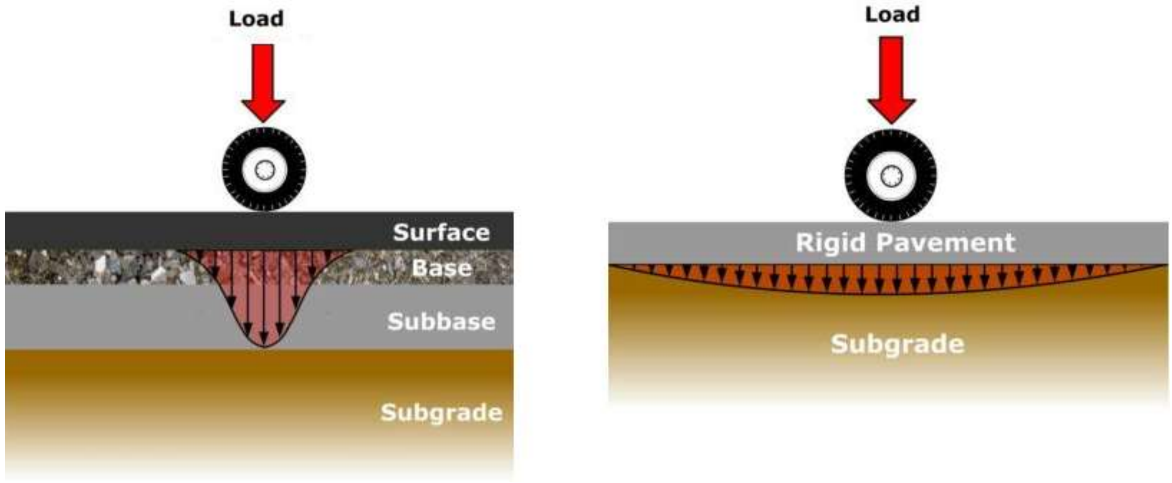
1.3. Scope

This thesis is organized into five main chapters and presented in the following order. Chapter 1 introduces the thesis and gives the objectives and the scope of the study. Chapter 2 provides background information on pavement type focusing on rigid pavements. Furthermore, literature review on design procedures and the basic design factors is presented in Chapter 2. Chapter 3 consists of methodology of the study, design approaches for fiber-reinforced concrete and comparison of design methodologies considered in the study. Chapter 4 is on the results and discussions of the comparative study, providing the results of required thickness in terms of each methodology in different subheadings. Chapter 5 summarizes the findings of the comparative study.

2. LITERATURE REVIEW

2.1. Types of Pavements

A pavement is a structure composed of superimposed layers above natural soil subgrade whose primary function is to distribute traffic loads to lower layers so that the bearing capacity of the soil is not exceeded. Pavements are divided into two main groups based on design considerations. These are flexible and rigid pavements. Flexible pavements are composed of several layers of granular material and considered to flex under traffic loading whereas rigid pavements are composed of Portland Cement Concrete (PCC) surface and they are considered to show stiffer behavior than flexible pavements (Delatte, 2008).



(a) (b)

Figure 2.1. Load distribution characteristics: (a) Flexible pavement; (b) Rigid pavement (Muench et al., 2003).

Rigid and flexible pavements have different load distribution characteristics. In flexible pavements, all layers carry the loading on the other hand in rigid pavements most of the loading is carried by slab only (See Figure 2.1) (Delatte, 2008). The details, advantages and disadvantages of both types of pavements are presented within this chapter.

2.1.1. Flexible Pavements

Flexible pavements which are commonly used for low to medium volume roads have a composition of different materials in a layered structure where stiffer materials are at the top and weaker ones are at the bottom (See Figure 2.2) (Kumar, 2017; Mallick and Tahar, 2018).

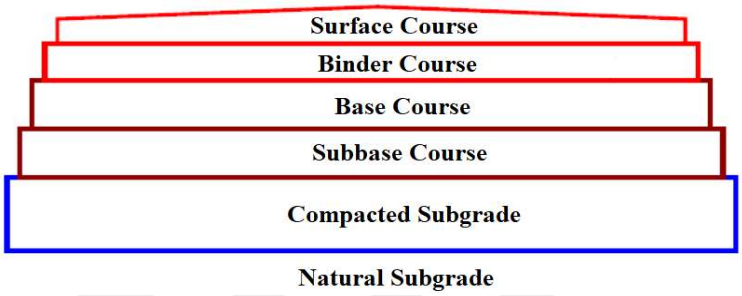


Figure 2.2. Typical cross-section of a flexible pavement (Huang, 2004).

Flexible pavements have very low flexural strength and they are considered to show flexible behavior under wheel loading. The load is transferred to the lower layers by granular structure and the stress decreases with increasing depth. Although flexible pavements are not so expensive, they have relatively high initial and maintenance costs (Adlinge and Gupta, 2009; Delatte, 2008; Mannering and Washburn, 2012). Flexible pavements have some advantages and disadvantages and Table 2.1 presents these in a summarized form.

Table 2.1. Advantages and disadvantages of flexible pavements (Harle, 2018).

Advantages of a flexible pavement
The cost of laying asphalt is less as compared to concrete.
Asphalt can be melted and reused again and again.
A small portion of asphalt can be repaired and relayed easily.
It provides more resistance and grip to the wheels of the vehicles.
Disadvantages of a flexible pavements
Rain, heat and snow have damaging effects on asphalt road and thus they require frequent repairs and maintenance.
Greenhouse gases and toxic gases are produced when the asphalt melts.
It has a relative short service life of 10 years.

2.1.2. Rigid Pavements

Rigid pavements which are generally used in weaker soils and in places where there is high traffic load are constructed from PCC slab over a granular base or subbase course (see Figure 2.3) (Deshmukh et al., 2017; Gill and Maharaj, 2015; Huang, 2004).

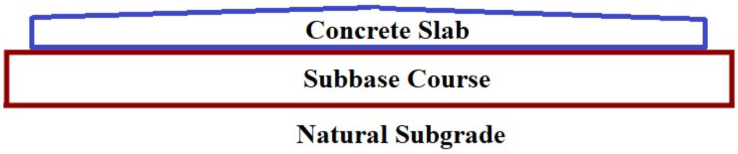


Figure 2.3. Typical cross-section of a rigid pavement (Huang, 2004).

In rigid pavements, the concrete slab which has high tensile strength carry most of the loading and distributes it over a wide area (Ioannides and Salsilli-Murua, 1989). Although the costs of rigid pavements are relatively high, they have less maintenance costs and longer service lives than flexible pavements (Jain et al., 2013). Rigid pavements also have advantages and disadvantages and they are summarized in Table 2.2.

Table 2.2. Advantages and disadvantages of a rigid pavement (Harle, 2018).

Advantages of a rigid pavement
It has a long service life of 40 years.
During the service life rigid pavement does not require frequent repair or patching work.
It does not deform under wheel loads, as a result, the fuel consumption decreases up to about 10%-20%.
It is not affected when exposed to oils, fuels etc. from automobiles.
It is unaffected by the action of rain and heat.
Since bitumen is burnt before paving, it produces pollution in the form of toxic gases. Rigid pavements do not incorporate such methods and thus pollution is reduced.
Since bitumen is produced from petroleum, which is becoming very scarce whereas rigid pavements are made from concrete (cement), which is abundant.
Disadvantages of a rigid pavement
It is relatively expensive.
The whole concrete slab is replaced if any failure or breakdown happens.
Due to rain and snow, vehicles generally slip on a rigid pavement.

2.1.3. Comparison of Flexible and Rigid Pavements

Flexible pavements have about 15 years of service life with low initial cost and they need periodic maintenance. On the other hand, rigid pavements have about more than 40 years of service life with high initial cost and they do not need much maintenance (Jain et al., 2013). It is observed that flexible pavements, which have less traffic volume, are more economical (Jain et al., 2013; Mohod and Kadam, 2016). When the structure of both pavement types are compared, it is seen that rigid pavements have stiffer structure than flexible pavements and they do not rupture under traffic loading (Jain et al., 2013). Tables 2.3 and 2.4 summarize properties and more comparable points between flexible and rigid pavements.

Table 2.3. Properties of flexible and rigid pavements (Medani et al., 2014).

Property	**Flexible Pavements**	**Rigid Pavements**
Deformation occurring in the subgrade is transferred to the upper layers.	yes	no
Design is based on	load-distributing characteristics of the component layers	flexural strength or slab action (rigid)
Flexural strength	low	high
Load transfer	grain to grain contact	flexural action
Construction cost depends on subgrade strength and traffic loading	yes	yes
Repairing cost	high	low
Life span	shorter	longer
The surfacing can be laid directly on the subgrade	no	yes
Thermal stresses	not critical	critical
Expansion joints needed	no	yes
Vehicles Fuel consumption	more	less

Table 2.4. Comparison of flexible pavements with rigid pavements (Mohod and Kadam, 2016).

Flexible pavements	**Rigid pavements**
Deformation in the subgrade is transferred to the upper layers	Deformation in the subgrade is not transferred to subsequent layers
Design is based on load distributing characteristics of the component layers	Design is based on flexural strength or slab action
Low flexural strength	High flexural strength
The load is transferred by grain to grain contact	No such phenomenon of grain to grain load transfer exists
Low completion cost, high repair cost	Low repair cost, high completion cost
Short service life (High Maintenance Cost).	Longer service life (Low Maintenance Cost).
Surfacing cannot be laid directly on the subgrade but a subbase is needed.	Surfacing can be directly laid on the subgrade.
No thermal stresses are induced as the pavement has the ability to contract and expand freely.	Thermal stresses are more vulnerable to be induced as the ability to contract and expand is very less in concrete.
Expansion joints are not needed.	Expansion joints are needed.
The strength of the road is highly dependent on the strength of the subgrade.	The strength of the road is less dependent on the strength of the subgrade.
Rolling of the surfacing is needed.	Rolling of the surfacing is not needed.
The road can be used for traffic within 24 hours.	The road cannot be used until 14 days of curing.
Force of friction is low.	Force of friction is high.

2.2. Types of Rigid Pavements

Elasticity and bending strength are high in hydraulic cement concrete that is used in rigid pavement slabs. Because of these mechanical properties, in rigid pavements, the applied loads are distributed to large areas. The concrete pavement slab can be applied to the formed base or subbase layer and sometimes directly applied to the available soil layer. Types of rigid pavements were briefly explained below (Delatte, 2008; Huang, 2004).

2.2.1. Conventional Rigid Pavements

In pavement construction, conventionally three different concrete pavement design types are commonly used, as given below.

2.2.1.1. Jointed Plain Concrete Pavements (JPCP). Jointed plain concrete pavement (JPCP) is the most widespread concrete pavement used in construction world. JPCP has an unreinforced structure and as a result, it is very economical. Joints are used to control the behavior under expansion and contractions and to restrain possible transverse cracks. In some applications, greased dowels can be used to prevent the breaking of aggregate interlocks due to heavy traffic loads (Delatte, 2008; Huang, 2004; Walubita et al., 2017). Overhead and side view of a typical JPCP is seen in Figure 2.4.

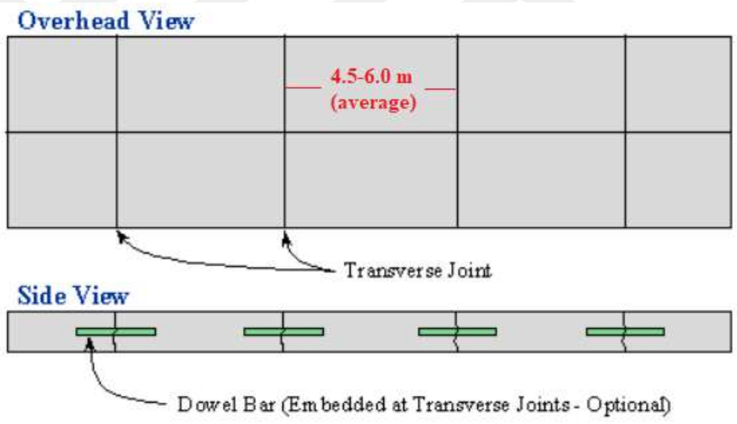


Figure 2.4. Jointed plain concrete pavement (Huang, 2004).

2.2.1.2. Jointed Reinforced Concrete Pavement (JRCP). Jointed reinforced concrete pavement (JRCP) is another type of rigid pavement. Unlike JPCP, there is a light reinforcement in this type of pavement and the slabs are longer than JPCP. JCRP contains reinforcement in the form of a welded wire fabric mesh or sometimes deformed bars can be used and this allows wider joint spacings. Steel dowel bars are used across transverse joints and steel tie bars are used across longitudinal joints (Delatte, 2008; Huang, 2004; Li et al., 2011). Overhead and side view of a JRCP is seen in Figure 2.5.

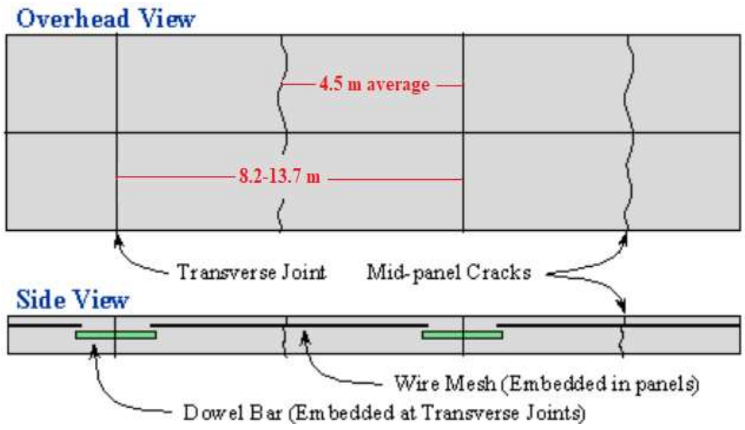


Figure 2.5. Jointed reinforced concrete pavement (Huang, 2004).

2.2.1.3. Continuously Reinforced Concrete Pavement (CRCP). Continuously reinforced concrete pavement (CRCP) is a structure that is preferred for the solutions to heavily loaded traffics. This type of pavement contains continuous, longitudinal steel reinforcement for managing the transverse cracking and does not have any transverse joints except some points like transitions to other pavement structures and bridge approaches (Roesler et al., 2016). Since CRCP has no joints, it provides a smoother ride than other types of pavements JPCP and JRCPs, however, it is rarely preferred because it is not very economical due to its heavy steel reinforcement composition (Delatte, 2008). Overhead and side view of a CRCP is seen in Figure 2.6.

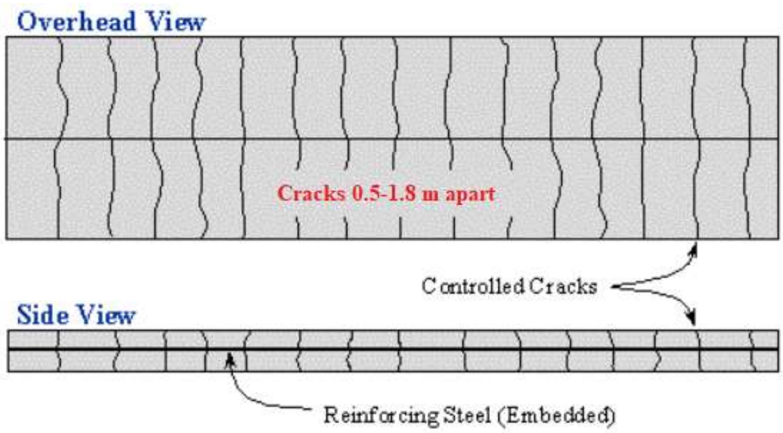


Figure 2.6. Continuously reinforced concrete pavement (Huang, 2004).

2.3. Roller Compacted Concrete Pavements

Roller compacted concrete is mainly compacted of cement, water, sand and aggregates like conventional concrete, but the cement content is less than that of conventional concrete. Apart from cement additional binders such as fly ash can also be added to this mixture. It is an economical concrete due to its low cement content. Roller compacted concrete has a much drier consistency than conventional concrete (Delatte, 2008). It requires high compression energy. RCC pavements are transported, spread and compacted by means of tools that are used for making bituminous slabs. With these features, RCC pavements can be constructed quickly and economically, thus it presents itself as an alternative superstructure. RCC pavements which were previously preferred in the world in industrial field floors where heavy loads were carried and went at low speed due to high surface roughness have been also used in urban roads and intercity highways in recent years. It emerges as an alternative to conventional concrete and asphalt roads. Roller compacted concrete production and construction are more practical than conventional concrete and can be used after compaction. It is laid and compacted in layers. In each layer, adequate and effective compaction should be ensured. This name is given to that concrete as it is usually compacted with vibratory rollers (Öztürk, 2018). Roller compacted concrete roads are manufactured using some common features of conventional concrete roads and asphalt roads. These common features are:

- The same materials are used as those used in conventional concrete roads. There are differences only in mixture ratios,
- The curing requirements are the same as with conventional concrete roads,
- As for the similarities with the asphalt roads, aggregate gradation, placement, compaction, etc. types of equipment are the same, and
- RCC pavements are generally used in roads with low-speed heavy traffic and in areas where strength, durability and economy are crucial such as airport runways and taxiways (Huang, 2004; Mallick and Tahar, 2018).

2.4. Fiber-Reinforced Concrete Pavements

The mechanical properties (such as flexural strength, tensile strength, compressive strength, creep behavior, impact resistance and toughness) of concrete pavements can be enhanced by using fiber reinforcement. In concrete road applications, fiber reinforcement can be used instead of steel meshes traditionally used for crack control. The main difference of fiber reinforcement with steel mesh reinforcement is the position, length and cross-sectional area along with the thickness of the pavement. The use of fiber reinforcements in concrete road applications can eliminate the costs and difficulties associated with the storage and placement of steel mesh and also allows the use of lower slab thickness (ACI Committee 544, 1996). LaHucik et al. (2017) states that the use of fiber reinforcements in concrete slabs (mostly for industrial grounds) has been going on for many years to increase the crack performance of concrete and the permissible joint spacing and reduce required slab thickness. In road applications, steel and synthetic fibers are generally used and regardless of the type of fiber used, it is crucial to design the fiber concrete appropriately to obtain the desired performance from the fiber reinforced concrete. Using a very low amount of fiber cannot improve the mechanical performance of the concrete, but the use of a high amount of fiber can increase the amount of void and thus also lead to a decrease in the strength and durability of the concrete. Therefore, the use of the appropriate amount of fiber has great importance in terms of fiber reinforced concrete performance (Öztürk, 2018). Regardless of the type of fiber, Roesler and Gaedicke (2004) state that in recent applications it has been aimed to use less than 0,5 % by volume of fiber for reasons such as economy and constructability. In their studies, Roesler et al. (2004) show that the ultimate load-carrying capacity of plain concrete pavement is significantly improved when the fiber is added and fiber type (fiber aspect ratio and fiber geometry) and fiber content (amount) are the most important factors affecting the ultimate load capacity of the slab. With respect to the performance of fiber-matrix interface and final concrete performance, design of the concrete matrix is very significant besides the parameters related with the fibers.

2.5. Pavement Design Factors

Many factors affect the design of the pavement. They can be divided into four groups as traffic and loading, structural models, subgrade type, material characterization and environment. These are going to be explained in detail in this section (Huang, 2004).

2.5.1. Traffic and Loading

The most important factor in the pavement design is the traffic. The most important factors are axle configuration, wheel load, contact pressure, moving loads, and load repetitions (Huang, 2004).

2.5.1.1. Axle Configuration. The axles are important parts that allow the wheels to rotate while moving. As the number of axles increases, the vehicle can carry more load. Thus, axle load also effects the pavement design. In the plate theory of rigid pavement design wheels on both sides are considered (Huang, 2004).

2.5.1.2. Wheel Load. To determine the pavement thickness to be applied, the wheel load on the pavement is a significant factor. The load of the wheels does not affect the subgrade soil when sufficient thickness is provided. Wheel configuration affects the stress distribution and deflection within a pavement. On most commercial vehicles, dual rear wheel is available, which enables the contact pressure is within the limits (Huang, 2004).

2.5.1.3. Contact Pressure. Since the contact area and contact pressure between the wheel and pavement surface are determined depending on it, tire pressure is an important factor. As it is simple to analyze, a circular area is generally considered although the shape of the contact area is elliptical (Huang, 2004).

2.5.1.4. Moving Loads. The vehicle's moving at creep speed will damage the pavement. Therefore, increasing the vehicle speed gradually results in less strain on the pavement (Huang, 2004).

2.5.1.5. Repetitions of Loads. Throughout its design life, many vehicles pass over the constructed pavement. Continuous repetitive wheel loads cause some deformation on the pavement. The sum of all wheel loads acting on the pavement gives the total deformation. Therefore, the load frequency is also considered in the pavement design. The single axle with dual wheels carrying 80 kN load is regarded as standard axle for the pavement design (Huang, 2004).

2.5.2. Structural Models

In determining the pavement responses like stresses, strains, and deflections on a pavement based on the application of wheel load, structural models are various analysis approaches. Plate theory is the most common structural model for rigid pavements. Portland Cement Concrete is used in the construction of rigid pavements. Since they are supposed to be elastic plates resting on viscous foundation, rigid pavements should be analyzed by the plate theory (Huang, 2004).

2.5.3. Subgrade Type

Subgrade soil must be tested to construct the pavement. The quality of the subgrade can be determined through the various tests such as CBR, Tri axial, etc. The required thickness for the pavement can be obtained by means of these tests. The pavement will be damaged easily if the subgrade soil is poor (Huang, 2004).

2.5.4. Material Characterization

For both flexible and rigid pavements, the following material properties are important. When pavements are considered as linear elastic, the modulus of elasticity, poisson's ratio of the subgrade and each component layer need to be specified. Also, for rigid pavements modulus of rupture has a significant impact on the required thickness design (Huang, 2004).

2.5.5. Environmental Factors

Performance of the pavement materials is affected by the environmental factors and these factors lead to various damages. Temperature and precipitation are the two types of environmental factors that affect the pavement and they are mentioned below (Huang, 2004).

2.5.5.1. Temperature. In the design of pavement, temperature is a crucial factor that needs to be taken into account. Temperature stresses or frictional stresses occur because of temperatural difference at the bottom and top of the slab in rigid pavements (Huang, 2004).

2.5.5.2. Precipitation. One of the primary reasons for pavement failure is the water under the pavement. Water coming either from precipitation or groundwater can lead the subgrade to become saturated or weaken. It can cause pavement pumping under heavy loads (Huang, 2004).

2.6. Failure Criteria of Rigid Pavements

Traditionally fatigue cracking of a pavement slab has been considered as the major, or only criterion for rigid pavement design. The allowable number of load repetitions that are going to fatigue cracking depends on the stress ratio between flexural tensile stress and concrete modulus of rupture. Other than pavement slab fatigue failure, pumping of foundation can be considered as another important critical criterion. The soil slurry ejects through the joints and cracks of cement concrete pavement during the downward movement of the slab under the heavy wheel loads. It is called “pumping”. Other major types of distress in rigid pavements include faulting, spalling, and deterioration (Delatte, 2008; Huang, 2004; Mallick and Tahar, 2018).

2.7. Concrete Pavement Design Approaches

There are two different approaches to the thickness design of rigid pavements: empirical approach (see Figure 2.7) and mechanistic-empirical approach (see Figure 2.8) These are explained as follows.

2.7.1. Empirical Design

Empirical design is a method based on observed performance rather than theoretical behavior (Selezneva et al., 2004). In empirical approach, the relationships among design inputs (such as loads, materials, layer configurations) were obtained based on experience and observations (Li et al., 2011). There is no need for complex computational capabilities or extensive material characterization for the design of pavement structures. In other words, complex cause-effect relationships between pavement design and observed pavement distresses are not needed to be defined theoretically. The correlation between pavement design input and pavement performance is established based on the observations of pavement responses to traffic loading and subgrade conditions (Boone, 2013). As an example, AASHTO design guide which is proposed in 1961 and regularly upgraded up until 1993, is a purely empirical design methodology.

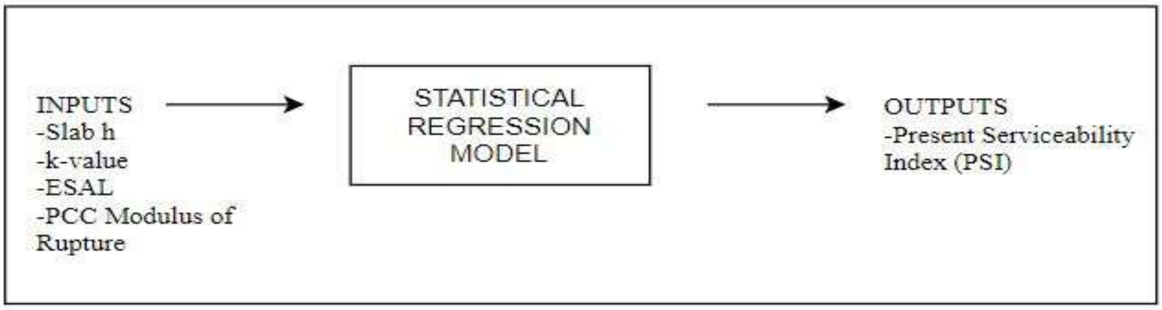


Figure 2.7. Illustration of the empirical approach to rigid pavement design. (U.S. Department of Transportation, 1992).

2.7.2. Mechanistic-Empirical Design

Mechanistic-empirical design links the observed performance with the theoretical behavior (Selezneva et al., 2004). This approach is considered more robust than the empirical approach, and since it involves engineering mechanics-based response terms its extrapolation is easy (Mashayekhi, Amini, Behbahani, and Nobakht, 2011). The mechanistic part of this approach calculates pavement structural responses (in terms of stress, strain and deflections) which result from traffic loading, environmental conditions and material properties. The empirical model then relates the pavement responses to the pavement performance (Boone, 2013).

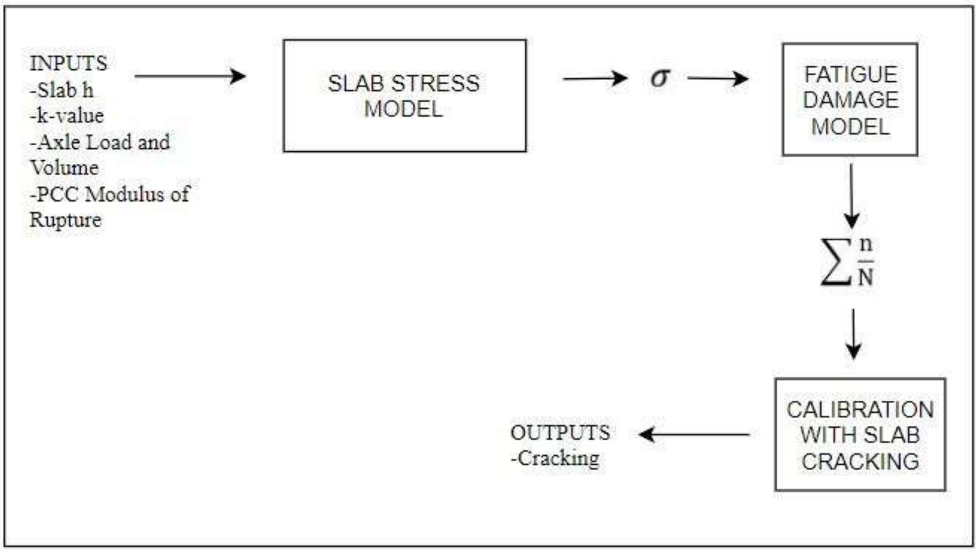


Figure 2.8. Illustration of the mechanistic-empirical approach to rigid pavement design (U.S. Department of Transportation, 1992).

Although empirical approaches have been widely used, the increased understanding of advantages of mechanistic-empirical design is becoming more prevalent (Mashayekhi et al., 2011). Some of the advantages of mechanistic-empirical design over empirical design are presented below (Timm et al., 1998).

- Consideration of changing load types,
- Better utilization and characterization of available materials,
- Improved performance predictions,
- Better definition of the role of construction by identifying the parameters that are most influential over pavement performance,
- Relation of material properties to actual pavement performance,
- Better definition of the existing pavement layer properties, and
- Accommodation of environmental and aging effects of materials.

2.8. Pavement Design Procedures

Three methods for pavement design procedures are described in this chapter: AASHTO (1993), IRC SP 46 (2013), IRC 58 (2011) and StreetPave v12 (2014) methodologies. The three methods discussed were chosen to provide an evolutionary perspective on the development of pavement design procedures and touch on the main key engines for developing existing methods.

2.8.1. AASHTO (1993)

The AASHTO (1993) pavement design method is based on empirical models developed by considering the effect of AASHO Road Test field results and the number of standard axle load repetitions on the performance of the slab (AASHTO, 1993). The AASHO Road Test consists of a series of experiments designed in the late 1950s and early 1960s in Ottawa, Illinois, to determine how the traffic load contributes to the deterioration of pavement and decrease in the service capability (Papagiannakis and Masad, 2008). Although the design guide was first launched in 1961, AASHTO updated its empirical design procedures in 1972, 1986, and 1993 (Delatte, 2008). The original empirical regression model based on the AASHO Road Test correlated the present serviceability index with slab thickness and axle load magnitude, type, and repetitions. This model was modified and extended by using Spangler’s corner stress formula in 1962, and by considering drainage, loss of support, effective k-value, variations of joint load transfer using the J factor, and incorporation of design reliability in 1986 (Ming-Jen et al., 1986; U.S. Department of Transportation, 1992). All of these published guidelines based on the same concept, but with the 1986 and 1993 editions, the transition from empirical to more theoretical-based design has been made in the design of the pavement structure. It should be noted that empirical performance equations, which are used to calculate the thickness of the pavement layer, have been developed under a certain climatic environment with a set of pavement materials, traffic and subgrade conditions (AASHTO, 1993). AASHTO (1993) provides different thickness design procedures for flexible and rigid pavement systems. The design procedure for rigid pavement involves solving an empirical equation and the thickness of the concrete slab is the only output that is obtained at the end of the iteration (Huang, 2004). AASHTO (1993) provides a design procedure for jointed plain concrete pavements, jointed reinforced concrete pavements, and continuously reinforced concrete pavements. The AASHTO design equation for rigid pavement is presented as follows (see Equation 2.1) (AASHTO, 1993; Delatte, 2008; Mallick and Tahar, 2018).

$$log_{10}W_{18} = Z_R * S_0 + 7.35 * log_{10}(D+1) - 0.06 + \frac{log_{10}\left[\frac{\Delta PSI}{4.5} - 1.5\right]}{1+\left[1.642*\frac{10^7}{(D+1)^{8.46}}\right]} + (4.22 - 0.32 *$$

$$\ldots P_t) * log_{10}\left[\frac{S_c' * C_D * [D^{0.75} - 1.132}{215.63 * J * [D^{0.75} - [18.42/(E_c/k^{0.25})]]}\right] \quad (2.1)$$

where

W_{18}: Predicted number of 18-kip (80kN) equivalent single axle load (ESAL) applications for the design period,

Z_R: Standard normal deviate,

S_0: Standard deviation,

ΔPSI: Design serviceability loss or (total change in serviceability index) (initial serviceability index (P_0) minus terminal serviceability index (P_t)),

P_t: Terminal serviceability index of the pavement,

D: The thickness of concrete pavement slab,

S_c': Modulus of rupture for Portland cement concrete (psi) (flexural strength of concrete),

J: Load transfer coefficient,

C_D: Drainage coefficient,

E_c: Modulus of elasticity for Portland cement concrete (psi), and

k: Modulus of subgrade reaction (pci).

AASHTO (1993) design guide classified the variables for design into four categories as follows.

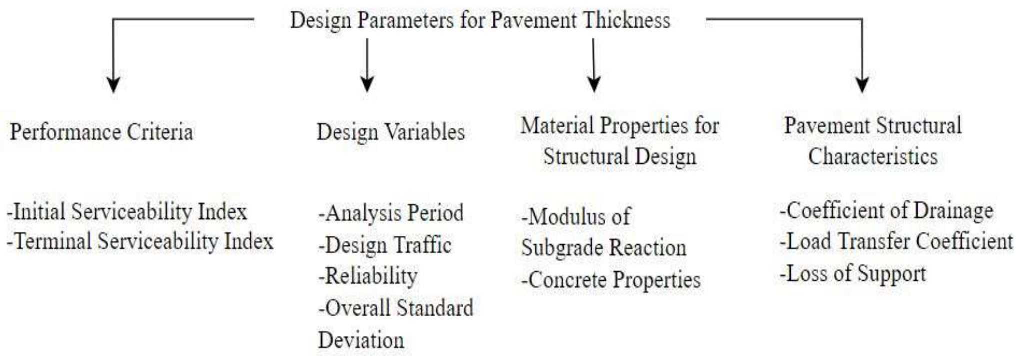


Figure 2.9. Summary of the design parameters for pavement thickness according to AASHTO (1993).

The parameters described below must be determined to complete a rigid pavement design using the AASHTO (1993) method.

- Serviceability

The pavement design approach of AASHTO was developed based on the serviceability concept. The ability of pavement to serve the type of traffic that uses the facility is stated as serviceability. Serviceability is rated on a scale of 0 (very rough impassible pavement) to 5 (perfectly smooth pavement) regarding the pavement's smoothness or rideability. A pavement's serviceability is represented with regard to the Present Serviceability Index (PSI) and the predicted loss in serviceability can be measured with the difference between initial and terminal serviceability as in the following equation (AASHTO, 1993).

$$\Delta PSI = P_0 - P_t \tag{2.2}$$

In the above equation, P_0 represents the initial serviceability index measured immediately after construction, and P_t is the terminal serviceability index at the time pavement loses its performance, i.e. when rehabilitation becomes necessary. It is recommended that $P_0 = 4.5$ for rigid pavements and $P_t = 2.5$ for major highways and $P_t = 2.0$ for highways of lesser traffic volumes (AASHTO, 1993).

- Analysis Period

The performance period is the time until a pavement design needs rehabilitation. A new, reconstructed or rehabilitated pavement structure has deformations from its initial serviceability to its terminal serviceability. And the performance period of a pavement structure is just that elapse time until these deformations occur. The total duration that the design strategy should include is called as "analysis period". It may be considered the same term as "performance period" but it is not exactly. Within the desired analysis period, realistic performance limitations can require planned rehabilitations. Thus, analysis period may include multiple performance periods. Such analysis time is the same as design life in the AASHTO (1993) guideline. AASHTO proposes different analysis periods for different road types depending on road conditions such as high volume urban or low volume paved (AASHTO, 1993).

- Traffic

Calculating a wheel or axle load for a single vehicle is not difficult but calculating the number and types of wheel or axle loads that will be applied to a given pavement during the design life becomes quite complicated. In the 1960s, in the complex empirical equations used to estimate pavement life, it was thought that using a single number to represent the entire traffic load would facilitate the procedure. Thus, in the AASHTO (1993) design procedure, traffic is characterized based on the predicted number of 18-kip (80 kN) single axle load during the design life for both highways and low-volume roads. This is also called Equivalent Single Axle Load (ESAL) and stated as W_{18} in the basic design equation given above (AASHTO, 1993; Huang, 2004). For design procedure, damage to the pavement resulted from the wheel load is a major concern rather than the wheel load. So, the ESAL concept is used to represent pavement damage caused by 18-kip single axle load (having dual tires on each side), which was the maximum legal load allowed during the AASHO Road Test in many states (Kawa et al., 1998). It is necessary to convert the total load applications due to mixed stream of traffic (varying axle loads and axle configurations) over the design life into the 18-kip ESAL (W_{18}) by using the recommended load equivalency factors (LEFs) (derived from statistical analysis of data taken during the AASHO Road Test) for each expected axle load. The equivalent effect of an 18-kip single axle load is obtained by multiplying the repetition number under each single or multiple axle load by its LEF (AASHTO, 1993; Huang, 2004; Kawa et al., 1998). In the guideline, the damage caused by the ESAL is described in terms of serviceability. Therefore, LEFs are a function of terminal serviceability (the terminal condition that is chosen to define the failure of the pavement structure, (P_t)) as well as pavement type (flexible or rigid) and slab thickness (D) as a structure capacity for rigid pavement. The design process is iterative, as it is necessary to make a thickness assumption to determine the equivalency factors. If the final design thickness is not sufficiently close (1 inch for rigid pavement) to the assumed thickness, the process must be repeated (AASHTO, 1993). Though LEFs provide a way of expressing equivalent damage levels between axles, it is more appropriate to express that damage in respect to the average amount of damage caused by a particular vehicle. The total amount of damage caused by one pass of a vehicle is obtained by adding the LEFs for each axle group of a particular vehicle. This summation represents a vehicle ESAL factor also known as truck factor defined as the average number of ESAL applications per vehicle (Mallick and Tahar, 2018). AASHTO (1993) suggests following basic equation to calculate the ESAL for each axle category.

$$ESAL_i = (Current\,Traffic * G * 365 * TF_i) \tag{2.3}$$

where

$ESAL_i$: ESAL for each axle category i;

i: truck class,

TF_i: Truck factor for truck class i, and

G: Growth factor.

In design, it is necessary to calculate a growth rate factor to convert a one-year traffic count to the total traffic the pavement will be exposed to during the design life. AASHTO (1993) provides the following equation for the expected growth rate (AASHTO, 1993).

$$Growth\,rate\,factor = \frac{(1+g)^n - 1}{g}, \quad \text{for } g \neq 0 \tag{2.4}$$

$$g = \frac{Annual\,Growth\,Rate}{100} \tag{2.5}$$

To obtain design ESAL it is necessary to sum up ESAL for each axle category as follows:

$$\acute{w}_{18} = Design\ ESAL = \sum_{i=1}^{n} ESAL_i \tag{2.6}$$

In the equation above Design ESAL ($\acute{w}_{18}$) represents the summation of ESAL for each axle category, and n is the number of truck classes. Design ESAL is the number of equivalent single axle loads considered for all lanes and both directions of travel. Truck traffic is found in all lanes on a multilane roadway, but only the lane that carries the majority of truck traffic is known as the design lane. And, for design purposes, this number must be multiplied by relevant factors and distributed to the lanes and directions. For determining traffic in the design lane (w_{18}), the following equation provided by AASHTO (1993) may be used (AASHTO, 1993; Mallick and Tahar, 2018):

$$w_{18} = D_D * D_L * \acute{w}_{18} \tag{2.7}$$

where

w_{18}: Traffic in the design lane in 18-kip ESAL

D_D: Directional distribution factor (trucks in design direction %),

D_L: Lane distribution factor for trucks,

$\acute{w}_{18}$: Design ESAL (the cumulative two-directional 18-kip ESAL)

The directional distribution factor used to determine the distribution of traffic relative to directions expressed as a percentage. It is usually assumed as 0.5 (50%) for each direction unless there are no special considerations such as more loaded trucks moving in one direction and more empty in the other. Also, the lane distribution factor is used to take into account the percentage of trucks in the design lane. It is stated as a percentage and varies depending on the number of lanes (AASHTO, 1993; Huang, 2004).

- Reliability

The reliability concept was introduced into the design process in 1986 to decrease the risk of premature structural deterioration below the acceptable level of serviceability (Ming-Jen et al., 1986). The reliability term generally refers to the probability level that the predicted design will exceed the required design, or in a simpler manner the probability that the strength of the material will exceed the stress on the material. The original AASHO equations were consisting of mean values that reduce the probability of the actual pavement to reach its design life by 50% (Mallick and Tahar, 2018). However, this level of risk is not acceptable for high volume and high-speed highways. AASHTO procedure incorporates a reliability coefficient to address the different levels of importance of a roadway. The more important its design is, the higher the reliability should be. It also depends on the traffic volume. As the traffic volume gets larger, the reliability should be increased. It allows the designer to set the level of certainty in the design. Typical reliability values for interstate highways are 90% or higher, whereas local roads can have reliability as low as 50% (AASHTO, 1993; Mannering and Washburn, 2012).

In the modified thickness design equation, the effect of reliability was taken into consideration by including Z_R and S_o parameters in the form of $Z_R * S_0$. Z_R means the standard normal deviate and S_0 means the overall standard deviation which is the combination of the standard error of the traffic prediction and performance prediction. S_o and Z_R terms are included within the design because there are always uncertainties in local traffic predictions. S_o should be selected to represent local conditions and assumed as 0.35 for rigid pavement (obtained from AASHO Road Test) (AASHTO, 1993; Mallick and Tahar, 2018). Standard normal deviate values corresponding to selected levels of reliability are presented in Figure 2.10.

Reliability, R (percent)	**Standard Normal Deviate, Z_R**
50	−0 000
60	−0 253
70	−0 524
75	−0 674
80	−0 841
85	−1 037
90	−1 282
91	−1 340
92	−1 405
93	−1 476
94	−1 555
95	−1 645
96	−1 751
97	−1 881
98	−2 054
99	−2 327
99 9	−3 090
99 99	−3 750

Figure 2.10. Standard normal deviate values corresponding to selected levels of reliability (AASHTO, 1993).

Besides, for a certain reliability level (R) the reliability design factor, F_R is explained as follows:

$$F_R = 10^{-Z_R * S_0} \tag{2.8}$$

where

F_R: Reliability design factor, $F_R \geq 1$

Z_R: Standard normal deviate

S_0: Overall standard deviation

The reliability design factor F_R explains the possible variations in both the design period traffic prediction (w_{18}) and the performance prediction (W_{18}). So, it provides a predetermined design reliability level (R%) that the pavement will carry the analysis period traffic. In fact, design-performance reliability is controlled by the following equation, which is actually embedded in the modified empirical thickness design equation (AASHTO, 1993).

$$W_{18} = w_{18} * F_R \tag{2.9}$$

where

W_{18}: Predicted number of 18-kip (80kN) equivalent single axle load applications

w_{18}: Traffic in the design lane in 18-kip ESAL

F_R: Reliability design factor

- Modulus of Subgrade Reaction

The type and magnitude of support the subgrade can provide should be determined because it will directly affect the design of the later stage. The effective modulus of subgrade reaction (k_{eff}) represents a measure of the support provided to the concrete slab by the underlying layers such as base, subbase, and subgrade (AASHTO, 1993; Huang, 2004). Typical cross-section for rigid pavement is given in Figure 2.11.

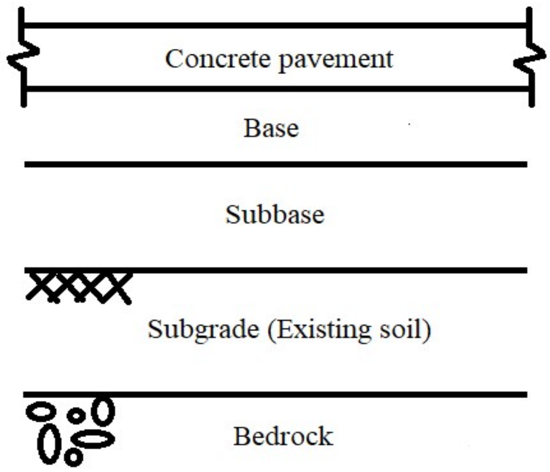


Figure 2.11. Cross-section for rigid pavement (Huang, 2004).

It is important that subgrade support must be uniform. Any improvements to the subgrade should be oriented toward providing uniformity rather than strength. According to the method presented in AASHTO (1993) guideline, the k_{eff} varies based on the subbase type, thickness (D_{SB}) and elastic modulus (E_{SB}), loss of support (LS) capability of the pavement, the dept from the slab to bedrock (rigid foundation), the moisture content and temperature of the subgrade. The values of strength and elastic modulus change according to the type of the subbase. Taking a subbase type into account to estimate k_{eff} provides a basis for evaluating its cost-efficiency in the design process. Potential design thickness should also be identified for each type of subbase to consider its cost-effectiveness. Also, the changes in temperature and moisture may affect the pavement performance in terms of the strength, durability and load-carrying capacity of the pavement and roadbed materials. Values of k depend on the season and differ throughout the year. So, relative damage parameter (u_r) which involves the influence of the seasonal change in the k-value should be evaluated. The relative damage u_r is described by the following empirical relationship (see Equation 2.10) (AASHTO, 1993; Huang, 2004). Also relative damage can be found by using relevant nomograph presented in AASHTO (1993).

$$u_r = (D^{0.75} - 0.39 * {k_{eff}}^{0.25})^{3.42} \tag{2.10}$$

where

u_r: Relative damage parameter,

D = Pavement slab thickness, and

k_{eff}: Effective modulus of subgrade reaction.

The average relative damage ($\bar{u}_r$) is given in Equation 2.11, which is a function of seasonal relative damage (u_r) and the number of seasons (n) (Huang, 2004):

$$\bar{u}_r = \frac{\sum u_r}{n} \tag{2.11}$$

In the design of the rigid pavements, the LS factor is included to consider potential support loss resulting from subbase erosion and deterioration caused by various vertical soil movements. The use of loss of support values has a very significant impact on the thickness design for concrete pavements because a specific set of pavement materials and one roadbed soil was used during the AASHO Road Test. In almost all cases at the AASHO Road Test where the concrete pavements fell below the minimum serviceability level, the cause of the failure was due to loss of support. Because the design equations were derived from this data, the reduction in serviceability is accounted for in the design procedure (Ming-Jen et al., 1986). In the design process, effective or composite k-value is reduced according to the size of the void that may develop under the slab. AASHTO (1993) offers some LS ranges (from 0 to 3) that vary depending on the type of material in respect of particularly its elastic modulus or stiffness. It is noted that the LS factor should also be taken into account for differential vertical soil movements that can lead voids under the pavement. Because a void may continue to develop, even if a non-erodible subbase is used, which decreases the pavement life (AASHTO, 1993).

The depth to the rigid foundation is another parameter that affects the design procedure. If bedrock lies within 3.048 m (10 feet) of the surface of the subgrade for any significant length along with the project, its effect on the overall k-value and the design slab thickness for that segment should be considered. The k_{eff} value should be modified, when the depth to a rigid foundation is less than 3.048 m (10 feet). However, when the concrete slab is located directly on subgrade (without subbase), AASHTO (1993) suggests the following theoretical relationship to obtain the composite modulus of subgrade reaction. The roadbed soil to be used for rigid pavement design is characterized by a subgrade reaction modulus k rather than a roadbed soil resilient modulus MR. Therefore, it is required to convert MR to k according to Equation 2.12 (AASHTO, 1993; Huang, 2004).

$$k = \frac{M_R}{19.4} \tag{2.12}$$

where

k: Composite modulus of subgrade reaction, and

M_R: Elastic modulus of the roadbed soil.

With the help of the relevant nomograms presented in AASHTO (1993), an effective modulus of subgrade reaction can be calculated with the following steps:

1. Assign roadbed soil resilient modulus (M_R) for each season,
2. Assign subbase resilient modulus (E_{SB}) for each season,
3. Determine composite k for each season,
    i. For $D_{SB} = 0$, use Equation 2.12,
    ii. For $D_{SB} > 0$, use relevant nomograph,
4. If depth to rigid foundation<3.048 m (10 feet), correct k for effect of rigid foundation near the surface using relevant nomograph, (This step should be ignored if the depth to a rigid foundation is greater than 3.048 m (10 feet)),
5. Estimate the required thickness of slab using relevant nomograph and determine relative damage (u_r) for each season,
6. Use average u_r to determine k_{eff} using relevant nomograph, and
7. Correct k_{eff} for potential loss of support (LS) using the relevant nomograph.

- Concrete Properties

On the other hand, in determining the strength of Portland Cement Concrete (PCC), the concrete modulus of rupture (S_c') and modulus of elasticity (E_c) are evaluated. 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 (AASHTO, 1993). Because concrete gains strength with age, the average 28-day strength is used for design purposes (Mannering and Washburn, 2012). If center point loading is used, a correlation with third-point loading should be established. Using the specified minimum construction strength will cause the design to be too conservative. Therefore, it is essential to correct the specified minimum strength to the design strength using the following Equation 2.13. (AASHTO, 1993; Delatte, 2008).

$$S_c'(mean) = S_c + z * (SD_s) \tag{2.13}$$

where

S_c': Estimated mean value for PCC modulus of rupture,

S_c: Construction specification on the concrete modulus of rupture

SD_s: Estimated standard deviation of concrete modulus of rupture, and

z: Standard normal variate

PS: The percent of the strength distribution

z=0.841, for PS=%20,

z=1.037, for PS=%15,

z=1.282, for PS=%10,

z=1.645, for PS=%5, and

z=2.327, for PS=%1

It is also needed a value for the concrete elastic modulus in the AASHTO design equation (Mannering and Washburn, 2012). The concrete modulus of elasticity is obtained from the stress-strain curve as taken in the elastic area. The modulus of elasticity for concrete (E_c) largely determined by the strength of the concrete. AASHTO suggests that elastic modulus ought to be measured according to the method defined in ASTM C 469 for such high stiffness materials like PCC (AASHTO, 1993). The value that is determined for E_c does not change significantly depending on the concrete strength range used in the construction of the slab and generally does not have an important effect on the design of the pavement thickness (Mallick and Tahar, 2018). To estimate the modulus of the elasticity E_c for Portland Cement Concrete, the American Concrete Institute equation which describes a relationship between the compressive strength of concrete and modulus of elasticity is recommended in AASHTO (AASHTO, 1993).

$$E_c = 57000 * (f_c')^{0.5} \tag{2.14}$$

where

E_c: Elastic modulus of PCC, and

f_c': Compressive strength of PCC defined using ASTM C39, AASHTO T22, or T140.

- Load Transfer Coefficient

To explain the concrete pavement structure's ability to transfer (distribute) loads across discontinuities such as joints or cracks, the load transfer coefficient J factor is used in rigid pavement design. J value is determined according to the type of pavement (JPCP, JRCP or CRCP); if transfer devices like dowels are used; and the type of shoulder (tied concrete or asphalt). For a specified set of conditions such as jointed concrete pavement with tied shoulders, generally, J value is going to increase with an increase in traffic volume as aggregate interlock decreases with load repetitions (AASHTO, 1993; Huang, 2004; Mallick and Tahar, 2018).

Higher coefficients ought to be used with low k – values, high thermal coefficients, and large temperature differentials as a general guide for the range of load transfer coefficients. Less support is provided to the pavement when higher load transfer coefficients are used. The pavement is expected to be more susceptible to faulty pumping, corner breaks and other load-related joint and corner deterioration with less support (AASHTO, 1993; Mallick and Tahar, 2018). The use of tied PCC shoulders (or widened outside lanes) on the other hand decreases the slab stress and increases the pavement's service life significantly lower J values can be used for both jointed and continuous pavement design in order to account for this. For tied shoulder, the lower J value supposes that traffic is not allowed to run on the shoulder (AASHTO, 1993).

The factors proposed for the design were obtained from experience (AASHO Road Test) and mechanistic stress analysis. The use of dowels at transverse joints exist in the nature of the procedure. Therefore, joint faulting was not a manifestation of distress at the Road Test. If undoweled joints are to be considered by the designer, an appropriate J factor (from the table given in AASHTO (1993)) may be developed or the design can be controlled by another institution's procedure such as PCA (1984) procedure (AASHTO, 1993). J=3.2, the most common value is for a JPCP or JRCP which is doweled but not tied to a concrete shoulder (AASHTO, 1993; Huang, 2004; Mallick and Tahar, 2018).

- Coefficient of Drainage

One of the primary reasons for pavement failure is the water under the pavement. Water coming either from precipitation or groundwater can lead the subgrade to become saturated or weaken. It can cause pavement pumping under heavy loads (AASHTO, 1993). The drainage coefficient characterizes the quality of drainage of subbase layers under the concrete slab. Water cannot reach saturation in the underlying layers with a good drainage supply; therefore, pumping is unlikely to occur (Mannering and Washburn, 2012).

Suggested C_d values about the quality of the drainage (i.e. the time required for the pavement to drain) and the amount of time during the year that the pavement structure is exposed to moisture levels approaching saturation are provided by AASHTO (1993) (AASHTO, 1993; Huang, 2004; Mallick and Tahar, 2018).

Drainage condition was included in the part of the performance equation that considers the strength of the slab, the stress and the support condition since the C_d affect the slab support and thus the overall stress condition in the slab (Seeds and Hicks, 1986).

Whereas slow draining layers that become saturated quite often are given a drainage coefficient as low as 0.8, generally quick-draining layers that almost never become saturated have a drainage coefficient as high as 1.2 according to the values given AASHTO (1993). For AASHO Road Test sub drainage conditions the C_d values are 1.0. As it was proved by the heavy pumping which occurred on some of the test sections at the AASHO Road Test, the pavement was not well-drained (AASHTO, 1993).

The drainage coefficient increases the required pavement thickness to compensate for the poor drainage. It is not always going to be an effective approach obviously. It is better to solve the problem if the drainage is really a problem instead of trying to build the pavement thicker to compensate (Delatte, 2008).

2.8.2. IRC SP 46 (2013)

An Indian guideline IRC SP 46 (2013) was chosen for thickness design in this study because the guideline offers a method of design for fiber-reinforced concrete pavements and includes various types of fibers. It is possible to summarize the design process provided in the guideline as follows. For design and quality control, flexural strength and toughness of FRC are the most important parameters. In the design of FRC pavement, the characteristic values of both flexural strength and equivalent flexural strength parameters (f_{ctk} and f_{e150k}) are used. If there is not enough data to determine the characteristic values, Equations 2.15 and 2.16 given in IRC SP 46 (2013) can be considered.

$$f_{e150k} = 0.7 * f_{e150m} \tag{2.15}$$

$$f_{ctk} = 0.7 * f_{ctm} \tag{2.16}$$

In equations above, f_{e150k} is characteristic equivalent flexural strength, f_{e150m} is mean equivalent flexural strength, f_{ctk} is characteristic flexural strength and f_{ctm} is mean flexural strength obtained from test results (IRC SP 46, 2013).

IRC SP 46 (2013) proposes a two-stage design approach with ultimate moment capacity control based on the third edition of TR 34 (3rd edition) (2003) and fatigue damage control based on IRC 58 (2011) (IRC SP 46, 2013). The method in the guideline can be outlined shortly, as given below.

2.8.2.1. Fatigue Damage Control based on IRC 58 (2015). IRC 58 (2015) guideline design approach focus on rationalizing the design process of PQC (Pavement Quality Concrete) slab by taking into account cumulative damage of fatigue due to the simultaneous action of tensile flexural stresses caused by single, tandem and tridem axle traffic loads and temperature gradient between the top and bottom fibers of concrete slab for various axle categories. However, the concept of warping caused by moisture changes along the slab depth is considered to be the inverse of the temperature change and is generally not regarded as critical for thickness design (IRC 58, 2015).

According to the guideline, carrying out hourly cumulative fatigue damage analysis is ideal, but no data is available to implement such an exercise. It is proposed that the maximum positive and negative temperature differentials separately may be accepted to be constant for the six-hour period between 10 AM and 4 PM during the day and for the six-hour period between 0 AM and 6 AM during the night hours. For the remaining 12 hours, the slab can be supposed to be free of curling stress for fatigue damage analysis because fatigue damage resulted from the combined load and temperature differential action during this period will be unimportant. In general, it is assumed that a pavement slab's top surface tends to be convex shape during the daytime since the temperature differences are positive and tends to be concave shape during the nighttime due to the negative temperature differentials. In fatigue damage check, the calculation of axle load stresses is crucial when the slab is in a curled form due to temperature difference in the day and night. During the day and night respectively, tensile and compressive stresses are generated in the bottom fibers due to the curling of the slab. Such two separate critical stress conditions can be explained below for day and nighttime. A critical stress condition is caused by the most severe combination of the various factors that trigger the maximum stress in the pavement (IRC 58, 2015).

First, the flexural stress at the bottom layer of the concrete slab becomes the maximum during the daytime when the axle loads operate in the center of the pavement slab, while a positive temperature gradient is observed. As illustrated in Figure 2.12 the temperature of the upper part of the concrete pavement slab during the day is higher than the lower part. It causes the slab to take a concave shape as given in Figure 2.12. So, this condition provides the possibility of bottom-up cracking (BUC). For single, tandem, and tridem axles, the maximum flexural stress location at the bottom of the pavement slab with or without tied concrete shoulder is equivalent, and the critical positions are shown in the Figure 2.13. It should be noted that the single axles trigger the highest stress followed respectively by tandem and tridem axles. Since the stress caused by tridem axles is low, stress analysis for bottom-up cracking wasn't taken into account (IRC 58, 2015).

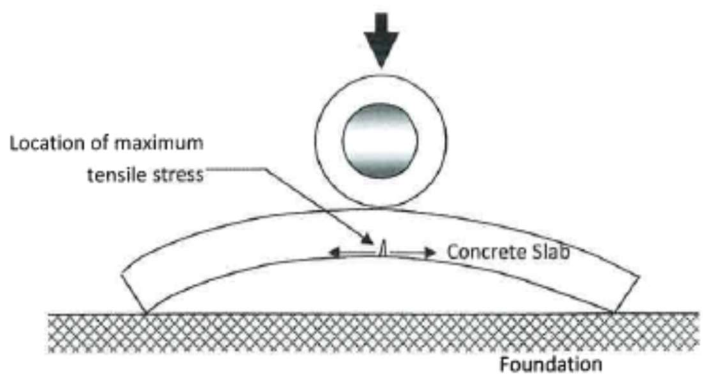


Figure 2.12. Axle load placed in the middle of the slab during daytime (IRC 58, 2015).

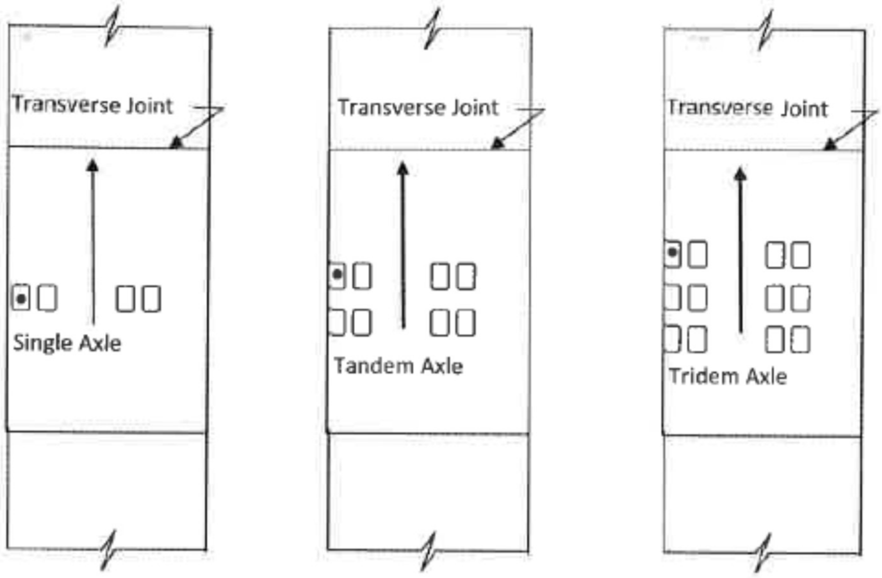


Figure 2.13. Placement of axles for maximum edge flexural stress at the bottom of slab without concrete shoulders (IRC 58, 2015).

Secondly, the top surface of the concrete pavement slab becomes cooler than the bottom surface during the night hours and the slab ends tend to curl up in a concave form resulting in support loss as can be seen in Figure 2.14. The temperature tensile stresses occur at the top of the pavement slab because of the restraint provided by the concrete self-weight and the dowel connections. Therefore, the flexural stress at the top surface of the slab becomes maximum during the nighttime when the axle loads operate near the transverse joints of the pavement slab, while a negative temperature gradient is observed. Because of the combined effect of high negative temperature gradient and the various axle loads, high tensile stress arises near the center of the critical longitudinal edge at the top of the slab and, thus, a top-down cracking (TDC) is initiated, as shown in Figure 2.14. Besides Figure 2.15 demonstrates the different axle configurations on the slab with successive axles positioned close to the transverse joints. Also, it is not expected that vehicles having axle spacing over 4,5 m will contribute to the top-down fatigue cracking (IRC 58, 2015).

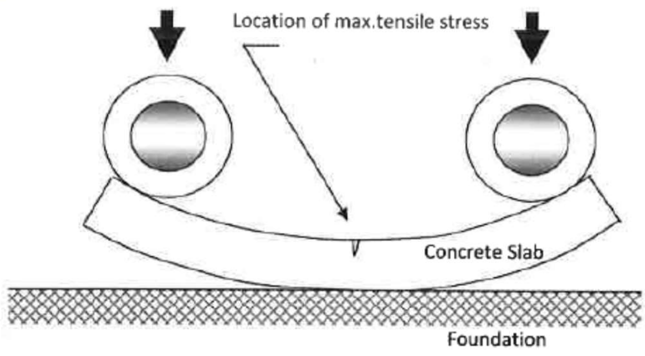


Figure 2.14. Placement of two axles of a commercial vehicle on a slab curled during nighttime (IRC 58, 2015).

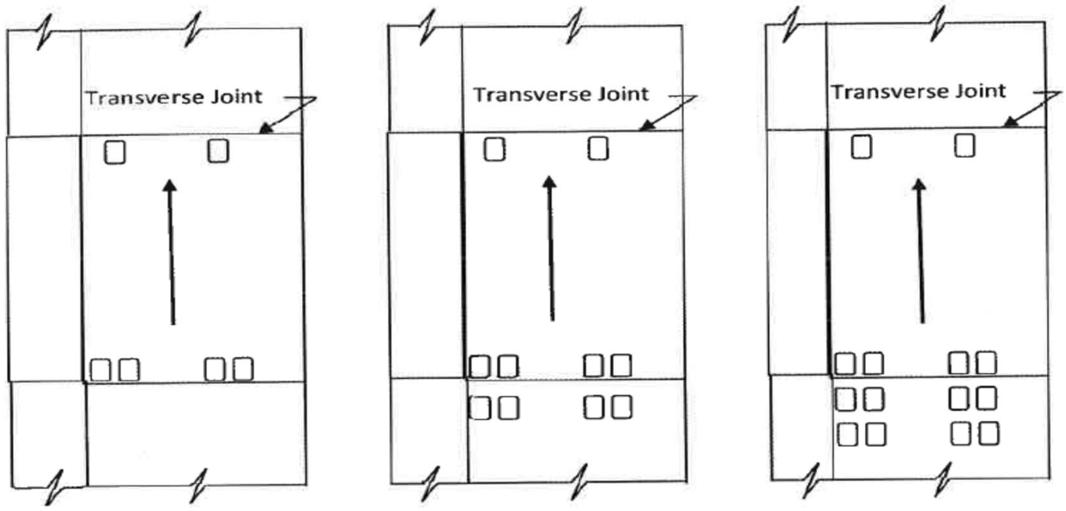


Figure 2.15. Different axle load positions causing tensile stress at the top fiber of the slab with tied concrete shoulder (IRC 58, 2015).

IRC 58 (2015) design takes into account the sum of cumulative fatigue damage due to wheel load (single, tandem, tridem axles) repetitions which caused flexural tensile stress at the bottom and top of the concrete pavement slab and the temperature differential between top and bottom of the slab for different axle categories. Repeated flexural stresses of various magnitudes gradually damage the slab, leading to crack propagation in the slab and ultimately to the development of microcracks under repeated loading and then failure. To calculate the maximum flexural stresses in the concrete slab in the edge region that arising from the combined effect of repeated loads and temperature difference, IRC 58 (2015) suggests various regression equations changing according to the different effective modulus of subgrade reaction values for the bottom-up and top-down cracking cases. By dividing the flexural stress obtained by regression analysis to the design flexural strength of cement concrete, a stress ratio (SR) is determined. If the stress ratio is less than 0,45 for plain concrete, the number of allowable axle load repetitions can be taken as infinite. It can be inferred here is that the higher the stress ratio, the less the number of repetitions of the load required for cracking. However, for stress ratio values greater than 0,45, the allowable repetitions of the different axle load groups can be found by the following relationships (IRC 58, 2015).

$$N = Unlimited\ for\ SR < 0.45 \tag{2.17}$$

$$N = \left(\frac{4.2577}{SR-0.4325}\right)^{3.268} \qquad 0.45 \le SR \le 0.55, \tag{2.18}$$

$$log_{10}N = \left(\frac{0.9718-SR}{0.0828}\right) \qquad For\ SR > 0.55 \tag{2.19}$$

where
N: Allowable number of load repetitions, and
SR: Maximum stress ratio.

Based on Miner's hypothesis, within IRC 58 (2015) these fatigue criteria are used to check the pavement slab's thickness adequacy. According to Miner's hypothesis, fatigue resistance is considered to be consumed not only by the repetition of one load but by the repetition of other loads. The above conservative fatigue criteria, which can be used to analyze bottom-up and top-down cracking have been developed by PCA 1984. It is possible to calculate fatigue damage to the slab by dividing the expected number of load repetitions to the allowable number of load repetitions of that load level. In the design, the cumulative fatigue damage (CFD) caused by the wheel loads and curling stresses in both the bottom and the top of the pavement slab is expected to be less than 1 (CFD (BUC)+CFD (TDC)≤1). The pavement should be considered safe if the CFD is less than 1 or is considered safe against large cracks if the CFD is equal to 1. Therefore, until this requirement is met, the design thickness should be increased (IRC 58, 2015).

As noted earlier, the analysis shows that in bottom-up and top-down cracking situations, contribution to CFD is only significant from 10 AM to 4 PM, and 0 AM to 6 AM respectively due to higher stresses because of the combined action of temperature gradient and wheel load. Cumulative fatigue damage expressions are demonstrated by Equations 2.20 and 2.21 for bottom-up and top-down cracking situations respectively (IRC 58, 2015).

For bottom-up cracking;

$$\sum_i^j \left(\frac{n_i}{N_i}\right) for\ 10\ AM\ to\ 4\ PM \tag{2.20}$$

For top-down cracking;

$$\sum_i^j \left(\frac{n_i}{N_i}\right) for\ 0\ AM\ to\ 6\ AM \tag{2.21}$$

where

N_i: Number of allowable load repetitions over the stated six-hour period for the i^{th} load category

n_i: Number of expected load repetitions over the stated six-hour period for the i^{th} load category

j: Total load category number

The limiting stress ratio value taken from IRC 58 (2015) and CFD procedure for plain concrete pavements are described above. The same procedure is also valid for fiber-reinforced concrete pavements mentioned in IRC SP 46 (2013) and the limiting stress values for fiber-reinforced concrete pavements are considered as follows. (IRC 58, 2015).

Pavements with fiber reinforced concrete (FRC) of low toughness ($f_{e150k} < 0.3*f_{ctk}$) shall be designed as non-reinforced pavements. In this case, fibers are considered to mainly contribute in controlling plastic shrinkage and temperature induced cracks (IRC SP 46, 2013). If polymeric fibers are used in dose less than 0.3 % by volume, for fatigue endurance, the limiting stress ratio should be taken as 0.45 (as in IRC 58 (2015)).

Significantly efficient fiber dosage is used to produce fiber reinforced concrete with significant toughness. When the polymeric fibers are used in FRC at a dose of 0.3% or more than 0.3% by volume of concrete for the mainly to control plastic shrinkage cracks or in FRC with steel or any other fibers with low toughness for fatigue strength, the limiting stress ratio can be taken as 0.5 instead of 0.45 as in plain concrete. So, the stress ratio values greater than 0.5, the allowable repetitions of the different axle load groups can be found by the following relationships (IRC SP 46, 2013).

$$N = Unlimited\ for\ SR < 0.5 \quad (2.22)$$

$$N = \left(\frac{3.7375}{SR-0.48}\right)^{3.333} \qquad 0.5 \leq SR \leq 0.576 \quad (2.23)$$

$$log_{10}N = \left(\frac{0.98-S}{0.07618}\right) \qquad For\ SR > 0.576 \quad (2.24)$$

High toughness FRC pavements will also be designed according to the procedure given in IRC 58 (2015) and the limiting stress ratio for the fatigue endurance shall be taken as 0.6. And, the allowable repetitions of the different axle load groups can be found by the following relationships (IRC SP 46, 2013).

$$N = Unlimited\ for\ SR < 0.6 \quad (2.25)$$

$$N = \left(\frac{2.9212}{SR-\ \ .57}\right)^{3.333} \qquad 0.6 \leq SR \leq 0.627, \quad (2.26)$$

$$log_{10}N = \left(\frac{0.99-SR}{0.06189}\right) \qquad For\ SR > 0.627 \quad (2.27)$$

<u>2.8.2.2. Ultimate Moment Capacity Check.</u> The described design approach in IRC SP 46 (2013) relies on the yield line analysis that provides a simple and quick assessment of the ultimate capacity of the concrete pavement slab following the procedure of the UK Concrete Society TR 34 (2003) (IRC SP 46, 2013).

Fibers used in normal dosages are supposed to have no significant impact on the first cracking strength of the concrete. Experimental studies show that the sub-base reaction, fiber type, and dosage are the main parameters that control the ultimate failure loads which may be higher by up to 60 %. Nevertheless, in terms of the fiber amount and fiber type used, the feasibility of inelastic analysis must be guaranteed. IRC SP 46 (2013), on the other hand, allows the ultimate moment due to the load to be determined by any suitable inelastic analysis if the $f_{e150k} \geq 0.3\ f_{ctk}$ is provided (IRC SP 46, 2013).

Figure 2.16 indicates the state of a single wheel load that is applied internally over a small circular area on a large concrete slab with ground support. In addition, in Figure 2.16, it is seen that the circumferential cracks are produced by the negative bending moment and the radial cracks by the positive bending moment. If the maximum negative circumferential moment is higher than the negative moment capacity of the slab, tensile cracking will take place at the top of the slab. Although the fiber increases the ductility of the concrete, the negative bending moment capacity does not increase, and thus, the first cracking stress is not affected. Accordingly, for design, the limiting criterion is assumed to be the development of visible circumferential cracks on the upper surface of the slab due to the negative bending moment. Besides, it can be concluded from the following Figure 2.16 that the cracked fiber concrete can resist the stresses resulting from the positive moment whereas the uncracked concrete strength can resist the stresses resulting from the negative moment (IRC SP 46, 2013).

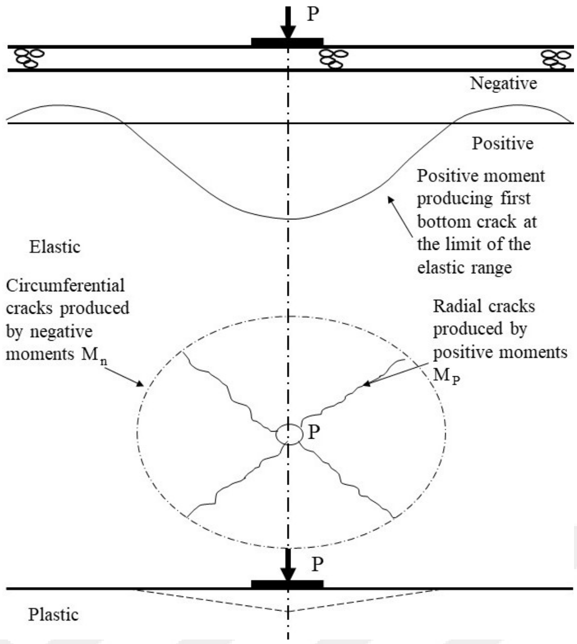


Figure 2.16. Development of radial and circumferential cracks in a concrete ground-supported slab (TR 34, 2003).

Hence, supposing redistribution of moments, the limit moment of resistance of the slab can be calculated as in the following equation.

$$M_0 = M_n + M_p \tag{2.28}$$

In Equation 2.28, M_0 is the limit moment of resistance, M_n is the negative moment of resistance and M_p is the positive moment of resistance of the slab (IRC SP 46, 2013).

Thus, in an interior location, the relation between the yielding moments for different loading cases can be represented as Equation 2.29:

$$\frac{M_n + M_p}{P * \gamma_f} = f\left(\frac{a}{l}\right) \tag{2.29}$$

where γ_f is the load factor (minimum 1.2), P is the applied load, a is the radius of the area under distributed load and l is the radius of relative stiffness (IRC SP 46, 2013).

The pavement has limited length and width, therefore the position of the load on the pavement surface has critical importance to the character or intensity of the maximum stress induced by the application of a particular traffic load (TR 34, 2003).

As shown in Figure 2.17, TR 34 (2003) considers three different loading positions for the design, namely internal, edge, and corner as critical load positions. Although the guide shows that the corner condition gives the worst load resistance performance theoretically, the experience has demonstrated the fact that the actual load capacity of the slab for corner loads is considerably higher than the theoretical one. Therefore, edge and internal loads should be given priority instead of corner loads for the design (TR 34, 2003).

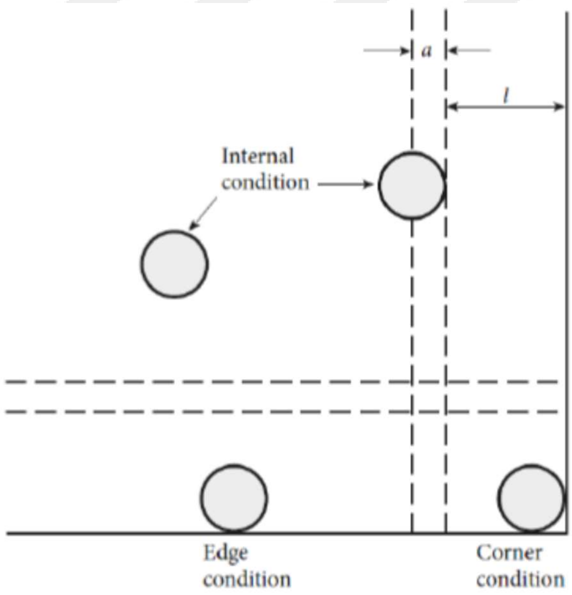


Figure 2.17. Definitions of loading locations (TR 34, 2003).

where a is the radius of the area under distributed load and l is the radius of relative stiffness which can be calculated by the following equation.

$$a = \sqrt{\frac{P}{\pi * \tau}} \tag{2.30}$$

$$l = \sqrt[4]{\frac{E * h^3}{12 * (1 - \mu^2) * k}} \tag{2.31}$$

In the Equations above 2.30 and 2.31, P represents the applied load, τ is tyre pressure, k is effective modulus of subgrade, E is modulus of elasticity of concrete, μ is poisson's ratio of concrete, and h is the slab thickness (TR 34, 2003).

A number of equations for each location are provided in TR 34 (2003) for the evaluation of the ultimate load capacity (P_u) of ground-supported concrete slabs to be exposed to a single wheel load. However, as already mentioned, the control of ultimate capacity is only considered in the case of edge and internal loading conditions (TR 34, 2003).

The following Equations 2.32 and 2.33 are proposed by TR 34 ((2003) third and (2016) fourth edition) to determine the ultimate load capacity of the concrete slab for an internal and edge loads, respectively. The maximum allowable load should be taken as a minimum of these two $P_u$values for the thickness design of fiber-reinforced concrete (TR 34, 2003).

$$P_u = \frac{M_p+M_n}{1-\frac{a}{3*l}} * 4 * \pi \tag{2.32}$$

$$P_u = \frac{\pi*(M_p+M_n)+4*M_n}{1-\frac{2*a}{3*l}} \tag{2.33}$$

In the equations above, P_u represents the ultimate load capacity, M_p and M_n are positive and negative moment capacity of the slab, respectively. Also, a is the radius of area under distributed load and l is the radius of relative stiffness (TR 34, 2003).

If the load applied in Figure 2.16 increases, the flexural stresses under the load will be equal to the concrete's flexural strength. Thus, the slab will start to yield due to the radial cracks produced by positive tangential moments on the base of the slab (TR 34, 2003).

In determining the positive moment capacity of the slab, the measurement of post-crack flexural strength taken into account to calculate the contribution of fibers to the flexural performance of concrete vary according to the utilized standards. In fact, IRC SP 46 (2013) recommends the use of ASTM C 1609, which employs beams in the four point loading arrangement based on TR 34 (3rd edition) (2003) to determine the post-cracking properties of the fiber-reinforced concrete. Nevertheless, EN 14651, based on the three point bending test, is now used for characterizing fiber-reinforced materials in the new version of TR 34 (2016). The following equations are proposed by TR 34 (2003) based on the four point bending test and TR 34 (2016) based on the three point bending test for the calculation of positive moment capacity, respectively (TR 34, 2003).

$$M_p = \frac{f_{e150}}{\gamma_m} * \frac{h^2}{6} \tag{2.34}$$

$$M_p = \frac{0.16 * f_1 + 0.29 * f_4}{\gamma_m} * \frac{h^2}{6} \tag{2.35}$$

where, f_{e150k} is the characteristic equivalent flexural strength, γ_m is the material safety factor, h is the slab thickness. Also, f_1 and f_4, obtained from the EN 14651 beam test, are characteristic residual flexural strengths of concrete representing flexural tensile stresses at a Crack Mouth Opening Displacement (CMOD) of 0.5 mm and 3.5 mm respectively.

As stated before, since the fibers do not affect the initial cracking stress of the concrete, TR 34 (2003) and TR 34 (2016) suggest the following equation for calculation of the negative moment capacity (TR 34, 2003).

$$M_n = \frac{f_{ctk}}{\gamma_m} * \frac{h^2}{6} \tag{2.36}$$

where M_n represents the negative moment capacity, f_{ctk} is characteristic flexural concrete strength, γ_m is the material safety factor and h is the slab thickness.

In the equations given above for the calculation of positive and negative moments, the strength values are divided by the partial material factor to obtain the design strengths (IRC SP 46, 2013; TR 34, 2003).

<u>2.8.2.3. Evaluation of Erosion.</u> Cracks are observed in many well-designed concrete pavements in a short 5-year duration after completion of construction. This is not due to fatigue cracks caused by structural deficiencies, but mainly due to support loss because of

granular subbase permanent deformation and erosion, the existence of water and heavy loads. AASHO Road Test states that in addition to fatigue cracking of concrete pavements, the deterioration of the foundation is a significant type of failure and should be taken into account in the design and maintenance for the adequate quality of the pavements. PCA 1980, using the AASHO Road Test data, showed that, in the presence of moisture the erosion of foundation was mostly caused by tandem and multi-axle vehicles, while fatigue cracking of the concrete pavement slabs was caused by single-axle vehicles. In particular, it is considered that the quality of subbase and subgrade, the climatic conditions, and the vehicles' gross weight are the main reasons for erosion. Although the guidelines suggest Dry Lean Concrete (DLC) (the non-erodible subbase), due to the very heavy commercial vehicles existing water may enter through cracks and joints. So, this condition results in the abrasion of granular materials and subbase causing longitudinal cracking along the wheel path on many highways. Considering the fact that tandem, tridem, and multi-axle vehicles constitute a large percentage of the total commercial vehicles on the roadways in India, it is clear that erosion is of crucial importance in designs and that the current design approach needs to be developed accordingly (IRC SP 46, 2013).

2.8.3. StreetPave v12 (2014)

StreetPave is a software used for determination of required pavement thickness (concrete or asphalt) for city, municipal, country and state roadways. The software was developed by American Concrete Pavement Association (ACPA). Concrete pavement design method used in StreetPave is mainly based on PCA method which was initially released in 1933 and updated in 1951, 1966 and 1984 (Oman and Grothaus, 2012; PCA, 1984). It should be noted here that, by using the software, asphalt pavement design and cost-benefit analysis can also be done, but they are not covered in the content of the thesis.

PCA (1984) is a mechanical and experimental based methodology. Deflection, strain and stress computations of the method generate the mechanical part, and usage of outcomes of the AASHO Road Tests generate the empirical part of the methodology. It should be noted that StreetPave software was created on the basis of the PCA (1984) design procedure; however, a number of additions were made to the PCA (1984) approach during the development of the software. The most important of these additions are the user-defined reliability (%), the percent of slabs cracked at the end of design life, and the contribution of fiber reinforcements to the flexural capacity of the concrete (Oman and Grothaus, 2012; Roesler et al., 2008).

StreetPave input interface consist of several parameters governed by the designer, including site-related and design-related variables. Traffic and subgrade parameters can be considered as site – related variables which can be listed as follows: traffic category, Average Daily Truck Traffic (ADTT), design lane and directional distribution factors, growth rate of traffic and modulus of subgrade reaction. On the other hand, design life, pavement material (concrete) properties (flexural strength, elastic modulus), percent of cracked slabs, reliability, load transfer mechanism at joints (doweled/undoweled) and inclusion of edge support can be considered as design-related variables. Decisions on such variables (design – related ones) have a major impact on pavement performance, constructability, long-term maintenance and rehabilitation requirements, initial and long-term costs, and many other associated issues (Oman and Grothaus, 2012; Roesler et al., 2008).

On the other hand, the sensitivity analysis module allows a user to analyze the results with respect to design life, k-value, reliability, concrete strength, and percent of slabs cracked at the end of design life. Besides, with the help of the ACPA procedure, the thickness design may be altered, taking into consideration the added structural advantage of the fibers (Oman and Grothaus, 2012; Purvis, 2013; Wimsatt et al., 2009).

PCA method (1984) considers two failure modes for design of concrete pavements, as fatigue failure of slab (due to repeated stress application of wheels) and erosion failure of foundation (due to repeated deflection of slab). According to PCA (1984) method required slab thickness is determined by considering safety against these two failure types (fatigue and erosion) (Oman and Grothaus, 2012; PCA, 1984). Details of the design approach followed in the methodology summarized below, in terms of fatigue and erosion analysis.

2.8.3.1. Fatigue Analysis. Fatigue is generally defined as the deterioration of the strength of a material due to repeated applications of load that is generally below the strength of the material. As a result of the repeated loads (much lower than the strength), progressive and permanent internal changes taking place in the material can lead to microcracks propagation.

Eventually, failure will occur because of the changes in the mechanical properties of the material at the macro level caused by repeated load applications (Ameen & Szymanski, 2006). The data obtained from beam fatigue tests at the beginning of 1950s and 1960s form the basis of the PCA fatigue model is employed in the ACPA's pavement design software, StreetPave. The aim of PCA's fatigue analysis is to prevent the first initiation of crack resulting from fatigue of concrete due to critical stress repetitions (PCA, 1984).

Titus-Glover et al. (2005) reported that the fatigue and erosion analysis procedures of PCA based on Miner's damage model (1945) which is the most common linear damage theory proposed for high-cyclic fatigue. Lee and Barr (2004) stated that for the pavement applications fatigue life is usually referred to high-cycle fatigue; between 10^5 and 10^6 loading cycles.

Fatigue analysis (controls fatigue cracking) mainly based on fatigue model that determines the allowable number of repetitions for each respective axle load group for a given stress ratio that can be calculated as in Equation 2.37.

$$Stress\ Ratio\ (SR) = \frac{\sigma_{eq}}{MOR} \tag{2.37}$$

In Equation 2.37, σ_{eq} is the flexural stress under traffic loading and MOR is the 28-day modulus of rupture (flexural strength) of the PCC (Portland Cement Concrete) (Papagiannakis and Masad, 2008; PCA, 1984; Titus-glover et al., 2005).

The pavement may endure more load cycles before failure when the stress ratio (actual flexural stress divided by the modulus of rupture) decreases. PCA reports that the concrete pavement can be subjected to infinite repetitions of the load if the stress ratio is less than 0,45 (ACI Committee 360, 2006).

Subsequently, the PCA procedure determines the maximum allowable load repetitions (N) based on the following relationship (Packard and Tayabji, 1985).

$$SR > 0{,}55, N = 10^{12{,}1*(0{,}972-S\ \ )} \tag{2.38}$$

$$0,45 < SR \leq 0,55, N = \left(\frac{4,258}{SR-0,4325}\right)^{3,268} \tag{2.39}$$

$$SR \leq 0,45, N = Unlimited \tag{2.40}$$

where

N: Allowable number of load repetitions to failure, and

SR: Maximum stress ratio.

In the PCA method, however, reliability is not considered as direct user input and based on models at a predetermined reliability level, which is around 90% or more (denote a high degree of reliability). Using such a high degree of reliability results in an excessive thickness in the design for low-volume and low-traffic roads or street pavements since it is too conservative (Titus-Glover et al., 2005).

Therefore, ACPA initiated research to expand and improve the existing PCA fatigue model through including reliability as a direct input for estimating fatigue damage of PCC. As a result of the research of Roesler et al. (2008), an enhanced fatigue model for StreetPave was introduced and employed in the software instead of PCA's fatigue equations (Roesler et al., 2008; Titus-Glover et al., 2005).

In order to calculate the percentage of fatigue damage for each regarding load levels and axle types that will occur during the design life, the PCA method which based on the Miner's cumulative fatigue damage assumption uses the expected number of load repetitions dividing by allowable load repetitions of that load level (ACI Committee 330, 2008; PCA, 1984; Titus-glover et al., 2005). Miner's rule is represented as Equation 2.41.

$$0 \leq \sum_{i=1}^{m}\left(\frac{n_i}{N_{if}}\right) = Fatigue\ damage\ \leq 1 \tag{2.41}$$

where

N_f: The max. number of allowable load repetitions to failure,

n_i: Expected number of load repetitions, and

m: Load sequence number before final failure.

When fatigue damage rate is less than 1, as with all cumulative damage forms, it still determines the level of damage, but does not indicate failure, however the damage fraction exceeds 1 failure occurs then the thickness of the pavement is regarded as insufficient (Christensen, 2008). Expected load repetitions, n, is estimated through past traffic predictions and the numerical forecasting techniques, while allowable load repetitions, N_f, is estimated relying on the ratio of equivalent stress resulting from traffic loading to PCC flexural strength (PCA, 1984; Titus-Glover et al., 2005).

The basis of the pavement design procedure is a comprehensive analysis of concrete stresses and deflections at pavement joints, corners and edges. Through fatigue analysis of PCA (1984) the maximum principle stresses occur when the wheel loads applied near the edge of the pavement and midway between the transverse joints. Although this is the most critical loading condition in terms of flexure, the severity of the critical stresses is significantly decreased if a concrete shoulder is tied on to the mainline pavement. PCA (1984) reported that with regards to fatigue life load configurations of tandem, tridem or quad (multiple) axles are less damaging than single axle configurations. Figure 2.18 shows the critical loading position for fatigue analysis.

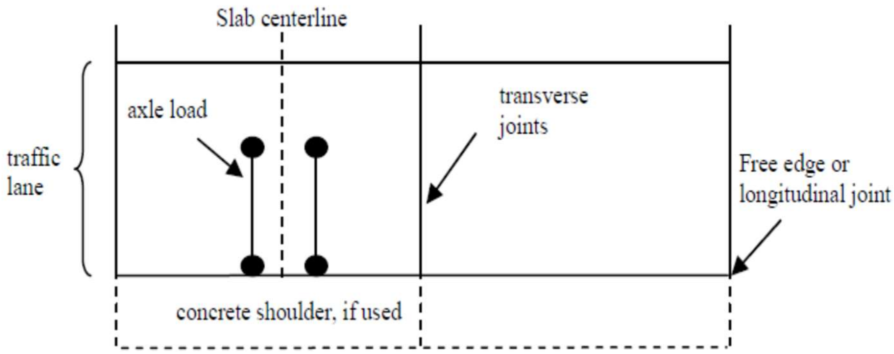


Figure 2.18. Most critical loading position for fatigue failure (Huang, 2004).

Analysis of fatigue assumes that about 6% of all truck loads pass close enough to the pavement edge to cause critical tensile stress. However, it is necessary to consider the existence of a tied concrete shoulder since it decreases critical edge stress dramatically. Concrete slabs are also exposed to warping and curling stresses besides traffic loading. The upward concave deformation of the slab because of differences in moisture content with slab depth is named as warping. Due to the fact that warping is a long-term phenomenon the resulting effect is influenced by the creep substantially. Curling means the slab behavior derived from temperature changes. Tensile restraint stresses occur at the bottom of the slab during the day when the upper surface is warmer than the bottom. The temperature distribution is reversed during the night and tensile restraint stresses occur at the surface of the slab. The distribution of temperature is usually not linear and varies constantly. There are also maximum daytime and nighttime temperature differentials for short periods. The combined effect of curling and warping stresses is generally subtractive from load stresses because the moisture content and temperature at the bottom of the slab exceed that at the top more than at the reverse. Therefore, PCA design process does not take into account stresses related to curling or warping (PCA, 1984). It should be noted that typically the design of light duty traffic pavements (irrespective of whether the joints are doweled) and the medium traffic pavements with doweled joints are governed by the fatigue analysis (Roesler et al., 2008).

2.8.3.2. Erosion Analysis. It is well-known by the pavement designers that a sub-base or base should be used under rigid pavements to avoid significant degradation and pumping of subgrade caused by frequent heavy truck traffic. Although concrete pavement systems are capable of providing a long service life, inappropriate design or mismanagement of the subgrade layer will result in a significant reduction in service life. The sub-base layer fulfills many vital roles such as fixed construction platform, uniform and consistent support, erosion resistance, drainage, increasing slab support in a concrete pavement system. As well as the fatigue cracks, it is therefore clear that the loss of foundation support arising from erosion, and the pumping of the subbase materials from under the pavement is a critical mode of distress that needs to be considered in the design stage. The erosion of the subbase is a foundation for understanding the process of major issues such as the faulting and punching of concrete pavement (Jung, 2010). StreetPave carries out pavement erosion anaysis using the empirical methods based on the field performance data obtained from sites in Wisconsin, Minnesota, North Dakota, Georgia, and California (Oman and Grothaus, 2012).

It is clear that, PCA and also StreetPave needs an erosion analysis to control some of the pavement distress modes that are not related to fatigue, such as erosion of material beneath the slab (the creation of voids beneath the pavement corners), joint faulting and pumping. The main purpose of the erosion analysis is to limit the pavement deflection effects at the slab corners, edges and joints thereby control the erosion of foundation and shoulder material (Mallick and Tahar, 2018; PCA, 1984). Erosion is caused by factors like the water existence, the rate at which water is discharged from the bottom, foundation layer erodibility, magnitude and number of the repeated loads, and the level of deflection. PCA has created an erosion model to limit the possibility of failure due to such reasons (Huang, 2004).

Erosion analysis is considered based on the pavement deflections produced by axle load located at the joint near to the corner that can be seen in Figure 2.19.

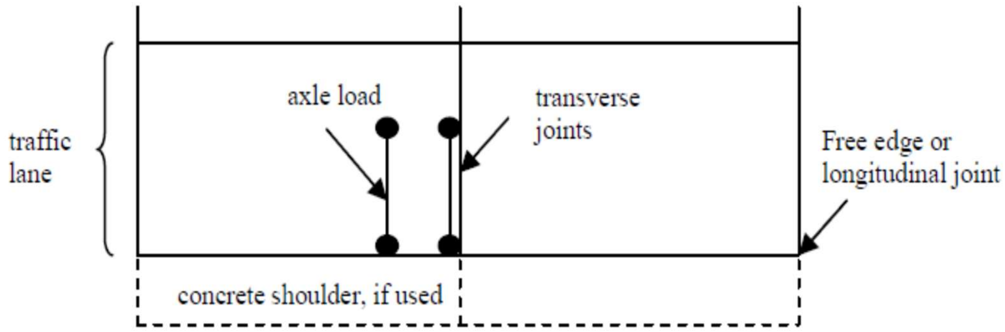


Figure 2.19. Most critical loading position for erosion failure (Huang, 2004).

Erosion analysis of PCA is affected by the existence of the dowel bars on the joints and the presence of concrete shoulder support since the use of such supports allows more efficient transfer of stresses caused by repeated loads, thereby minimizing the problems like foundation erosion and degradation of the joints. Besides, several important advantages can be obtained by the use of a lean concrete subbase that reduces deflections and stresses of pavement, like providing significant support for trucks passing through the joints and providing resistance to sub-base erosion due to repetitive pavement deflections (Lee and Carpenter, 2001; Papagiannakis and Masad, 2008; PCA, 1984). PCA reported that generally the design of medium and high-duty traffic pavements without doweled joints (loads transfer via aggregate-interlock) and the heavy-duty pavements with doweled joints are governed by the erosion analysis (Parjoko, 2012; PCA, 1984).

On the other hand, during the AASHO (American Association of State Highway Officials) Road Test pumping and degradation (erosion) of the granular subbase beneath concrete shoulder was the fundamental mode of failure of concrete pavements. Since satisfactory correlations could not be obtained among the corner deflections and the performance of the AASHO Road Test pavement sections, it was found that deflection alone was a poor predictor of pavement life. Therefore, erosion damage is associated with the concept of power or rate of work in the PCA model. The concept is that for a unit area a thinner pavement with its shorter deflection basin received a faster punch than a thicker slab did. (the more concentrated the load, the faster the load is applied and released as a wheel rolls over the pavement) (Huang, 2004; Lee and Carpenter, 2001; Parjoko, 2012; PCA, 1984).

The rate of work or power equation is represented below:

$$Power = 268{,}7 * \left(\frac{k^{1{,}27} * \delta_{eq}^2}{h}\right) \tag{2.42}$$

where
k: Modulus of subgrade reaction,
δ_{eq}: Corner deflection, and
h: Pavement thickness.

To determine the allowable number of load repetitions for the analysis of erosion the equation below is recommended.

$$\log N_e = 14{,}524 - 6{,}777 * (C_1 * P - 9)^{0{,}103} \tag{2.43}$$

In Equation 2.33, N_e is the allowable load repetitions for erosion damage, $C_1$is an adjustment factor (it is assumed for stabilized subbase 0,9 and 1 for untreated subbase) and P is the power (rate of work) (Huang, 2004; PCA, 1984).

Cumulative erosion damage should be found after the allowable load repetitions for erosion damage are found. The PCA design method uses the Miner's cumulative damage concept, which is also used for calculating fatigue damage to calculate erosion damage (PCA, 1984). It should be noted that the truck wheel loads which positioned on the outer edge of the pavement generate severe cases than any other positions of load. The effects reduce significantly as the truck placement moves a few inches inward from the edge. So, according to PCA design procedure, the most critical situation occurs when 6% of trucks are on the edge (PCA, 1984).

Before determining cumulative erosion damage in erosion analysis, it is necessary to know if there is a concrete shoulder. If the concrete shoulder is not available, corner loads generated by 6% of trucks riding right against the edge are critical and erosion damage is calculated as follows.

$$0 \leq D_{erosion} = \sum_{i=1}^{m} \left(\frac{0{,}06 * n_i}{N_{ei}} \right) \leq 1 \tag{2.44}$$

In the case of existing a concrete shoulder, the corner deflection is not significantly affected by the placement of the wheel loads, so the corner loads produced by 94% of trucks that don’t encroach on the shoulder are critical (PCA, 1984).

$$0 \leq D_{erosion} = \sum_{i=1}^{m} \left(\frac{0{,}94 * n_i}{N_{ei}} \right) \leq 1 \tag{2.45}$$

where
N_{ei}: Allowable number of repetitions for erosion at i^{th} load group,
n_i: Predicted number of repetitions for erosion at i^{th} load group, and
m: Total number of loads.

Briefly, it can be inferred that corner loads (6% of trucks) are critical in the absence of concrete shoulder; and where there is a concrete shoulder, the greater number of loadings inward from the pavement corner (94% of trucks) are critical. It should be noted that because the deflections are much smaller when the pavement is supported by a shoulder, the N_i values will not be the same as those calculated when there is no shoulder support. The cumulative fatigue damage steps described previously are followed to determine the cumulative erosion damage and if the erosion damage ratio exceeds 1, the thickness of the pavement is considered to be insufficient (Huang, 2004; PCA, 1984).

3. METHODOLOGY

In the scope of this study, the required thickness values for plain and fiber reinforced concretes with different properties were evaluated by a comparative study. As mentioned in the introduction part, AASHTO (1993), IRC SP 46 (2013), and StreetPave v12 (2014) (based on PCA (1984)) methodologies were used for the thickness design. To understand better the differences in outcomes between each design approach, the basis of the equations must be understood, how and when different material, environmental, structural properties affect the performance of the pavement. Therefore, a comprehensive review of design methodologies was conducted in the literature and the differences in the design input parameters for each methodology were evaluated.

The thickness of the slab could be decreased by increasing the flexural strength of the concrete for example reducing the water to cement ratio or increasing the cement content. But this would make the system more brittle and there would be a higher cracking probability compared to the fiber-reinforced concrete system. Also, it should be noted that the amount of contribution to be obtained using fibers can vary significantly according to fiber type, fiber amount, and concrete matrix properties. Therefore, thickness design was carried out using different types of fibers (synthetic, steel and carbon) and different types of concretes (RCC and conventional PCC) to observe the effects of these parameters on both plain and fiber reinforced concrete. The material parameters for plain and fiber reinforced concrete used in RCC pavements were taken from the two previous studies found in the literature carried out by Öztürk (2018) and LaHucik et al., (2017); in conventional PCC pavements, these parameters were taken from the previous study found in the literature carried out by Mulheron, (2015). Also, traffic and foundation parameters obtained from IRC SP 46 (2013) were used for all design methodologies. Therefore, all differences between thicknesses would be a function of the design methodologies and material properties. The assumptions made in the design approaches were explained later in the study.

One of the design procedures used in this study, AASHTO (1993) does not consider fibers in the design process, but the positive effects of the fibers can be considered by the approach described in detail below. StreetPave and IRC SP 46 (2013) use different approaches when considering fibers. In the fiber reinforced concrete pavement thickness design, it should be noted that the parameters used to calculate the contribution of fibers to the flexural performance of the concrete vary according to the used approach. So, how the design methodologies used in this study consider the effects of fibers are explained in detail below. The accepted methodology for the design can be summarized as follows.

1. Using the traffic, foundation and road parameters obtained from IRC SP 46 (2013), traffic data, which is considered as a different way for each methodology and which affects the pavement thickness in the design process, is converted to the desired format described in detail in the literature review (18-kip ESAL for AASHTO (1993), 6-hour axle load repetition for daytime and night time for IRC 58 (2011) , axle/1000 trucks for StreetPave v12 (2014)).
2. The material parameters for plain and fiber reinforced concrete used in RCC pavements were taken from the two previous studies found in the literature carried out by Öztürk (2018) and LaHucik et al., (2017); in conventional PCC pavements, these parameters were taken from the previous study found in the literature carried out by Mulheron, (2015).
3. The pavement thickness was calculated according to each methodology for both plain and fiber reinforced concrete using these material and traffic parameters.
4. Finally, the results were compared in terms of performance of the fibers, shortcomings of methodologies, and performance of the concrete matrix.

During the design process, parameters related with foundation, road and traffic were kept constant to observe the effects of different type and amount of fibers and different type of concretes. Furthermore, the tables used to determine the thickness requirements according to the three design methodologies were given in Appendix A for a sample concrete mixture.

3.1. Fiber-Reinforced Concrete Pavement Design Approaches

Studies from the literature show that fiber reinforcement in concrete, will improve the flexural strength, flexural toughness and fatigue resistance of concrete pavement. Thus, fiber addition can decrease the thickness requirement of concrete pavement depending on the fiber type, fiber amount and concrete matrix properties. In thickness design procedures, the contribution of fibers is considered in several ways. These approaches are summarized below. It should be noted here that, although AASHTO (1993) doesn't consider the fibers in the design of concrete pavement, the contribution of the fibers was evaluated with the approach of Altoubat et al. (2008).

- Altoubat et al. (2008) approach;

Altoubat et al. (2008) have stated that the most important concrete pavement material parameter used in the design of concrete pavements is the flexural strength (MOR) value. However, MOR alone cannot explain the contribution of fibers to flexural behavior of the concrete since the toughness benefit of fibers come up with the post cracking behavior. Thus, increased flexural strength (effective flexural strength) (MOR') is suggested as design flexural strength to consider the fiber contribution, as given in Equation 3.1. According to this approach, the flexural strength (MOR) of the concrete is increased by the equivalent flexural strength ratio ($R_{e,3}$) value which represents the equivalent flexural strength ratio. Altoubat et al. (2008) stated that this approach could be used in the Portland Cement Association's design method for airports (1973) and AASHTO (1993) design method. It should be noted that this approach is recommended for concrete pavement with a low volume fraction of fibers (below 0.5% by volume) (providing equivalent flexural strength ratio between 20 % and 50%).

$$MOR' = MOR * (1 + R_{e,3}) \quad (3.1)$$

In the equation above, $R_{e,3}$ is equivalent flexural strength ratio determined from the results of third point bending test based on ASTM C1609 and calculated with the help of Equation 3.2. The graphic of the equivalent flexural strength value can be seen in Figure 3.1.

$$R_{e,3} = R_{T,150}^{D} = \frac{L * T_{150}^{D}}{3 * f_1 * b * d^2} \quad (3.2)$$

where

L: Span length (mm),

b: Width of the specimen (mm),

d: Depth of the specimen (mm),

f_1: Flexural strength (N/mm^2), and

T_{150}^{D}: Flexural toughness of the specimen (Nmm) (calculated from load-midspan deflection curve up to a net deflection of L/150).

- IRC SP 46 (2013) approach;

Design methodology of IRC SP 46 (2013) is mainly based on IRC 58 (2011) (used to design plain concrete pavements). However, IRC SP 46 (2013) modified the limiting stress ratio values given in IRC 58 (2011) to consider the positive effect of fibers. According to the IRC 58 (2011) and IRC SP 46 (2013), the stress ratio values (for plain and fiber reinforced concrete) in which the fatigue life of the pavement can be taken as infinity are stated below. As can be seen below, fatigue strength is increased according to the toughness contribution of fiber to the behavior of concrete. Also, if the stress ratio is above the stated limit values, calculation of the allowable number of repetitions were given in IRC 58 (2011) and IRC SP 46 (2013).

- Limiting stress ratio for plain concrete pavement is 0.45 (IRC 58, 2015).
- Pavements with fiber reinforced concrete (FRC) of low toughness (f_{e150k}<0.3 f_{ctk}) shall be designed as non-reinforced pavements. In this case, fibers are considered to mainly contribute in controlling plastic shrinkage and temperature induced cracks (IRC SP 46, 2013).
- If polymeric fibers are used in dose less than 0.3 % by volume, for fatigue endurance, the limiting stress ratio should be taken as 0.45 (as in IRC 58 (2015)).
- If polymeric fibers are used in dose more than 0.3 % by volume (mainly to control plastic shrinkage cracks), or in FRC with steel or any other fibers having low toughness (fe150k<0.3 fctk), for fatigue endurance, the limiting stress ratio should be taken as 0.50 (IRC SP 46, 2013).
- Pavements with FRC having high toughness shall be designed as per the procedure given in IRC 58. For fatigue endurance, the limiting stress ratio will be taken as 0.6 (in place of 0.45 in IRC 58 (2015)) (IRC SP 46, 2013).

here

f_{e150k}: the characteristic equivalent flexural strength, and

f_{ctk}: characteristic flexural strength of concrete

IRC SP 46 (2013) guideline considers the contribution of fibers to the fatigue life of the concrete pavement as described above. However, although erosion damage is as important as fatigue damage in concrete pavement design, erosion is not considered in the current design standard. Therefore, there is a need to develop a design approach in order to consider erosion damage and the contribution of fibers to erosion resistance.

- StreetPave v12 (2014) approach;

In StreetPave v12 (2014), the contributions of fibers to the thickness requirement is taken into consideration in a similar way with the Altoubat et al. (2008). The only difference between these two approaches is the parameters that they use to increase the flexural strength. Instead of the equivalent flexural strength ratio value ($R_{e,3}$) used in Altoubat et al. (2008), the StreetPave uses the residual flexural strength ratio value. The residual flexural strength ratio specified herein is determined by the ratio of the residual flexural strength value which is obtained for 3 mm displacement as a result of a third point bending test in accordance with ASTM C1609 to the flexural strength value. The increased flexural strength is calculated by Equation 3.3

$$MOR' = MOR * (1 + residual\ flexural\ strength\ \ ratio) \qquad (3.3)$$

As stated previously, in PCA (1984) thickness design procedure, thickness is determined according to the critical thickness case by doing calculations respectively for the fatigue and erosion damage. In StreetPave v12 (2014), the increased flexural strength value (MOR') is merely used to calculate the fatigue damage of the slab and the contributions of fiber reinforcements are not taken into consideration in any way while calculating the erosion damage. However, the fibers in joint areas improves the fracture control and the micro-dowel action increases the load transfer capacity. So, it can minimize the amount of potential damage beneath the slab. Thus, a design approach needs to be developed in order to consider the contribution of fibers to the erosion resistance.

Equivalent flexural strength and residual flexural strength values in Altoubat et al. (2008) and StreetPave v12 (2014) are shown on the graph in Figure 3.1.

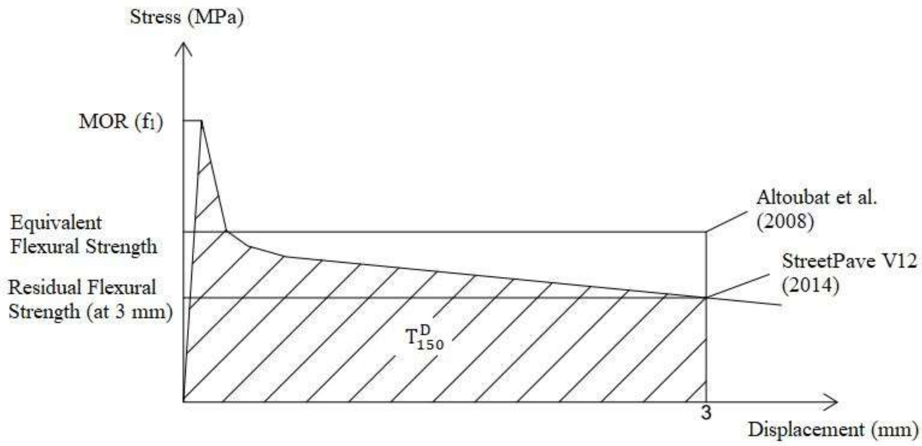


Figure 3.1. Equivalent flexural strength and residual flexural strength (ASTM C1609).

Table 3.1 shows the type of analysis considered in the methodologies. It also shows the standards which considers the effects of fibers in design.

Table 3.1. Summary of the design methodologies in terms of analysis type.

				Contribution of fibers	
	Fatigue	Erosion	Loss of Serviceability (Fatigue &	Fatigue	Erosion
AASHTO (1993)	yes	yes	yes	no	no
PCA (1984)	yes	yes	no	no	no
StreetPave v12 (2004)	yes	yes	no	yes	no
IRC 46 SP (2013)	yes	no	no	yes	no

As can be seen from the Table 3.1. all standards have different approaches and limitations which result in different results in thickness.

3.2. Flow Chart of Rigid Pavement Thickness Design

In the scope of the study, the pavement thicknesses were found according to 3 different design methodologies. Since each methodology has its own specific parameters and design procedure, it is important to present the procedures in graphical terms for understanding the designs. The flowchart of the concrete pavement thickness design using the AASHTO (1993), PCA 1984 (which is the basis of StreetPave), and IRC SP 46 (2013) guidelines were given below from Figure 3.2 to 3.4.

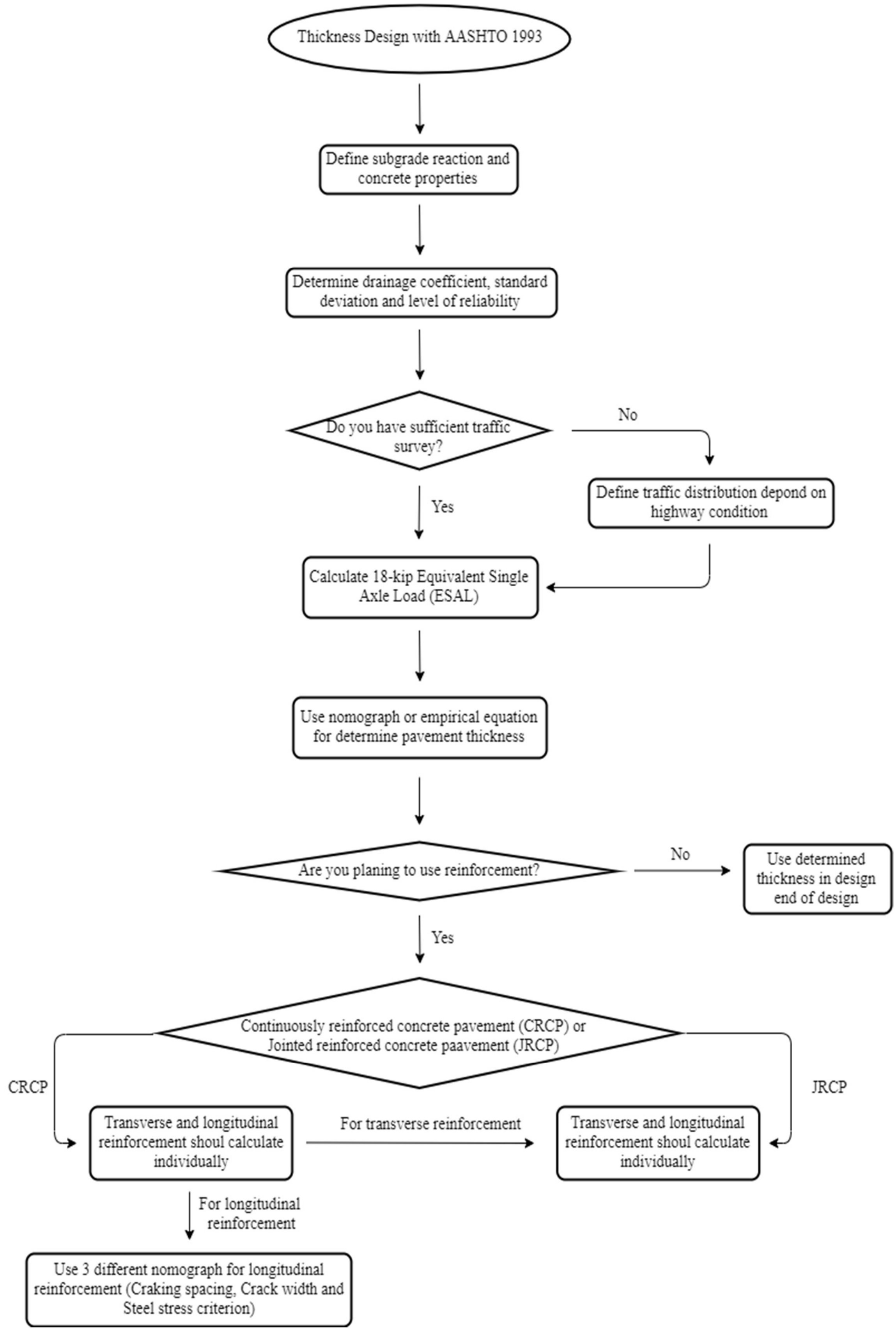


Figure 3.2. AASHTO (1993) flow chart (Kıcı and Tigdemir, 2017).

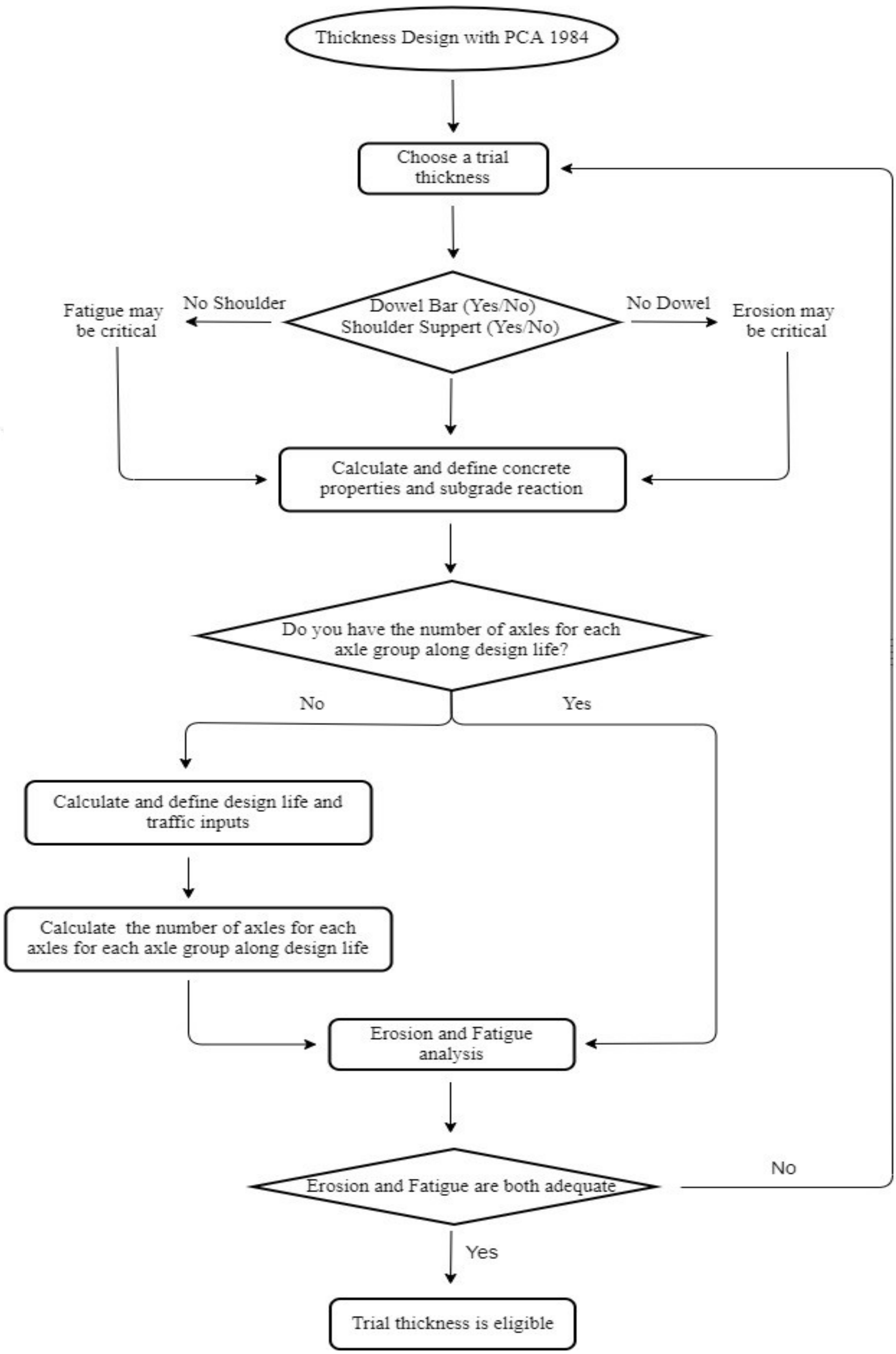


Figure 3.3. PCA (1984) flow chart (Kıcı and Tigdemir, 2017).

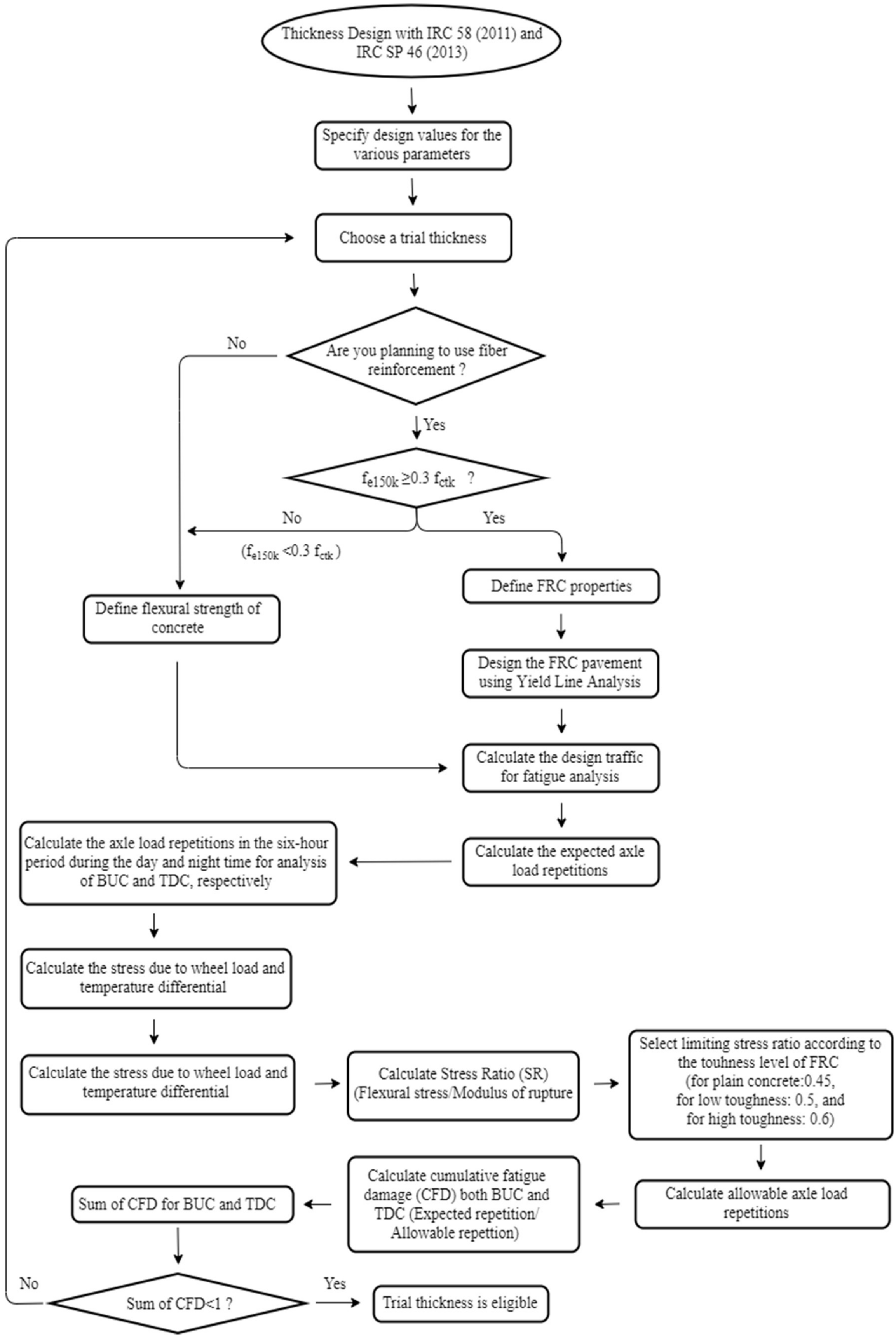


Figure 3.4. IRC 58 (2011) and IRC SP 46 (2013) flow charts.

3.3. Comparison of Design Methodologies

The purpose of this section is to evaluate the differences and similarities of rigid pavement design in terms of the Indian and American approaches. In general, with the number and complexity of the distress modes increasing, additional inputs are required to provide all necessary variables. Although some of the parameters used in these methodologies are the same, most of them are different from each other. This makes it difficult to compare methodologies directly.

- Distress modes and Design methods

Each thickness design methodology follows either empirical or mechanistic-empirical design methods to predict the pavement performance. There is one or more performance criteria used to define the end of the performance life of the pavement (Selezneva et al., 2004). Since AASHTO (1993) uses a slab thickness design equation based on performance data from the AASHO Road Test, the design procedure is strictly empirical. AASHTO (1993) evaluates the performance in terms of loss of serviceability caused by 18-kip single axle load applications. Also, the type of hazard is not distinguished as individual distresses like fatigue or erosion (AASHTO, 1993). However, StreetPave v12 (2014) (also PCA 1984), evaluates the performance with the cumulative fatigue (considering only bottom-up cracking) and erosion damage analysis based on Miner's hypothesis. While StreetPave evaluates fatigue through a mechanistic-empirical design, erosion model is based mainly on empirical relationships obtained from field studies. Thus, StreetPave is both mechanistic and empirical. StreetPave also considers the terminal serviceability and percentage of cracked slabs as failure criteria which should be selected based on policy and experience. In fact, terminal serviceability value does not change the thickness requirement for StreetPave design. It is only used for the calculation of Rigid ESALs by AASHTO (1993) method, which is another output of the software (Oman and Grothaus, 2012). IRC SP 46 (2013) is mainly mechanistic-empirical design focusing on slab stress predictions caused by both wheel load and temperature differential. Also, IRC SP 46 (2013) evaluates the performance through the cumulative fatigue damage analysis (considering bottom-up and top-down cracking) based on Miner's hypothesis (IRC SP 46, 2013).

- Traffic

The number of vehicles and axle weights that will pass through the lifetime of the pavement is very important in terms of design. When compared to other vehicles, the effect of heavy vehicle factor is considerably high on the design of pavement thickness. For this reason, for the proper pavement thickness design, the heavy vehicle factor is very important (Huang, 2004). Therefore, in the design methodologies considered in this study, only truck loads are considered for traffic loads.

Traffic is considered in different ways throughout these design procedures. In AASHTO (1993), the mixed traffic stream is converted to the 18-kip ESAL by using the load equivalency factors to estimate the relative damage (AASHTO, 1993). However, for StreetPave, the axle load spectrum which requested in the form of axles/1000 trucks is considered (Oman and Grothaus, 2012). IRC SP 46 (2013) also uses the axle load spectrum which consists of axle load classes and frequency data (with respect to axle load class and axle type) (IRC SP 46, 2013).

- Reliability

In design process, the reliability concept is used to reduce the risk of premature structural deterioration. IRC SP 46 (2013) does not employ any reliability approach to estimate the pavement performance. Although AASHTO (1993) and StreetPave consider reliability as a parameter in design, by which method they take it into account varies greatly. The reliability used in AASHTO (1993) effectively scales the number of ESAL and is in the form of $Z_R * S_0$ in design equation. Z_R represents the normal deviate for a given reliability and S_0 is the standard deviation in the design equation. Also, loss of support (LS) is an additional safety factor applied to the design to reduce the k-value (AASHTO, 1993). On the other hand, StreetPave applies the reliability to the fatigue equation to calculate allowable repetitions. And, reliability used in StreetPave effectively shifts the fatigue curve to be more or less conservative (Oman and Grothaus, 2012; Rodden et al., 2012).

- Temperature and Moisture

A concrete pavement is subjected to curling (caused by temperature changes) and warping (caused by differences in moisture content) stresses as well as stresses caused by wheel loads during its design life (Huang, 2004). The warping stresses are ignored by considered methodologies in this study. The curling stresses are considered in the IRC SP

46 (2013) method however, there is no check for such stresses in the StreetPave (also PCA 1984) and ASSHTO (1993) methodologies. IRC SP 46 (2013) considers the most important temperature stresses since India has an extreme type of climate. The curves corresponding to the stress computation in the IRC SP 46 (2013) method is based on temperature as well as load stresses for a particular modulus of subgrade reaction.

- Environmental effects

Since the AASHO Road Test design equations are based on traffic test results over a two-year period, the effects of long-term temperature and moisture on loss of serviceability were not considered. Environmental effects were considered in three ways in the design method: drainage coefficients, serviceability loss due to environmental conditions, and estimation of an effective subgrade resilient modulus that reflects seasonal variations. StreetPave does not consider climate. However, StreetPave supposes that thanks to its design features such as free draining subbase layer as necessary, with or without drainage is somehow accounted for in a way that does not have a negative impact on pavement performance (Rodden et al., 2012). IRC SP 46 (2013) considers only the temperature difference that occurs at the top and bottom of the concrete slab as environmental impact.

- Edge support and dowel bar considerations

In the design, whether the concrete shoulder support is present, or the joints are doweled affects the pavement thickness. Such supports are often included in the design to prevent deterioration resulting from erosion and methodologies take into account their existence in different ways (Huang, 2004). In AASHTO (1993), the presence of load transfer devices such as dowel bars and the tied shoulder is only considered when choosing the load transfer coefficient, J. The experience of the designer has an important role in the determination of J. The pavement thickness and performance are affected directly by the value of J. StreetPave considers the presence of tied concrete shoulder in the design and offers a thickness value for two cases at the end of the design, with or without dowel at joints (Oman and Grothaus, 2012). IRC SP 46 (2013) considers the presence of tied concrete shoulder in the stress equations varying according to axle type and modulus of subgrade reaction. Also, IRC SP 46 (2013) recommends the use of load transfer devices such as dowels. It states that the use of dowel bars doesn't reliable for pavement thickness less than 200 mm (IRC SP 46, 2013).

- Additional inputs

AASHTO (1993) rigid pavement design contains different parameters as compared to IRC SP 46 (2013) and StreetPave such as load transfer coefficient (J) mentioned above, and drainage coefficient (C_d). The purpose of the drainage coefficient is to increase the thickness of the pavement required to make up for the poor drainage. Although drainage is an important factor in pavement performance especially in terms of erosion, it has been overlooked in current design methodologies except ASSHTO (1993).

- Concrete properties and modulus of subgrade reaction

On the other hand, there are some parameters evaluated in the same manner in the design process such as modulus of rupture of concrete, modulus of subgrade reaction and modulus of elasticity of concrete. It is necessary to use the 28-day modulus of rupture determined from four point bending test as the design strength. The methodologies considered in this study allow the designer to consider all the layers to be placed under the concrete slab in the rigid pavement design by using the effective modulus of subgrade reaction. Also, each methodology allows conversion from CBR or R-value to obtain modulus of subgrade reaction. In addition, when determining subgrade reaction, AASHTO (1993) considers the effect of loss of support of underlying materials due to erosion or degradation with the Loss of Support (LS) factor.

- Contribution of fibers

While IRC SP 46 (2013) and StreetPave allows considering the contribution of fibers in design, AASHTO (1993) does not have provisions for the use of fibers. It should be noted that, in fiber- reinforced concrete pavement thickness design, the parameters used to calculate the contribution of fibers to the flexural performance of the concrete vary according to the used approach. The approaches considered in the design of the fiber-reinforced concrete pavement in this study were explained in detailed previously.

The design parameters considered by each design methodology are compared in Table 3.2. As noted in Table 3.2, none of the models consider material-related distress; as structural thickness models, it is assumed that all materials used in the construction of the pavement will be of a quality such that the pavement will fail from a distress mode modeled in the design approach before it fails from material-related distress (Rodden et al., 2012).

Table 3.2. Comparison of design variables (Rodden et al., 2012).

		AASHTO (1993)	IRC SP 46 (2013)	StreetPave v12 (2014)
Subgrade	California Bearing Ratio (CBR)	X	X	X
	Resistance Value (R-Value)	X	X	X
	Resilient Modulus of the Subgrade (M_{RGB})	X	X	X
	Thickness of Treatment	X	X	X
Subbase	Layer Modulus (Elastic or Resilient)	X	X	X
	Layer Thickness	X	X	X
	Drainage	X		
Temperature Differential			X	
Composite Support Value (k-value)		Static	Static	Static
Concrete	Flexural Strength	X	X	X
	Modulus of Elasticity	X	X	X
	Fibers		X	X
	Fiber contribution to distresses types		Fatigue	Fatigue
Structure	Doweled or Undoweled	X	X	X
	Widened Slab		X	X
	Edge Support	X	X	X
Traffic	Design Life	X	X	X
	Applied Load	X	X	X
	Traffic Spectrum		X	X
	18-kip ESAL	X		
	Load Configuration	X	X	X
	Trucks/Day	X	X	X
	Growth Rate	X	X	X
	Directional Distribution	X	X	X
	Design Lane Distribution	X	X	X
Safety Factors	Reliability	X		X
	Overall Standard Deviation	X		
	Standard Normal Deviate	X		
	Loss of Support	X		
Percentage of slabs cracked at the end of design life				X
Distress Modes	Fatigue		X	X
	Erosion			X
	Serviceability Loss	X		
	Material-Related Distresses			

4. RESULTS AND DISCUSSIONS

This chapter presents the study on parameters or variables used by AASHTO (1993), IRC SP 46 (2013) and StreetPave v12 (2014) to complete the design of pavement thickness. The thickness design results of both plain and fiber reinforced RCC and conventional PCC pavements are examined under the following three headings.

1. The results of thickness design for RCC and conventional PCC pavements were first evaluated in terms of plain and fiber-reinforced within each methodology.
2. The thickness values obtained for the RCC and conventional PCC pavements were then evaluated separately for design methodologies to observe the causes of thickness similarities or differences according to methodologies. In addition, the performance of the fibers in the concrete type with different consistency (conventional PCC and RCC) was examined.
3. Finally, the effects of the presence of edge support and load transfer devices such as dowel bars on the thicknesses calculated for RCC and conventional PCC were evaluated in terms of design methodologies. It was also examined which failure mode governed the design.

4.1. Thickness Design

As mentioned in the introduction part, thickness designs performed for the study were done by using AASHTO (1993), IRC SP 46 (2013) and StreetPave v12 (2014) methodologies. These design methodologies described in detail in the literature review can be summarized as follows.

As stated in the literature review, AASHTO (1993) is based on the concept of loss of serviceability which includes distresses such as fatigue and erosion. The empirical formula obtained from AASHO Road Test was used in the thickness design. Therefore, in the design for AASHTO, using the empirical equation, a single thickness value including various failure modes such as erosion and fatigue was obtained. It should be noted here that, although AASHTO (1993) does not consider the fibers in the design of concrete pavement, the contribution of the fibers was evaluated with the approach of Altoubat et al. (2008). According to this approach, the flexural strength (MOR) of the concrete is increased by the equivalent flexural strength ratio ($R_{e,3}$) value which represents the equivalent flexural strength ratio.

The design methodology of IRC SP 46 (2013) is mainly based on IRC 58 (2011) (used to design plain concrete pavements). However, IRC SP (46) 2013 modified the limiting stress ratio values given in IRC 58 (2011) to consider the positive effect of fibers. IRC SP 46 (2013) guideline proposes a two-stage design, moment capacity check based on TR 34-third edition (2003) and fatigue damage control based on IRC 58 (2011). First, fatigue damage control was carried out for both plain and fiber reinforced concrete by considering the use of appropriate stress ratios which were explained in the literature. Then the ultimate moment capacity check was made only for high-toughness FRCs.

StreetPave v12 (2014) software was proposed by ACPA, and its design method was developed based on PCA (1984) method. Accordingly, StreetPave calculates the total fatigue and erosion damage separately using the detailed axle load distribution data and determines the required thickness for the critical situation (for either fatigue or erosion). It should be noted here that; although, StreetPave was developed by considering the PCA (1984) method, during the development of the software some additions were made; such as user defined reliability (%), percent of allowed slabs cracked at the end of design life and consideration of fibers to the required thickness. As explained before, the residual flexural strength ratio for 3 mm net deflection is used to increase the MOR value to consider the contribution of fibers. However, it is only valid for fatigue analysis case, and effects of fibers are not considered in the erosion analysis done in the software.

The thickness design calculations were made for a sample road according to 3 design methodologies by using different types of fibers (synthetic, steel and carbon) and concrete (conventional and roller compacted concrete). Also, calculations were carried out for both plain and fiber-reinforced concrete pavements; and the results were compared. In this way, it was investigated how the thickness requirement for plain concrete roads were affected according to 3 design methodologies and mixtures with different consistency. As mentioned before; contribution that can be taken from the addition of fiber highly depends on the type and amount of used fiber, as well as properties of concrete matrix. Therefore, the results obtained for fiber-reinforced concrete were examined to see the effects of these parameters on thickness requirements. Also, in the scope of the study, thickness design calculations were made for both with and without tied concrete shoulder to evaluate the effects of the presence of edge support for RCC and conventional PCC pavements on the pavement thickness requirement. As mentioned in the literature review, since RCC pavements don't use dowel bars, the effects of dowel bars on the thickness requirement were only evaluated for conventional PCC pavements.

First, the thickness design results obtained using three design methodologies were given separately to evaluate the design results in terms of plain and fiber-reinforced conditions within themselves. Then, the overall results were compared in terms of RCC and conventional PCC pavements to see the differences and similarities arising from the methodologies. The subgrade, road, traffic and material related parameters used by the 3 design methodologies were given from Table 4.1 to 4.3. Also, the detailed information about the parameters and variables used in the 3 different design methodologies were given the following sections.

4.1.1. Traffic and Foundation Parameters

Traffic and foundation parameters to be used for thickness design conducted in this study were obtained from IRC SP 46 (2013). Road, traffic, and foundation parameters used were given in Table 4.1, and the axle load spectrum was given in Table 4.2. It should be noted that one of the basic parameters in pavement design is traffic. The conversion of the traffic calculation needs to be carried out for the thickness design to be performed correctly. In the design process, all three methodologies use different traffic units which are summarized below.

In AASHTO (1993) methodology, total load applications due to mixed stream of traffic over the design life is requested into the 18-kip ESAL (W_{18}). Therefore, after determining the appropriate LEFs (Load Equivalency Factors) considering the each axle load class and the each axle type (single, tandem, and tridem) given in the Table 4.2, design ESAL

(Equivalent Single Axle Load) was calculated taking into account all proportion of the axle category (steering, rear single, rear tandem, and rear tridem).

According to IRC SP 46 (2013), thickness design was carried out considering the temperature gradient. Therefore, calculated design lane axle load repetitions during the design period are evaluated in terms of daytime (from 10 AM to 4 PM) and nighttime (from 0 AM to 6 AM) which has the probability of occurring bottom-up cracking and top-down cracking, respectively. The cumulative fatigue damage is checked for these six-hour periods in which high flexural stresses occur due to axle load and temperature difference.

It should be noted that in the StreetPave v12 (2014) software, traffic spectrum data is requested in the form of axle/1000 trucks (as in PCA (1984)). Therefore, in the Table 4.2, the axle load class distributions for the axle type are converted into axle/1000 trucks.

In the IRC SP 46 (2013) and StreetPave design, the damage caused by the front (steering) single axles was neglected because the damage would be lower than the rear axles; AASHTO (1993) considers all proportions of axles in determining the relative damage caused by mixed traffic flow. Also, in the design of 3 methodologies, it is considered that the damage caused by heavy vehicles is considerably high compared to other vehicles like cars, and only truck loads are considered as traffic loads.

On the other hand, Effective CBR of compacted subgrade for the application site was given as 8% in the example given in IRC SP 46 (2013). Additionally, modulus of sub-grade reaction was 50.3 MPa/m and dry lean concrete (DLC) sub-base with a thickness of 150 mm and minimum 7-day compressive strength of 10 MPa was provided. And, effective modulus of sub-grade reaction of combined foundation of compacted subgrade and DLC sub-base is calculated as 285 MPa/m by using these parameters. To provide a de-bonding layer, a polyethene sheet is placed between the DLC and the FRC plate.

Table 4.1. Parameters related to traffic, road and foundation.

Parameters		**Value**
Design period		30 years
Total number of lanes		4
Number of lanes for one direction		2
Lane width		3.5 m
Transverse joint spacing		4.5 m
Two-way annual daily traffic (ADT)		1000 vehicles/day
Percent of traffic in the predominant direction		50%
Percent of traffic travelling during night 12 hours		60%
Percent of traffic travelling during day 12 hours		40%
The vehicles with spacing between the front axle and the first rear axle less than 4.5		55%
The annual rate of growth of traffic		7.5%
Axle load category	Front (Steering) single	45%
	Rear single	15%
	Tandem	25%
	Tridem	15%
The average number of axles per commercial vehicle		2.35
Subgrade CBR (%)		8%
Composite modulus of subgrade reaction		285 MPa/m
Maximum day time temperature differential in slab		16.8 °C
Maximum nighttime temperature differential in slab		13.4 °C

Table 4.2. Axle load spectrum.

Single Axle		**Tandem Axle**		**Tridem Axle**	
Axle Load Class (kN)	**Frequency (% of single axles)**	**Axle Load Class (kN)**	**Frequency (% of tandem axles)**	**Axle Load Class (kN)**	**Frequency (% of tridem axles)**
185-195	18.15	380-400	14.50	530-560	5.23
175-185	17.43	360-380	10.50	500-530	4.85
165-175	18.27	340-360	3.63	470-500	3.44
155-165	12.98	320-340	2.50	440-470	7.12
145-155	2.98	300-320	2.69	410-440	10.11
135-145	1.62	280-300	1.26	380-410	12.01
125-135	2.62	260-280	3.90	350-380	15.57
115-125	2.65	240-260	5.19	320-350	13.28
105-115	2.65	220-240	6.30	290-320	4.55
95-105	3.25	200-220	6.40	260-290	3.16
85-95	3.25	180-200	8.90	230-260	3.10
<85	14.15	<180	34.23	<230	17.58
Total	100	Total	100	Total	100

4.1.2. Material Parameters

The material parameters for plain and fiber reinforced concrete used in RCC pavements were taken from the two previous studies carried out by LaHucik et al., (2017) and Öztürk (2018); in conventional PCC pavements these parameters were taken from the previous study carried out by Mulheron, (2015). In addition, using the previous study conducted by Öztürk (2018), different RCC pavement material data was obtained from the master thesis prepared as a result of the experiments carried out in 2018 by Onur Öztürk in the laboratory of Boğaziçi University. On the other hand, material properties were taken from the different studies in the literature for different concrete types to see the effects of the concrete matrix on the effectiveness of fibers. The material parameters for each mixture used in the designs are given in Table 4.3. As given in Table 4.3, FRC mixtures that include different types of fibers (steel, synthetic, carbon) in different amounts (0.2-1.0%) were selected to see the effects of types and amounts of the used fibers. It can be seen from Table 4.3 that different flexural strength, residual flexural strength, equivalent flexural strength ratio and modulus of elasticity values (parameters used in thickness design) were reported by the authors for the usage of different types of fibers in different amount.

The mechanical properties of concrete cannot be improved using lower volume of fibers. In contrast, the use of high fiber volume (>0.5 by volume) leads to an increase in the amount of void, resulting in a decrease in the strength and durability of the concrete. Therefore, it is very crucial to determine the optimum fiber content in order to maximize the use of the fiber.

As shown in Table 4.3, the contribution of each of the different types of fibers to the flexural strength and the residual flexural strength ratio (at 3 mm) is different. For example, the Hook-60 (steel) type fibers used in the study conducted by LaHucik et al (2017) increases the flexural strength when used by 0.2% by volume, slightly decreases the flexural strength when used by 0.4% by volume, but the residual flexural strength ratio value significantly increases. In addition, by using 0.5 % by volume of PP Macro type fibers used in the study conducted by Mulheron (2015), it is seen that there is a maximum increase in flexural strength compared to the usage of 0.7% and 1% by volume. However, it is seen that when it is used by 1% by volume the increase in flexural strength of concrete is not as much as when it is used by 0,5% by volume, but the residual flexural strength ratio value significantly increases. This is because increasing the amount of fiber after a certain ratio (around 0.5 volume %) causes a decrease in the maximum dry density. As a result of the decrease in dry density, it is seen that mechanical performance and durability of materials are negatively affected.

In addition, the “Control” data given in Table 4.3 for RCC and conventional PCC pavements represent plain concrete. Since the material parameters of the two studies were used for RCC mixtures, plain RCC in the study carried out by Öztürk (2018) was named RCC Control-1 and plain RCC in the study conducted by LaHucik et al. (2017) was named RCC Control-2. Also, it should be noted that the flexural strength values, the key parameter in the design, differ in each mixture. Thickness design for each of the concrete mixtures given in the Table 4.3 was done, and results were compared in the following sections.

It should be emphasized here that contribution of fibers to the pavement thickness requirement were investigated in the scope of the study presented here, and contributions of fibers to the durability properties of concrete pavements weren’t mentioned. More durable concrete pavements with lower thickness can be constructed by using the fibers in concrete pavement applications.

Table 4.3. Material parameters for RCC and conventional PCC mixtures.

Study	Mixture	Fiber Length (mm)	Fiber Length/Diameter	Fiber Amount Volume (%)	Modulus of Elasticity (MPa)	Flexural Strength (MPa)	Residual Flexural Strength at 3 mm (MPa)	Residual Flexural Strength Ratio (%)	Equivalent Flexural Strength Ratio ($R_{e,3}$) (%)	UW (kg/m^3)
Öztürk, 2018 (RCC)	RCC Control-1	-	-	-	40500	7.26	-	-	-	2524
	RCC-S54 (synthetic)	54	98	0.5	36700	6.23	0.48	7.70	21.03	2517
	RCC-S36 (synthetic)	36	90	0.5	33000	5.93	0.39	6.58	22.43	2499
LaHucik et al., 2017 (RCC)	RCC Control-2	-	-	-	31600	4.65	-	-	-	2529
	Emboss - 0.2 (synthetic)	48	67	0.2	33000	4.15	0.40	9.64	17.4	2609
	Emboss - 0.4 (synthetic)	48	67	0.4	29000	4.35	1.05	24.14	32.5	2610
	Smooth - 0.2 (synthetic)	40	90	0.2	30200	3.95	0.25	6.33	14.2	2588
	Smooth - 0.4 (synthetic)	40	90	0.4	30400	4.00	0.55	13.75	25.7	2588
	Hook - 0.2 (steel)	60	55	0.2	32000	5.05	1.35	26.73	36.6	2595
	Hook - 0.4 (steel)	60	55	0.4	30800	4.50	1.95	43.33	59.8	2594
Mulheron, 2015 (PCC)	PCC Control	-	-	-	31357	5.55	-	-	-	2323
	Polypropylene Macro – 0.5	54	-	0.5	34001	6.79	1.45	21.35	36.7	2355
	Polypropylene Macro – 0.7	54	-	0.7	34494	6.48	2.14	33.02	60.1	2371
	Polypropylene Macro – 1.0	54	-	1.0	31016	6.17	2.65	42.94	55.0	2355
	Steel – 0.9	50	-	0.9	31096	5.38	4.07	75.68	80.8	2307
	Carbon – 0.3	102	-	0.3	31231	6.52	1.14	17.50	35.3	2323
	Carbon – 0.7	102	-	0.7	32348	7.17	2.31	32.22	60.0	2323
	Carbon – 1.0	102	-	1.0	31632	6.86	2.48	36.15	38.9	2355

4.2. Summary of AASHTO (1993) Methodology

- The AASHTO (1993) pavement design method is based on empirical models developed by considering the effect of the AASHO Road Test field results.
- AASHTO (1993) evaluates the performance in terms of loss of serviceability caused by 18-kip single axle load applications. And, the type of hazard is not distinguished as individual distresses like fatigue or erosion.
- In AASHTO (1993), the mixed traffic stream is converted to the 18-kip ESAL by using the load equivalency factors to estimate the relative damage.
- Since the AASHO Road Test design equations are based on traffic test results over a two-year period, the effects of long-term temperature and moisture on the loss of serviceability were not considered. Environmental effects were considered in three ways in the design method: drainage coefficients, serviceability loss due to environmental conditions, and estimation of an effective subgrade resilient modulus that reflects seasonal variations.
- In AASHTO (1993), the presence of load transfer devices such as dowel bars and the tied shoulder is only considered when choosing the load transfer coefficient, J. The experience of the designer has an important role in determining the J value. Also, the pavement thickness and performance are affected directly by the value of J.
- AASHTO (1993) doesn't have provisions for the use of fibers. However, in the scope of this study effects of fibers were considered within the design process via Altoubat et al. (2008) approach.
- Most effective parameters: flexural strength and modulus of elasticity of concrete, load transfer coefficient (J).

4.2.1. Thickness Design According to AASHTO (1993)

For thickness design using AASHTO (1993), selection of the design parameters were made based on the recommendations given in the guideline. So, according to roadway classification, reliability is taken as 85%. Also, AASHTO (1993) recommends the total standard deviation (S_0) values from 0.3 to 0.4 for rigid pavements and it was taken as 0.35 which represents the average conditions. For design it is necessary to select both an initial and terminal serviceability index. The initial serviceability index represents the pavement smoothness immediately after construction and is taken as 4.5. The terminal serviceability index represents the lowest acceptable level before rehabilitation. In the guide, for major highways it is suggested 2.5 or 3.0 and for this study it is taken as 2.5. The drainage coefficient (C_d) is assumed as 1 as in the AASHO Road Test. So, the drainage coefficient factor didn't have any significance in the design process. However, the load transfer coefficient (J) was determined as follows considering the presence of dowel bar and edge support according to the given chart in the ASSHTO 1993 design guide.

As stated in the literature review, RCC mixtures are drier compared to conventional PCC (due to their high dose of fine aggregate, low cement and water content). Also, since RCC has zero slump, the mixture is not sufficiently fluid to be shaped by conventional PCC pavers. So, in conventional PCC pavements, compaction takes place internally, while in the RCC pavements the main compaction is performed by externally compacting the concrete with rollers. Compaction has been observed to increase the aggregate interlock of RCC. In addition, it is not possible to place dowel bars at joints due to the compaction method on RCC pavements.

Therefore, in Table 4.4, the column "With dowel bar" is shown as "-" for the RCC. Also, for the RCC pavements, the J coefficient was determined by considering the relatively increased aggregate interlock at load transfer. It should be noted that the higher J represents the lower support condition. Table 4.4 shows the assumptions made for the load transfer coefficient used in the empirical formula.

Table 4.4. Determination of load transfer coefficient.

Type of Concrete	Without Tied Concrete		With Tied Concrete	
	With dowel bar	Without dowel bar	With dowel bar	Without dowel bar
PCC	3.2	4.2	2.8	3.8
RCC	-	4	-	3.6

One of the most critical inputs of the AASHTO (1993) design method is to estimate the ESALs that a particular pavement will be subject to over its design life. To determine the total ESAL, load equivalency factors (LEFs) were selected for each axle type and load class from the LEF tables given in the design guide and multiplied with each axle load (LEFs were selected from tables considering terminal serviceability: 2.5 and D: 10 inches). Then, design lane traffic was obtained by multiplying the total ESAL with directional and lane distribution coefficients which were taken as 0.5 and 0.9, respectively (in accordance with the recommendations in the guideline). A spreadsheet was created to solve the rigid pavement design equation given in the literature review. Thus, RCC and conventional PCC pavement thickness values were calculated considering all the assumed design parameters mentioned herein.

It should be noted that AASHTO (1993) does not consider the contribution of fibers in the design. However, Altoubat et al. (2008) stated that the approach they developed to consider the contribution of fibers to the toughness of the concrete can be used in AASHTO (1993). Therefore, the flexural strength values of the fiber reinforced concretes used in the design were increased with the approach of Altoubat et al. (2008). In the previous section, this approach was described in detail.

The parameters related to the road, traffic, foundation and material used as common by the 3 design methodologies were given in the previous tables. The following table shows the values of the parameters that are specific to AASHTO and were used in the design process.

Table 4.5. Design variables for AASHTO (1993).

Parameter	**Value**
Number of ESALs (W18)	620673155
Reliability (R) (%)	85
Normal Standard Deviation (ZR)	-1.037
Total Standard Deviation (S0)	0.35
Initial Serviceability (PI)	4.5
Terminal Serviceability (PT)	2.5
Drainage Coefficient (Cd)	1
Directional Distribution Coefficient	0.5
Lane Distribution Coefficient	0.9

It should be noted that thickness values given from Table 4.6 to 4.8 were obtained by the empirical equation for loss of serviceability (ΔPSI) caused by 18-kip equivalent single axle load applications (concrete road failures such as fatigue or erosion are not individually distinguished), which is the performance criterion of AASHTO.

<u>4.2.1.1. Results for Plain RCC and Conventional PCC.</u> In this section, the slab thicknesses were compared in terms of plain RCC and conventional PCC pavements both with and without tied concrete shoulder. As mentioned before, since dowel bars aren't used in RCC pavements, the thickness comparison was made for the "without dowel bar" situation. Table 4.6 shows the change in thickness requirement for three studies with different material parameters.

Table 4.6. Thickness requirements of plain RCC and conventional PCC in terms of AASHTO (1993).

Study	Mixture	Required Thickness (cm)	
		Without Tied Concrete Shoulder	With Tied Concrete Shoulder
		Without Dowel Bar	Without Dowel Bar
Öztürk, 2018 (RCC)	RCC Control-1	38.1	35.9
LaHucik et al., 2017 (RCC)	RCC Control-2	47.8	45.2
Mulheron, 2015 (PCC)	PCC Control	44.6	42.2

One of the main parameters affecting the required thickness is the flexural strength value of concrete. Also, RCC has a flexural strength value equivalent to that of conventional PCC. Since plain concretes are compared here, it should be noted that the differences between the thickness values only results from the differences in the flexural strength (MOR) and elasticity modulus of the concretes (E).

When Table 4.6 were examined, it was seen that RCC Control-1 pavement which has relatively highest flexural strength (7.26 MPa) (see Table 4.3) taken from the study conducted by Öztürk (2018) gives the lowest thickness value. However, the highest pavement thickness was obtained by RCC Control-2 pavement, which has relatively low flexural strength (4.65 MPa) (see Table 4.3) taken from the study conducted by LaHucik et al. (2017). Thus, it was seen that an increase in flexural strength of 36% could reduce the thickness requirement by up to 20%.

In addition, the results given in Table 4.6 for the pavement slab with and without tied concrete shoulder showed that the thickness could be reduced by up to approximately 6% by using the concrete shoulder. It was stated in the literature that the stresses that the concrete road is subjected to during the design life could be reduced by the presence of edge support or load transfer means. The presence of such supports therefore reduces the thickness requirement.

4.2.1.2. Results for Fiber-Reinforced Conventional PCC. The slab thicknesses were compared in terms of fiber-reinforced conventional PCC both with and without tied concrete shoulder. Also, the thickness requirement was evaluated in terms of the presence of dowel bars. Table 4.7 shows the change in thickness requirement for the conventional PCC mixtures with different material parameters.

Table 4.7. Thickness requirements of fiber-reinforced conventional PCC in terms of AASHTO (1993).

Study	Mixture	Required Thickness (cm)			
		Without Tied Concrete Shoulder		With Tied Concrete Shoulder	
		With Dowel Bar	Without Dowel Bar	With Dowel Bar	Without Dowel Bar
Mulheron, 2015 (PCC)	PCC Control	38.5	44.6	35.8	42.2
	Polypropylene Macro – 0.5*	29.0	33.8	26.8	32.0
	Polypropylene Macro – 0.7*	27.2	31.9	25.0	30.1
	Polypropylene Macro – 1.0*	28.2	33.0	26.0	31.2
	Steel – 0.9*	27.9	32.8	25.8	31.0
	Carbon – 0.3*	29.6	34.6	27.4	32.7
	Carbon – 0.7*	25.4	29.8	23.3	28.2
	Carbon – 1.0*	28.3	33.1	26.2	31.3

* represents the used fiber amount by volume (%)

When the required thickness values given in Table 4.7 were examined, it was seen that the slab thickness requirement can be reduced up to 34% by using fiber, regardless of whether there was an edge support or dowel bar (this value may show an increase or decrease depending on the properties and amounts of fiber materials).

As expected, it was seen that the highest performance can be achieved with Carbon - 0.7 PCC mixture which doesn't decrease the flexural strength value. Although Steel - 0.9 has the maximum contribution to toughness (~76%), it was seen that the expected contribution could not be obtained from the fibers since the flexural strength value decreased significantly. As a result of this, it was observed that the use of a fiber-reinforced concrete with poorly optimized mixtures gives the required slab thickness values similar to or higher than that of plain concrete. (Steel - 0.9 did not give as much thickness as the plain concrete case but gave the worst results among other fiber-reinforced concrete mixtures.) Therefore, in order to achieve the desired contribution by using fiber in concrete road applications the selection of fiber type and amount, as well as the importance of the design of the concrete matrix are clear.

In addition to all of these, the results given in Table 4.7 for the pavement thickness with and without tied concrete shoulder showed that the thickness could be reduced by up to ~8% for with dowel bar and ~5.5% for without dowel bar conditions in the case of slab edge was supported. On the other hand, when the values given in Table 4.7 were examined, the importance of load transfer with the dowel bar was clearly seen. Accordingly, the use of dowel bars resulted in a reduction of approximately 16% in the obtained thickness values for both with and without concrete shoulder conditions.

4.2.1.3. Results for fiber-reinforced RCC. In this section, the thickness values obtained for 2 different RCC studies were compared separately in terms of fiber-reinforced RCC both with and without tied concrete shoulder. It should be noted that, the thickness requirement was evaluated in terms of the presence of dowel bars since the dowel bars aren't used in RCC. Table 4.8 shows the change in thickness requirement for 2 different RCC mixtures with different material parameters.

Table 4.8. Thickness requirements of fiber-reinforced RCC for AASHTO (1993).

Study	Mixture	Required Thickness (cm)	
		Without Tied Concrete Shoulder	With Tied Concrete Shoulder
		Without Dowel Bar	Without Dowel Bar
Öztürk, 2018 (RCC)	RCC Control-1	38.1	35.9
	RCC-S54 (synthetic) *b	37.1	35.1
	RCC-S36 (synthetic) *b	37.7	35.6
LaHucik et al., 2017 (RCC)	RCC Control-2	47.8	45.2
	Emboss - 0.2*a (synthetic)*b	46.7	44.1
	Emboss - 0.4*a (synthetic) *b	42.4	40.0
	Smooth - 0.2*a (synthetic) *b	48.3	45.7
	Smooth - 0.4*a (synthetic) *b	45.7	43.2
	Hook - 0.2*a (steel) *b	38.6	36.4
	Hook - 0.4*a (steel) *b	37.7	35.6

*a represents the used fiber amount by volume (%)

*[b] represents the used fiber type

When the required thickness values given in Table 4.8 were examined in terms of the study conducted by Öztürk (2018), it is seen that the slab thickness requirement could be reduced only up to ~3% by using fibers regardless of whether there is edge support.

When the material properties were examined, it was seen that the fibers used in the mixture contribute about 7% to the toughness. Also, because of the low performance of the fibers used here, some reduction in the flexural strength of the concrete occurs. As mentioned before, the flexural strength values are increased by the approach of Altoubat et al (2008) to consider the positive contributions of the fibers. However, since the increased flexural strength values are too close to the flexural strength of the plain concrete, the calculated thicknesses are very close to each other.

In addition to all of these, the results given in Table 4.8 for the pavement thickness with and without tied concrete shoulder showed that the thickness could be reduced up to ~5.5% in the case of slab edge was supported.

On the other hand, when the required thickness values given in Table 4.8 were examined in terms of the study conducted by LaHucik et al. (2017), it was seen that the slab thickness requirement could be reduced up to 21% by using fiber, regardless of whether there is edge support (these values may show an increase or decrease depending on the properties and amounts of fiber materials).

As expected, the highest performance was achieved with the Hook - 0.4 mixture, which has the highest flexural strength and the highest contribution to the toughness. However, since Smooth - 0.2 has little contribution (6%) to the toughness and causes a reduction in flexural strength, the use of these fibers leads to a greater thickness than the plain concrete mixture. As a result, it was inferred that the use of fiber-reinforced concrete with poorly optimized mixtures provides a higher or similar required slab thickness than plain concrete.

Therefore, as in conventional PCC, to achieve the desired contribution by using fiber in concrete road applications the selection of fiber type and amount, as well as the importance of the design of the concrete matrix are clear. In addition to all of these, the results given in Table 4.8 for the pavement thickness with and without tied concrete shoulder showed that the thickness could be reduced up to ~6% in the case of slab edge was supported.

When Table 4.8 was examined in terms of both different RCC studies, it was seen that the thicknesses obtained from the study conducted by Öztürk (2018) gave lower values than the other. In the study conducted by Öztürk, although the fibers performed poorly and caused a decrease in flexural strength, it was seen that the flexural strength was still higher than the flexural strength of all mixtures in the other study. It showed that the difference in thickness values between two different RCC studies was due to flexural strength.

4.2.1.4. Preliminary Conclusion for AASHTO (1993). Dominant parameters and their effects: In the design, it was observed that the flexural strength of concrete (MOR) and load transfer coefficient (J) influenced the obtained thickness values. The higher the flexural strength, the lower the thickness obtained. Furthermore, the smallest load transfer coefficient has been chosen for “with dowel bar” and “with tied concrete shoulder” case for conventional PCC. The lowest thicknesses were obtained for this case.

Slab edge support: Required thickness values could be reduced by up to ~5.5% for “without dowel bar” situation and ~8% for “with dowel bar” situation in the case of slab edge was supported.

Use of dowel bars: The use of dowel bar is only applicable to conventional PCC, and the use of dowel bar provides a reduction in thickness of about 15%.

Use of fibers: It was seen that the use of fiber can reduce the pavement thickness requirement by up to 35% (these values may increase or decrease depending on the materials). As expected, it was seen that the highest performance can be achieved with mixtures that do not decrease the flexural strength value and also contribute to the toughness. However, it was seen that the use of a fiber-reinforced concrete with poorly optimized mix yields the required pavement thickness values similar to that of plain concrete.

4.3. Summary of IRC 58 (2011) and IRC SP 46 (2013) Methodologies

- The design methodology of IRC SP 46 (2013) is mainly based on IRC 58 (2011) (used to design plain concrete pavements). However, IRC SP (46) 2013 modified the limiting stress ratio values given in IRC 58 (2011) to consider the positive effect of fibers.
- IRC SP 46 (2013) is mainly mechanistic-empirical design focusing on slab stress predictions caused by both wheel load and temperature differential.
- IRC SP 46 (2013) evaluates the performance through the cumulative fatigue damage analysis (considering bottom-up and top-down cracking) based on Miner's hypothesis.
- IRC SP 46 (2013) uses the axle load spectrum which consists of axle load classes and frequency data (with respect to axle load class and axle type).
- IRC SP 46 (2013) considers only the temperature difference that occurs at the top and bottom of the concrete slab as an environmental effect.
- IRC SP 46 (2013) recommends the use of load transfer devices such as dowels. It states that the use of dowel bars isn't reliable for pavement thickness less than 200 mm.
- IRC SP 46 (2013) guideline considers the contribution of fibers to the fatigue life of the concrete pavement via increasing the limiting stress ratios depending on the toughness contribution of fiber to the behavior of concrete. Also, if the stress ratio is above the stated limit values, calculation of the allowable number of repetitions was given in IRC 58 (2011) and IRC SP 46 (2013).

4.3.1. Thickness Design According to IRC SP 46 (2013)

As stated in the literature review, a two-stage design is recommended, which consists of moment capacity control based on TR 34 – third edition (2003) and fatigue damage control based on IRC 58 (2011). The parameters related to the subgrade, road, traffic and material used in the design were given from Table 4.1 to 4.3.

It should be noted that the methodology of IRC SP 46 (2013) is mainly based on IRC 58 (2011) which is used for the design of plain concrete pavements. However, IRC SP 46

(2013) modified the limiting stress ratio values adopted in IRC 58 (2011) to consider the effects of fibers. In addition to the fatigue analysis carried out in IRC 58 (2011), IRC 46 (2013) performs the yield line analysis check for the heaviest load of the single axle load class that is considered as critical for fatigue damage if a high-toughness fiber is used ($f_{e150k} \geq 0.3 * f_{ctk}$)(IRC SP 46, 2013). Pavements with FRC having high toughness shall be designed as per the procedure given in IRC 58.

In addition, since erosion is not a failure mode considered in the present design procedure the presence or absence of dowel bars at the joints is not regarded as a parameter affecting the design thickness. However, in the design guide IRC 46 (2013), the use of dowel bars is recommended for the following situations. If the thickness of the design is less than 200 mm, the use of dowel bars is not considered as reliable.

i) Thickness ≥ 200 mm in the design using fibers with low contribution to toughness,

ii) Thickness ≥ 150 mm in the design using fibers with high contribution to toughness.

IRC 58 (2015), on the other hand recommends the use of dowel bars for heavy traffic (>450 trucks/day) (due to insufficient aggregate interlock in the joint transfer of problems caused by repeated heavy loads) and edge support (to protect the edge of concrete pavement on high volume roads).

4.3.2. Design for Fatigue Resistance (based on IRC 58 (2011))

Since the designed pavement has a high traffic volume (>500 trucks/day), the analysis of fatigue is necessary. Table 4.9 includes the road parameters computed for fatigue analysis. Table 4.10 also displays the category–wise axle load repetitions for bottom-up and top-down fatigue cracking analysis. For fatigue damage, the thickness requirements for fiber reinforced RCC and conventional PCC pavements were determined according to IRC SP 46 (2013), using the traffic parameters given below and the material properties obtained from the three studies in the literature (Öztürk (2018), LaHucik (2017) and Mulheron (2015)). Thickness design procedure details for fatigue damage are given below.

First, the limiting stress ratio for plain RCC and conventional PCC was defined as 0.45. However, the limiting stress ratio was defined 0.5 (low toughness, $f_{e150k}<0.3*f_{ctk}$) and 0.6 (high toughness, $f_{e150k}\geq 0.3*f_{ctk}$), considering the toughness provided by fibers for fiber reinforced RCC and conventional PCC pavements. Then, respectively, the expected axle load repetitions, the flexural stresses caused by the axle loads and temperature differential, the stress ratio (flexural stress / 90-day characteristic flexural strength of concrete), the number of allowable axle load repetitions and the fatigue damage caused by repeated axle loads (expected axle load repetitions/ allowable axle load repetitions) for bottom-up and top-down cracking were calculated. All design values were calculated by the equations specified in IRC SP 46 (2013) and IRC 58 (2015) guidelines.

Table 4.9. Road parameters for fatigue analysis.

Parameter	**Value**
Two-way annual daily traffic (vehicles/day)	1000
The annual rate of growth	0.075
Total two-way commercial vehicles (vehicles/30 years)	37740782
Average axle number	2.35
Total two-way axial load repetitions (vehicles/30 years)	88690838
Percent of traffic in the predominant direction	0.5
Number of vehicles per a direction	44345419
Number of vehicles after lateral placement of axles	11086355
Day-time traffic	0.4
Design axle load repetition (during day times (12 hours))	4434542
Design axle load repetitions for bottom up cracking analysis (6 hours)	2217271
Night-time traffic	0.6
Design axle load repetition (during night times (12 hours))	6651813
Design axle load repetitions for top down cracking analysis (6 hours)	3325906
(For top down cracking) percent of vehicles with spacing between the front and the first rear	0.55
Design number of axle load repetitions for top down cracking	1829249

Table 4.10. Category-wise axle load repetitions for bottom-up and top-down fatigue cracking analysis.

Axle Category	**Axle Category**	**Category Wise Axle Repetitions for Bottom-up Cracking Analysis**	**Category Wise Axle Repetitions for Top-Down Cracking Analysis**
Front (steering)	0.45	997772	823162
Rear Single	0.15	332591	274387
Tandem	0.25	554318	457312
Tridem	0.15	332591	274387

4.3.2.1. Results for Plain RCC and Conventional PCC. In this section, the slab thicknesses were compared in terms of plain RCC and conventional PCC pavements both with and without tied concrete shoulder. In the design process of IRC 58 (2011), presence of dowel bars in conventional PCC doesn't change the calculated required thickness since erosion isn't considered as a failure mode in the current design guide. Also, since dowel bars aren't used in RCC pavements, the thickness comparison was made for the "without dowel bar" situation. Table 4.11 shows the change in thickness requirement for 3 studies with different material parameters.

Table 4.11. Comparison of plain RCC and conventional PCC in terms of IRC 58 (2011).

Study	Mixture	Required Thickness (cm)		Standard (Stress Ratio) *
		Without Tied	With Tied	
		Fatigue	Fatigue	
Öztürk, 2018 (RCC)	RCC Control-1	19.2	16.1	IRC 58 (0.45)
LaHucik et al., 2017 (RCC)	RCC Control-2	29.4	25.3	
Mulheron, 2015 (PCC)	PCC Control	26	22	

* The design was made for plain concrete according to IRC 58 (2011) with the stress ratio of 0.45.

Flexural strength value of concrete is one of the main parameters affecting the required thickness. Also, RCC has a flexural strength value equivalent to that of conventional PCC. Since plain concretes were compared here, it should be noted that the differences between the thickness values only result from the differences in the flexural strength (MOR) and elasticity modulus of the concretes (E). Therefore, the results obtained previously for the AASHTO design method could be seen here.

Briefly, a thinner slab was obtained by using the plain RCC mixture (RCC Control-1) taken from the study conducted by Öztürk (2018) which has a relatively high flexural strength, while a thicker slab is obtained by using the plain mixture (RCC Control-2) taken from the study conducted by LaHucik et al. (2017) which has a relatively low flexural strength. Thus, it was seen that an increase in flexural strength of 36% could reduce the thickness requirement by up to 35%.

In addition, the results given in Table 4.11 for the pavement slab with and without tied concrete shoulder showed that the required thickness values could be reduced by up to 16% if the slab edge was supported.

4.3.2.2. Results for Fiber-Reinforced Conventional PCC. The slab thicknesses were compared in terms of fiber-reinforced conventional PCC both with and without tied concrete shoulder. As mentioned before, since erosion isn't considered as a failure mode in the current design guide, the presence of dowel bars in the design process does not make any changes in required thickness values. Also, when the Table 4.12 was examined, it was seen that the obtained thicknesses were higher than the limit thickness value recommended by IRC SP 46 (2013) for the use of dowel bar. So, the obtained thickness values were assumed as with dowel bars according to the recommendation in the guide (use dowel bars if the thickness of the plain concrete higher than 200 mm or use dowel bars if the thickness of the FRC having high toughness higher than 150 mm). Table 4.12 shows the change in thickness requirement for the conventional PCC mixtures with different material parameters.

Table 4.12. Thickness comparison of fiber-reinforced conventional PCC in terms of IRC 58 (2011) and IRC SP 46 (2013).

Study	Mixture	Required Thickness (cm)		Standard (Stress Ratio) *[b]
		Without Tied Concrete Shoulder	With Tied Concrete Shoulder	
		Fatigue	Fatigue	
Mulheron, 2015 (PCC)	PCC Control	26.0	22.0	IRC 58 (0.45)
	Polypropylene Macro – 0.5*[a]	20.8	17.4	IRC 46 (0.5)
	Polypropylene Macro – 0.7*[a]	19.7	16.4	IRC 46 (0.6)
	Polypropylene Macro – 1.0*[a]	21.4	17.8	IRC 46 (0.6)
	Steel – 0.9*[a]	23.3	19.4	IRC 46 (0.6)
	Carbon – 0.3*[a]	22.3	18.7	IRC 46 (0.5)
	Carbon – 0.7*[a]	19.1	15.9	IRC 46 (0.6)
	Carbon – 1.0*[a]	19.8	16.5	IRC 46 (0.6)

*[a] represents the used fiber amount by volume (%)

*[b] represents the design guidelines and stress ratios used in the design process. (IRC 58 for plain concrete or fibers are used less than 0.3% by volume (with 0.45 stress ratio); IRC 46 for fibers are used in dose more than 0.3% by volume (mainly control plastic shrinkage) or fibers having low toughness (with 0.5 stress ratio), and for fibers having high toughness (with 0.6 stress ratio)).

When the required thickness values given in Table 4.12 were examined, it was seen that the required thickness could be reduced by up to 27% by using fiber, regardless of whether there is edge support. (this value may show an increase or decrease depending on the properties and amounts of fiber materials).

As expected, it was seen that the highest performance could be achieved with Carbon – 0.7 PCC mixture, which doesn't decrease the flexural strength value. Although Steel – 0.9 has the maximum contribution to toughness (~76%), it was seen that the expected contribution could not be obtained from the fibers since the flexural strength value decreases significantly. As a result of this, it was observed that the use of fiber-reinforced concrete with poorly optimized mixtures gives the required slab thickness values similar to or higher than that of plain concrete. (Steel – 0.9 fiber did not give as much thickness as a plain concrete case but gave the worst results among other fiber-reinforced concrete mixtures.) Therefore, in order to achieve the desired contribution by using fiber in concrete road applications the selection of fiber type and amount, as well as the importance of the design of the concrete matrix are clear.

In addition to all of these, the results given in Table 4.12 for the pavement thickness with and without tied concrete shoulder showed that the thickness could be reduced up to ~15% in the case of slab edge was supported.

<u>4.3.2.3. Results for Fiber-Reinforced RCC.</u> In this section, the thickness values obtained for 2 different RCC studies were compared separately in terms of fiber-reinforced RCC both with and without tied concrete shoulder. It should be noted that, the thickness requirement wasn't evaluated in terms of the presence of dowel bars since the dowel bars aren't used in RCC. Table 4.13 shows the change in thickness requirement for 2 different RCC mixtures with different material parameters.

Table 4.13. Thickness comparison of fiber reinforced RCC for IRC 58 (2011) and IRC SP 46 (2013).

Study	Mixture	Required Thickness (cm)		Standard (Stress Ratio) *c
		Without Tied Concrete Shoulder	With Tied Concrete Shoulder	
		Fatigue	Fatigue	
Öztürk,2018 (RCC)	RCC Control-1	19.2	16.1	IRC 58 (0.45)
	RCC-S54	22.2	18.7	IRC 46 (0.5)
	RCC-S36	24.3	20.4	IRC 46 (0.5)
LaHucik et al., 2017 (RCC)	RCC Control-2	29.4	25.3	IRC 58 (0.45)
	Emboss - 0.2*a (synthetic)*b	31.4	27.6	IRC 58 (0.45)
	Emboss - 0.4*a (synthetic) *b	30.6	26.0	IRC 46 (0.5)
	Smooth - 0.2*a (synthetic) *b	34.3	30.5	IRC 58 (0.45)
	Smooth - 0.4*a (synthetic) *b	31.7	27.5	IRC 46 (0.5)
	Hook - 0.2*a (steel) *b	26.1	22.0	IRC 46 (0.5)
	Hook-0.4*a (steel) *b	26.3	22.1	IRC 46 (0.6)

*a represents the used fiber amount by volume (%)

*b represents the used fiber type

*c represents the design guidelines and stress ratios used in the design process. (IRC 58 for plain concrete or fibers are used less than 0.3% by volume (with 0.45 stress ratio); IRC 46 for fibers are used in dose more than 0.3% by volume (mainly control plastic shrinkage) or fibers having low toughness (with 0.5 stress ratio), and for fibers having high toughness (with 0.6 stress ratio)).

When the required thickness values given in Table 4.13 were examined in terms of the study conducted by Öztürk (2018), it was seen that the slab thickness requirement increased up to ~26% by using fibers. It should be noted that this value may show an increase or decrease depending on the properties of fiber materials. Despite the use of fibers in the concrete mixture, an increase in the required slab thickness is an undesirable situation.

When the material properties are examined (see Table 4.3), it was seen that the contribution of fibers used in the mixture to the toughness of concrete was low ($f_{e150k}<0.3*f_{ctk}$). Due to the poor performance of the fibers used in both the RCC-S54 and RCC-S36 mixtures, a significant reduction occurred in the flexural strength of the fiber-reinforced concrete compared to the plain concrete. Since one of the main parameters influencing the design is the flexural strength of the concrete, the decrease in strength led to an increase in the required thickness of the concrete slab. It should be noted that, the fiber-reinforced concrete mixtures performed poorly even than the plain concrete mixture. It is obvious that the contribution that can be obtained from the addition of fiber highly depends on the type and amount of used fiber, as well as properties of concrete matrix.

In addition to all of these, the results given in Table 4.13 for the pavement thickness with and without tied concrete shoulder showed that the thickness could be reduced by up to ~16% in the case of slab edge was supported.

On the other hand, when the required thickness values given in Table 4.13 were examined in terms of the study conducted by LaHucik et al. (2017), it was seen that the slab thickness requirement could be reduced by up to 13% by using fiber (these values may show an increase or decrease depending on the properties and amounts of fiber materials).

As expected, the highest performance was achieved with the Hook - 0.2 mixture having the highest flexural strength value and the relatively high contribution to toughness. However, since Smooth - 0.2 fiber contributes very little to the toughness (f_{e150k}<0.3*f_{ctk}) and causes a significant reduction in flexural strength, the use of these fibers led to a greater thickness than the plain concrete mixture (up to %20 increase in required thickness). As a result, it was inferred that the use of fiber-reinforced concrete with poorly optimized mixtures provides a higher or similar required slab thickness than plain concrete. Therefore, as in conventional PCC, to achieve the desired contribution by using fiber in concrete road applications the selection of fiber type and amount, as well as the importance of the design of the concrete matrix are clear.

In addition to all of these, the results given in Table 4.13 for the pavement thickness with and without tied concrete shoulder showed that the thickness could be reduced by up to ~15% in the case of slab edge was supported.

When Table 4.13 was examined in terms of both 2 different RCC studies, it was seen that the thicknesses obtained from the study conducted by Öztürk (2018) gave lower values than the other. Although the fibers used in the study conducted by Öztürk (2018) showed poor performance and caused a decrease in the flexural strength values, it was found that the flexural strength values were still higher than the flexural strength values of all mixtures in the other study conducted by LaHucik et al. (2017). It resulted in lower required thickness values with the mixtures in the study conducted by Öztürk (2018) compared to the other RCC study.

4.3.3. Design for Ultimate Moment Resistance (based on TR 34 -third edition (2003))

In addition to fatigue analysis made for plain and fiber-reinforced concrete, the ultimate moment capacity check was performed for the pavements with FRC having high toughness according to the recommendation of IRC 46 (2013). IRC SP 46 (2013) recommends the use of ASTM C 1609 based on TR 34 (2003) for the characterization of fiber reinforced material. The results of the four point bending test conducted according to ASTM C 1609, f_l (characteristic flexural strength) and f_{e150k} (characteristic equivalent flexural strength), were used for the ultimate moment capacity check. So, the thickness design in terms of ultimate moment resistance was made for the pavements with FRC having high toughness ($f_{e150k} \geq 0.3\ f_{ctk}$) and the results were compared. It should be noted that equations that were used to calculate the results given in the Table 4.14 were given in literature review. The following table shows the thicknesses according to fatigue and ultimate moment resistance.

Table 4.14. Thickness requirements for ultimate moment resistance and fatigue resistance.

Study	Mixture	Required Thickness (cm)		
		Ultimate Moment Resistance (TR 34-third edition (2003))	Fatigue Resistance	
			Without Tied Concrete Shoulder	With Tied Concrete Shoulder
LaHucik et al., 2017 (RCC)	Hook – 0.4 (steel)	12.7	26.3	22.1
Mulheron,2015 (PCC)	Polypropylene Macro – 0.7	10.2	19.7	16.4
	Polypropylene Macro – 1.0	10.3	21.4	17.8
	Steel – 0.9	10.7	23.3	19.4
	Carbon – 0.7	9.4	19.1	15.9
	Carbon – 1.0	9.6	19.8	16.5

As can be seen from the Table 4.14, fatigue resistance of the pavements is the critical case for both with and without concrete shoulder. In addition, when Table 4.14 was examined, it was seen that the lowest thickness value in terms of ultimate moment capacity was obtained for Carbon-0.7 which has the highest flexural strength, and the highest thickness value was obtained for Hook-0.4 which has the lowest flexural strength. Table 4.14 also shows that different results are obtained when different fibers of different types, geometry, surface properties, and dimensions are used.

4.3.4. Preliminary Conclusion for IRC 58 (2011) and IRC SP 46 (2013)

- Dominant parameters and their effects: In the design, it was observed that the flexural strength of concrete (MOR) and stress ratio (which is the function of flexural strength and the equivalent stress caused by the axle load) influenced the obtained thickness values. The higher the flexural strength, the lower the thickness obtained. Also, stress ratios (0.45,0.5 or 0.6) used in the design determined according to the obtained toughness via fibers. The higher the stress ratio, the less the number of repetitions of the load required for cracking. In this study, since the traffic is constant, the expected axle load repetitions are constant. Therefore, an increase in the stress ratio provides a decrease in the required thickness. As a result, relatively low required thicknesses were found in designs using high stress ratio and high flexural strength.

- Slab edge support: Required thickness values could be reduced by up to ~17% in the case of slab edge was supported.

- Use of dowel bars: Since erosion is not a failure mode considered in the present design procedure the presence or absence of dowel bars at the joints is not regarded as a parameter affecting the design thickness. However, in the design guide IRC 46 (2013), the use of dowel bars is recommended for the following situations. If the thickness of the design is less than 200 mm, the use of dowel bars is not considered as reliable.

i) Thickness ≥ 200 mm in the design using fibers with low contribution to toughness,
ii) Thickness ≥ 150 mm in the design using fibers with high contribution to toughness

- Use of fibers: It was seen that the use of fiber could reduce the pavement thickness requirement by up to 28% (these values may increase or decrease depending on the materials). As expected, it was seen that the highest performance could be achieved with mixtures that do not decrease the flexural strength value and also contribute to the toughness. However, it was seen that the use of fiber-reinforced concrete with a poorly optimized mix yields the required pavement thickness values similar to that of plain concrete. (slab thickness requirement increased up to ~26% by using fibers).

4.4. Summary of StreetPave v12 (2014) Methodology

- StreetPave evaluates fatigue through a mechanistic-empirical design, erosion model is based mainly on empirical relationships obtained from field studies. Thus, StreetPave is both mechanistic and empirical.
- StreetPave uses the axle load spectrum which requested in the form of axles/1000 trucks.
- StreetPave considers the presence of tied shoulder in the design and at the end of the design offers a thickness value for two cases with or without dowel bar at joints.
- StreetPave considers the effect of fibers by increasing the flexural strength of concrete by the approach proposed by Roesler et al. (2008).

4.4.1. Thickness Design According to StreetPave v12 (2014)

For the thickness design that was done by using StreetPave v12 (2014), based on the recommendations given in the software, reliability was taken as 85% (depends on the type of roadway). Briefly, StreetPave incorporates the user-entered reliability via probabilistic models, based on accumulated fatigue damage in beam tests, to estimate the probability of obtaining a certain percentage of cracking at a given level of reliability. So, relatively high reliability is used for high traffic, high-speed roadways, while low traffic, low-speed roads typically need a low level of reliability. It should be noted that as reliability increases, the probability of failure decreases. Also, according to terminal serviceability and the type of roadway the percentage of slab cracked at the end of the design life was taken as 5%. Also, terminal serviceability (that is the point at which the pavement needs rehabilitation of some sort) was taken as 2.5 however, the value of this input does not impact the required thickness for StreetPave rigid pavement designs; this input is only used to calculate the equivalent single axle loads (ESALs) as in AASHTO (1993) guideline. To complete the design according to software traffic, foundation, road, and material related parameters were taken from the tables mentioned previously. The values of other parameter used in the design process of StreetPave were given in the table below.

Table 4.15. Design variables for StreetPave v12 (2014).

Parameter	Value
Reliability (R) (%)	85
Percentage of slabs cracked at the end of design life (%)	5
Terminal Serviceability (P_t)	2.5

The software suggests thickness results for the case with and without dowel bars depending on the presence of the edge support. In this study, to see the effect of usage of concrete shoulder on the thickness requirement of the pavement, thickness requirements were determined for both with and without concrete shoulder cases. As mentioned earlier, StreetPave considers the effect of fibers by increasing the flexural strength of concrete by the approach proposed by Roesler et al. (2008).

In the PCA (1984) design guideline, it was stated that the fatigue resistance of pavement will be critical case for the roads that carry low (with or without dowel bars) or medium (with dowel bars) traffic; and erosion resistance of the pavement will be the critical case for the roads that carry medium (without dowel bars) or high (with or without dowel bars) traffic loads. In the design (based on PCA (1984)), the pavement structure is controlled according to both failure modes (fatigue and erosion) and the thickness design is made according to the critical value.

The thickness values given from Table 4.16 to 4.18 indicated that the critical failure mode for each mixture (for RCCs and conventional PCC) was erosion (pumping/corner deflection/faulting) not fatigue (traditional load-related cracking). This is because the higher thickness requirement values were found for erosion analysis.

However, correspondence with the ACPA during the thesis study indicates that the erosion model was developed before the fibers were widely used in full depth concrete pavements, and thus the effects of fibers were not considered within the model. Furthermore, when the thickness values given in Table 4.16 to 4.18 are examined, it can be observed that the use of fibers for both RCC and conventional PCC has no effect on erosion control. It is clear that StreetPave considers the positive effect of fibers only in case of fatigue failure by increasing the flexural strength of concrete.

When the thickness values given in Tables 4.16 to 4.18 were examined, it was seen that thickness values were calculated for fatigue. As mentioned above, StreetPave gives the required thickness value for fatigue or erosion which constitutes the critical situation (erosion for all mixtures in this study). However, to see the effects of fibers in terms of fatigue resistance, the thickness requirements based on fatigue analysis were also separately determined using a spread sheet. To create the spreadsheet, the equations in the study conducted by Lee and Carpenter (2001) were used. In addition, to consider the fiber effect in fatigue analysis the approach proposed by Roesler et al. (2008), also used by StreetPave, was considered when creating the spreadsheet. It should be noted that the equations in the study conducted by Lee and Carpenter (2001) were established to create tables and charts developed by PCA (1984).

4.4.1.1. Results for Plain RCC and Conventional PCC. The slab thicknesses were compared in terms of plain RCC and conventional PCC pavements both with and without tied concrete shoulder. Since erosion resistance of the pavements were found as critical design case for all the mixtures, thickness values were evaluated in terms of erosion. In addition, although StreetPave provides both doweled and undoweled design thicknesses, the studies were compared on undoweled design thicknesses since dowel bars aren’t used in RCC pavements. So, the contribution of dowel bars to the load transfer doesn’t examined here. In fact, StreetPave proposes to use 5 inches as the minimum undoweled pavement thickness and 6 inches as the minimum thickness for doweled concrete to ensure coverage over the dowel bars.

Table 4.16. Comparison of plain RCC and Conventional PCC in terms of StreetPave.

Study	Mixture	Required Thickness (cm)	
		Without Tied Concrete Shoulder	With Tied Concrete Shoulder
		Without Dowel Bar	Without Dowel Bar
		Erosion	Erosion
Öztürk,2018 (RCC)	RCC Control-1	36.3	29.6
LaHucik et al., 2017 (RCC)	RCC Control-2	38.0	31.0
Mulheron, 2015 (PCC)	PCC Control	38.1	31.0

It was mentioned earlier that the equations in the study conducted by Lee and Carpenter (2001) were used in spreadsheets created to consider the fiber effect in fatigue analysis. In the same study, the equation set for erosion was also given. These equations established to create the tables and charts developed by PCA 1984. It should be noted that, design method of StreetPave software was developed based on the PCA 1984. Therefore, it is appropriate to consider the equations in the study conducted by Lee and Carpenter (2001) when evaluating the required thickness values calculated for StreetPave. It is possible to accurately evaluate the thickness values given in Table 4.16 by examining the set of equations given for erosion in the study conducted by Lee and Carpenter (2001).

As mentioned earlier, erosion is a mode of failure mainly caused by the deformation of the layer under the concrete pavement slab. Therefore, although the flexural strength of concrete is an important parameter directly affecting the required thickness in fatigue analysis, it has no effect on the required thickness in erosion analysis. When the erosion equation was examined, it was seen that the parameters that affect the required thickness values depend mainly on the presence of edge support and dowel bar, modulus of subgrade reaction, applied axle load, and radius of relative stiffness. When the Table 4.16 were examined, it was seen that the required thickness values for plain concrete mixtures of 3 different studies were almost the same. Minor differences in thicknesses were due to differences in the modulus of elasticity of concrete used to calculate the radius of relative stiffness, which is one of the parameters affecting the erosion analysis.

In addition, the results given in Table 4.16 for the pavement slab with and without tied concrete shoulder showed that the thickness could be reduced by up to 19% if the slab edge was supported.

4.4.1.2. Results for Fiber-Reinforced Conventional PCC. The slab thicknesses were compared in terms of fiber-reinforced conventional PCC both with and without tied concrete shoulder. Also, the thickness requirement was evaluated in terms of the presence of dowel bars. The required thickness values obtained by the spreadsheet were given in the table below to observe the contribution of the fibers used in different types and amounts. The required pavement thickness values (for with and without concrete shoulder) determined for each mixture (for traffic and foundation parameters used) using StreetPave v12 (2014) were given in Table 4.17. It should be emphasized here that results of the fiber reinforced PCC mixtures were compared with PCC control mixture.

Table 4.17. Comparison of fiber reinforced conventional PCC in terms of StreetPave.

Study	Mixture	Required Thickness (cm)					
		Without Tied Concrete			With Tied Concrete Shoulder		
		With Dowel Bar	Without Dowel Bar	With or Without Dowel Bar	With Dowel Bar	Without Dowel Bar	With or Without Dowel Bar
		Erosion	Erosion	Fatigue	Erosion	Erosion	Fatigue
Mulheron, 2015 (PCC)	PCC Control	31.4	38.1	22.1	24.4	31.0	19.8
	Polypropylene Macro – 0.5*	30.8	37.5	19.3	24.0	30.6	17.2
	Polypropylene Macro – 0.7*	30.8	37.5	20.0	24.0	30.5	17.9
	Polypropylene Macro – 1.0*	31.3	38.1	20.4	24.4	31.0	18.2
	Steel – 0.9*	31.4	38.1	22.5	24.5	31.1	20.2
	Carbon – 0.3*	31.4	38.2	19.6	24.5	31.0	17.5
	Carbon – 0.7*	30.8	37.4	18.4	24.0	30.5	16.3
	Carbon – 1.0*	31.3	38.1	19.2	24.4	31.0	17.0

* represents the used fiber amount by volume (%)

First, when Table 4.17 was examined, it was observed that the design for all mixtures in StreetPave were driven by the erosion analysis (as higher thickness values were found for erosion analysis). Also, it was seen that similar thickness requirement values obtained for plain and fiber reinforced concrete mixtures in terms of erosion analysis. This is because the fiber effect was not considered in the erosion calculations used in the StreetPave design approach. However, the values given in the table for the erosion failure mode "with and without dowel bar" cases reveal the importance of dowel bars and load transfer in terms of erosion. The small differences in thicknesses obtained from the erosion analysis resulted from the differences in the modulus of elasticity (E) of the concrete used to calculate the radius of relative stiffness (l), which is one of the parameters affecting the erosion analysis. However, the modulus of elasticity (E) of the concrete doesn't truly represent the contribution of fibers, as the fibers contribute especially in the post cracking region. Therefore, one could expect fibers to act like dowels in pavements. When Table 4.17 was examined, it was seen that the presence of the dowel bar results in a reduction of approximately 18% for "without tied concrete shoulder" case and 22% for "with tied concrete shoulder" case in required thickness values. Dowel bars may be eliminated or decreased if fiber reinforced concretes are employed in design. Considering all of these, it is clear that StreetPave v12 (2014) should be revised in terms of erosion resistance analysis of FRC pavements by considering load transfer (micro-dowel) action of fibers.

Also, as mentioned previously, thickness values for fatigue failure were calculated by generated spreadsheet only to see the contribution of fibers to that failure mode. When the required thickness values given in Table 4.17 determined for fatigue damage are examined, it is seen that the thickness requirement can be reduced by up to 17% by using fiber (for the traffic and material properties used in this study). As expected, the highest performance can be achieved with Carbon – 0.7, a mixture that significantly contributes to flexural strength (MOR) and having relatively high residual flexural strength value. However, it was seen that the use of Steel - 0.9 which was a poorly optimized mix gave the required slab thickness values similar to that of plain concrete. Therefore, in order to achieve the desired contribution by using fiber in concrete road applications, the selection of fiber type and amount, as well as the importance of the concrete matrix design are clear.

In addition to all of these, the thickness requirement values given for "with and without tied concrete shoulder" showed that the required thickness could be reduced slightly in the case of the slab edge was supported. (For erosion damage analysis (~20%)), and for fatigue damage analysis (~10%)).

4.4.1.3. Results for Fiber-Reinforced RCC. In this section, the thickness values obtained for 2 different RCC studies were compared separately in terms of fiber-reinforced RCC both with and without tied concrete shoulder. It should be noted that, the thickness requirement wasn't evaluated in terms of the presence of dowel bars since the dowel bars aren't used in RCC. The required pavement thickness values (for with and without concrete shoulder) determined for each mixture (for traffic and foundation parameters used) using StreetPave v12 (2014) were given in Table 4.18. It should be emphasized here that results of the fiber reinforced RCC mixtures were compared with RCC control mixture.

Table 4.18. Comparison of fiber-reinforced RCC in terms of StreetPave.

Study	Mixture	Required Thickness (cm)			
		Without Tied Concrete Shoulder		With Tied Concrete Shoulder	
		Without Dowel Bar	With or Without Dowel Bar	Without Dowel Bar	With or Without Dowel Bar
		Erosion	Fatigue	Erosion	Fatigue
Öztürk, 2018 (RCC)	RCC Control-1	36.3	18.8	29.6	16.8
	RCC-S54 (synthetic) *[b]	37.0	19.7	30.1	17.6
	RCC-S36 (synthetic) *[b]	37.7	20.3	30.7	18.1
LaHucik et al., 2017 (RCC)	RCC Control-2	38.0	25.1	31.0	22.8
	Emboss - 0.2*[a] (synthetic)*[b]	37.7	25.7	30.7	23.3
	Emboss - 0.4*[a] (synthetic) *[b]	38.7	22.3	31.4	20.0
	Smooth - 0.2*[a] (synthetic) *[b]	38.4	26.9	31.2	24.4
	Smooth - 0.4*[a] (synthetic) *[b]	38.3	25.4	31.2	23.0
	Hook - 0.2*[a] (steel) *[b]	38.0	20.0	30.9	17.8
	Hook - 0.4*[a] (steel) *[b]	38.3	19.8	31.1	17.6

*[a] represents the used fiber amount by volume (%)

*[b] represents the used fiber type

First, when the required thickness values given in Table 4.18 were examined in terms of the study conducted by Öztürk (2018), it was observed that the design for all mixtures in StreetPave were driven by the erosion analysis. (as higher thickness values were found for erosion analysis). Also, it was seen that similar thickness requirement values obtained for plain and fiber reinforced concrete mixtures in terms of erosion analysis. This is because the fiber effect is not considered in the erosion calculations used in the StreetPave design approach. As mentioned before, the small differences in thicknesses obtained from the erosion analysis resulted from the differences in the modulus of elasticity (E) of the concrete used to calculate the radius of relative stiffness (l), which is one of the parameters affecting the erosion analysis. However, the modulus of elasticity (E) of the concrete doesn't truly represent the contribution of fibers, as the fibers contribute especially in the post cracking region. Considering all of these, it is clear that StreetPave v12 (2014) should be revised in terms of erosion resistance analysis of FRC pavements by considering load transfer (micro-dowel) action of fibers.

Also, as mentioned previously, thickness values for fatigue failure were calculated by generated spreadsheet only to see the contribution of fibers to that failure mode. When the required thickness values given in Table 4.18 for fatigue damage were examined, it was seen that the thickness requirement increases (8%) with the use of fibers contrary to expectations. Although a high volume of fiber (0.5%) was used, the highest thickness value was obtained with RCC-S36, a mixture which significantly reduced the flexural strength value compared to the flexural strength of the plain concrete. It was seen that the use of a fiber-reinforced concrete with poorly optimized mixtures resulted in similar to or higher required pavement thickness than that of plain concrete. Therefore, in order to achieve the desired contribution by using fiber in concrete road applications, the selection of fiber type and quantity, as well as the importance of the concrete matrix design are clear.

On the other hand, when the required thickness values given in Table 4.18 were examined in terms of the study conducted by LaHucik et al. (2017), it was observed that the design for all mixtures in StreetPave were driven by the erosion analysis. (as higher thickness values were found for erosion analysis). Also, it was seen that similar thickness requirement values obtained for plain and fiber reinforced concrete mixtures in terms of erosion analysis. This is because the fiber effect is not considered in the erosion calculations used in the StreetPave design approach. As mentioned above, the small differences in thicknesses obtained from the erosion analysis resulted from the differences in the modulus of elasticity (E) of the concrete used to calculate the radius of relative stiffness (l), which is one of the parameters affecting the erosion analysis. However, the modulus of elasticity (E) of the concrete doesn’t truly represent the contribution of fibers, as the fibers contribute especially in the post cracking region. Considering all of these, it is clear that StreetPave v12 (2014) should be revised in terms of erosion resistance analysis of FRC pavements by considering load transfer (micro-dowel) action of fibers.

Also, as mentioned previously, thickness values for fatigue failure were calculated by generated spreadsheet only to see the contribution of fibers to that failure mode. When the required thickness values given in Table 4.18 determined for fatigue damage were examined, it was seen that the thickness requirement could be reduced by up to 23% by using fiber (for the traffic and material properties used in this study). As expected, the highest performance can be achieved with Hook - 0.4, a mixture that does not reduce the flexural strength value slightly and also contributes to a high degree of toughness. However, it was seen that the use of a fiber-reinforced concrete with poorly optimized mixtures results in similar to or higher required pavement thickness than that of plain concrete. In this study, the Smooth-0.2 fibers used in the study clearly demonstrate this situation. When the previously given material parameters table for RCC is examined, it was seen that Smooth-0.2 fiber was used in low volume (0.2%) and its contribution to toughness was low. Thus, the Smooth-0.2 fiber could not achieve the desired performance because it caused a decrease in the flexural strength of concrete and was a poorly optimized mixture. Therefore, in order to achieve the desired contribution by using fiber in concrete road applications, the selection of fiber type and quantity, as well as the importance of the concrete matrix design are clear.

In addition to all of these, the thickness requirement values given for "with and without tied concrete shoulder" showed that the required thickness can be reduced slightly in the case of the slab edge was supported. (For erosion damage analysis (~20%)), and for fatigue damage analysis (~10%)).

4.4.1.4. Preliminary Conclusion for StreetPave v12 (2014). Dominant parameters and their effects: It should be noted that, the design for all mixtures in StreetPave were driven by the erosion analysis (as higher thickness values were found for erosion analysis). So, the small differences in thicknesses obtained from the erosion analysis resulted from the differences in the modulus of elasticity (E) of the concrete. However, if the failure mode were fatigue, the most important parameter affecting the design would be flexural strength of concrete (MOR).

Slab edge support: Required thickness values could be reduced by up to ~20% for erosion analysis and ~10% for fatigue analysis in the case of slab edge was supported.

Use of dowel bars: At the end of analysis the software proposes a required thickness for both doweled and undoweled case. In fact, StreetPave proposes to use 5 inches as the minimum undoweled pavement thickness and 6 inches as the minimum thickness for doweled concrete to ensure coverage over the dowel bars. It should be noted that, RCC aren't used dowel bars.

Use of fibers: It should be noted that the fiber effect is not considered in the erosion calculations used in the StreetPave design approach. So, thickness values for fatigue failure were calculated by generated spreadsheet only to see the contribution of fibers to that failure mode. It was seen that the thickness requirement could be reduced by up to 23% by using fiber (for the traffic and material properties used in this study). As expected, it was seen that the highest performance could be achieved with mixtures that do not decrease the flexural strength value and also contribute to the toughness. However, it was seen that the use of a fiber-reinforced concrete with poorly optimized mix yields the required pavement thickness values similar to that of plain concrete (slab thickness requirement increased up to ~7% by using fibers).

4.5. Thickness Requirements and Comparison of Results in terms of 3 Design Methodologies

Table 4.19 shows a comparison of thickness design among the StreetPave v12 (2014), IRC SP 46 (2013), and AASHTO (1993) based on the studies of Öztürk, 2018, LaHucik,2017 and Mulheron,2014 which consist of RCC and PCC pavements for plain and various fiber volume. Thickness comparisons for pavements were given in parts for each methodology for clarity in the previous sections. Here again, the calculated thickness values were compared in terms of 3 methodologies. Also, considerations in design and comparisons are explained under the "design notes".

Table 4.19. Thickness design requirements (StreetPave v12 (2014), IRC 58 (2011)-IRC SP 46 (2013) and AASHTO (1993)).

Study	Mixture	STREETPAVE V12 (2014)						IRC 58 (2011) -IRC SP 46 (2013)			AASHTO (1993)			
		Without Tied Concrete Shoulder			With Tied Concrete Shoulder			Without Tied Concrete Shoulder	With Tied Concrete Shoulder	Standard (Stress Ratio)	Without Tied Concrete Shoulder		With Tied Concrete Shoulder	
		With Dowel Bar	Without Dowel Bar	With or Without Dowel Bar	With Dowel Bar	Without Dowel Bar	With or Without Dowel Bar	With or Without Dowel Bar	With or Without Dowel Bar		With Dowel Bar	Without Dowel Bar	With Dowel Bar	Without Dowel Bar
		Required Thickness (cm)						Required Thickness (cm)			Required Thickness (cm)			
		Erosion	Erosion	Fatigue	Erosion	Erosion	Fatigue	Fatigue	Fatigue		Fatigue & Erosion	Fatigue & Erosion	Fatigue & Erosion	Fatigue & Erosion
Mulheron, 2015 (PCC)	PCC Control	31.4	38.1	22.1	24.4	31.0	19.8	26.0	22.0	IRC 58 (0.45)	38.5	44.6	35.8	42.2
	Polypropylene Macro – 0.5	30.8	37.5	19.3	24.0	30.6	17.2	20.8	17.4	IRC 46 (0.5)	29.0	33.8	26.8	32.0
	Polypropylene Macro – 0.7	30.8	37.5	20.0	24.0	30.5	17.9	19.7	16.4	IRC 46 (0.6)	27.2	31.9	25.0	30.1
	Polypropylene Macro – 1.0	31.3	38.1	20.4	24.4	31.0	18.2	21.4	17.8	IRC 46 (0.6)	28.2	33.0	26.0	31.2
	Steel – 0.9	31.4	38.1	22.5	24.5	31.1	20.2	23.3	19.4	IRC 46 (0.6)	27.9	32.8	25.8	31.0
	Carbon – 0.3	31.4	38.2	19.6	24.5	31.0	17.5	22.3	18.7	IRC 46 (0.5)	29.6	34.6	27.4	32.7
	Carbon – 0.7	30.8	37.4	18.4	24.0	30.5	16.3	19.1	15.9	IRC 46 (0.6)	25.4	29.8	23.3	28.2
	Carbon – 1.0	31.3	38.1	19.2	24.4	31.0	17.0	19.8	16.5	IRC 46 (0.6)	28.3	33.1	26.2	31.3

Table 4.19. Thickness design requirements (StreetPave v12 (2014), IRC 58 (2011)-IRC SP 46 (2013) and AASHTO (1993)) cont.

Öztürk, 2018 (RCC)	RCC Control-1	-	36.3	18.8	-	29.6	16.8	19.2	16.1	IRC 58 (0.45)	-	38.1	-	35.9
	RCC-S54 (synthetic)	-	37.0	19.7	-	30.1	17.6	22.2	18.7	IRC 46 (0.5)	-	37.1	-	35.1
	RCC-S36 (synthetic)	-	37.7	20.3	-	30.7	18.1	24.3	20.4	IRC 46 (0.5)	-	37.7	-	35.6
LaHucik et al., 2017 (RCC)	RCC Control-2	-	38.0	25.1	-	31.0	22.8	29.4	25.3	IRC 58 (0.45)	-	47.8	-	45.2
	Emboss - 0.2 (synthetic)	-	37.7	25.7	-	30.7	23.3	31.4	27.6	IRC 58 (0.45)	-	46.7	-	44.1
	Emboss - 0.4 (synthetic)	-	38.7	22.3	-	31.4	20.0	30.6	26.0	IRC 46 (0.5)	-	42.4	-	40.0
	Smooth - 0.2 (synthetic)	-	38.4	26.9	-	31.2	24.4	34.3	30.5	IRC 58 (0.45)	-	48.3	-	45.7
	Smooth - 0.4 (synthetic)	-	38.3	25.4	-	31.2	23.0	31.7	27.5	IRC 46 (0.5)	-	45.7	-	43.2
	Hook - 0.2 (steel)	-	38.0	20.0	-	30.9	17.8	26.1	22.0	IRC 46 (0.5)	-	38.6	-	36.4
	Hook - 0.4 (steel)	-	38.3	19.8	-	31.1	17.6	26.3	22.1	IRC 46 (0.6)	-	37.7	-	35.6

4.5.1. Design Notes

- Dowel bars aren't used in RCC pavements. Since the thickness values for "with dowel bar" column of RCC was represented as "-".
- StreetPave calculates thickness only for critical failure mode (either fatigue or erosion). The failure mode of all mixtures in this study was determined as erosion. Since it is known that fibers only benefit fatigue failure, thicknesses for the fatigue failure were calculated to observe the effect of fibers on thickness requirements with the help of a generated spreadsheet.
- The "Standard and Stress Ratio" column in Table 4.19 represents the design guidelines and stress ratios used for each mixture in design process.
    - Limiting stress ratio for plain concrete pavement is 0.45 (IRC 58,2015),
    - Pavements with FRC of low toughness ($f_{e150k}<0.3*f_{ctk}$) shall be designed as non-reinforced pavements (IRC SP 46,2013),
    - If polymeric fibers ere used in dose less than 0.3% by volume, for fatigue endurance, the limiting stress ratio should be taken as 0.45 (as in IRC 58,2015),
    - If polymeric fibers are used in dose more than 0.3% by volume, or in FRC with steel or any other fibers having low toughness ($f_{e150k}<0.3*f_{ctk}$), for fatigue endurance, the limiting stress ratio should be taken as 0.5 (IRC SP 46,2013),
    - Pavements with FRC having high toughness ($f_{e150k}>0.3*f_{ctk}$) shall be designed as per the procedure given in IRC 58. For fatigue endurance, the limiting stress ratio will be taken as 0.6 (IRC SP 46,2013).
- IRC SP 46 (2013) suggests the use of load transfer devices such as dowel bars (to reduce the stresses due to edge load) in the case of as follows;
    - if the pavement thickness ≥ 200 mm for pavement with FRC having low toughness ($f_{e150k}<0.3*f_{ctk}$) or,
    - if the pavement thickness ≥ 150 mm for pavement with structural FRC having high toughness ($f_{e150k}>0.3*f_{ctk}$),
    - if the pavement thickness < 200 mm the use of such devices isn't considered as reliable.
- When Table 4.19 is examined in terms of IRC designs it is seen that;
    - All the calculated thickness values for control concrete and having low toughness FRCs ≥ 200 mm and,

- All the calculated thickness values for having high toughness FRCs ≥ 150 mm

It should be noted that presence of dowel bars doesn't change the required thickness values. It means that IRC considers the dowel bars have no impact in the design procedure of fatigue.

- In the design procedure of IRC 58 (2011), 28-day mean flexural strength values of concrete were reduced by 30 percent (in the absence of enough data for the determination of the characteristic value) and characteristic flexural strength values were used. Also, design guide recommended the use of 90-day characteristic flexural strength for thickness calculation. Increasing the 28-day flexural strength by a factor of 1.1 may be used to get 90-day strength. Also, design guide emphasized that in no case should 28-day flexural strength of pavement quality concrete be less than 4.5 MPa. However, in the study conducted by LaHucik (2017) and Mulheron (2015) for RCC and conventional PCC, 28-day flexural strength values of some concretes were found to be less than 4.5 MPa. In the design, the average flexural strength value was used, since the use of characteristic flexural strength would further reduce the flexural strengths even below 4.5 MPa. Therefore, mean flexural strength was used instead of characteristic flexural strength throughout the design in order to make the design properly.
- In the design procedure of AASHTO (1993) it is stated that in order to apply properly the reliability factor (R), the inputs in the design equation should be the mean value, without any adjustment designed to make the input "conservative."
- It should be noted that Altoubat et al. (2008) approach is recommended for concrete pavement with a low volume fiber fraction (below 0.5%) (providing equivalent flexural strength ratio between 20 % and 50%). However, in the thesis study some of the FRC materials used (such as Polypropylene Macro-0.7, Polypropylene Macro-1, Steel-0.9 and Carbon-0.7 which have $R_{e,3}$ value 60.1, 55, 80.8 and 60 from the study conducted by Mulheron, (2014) respectively and Hook-0.4 which have $R_{e,3}$ value 59,8 from the study conducted by LaHucik et al., (2017)) give equivalent flexural strength ratio ($R_{e,3}$) higher than 50%. To observe the effects of using that have high toughness FRC within the approach of Altoubat et al. these FRCs were employed.

4.5.2. Evaluation of the Results in terms of 3 Design Methodologies

In the previous sections, the required thickness values obtained for conventional PCC and RCC pavements were evaluated separately for each methodology. The assessment was made for each methodology based on the following parameters;

- the effect of using different types of concrete on thickness requirement,
- the effect of using different types and amounts of fiber on thickness requirements,
- the effect of using dowel bar and edge support on thickness requirement, and
- dominant parameters in each methodology

In this section, the reasons for the differences in calculated thickness values for conventional PCC and RCC mixtures based on the considered methodology were examined. This comparison aims to explore the reasons for the required thickness values varying according to the methodologies despite the use of the same road, traffic, foundation, and material properties. Therefore, to make a comparison, the calculated required thicknesses according to 3 methodologies were given in the same table. Also, comparisons were made separately in terms of conventional PCC and RCC pavements.

4.5.2.1. Results for Fiber-Reinforced Conventional PCC. The required thickness values calculated for conventional PCC pavements were compared in terms of 3 design methodologies. For clarity of comparison, the required thickness values were given in 2 tables for the presence or absence of edge support. In addition, IRC SP 46 (2013) design guide recommends the use of dowel bars for slab thicknesses of more than a particular thickness value. Therefore, the thickness comparisons were made for the "with dowel bar" case. Table 4.20 shows the thickness values obtained as a result of the design made according to 3 methodologies for conventional PCC for with tied concrete shoulder case. In addition, Table 4.21 shows the thickness values obtained as a result of the design made according to 3 methodologies for conventional PCC for without tied concrete shoulder case. To understand the difference in results between each design method, it is necessary to understand the basis of equations how and when various material, structural, environmental, etc. properties affect pavement performance, and the basis for any data used to calibrate the design equations.

Table 4.20. Thickness requirements for conventional PCC with tied concrete shoulder.

		STREETPAVE v12 (2014)	**IRC 58 (2011) – IRC SP 46 (2013)**	**AASHTO (1993)**
Study	Mixture	Required Thickness (cm)		
		With Tied Concrete Shoulder	With Tied Concrete Shoulder	With Tied Concrete Shoulder
		With Dowel Bar	With or Without Dowel Bar	With Dowel Bar
		Erosion	Fatigue	Erosion & Fatigue
Mulheron, 2015 (PCC)	PCC Control	24.4	22.0	35.8
	Polypropylene Macro – 0.5	24.0	17.4	26.8
	Polypropylene Macro – 0.7	24.0	16.4	25.0
	Polypropylene Macro – 1.0	24.4	17.8	26.0
	Steel – 0.9	24.5	19.4	25.8
	Carbon – 0.3	24.5	18.7	27.4
	Carbon – 0.7	24.0	15.9	23.3
	Carbon – 1.0	24.4	16.5	26.2

Table 4.21. Thickness requirements for conventional PCC without tied concrete shoulder.

		STREETPAVE v12 (2014)	**IRC 58 (2011) – IRC SP 46 (2013)**	**AASHTO (1993)**
Study	Mixture	Required Thickness (cm)		
		Without Tied Concrete Shoulder	Without Tied Concrete Shoulder	Without Tied Concrete Shoulder
		With Dowel Bar	With or Without Dowel Bar	With Dowel Bar
		Erosion	Fatigue	Erosion & Fatigue
Mulheron, 2015 (PCC)	PCC Control	31.4	26.0	38.5
	Polypropylene Macro – 0.5	30.8	20.8	29.0
	Polypropylene Macro – 0.7	30.8	19.7	27.2
	Polypropylene Macro – 1.0	31.3	21.4	28.2
	Steel – 0.9	31.4	23.3	27.9
	Carbon – 0.3	31.4	22.3	29.6
	Carbon – 0.7	30.8	19.1	25.4
	Carbon – 1.0	31.3	19.8	28.3

In Table 4.20 and 4.21, when the results obtained for plain concretes according to all 3 design methodologies were examined, it was seen that the maximum thickness values were obtained by AASHTO (1993) and the lowest thickness values were obtained by IRC guidelines. The main reason for this is that the 28-day mean flexural strength was used in AASHTO (1993) guide, whereas the 90-day mean flexural strength was used in IRC methodologies. It should be noted that the strength of concrete increases with age. However, obtaining higher thickness values for AASHTO was supported by a similar study conducted by Li et al (2011). Accordingly, Li et al. (2011) concluded that AASHTO (1993) provided reasonable structural designs for flexible pavements using a more realistic layer coefficient for flexible pavements, but that rigid pavements exhibited extremely thick slabs for the inputs given in the AASHTO (1993) guideline. It was also seen that the critical failure mode for StreetPave was erosion. As previously mentioned, the thickness values for erosion analysis only show the effect of fibers on the elastic modulus (E) of the concrete. Another reason why the thicknesses for plain concretes were different from each other is that the methodologies consider different performance criteria. When Tables 4.20 and 4.21 were examined, it was seen that each methodology provided the required thickness value for different critical failure modes as follows.

- IRC guidelines only calculates thickness for fatigue failure mode,
- StreetPave calculates thickness for both fatigue and erosion and gives a single value for critical situation, and
- AASHTO gives a single thickness value calculated by empirical formula based on loss of serviceability concept, including distress modes such as fatigue and erosion.

Therefore, the thicknesses calculated for the different critical failure modes for each methodology were compared with each other.

On the other hand, when Table 4.20 and 4.21 are examined, it was seen that the required slab thicknesses calculated according to StreetPave were similar, although different types and amounts of fibers were used. This is because the StreetPave does not consider the contribution of fibers in the erosion analysis. Therefore, the thickness values given for the erosion analysis show only the effects of the fibers on the modulus of elasticity (E) of the concrete. However, it should be noted that the modulus of elasticity (E) of the concrete doesn’t show the contribution of the fibers to the behavior of the concrete, since the fibers contribute to the behavior of the concrete, especially in the post cracking region. However, if the contribution of the fibers to erosion analysis were considered, the required thicknesses would be reduced relative to the performance of the fibers.

The IRC SP 46 (2013) guideline considers the contribution of fibers to the fatigue life of the concrete slab by increasing the stress ratio relative to the toughness provided by the fibers. As mentioned above, the 90-day mean flexural strength of concrete was used in the calculation of the thicknesses obtained for the IRC shown in the table. Therefore, the required thickness values calculated according to IRC guidelines are less than that of StreetPave and AASHTO, since the 28-day mean flexural strength of concrete is used instead of 90-day. Each thickness value calculated for IRC design guides was calculated for different stress ratios (0.45 for plain concrete, 0.5 or 0.6 for fiber-reinforced concrete), which were determined according to the performance of the fibers.

It should be noted that the required thickness values for AASHTO (1993) represents both erosion and fatigue. In addition, when Table 4.20 was examined, it was observed that although the effect of fibers was taken into consideration by approach of Altoubat et al., it still gave the highest required thickness. For fiber-reinforced concrete, the thickness values calculated according to AASHTO (1993) still gave the highest values (as in plain concrete), although the effect of the fibers was considered.

When the thickness values obtained using StreetPave were examined, it was seen that similar results were obtained with the thickness values obtained using AASHTO (1993).

However, in AASHTO (1993) the contribution of fibers was taken into account by the Altoubat et al. approach, while the contribution of fibers in StreetPave was not taken into account in the critical failure mode of erosion analysis. If StreetPave had considered fiber contribution in the erosion analysis, the thickness values to be obtained would be close to those calculated by IRC guidelines.

Furthermore, when the thickness values in Tables 4.20 and 4.21 were evaluated together, the following results were observed.

If the slab edge was supported;

- the required thickness values calculated according to StreetPave could be reduced by up to ~ 22%,
- the required thickness values calculated according to IRC design guidelines could be reduced by up to ~ 17%,
- the required thickness values calculated according to AASHTO (1993) design guidelines could be reduced by up to ~ 8%.

4.5.2.2. Results for Fiber-Reinforced RCC. The required thickness values calculated for 2 different RCC pavements were compared in terms of 3 design methodologies. For clarity of comparison, the required thickness values were given in 2 tables for the presence and absence of edge support. Table 4.22 shows the thickness values obtained as a result of the design made according to 3 methodologies for 2 different RCC pavements for with tied concrete shoulder case. In addition, Table 4.23 shows the thickness values obtained as a result of the design made according to 3 methodologies for 2 different RCC pavements for without tied concrete shoulder case. It should be noted that since dowel bars aren't used in RCC pavements, comparisons were made for "without dowel bar" case.

Table 4.22. Thickness requirements for RCC with tied concrete shoulder.

Study	Mixture	**STREETPAVE v12 (2014)**	**IRC 58 (2011) – IRC SP 46**	**AASHTO (1993)**
		Required Thickness (cm)		
		With Tied Concrete	With Tied Concrete	With Tied Concrete
		Without Dowel Bar	With or Without Dowel Bar	Without Dowel Bar
		Erosion	Fatigue	Erosion & Fatigue
Öztürk, 2018 (RCC)	RCC Control-1	29.6	16.1	35.9
	RCC-S54 (synthetic)	30.1	18.7	35.1
	RCC-S36 (synthetic)	30.7	20.4	35.6
LaHucik et al., 2017 (RCC)	RCC Control-2	31.0	25.3	45.2
	Emboss - 0.2	30.7	27.6	44.1
	Emboss - 0.4	31.4	26.0	40.0
	Smooth - 0.2	31.2	30.5	45.7
	Smooth - 0.4	31.2	27.5	43.2
	Hook - 0.2 (steel)	30.9	22.0	36.4
	Hook - 0.4 (steel)	31.1	22.1	35.6

Table 4.23. Thickness requirements for RCC without tied concrete shoulder.

Study	Mixture	STREETPAVE V12 (2014)	IRC 58 (2011) – IRC SP 46 (2013)	AASHTO (1993)
		Required Thickness (cm)		
		Without Tied Concrete Shoulder	Without Tied Concrete Shoulder	Without Tied Concrete Shoulder
		Without Dowel Bar	With or Without Dowel Bar	Without Dowel Bar
		Erosion	Fatigue	Erosion & Fatigue
Öztürk, 2018 (RCC)	RCC Control-1	36.3	19.2	38.1
	RCC-S54	37.0	22.2	37.1
	RCC-S36	37.7	24.3	37.7
LaHucik et al., 2017 (RCC)	RCC Control-2	38.0	29.4	47.8
	Emboss - 0.2 (synthetic)	37.7	31.4	46.7
	Emboss - 0.4 (synthetic)	38.7	30.6	42.4
	Smooth - 0.2 (synthetic)	38.4	34.3	48.3
	Smooth - 0.4 (synthetic)	38.3	31.7	45.7
	Hook - 0.2 (steel)	38.0	26.1	38.6
	Hook - 0.4 (steel)	38.3	26.3	37.7

When Table 4.22 and 4.23 were examined, it was seen that the same results were obtained with the conventional PCC evaluated in the previous section. Furthermore, when the thickness values in Tables 4.22 and 4.23 were evaluated together, the following results were observed.

If the slab edge was supported;

- the required thickness values calculated according to StreetPave could be reduced by up to ~ 19%,
- the required thickness values calculated according to IRC design guidelines could be reduced by up to ~ 16%,
- the required thickness values calculated according to AASHTO (1993) design guidelines could be reduced by up to ~ 6%.

<u>4.5.2.3. Results for Fiber-Reinforced Conventional PCC and RCC.</u> As mentioned earlier, the material properties used in this study were taken from three different studies conducted

for RCC and conventional PCC in the literature to see the effects of concrete matrix on the effectiveness of fibers. In this section, the thickness values obtained for fiber-reinforced RCC and conventional PCC according to 3 design methodologies were evaluated in terms of two different concrete types.

Since StreetPave doesn't consider the contribution of fibers in erosion design, similar thickness values were found for both fiber-reinforced RCC and conventional PCC. So, to see the contribution of fibers, required thickness values in terms of fatigue resistance given in Table 4.19 was examined. Accordingly, it was seen that required thickness of the concrete pavement could be reduced by up to 23% (LaHucik et al. 2017). However, despite the use of fibers, the required thickness of the concrete pavement increased by up to 8% (Öztürk, 2018) due to the poor performance of the fibers.

Each thickness value calculated according to IRC design guides was calculated for different stress ratios (0.45 for plain concrete, 0.5 or 0.6 for fiber-reinforced concrete), which were determined according to the performance of the fibers. Therefore, when Table 4.19 was examined, it was seen that different thickness values were obtained for different concrete types depending on the considered stress ratio. Accordingly, it was seen that required thickness of the concrete pavement could be reduced by up to 28% (Mulheron, 2015). However, it was also seen that the required thickness of the concrete pavement increased by up to 26% (Öztürk, 2018) due to the poor performance of the fibers.

As stated before contribution of fibers were considered with the approach of Altoubat et al. (2008) within AASHTO (1993). When Table 4.19 was examined, it was seen that required thickness of the concrete pavement could be reduced by up to 35% (Mulheron, 2015). However, it was also seen that the required thickness of the concrete pavement could be reduced up to 3% (Öztürk, 2018) unlike the other two methodologies.

When the results given for RCC and PCC compared, it is clear that for both types of concretes a variety of contributions can be obtained by using different types of fibers in different amounts. Results showed that, maximum contributions were taken from the mixtures that significantly contributes to flexural strength (MOR) and toughness of the concrete. Also, it was seen that the use of a fiber-reinforced concrete with poorly optimized mixtures resulted in similar to or higher required pavement thickness than that of plain concrete. Therefore, in order to achieve the desired contribution by using fiber in concrete road applications, the selection of fiber type and quantity, as well as the importance of the concrete matrix design are clear. It should be noted that, the contribution of fibers varies depending on how the fibers are taken into account in the considered methodology.

4.5.2.4. Results for Plain RCC and Conventional PCC in terms of Temperature Consideration. As stated before, temperature differential between the top and bottom fibers of concrete pavements causes the concrete slab to curl, giving rise to stresses. Since IRC design guides are the only design methodologies that take the temperature effect into consideration on thickness design among the other design methodologies considered in this study, there was a need to make an evaluation according to the values given in Table 4.24. As mentioned earlier, StreetPave and IRC design methodologies use the same fatigue equation. Therefore, in order to see the effect of temperature, the thickness values for plain concrete mixtures calculated according to these methodologies were compared. Required thickness values evaluated in terms of with and without tied concrete shoulder for RCC and conventional PCC pavements.

Table 4.24. Thickness requirements for plain RCC and conventional PCC.

Study	**Mixture**	**STREETPAVE V12 (2014)**		**IRC 58 (2011) -IRC SP 46 (2013)**		Standard (Stress Ratio)
		Without Tied Concrete Shoulder	With Tied Concrete Shoulder	Without Tied Concrete Shoulder	With Tied Concrete Shoulder	
		With or Without Dowel Bar	With or Without Dowel Bar	With or Without Dowel Bar	With or Without Dowel Bar	
		Required Thickness (cm)				
		Fatigue	Fatigue	Fatigue	Fatigue	
Öztürk, 2018 (RCC)	RCC Control-1	18.8	16.8	19.2	16.1	IRC 58 (0.45)
LaHucik et al., 2017 (RCC)	RCC Control-2	25.1	22.8	29.4	25.3	IRC 58 (0.45)
Mulheron, 2015 (PCC)	PCC Control	22.1	19.8	26.0	22.0	IRC 58 (0.45)

When Table 4.24 was examined in terms of without tied concrete shoulder for StreetPave and IRC design methodologies, it was seen that required thickness of the concrete pavement increased up to 17% due to the extra stress caused by the temperature. In addition, when Table 4.24 was examined in terms of with tied concrete shoulder for StreetPave and IRC design methodologies, it was seen that required thickness of the concrete pavement increased up to 11% due to the extra stress caused by the temperature. It is obvious that the temperature stresses should be considered in the design process for regions have an extreme type of climate such as India.

4.5.2.5. Results for Presence of Dowel Bar and Concrete Shoulder and Fiber Usage. The rates of change in thickness calculated according to the presence of concrete shoulder, dowel bars and the use of fibers of different amounts and types were given from Table 4.25 to 4.27 for RCC and conventional PCC.

As stated before, tied concrete shoulders are recommended to protect the edge of high-volume highway pavements. Also, IRC 58 (2015) stated that concrete shoulder reduces the edge flexural stress by 20 to 30%. This will result in reduction of pavement thickness. On the other hand, due to additional amount of concrete will be similar with and without concrete shoulder cases. Each methodology considered in this study takes into account the presence of the tied concrete shoulder in a different way. Therefore, the effect of the tied concrete shoulder on the required thickness varies. These differences were presented from Table 4.25 to 4.27.

Load transfer at joints is provided by means of dowel bars. Dowel bars enable good riding quality to be maintained by preventing faulting at the joints. Also, it is known that the use of load transfer devices such as dowel bars reduces the stresses due to edge load. It should be noted that dowel bars aren't used in RCC and the load transfer at joints is provided by aggregate-interlock. On the other hand, for conventional PCC presence of dowel bars are considered in each methodology with a different way. So, the effect of dowel bar on the required thickness varies as can be seen in Table 4.25.

For AASHTO (1993), the presence of tied concrete shoulder and dowel bars are only considered to make an adjustment of the load transfer coefficient (J) according to the table given in the guide.

For IRC design guidelines while the presence of tied concrete shoulder is considered within the stress equation, the presence of dowel bars at the joints is not regarded as a parameter affecting the design thickness. In the current design guidelines, erosion isn’t considered as a failure mode and in the design guide there are some recommendations for the usage of dowel bars.

For StreetPave, edge support as existent in designs for tied concrete shoulder or curb and gutter or, alternatively, a widened lane condition. Also, at the end of analysis, the software proposes a required thickness for both doweled and undoweled case.

On the other hand, each methodology provided the required thickness value for different critical failure modes as follows.

- IRC guidelines only calculates thickness for fatigue failure mode,
- StreetPave calculates thickness for both fatigue and erosion and gives a single value for critical situation.
- AASHTO gives a single thickness value calculated by empirical formula based on loss of serviceability concept, including distress modes such as fatigue and erosion.

Therefore, the thicknesses calculated for the different critical failure modes for each methodology were compared with each other.

Table 4.25. Summary of the results for conventional PCC.

	Mulheron,2015-Conventional PCC		
	AASHTO (1993)	IRC 58 (2011) & IRC SP 46 (2013)	StreetPave v12 (2014)
Fiber	Decrease up to ~35%	Decrease up to ~28%	-No effect was seen in erosion analysis. -Decrease up to ~18% in fatigue analysis.
Shoulder	Decrease up to ~8%	Decrease up to ~15%	Decrease up to ~20% for erosion analysis and ~10% for fatigue analysis
Dowel Bars	Decrease up to ~16%	The presence or absence of dowel bars at the joints is not regarded as a parameter affecting the design thickness	Decrease up to ~22%
Critical Failure Mode for Thickness Requirement	Fatigue and Erosion	Fatigue	Erosion

When Table 4.25 given for conventional PCC pavements was examined, it was seen that AASHTO (1993) is the methodology that considers the effect of the presence of tied concrete shoulder on the required thickness in the least amount. Also, it was seen that StreetPave considers the effect of presence of dowel bars on the required thickness more than other methodologies. It should be noted that, IRC design guides don’t consider the effect of dowel bars on the required thickness. Also, when Table 4.25 was examined, it was seen that the effect of fibers was more pronounced in AASHTO (1993).

Table 4.26. Summary of the results for RCC (LaHucik et al.,2017).

	LaHucik et al.,2017-RCC		
	AASHTO (1993)	IRC 58 (2011) & IRC SP 46	StreetPave v12 (2014)
Fiber	Decrease up to ~21%	Decrease up to ~13%	Decrease up to ~23%
Shoulder	Decrease up to ~5.5%	Decrease up to ~16%	Decrease up to ~20% for erosion analysis and ~10% for fatigue analysis
Dowel Bars	Dowel bars aren’t used in RCC.		
Critical Failure Mode for Thickness Requirement	Fatigue and Erosion	Fatigue	Erosion

When Table 4.26 given for RCC pavements (LaHucik et al.,2017) was examined, it was seen that AASHTO (1993) is the methodology that considers the effect of the presence of tied concrete shoulder on the required thickness in the least amount. It should be noted that, dowel bars aren't used in the RCC pavements. Also, when Table 4.26 was examined, it was seen that the effect of fibers was more pronounced in StreetPave.

Table 4.27. Summary of the results for RCC (Öztürk,2018).

	Öztürk, 2018-RCC		
	AASHTO (1993)	IRC 58 (2011) & IRC SP 46 (2013)	StreetPave v12 (2014)
Fiber	Decrease up to ~3%	Increase up to ~26%	Increase up to ~8%
Shoulder	Decrease up to ~5.5%	Decrease up to ~16%	Decrease up to ~20% for erosion analysis and ~10% for fatigue analysis
Dowel Bars	Dowel bars aren't used in RCC.		
Critical Failure Mode for Thickness Requirement	Fatigue and Erosion	Fatigue	Erosion

When Table 4.27 given for RCC pavements (Öztürk, 2018) was examined, it was seen that AASHTO (1993) is the methodology that considers the effect of the presence of tied concrete shoulder on the required thickness in the least amount. It should be noted that, dowel bars aren't used in the RCC pavements. Also, when Table 4.27 was examined, it was seen that the effect of fibers was more pronounced in AASHTO (1993).

4.5.3. Concrete Pavements in Turkey

In this section, the studies that have been done so far on the rigid pavements which are extremely advantageous and inevitable for our country in terms of both source and engineering were mentioned and given information about the latest methods available for the design of rigid pavements. Concrete roads (rigid pavements) and asphalt roads (flexible pavements) are significant alternatives for each other in many countries around the world. Making the right choice for the road to be built depends on many factors such as the country's resources and budget, the traffic volume, and the foundation characteristics, etc. (Acıkök et al., 2019). Due to the need for the roads that are long-lasting and requiring less maintenance, concrete roads have been frequently become a current issue in recent years in our country as in the world (Cömert et al., 2019). Although the cement industry is developed in our country, flexible pavements are widely used in road constructions. This leads to external dependence, and besides, the road construction costs are constantly changing depending on the global economic data (Özkan et al., 2019).

Since its establishment in 1950, General Directorate of Highways of Turkey (GDH) has only built asphalt roads. As GDH personnel and road contractors are accustomed to asphalt, there is some timidity and indecision against a new technology such as concrete road construction. In addition to this psychological effect, the misconception that the concrete road is more expensive than the asphalt, has prevented the construction of the concrete roads in our country so far. However, using concrete roads provides durable, economical, and long-lasting solutions and contributes to competition by creating an alternative to asphalt roads. It is possible to save with the increasing competition with the use of concrete roads and emphasized that the concrete roads are the most economical choice considering the life cycle cost even if the initial investment cost of concrete road is more than asphalt roads. In recent years, especially in rural roads, rigid pavements have been preferred and successful applications are being made. Although the number of applications has increased, it is still limited regarding the volume of new roads being built. In rural roads, Roller Compacted Concrete (RCC) road is used as well as conventional concrete road. Today, RCC road construction is a good option for this type of roads and in recent years, it has started to be preferred especially by the municipalities and provincial private administrations. The most important advantage of RCC roads is that they can be built with conventional asphalt road construction equipment (asphalt paver and rollers) (Ünverdi et al., 2019).

In our country, the road network can be analyzed in two main groups. The first of these is the road network under the responsibility of GDH it is 67,333 km in total as of January 1, 2019. The second group is the road network of local governments and although there is no clear statistical information, the total length of these roads is estimated to be more than 350.000 km. The majority of the roads within the responsibility area of GDH are bituminous surface treatments and flexible pavement and the ratio of concrete roads is quite low (Özkan et al., 2019). GDH does not have tangible and sufficient data on project planning, pricing, construction and maintenance costs between asphalt and concrete pavement.

Likewise, road construction companies have limited experience in this field. For this reason, in order to eliminate the concerns about whether concrete pavement construction will be successful in our country, it is aimed to monitor the behavior of concrete road by constructing test sections on various roads including different traffic categories, topographical structure and climatic differences, consequently, to clarify the opinions of the concrete road. Also, it is aimed to provide the necessary information transfer from the experts of these countries by making investigations in countries where concrete roads are widely applied. Within the scope of concrete road productions, first started in 2004, 2 km of Afyonkarahisar-Emirdağ road, 3.5 km of Hasdal-Kemerburgaz road, 1 km of Ordu-Ulubey road and 1.6 km of Izmit-Karamürsel road, a total of 8.1 km of concrete roads were manufactured. As of January 1, 2019, only 8.1 km of the total road network of 67.333 km is concrete road. In other words, only one in ten thousand of our road network consists of concrete roads. Jointed plain concrete pavement (JPCP) was preferred in all of the test roads applied on the roads in the GDH road network in our country. The design of these concrete pavement roads was carried out using the AASHTO (1993) empirical design method. The performances of the test sections have been carried out both on-site observationally and by means of functional status measurement devices in certain periods, and performance monitoring is still going on (Komut et al., 2019).

On the other hand, in the road networks of local governments, the interest in rigid pavements has increased in recent years and local administrations such as Antalya, Samsun, Manisa and Kocaeli Metropolitan Municipalities have produced concrete roads especially on neighborhood roads (Özkan et al., 2019). In addition, the road superstructure of the "Kemaliye-Dutluca Tunnels and Connection Roads and Kemaliye and Kozlupınar Viaducts" project, which was tendered in 2017 and has a total length exceeding 21 km, has been designed as rigid. This development has been an important step in the spread of concrete roads in our country. Therefore, in the light of the developments in Turkey and abroad, GDH has accelerated its work on concrete roads, and in 2016, it has issued the "Technical Specification for Concrete Roads". In 2017, "Highways Concrete Road Superstructures Design Guide" preparation studies were started within the body of General Directorate of Highways R & D Department and Superstructure Development Department. "Highways Concrete Road Superstructures Project Planning Guide" was completed in 2019 and come into use as a resource that can be utilized by superstructure engineers, sector representatives of relevant institutions, organizations and all stakeholders. In the project planning guide, the project design method which was developed by AASHTO was based on. As a result of the assessment made by GDH, in the preparation of the specification to be prepared for our country, highly durable, especially exposed to heavy traffic, widely used and thought to be both economic and efficient at the same time, the "Jointed Plain Concrete Pavement" type has been dealt with (Kaşak et al., 2019).

While preparing the specification, our national needs and opportunities are prioritized, and national and international standards and principles are taken into consideration. One of the sources used among the many documents examined in this context is the book of Concrete Pavements prepared by the Federal Association of the German Cement Industry. The documents published by the German Road and Transport Research Association, including the technical directives and requirements for materials and material mixtures for hydraulically bonded base layers and concrete pavements are the other sources that are utilized. In addition, the publications of the United States, a pioneer and advanced in road construction, the U.S Department of Transportation Federal Highway Administration, a department of the Ministry of Transport, specializing in road transport, the American National Concrete Pavement Technology Center's publications were also utilized. Likewise, the Belgian specification "CCT Qualiroutes", which has a lot of concrete road experience, is among the documents that were examined. The main source of this specification is the "Highway Technical Specification" which is used in all technical works to be performed in Central and Regional Organizations of General Directorate of Highways. In addition, harmonized Turkish Standards on "Concrete Pavements" have been used to form the specification content. The specification, in general terms, consists of properties of the constituent materials of coating concrete and desired criteria, performance values expected from the concrete pavement, concrete design and requirements, production of the concrete, construction of test roads, manufacturing of the concrete road, the quality control inspections to be carried out by both the contractor company and the administration, and finally the payment deductions to be made by the administration if necessary as a result of these inspections (Cömert et al., 2019).

The studies on the "Technical Specification for Concrete Roads" and "Rigid Superstructures Design Guide" are still going on. The highways in Turkey is compatible with the AASHTO (1993) design criteria. AASHTO (1993) design criteria were taken as basis in the project planning guide prepared by GDH for flexible superstructures in 2008 and preliminary preparations were made for the transition to mechanistic-empirical design method. AASHTO (1993) project design criteria were also taken as basis in the project planning guide for rigid superstructures in 2019. However, with the developing technological accumulation, especially in the USA, important steps are taken in the transition from empirical design method (AASHTO, 1993) to mechanistic design method in order to create more realistic and sustainable designs by making important studies on road superstructure design. Concrete road is a new spreading pavement type in our country. Therefore, more mechanical knowledge and experience about these roads needs to be increased in a short time.

As mentioned earlier, Roller Compacted Concrete (RCC) is a zero-slump concrete produced in a concrete mixing station with conventional concrete materials and applied with asphalt road equipment. The superstructures made with RCC road technology can be used safely even on the subbase surfaces with low carrying capacity. It is preferred by many international road authorities due to its rapid construction with classical road construction elements, opening to light vehicle traffic in a short time and long life, and its advantages have been utilized for many years (Özkan et al., 2019). In the last decade, RCC road applications that were launched with the efforts of Turkey Cement Manufacturers' Association (TCMA) have been widely used in many of our provinces due to its advantages and the studies are going on successfully. In recent years, RCC road applications have started to be preferred by local governments in our provinces such as Samsun, Kocaeli, Denizli, Tekirdağ, Bartın, Kırklareli, Edirne, Sinop, Kastamonu, Erzurum, Osmaniye, Afyon and Bolu and have started to be used in urban and rural road superstructures. As a result of these successful implementations, due to the technical and economic superiority of the RCC pavements compared to its alternatives, the General Directorate of Local Administrations determined the use of concrete roads as the main target as an alternative to other types of covering on rural roads with its letter dated 27.03.2018. In addition, “Roller Compacted Concrete Roads Technical Specifications” was prepared and made available to road engineers and administrations at the end of 2017. In 2018, “Roller Compacted Concrete Roads Design Guide” was prepared. It was stated that it would be appropriate to evaluate this guideline together with “Roller Compacted Concrete (RCC) Roads Technical

Specification". This project planning guide shows information about road pavements using RCC road technology and how the superstructure layer thicknesses are determined. It is stated that the general design approach of the guide is in compliance with the "StreetPave" software, which was prepared by the American Concrete Pavement Association in 2014 and made available internationally. On the other hand, calculations and evaluations have been made by considering the engineering characteristics of road construction sector, traffic composition and road construction materials of our country. However, when the design guide was examined, it was seen that the project design guide was prepared using the "StreetPave" software program calculation method in accordance with the design specified in AASHTO (1993) Rigid Pavement Design Guide. Also, it is stated that the total equivalent traffic passing through the heavy traffic lane should be determined during the life of the project by converting the single, tandem and tridem axle loads passing through the road to be designed for thickness to the equivalent number of standard axle loads. Also, it is stated that StreetPave software project traffic account principles have been accepted in the guide (TÇMB, 2018).

5. SUMMARY AND CONCLUSIONS

5.1. Overview

In this study, the required thickness values for plain and fiber reinforced concretes with different properties were evaluated by a comparative study. AASHTO (1993), IRC SP 46 (2013), and StreetPave v12 (2014) (based on PCA 1984) methodologies were used for the thickness design. Because of modeling and computer advancements over the last 50 years, concrete pavement thickness design has progressed from strictly empirical designs (e.g., AASHTO 93) to mechanistic-empirical designs (e.g., StreetPave, IRC 58 (2011), and IRC SP 46 (2013)). To understand better the differences in outcomes between each design approach, the basis of the equations must be understood, how and when different material, environmental, structural properties affect the performance of the pavement. Therefore, a comprehensive review of design methodologies was conducted in the literature and the differences in the design input parameters for each methodology were evaluated.

The thickness design based on three methodologies was carried out for a sample road using material parameters retrieved from the literature. The material parameters for plain and fiber reinforced concrete used in RCC pavements were taken from the two previous studies found in the literature carried out by Öztürk (2018) and LaHucik et al., (2017); in conventional PCC pavements, these parameters were taken from the previous study found in the literature carried out by Mulheron, (2015). Also, traffic and foundation parameters obtained from IRC SP 46 (2013) were used for all design methodologies. Therefore, all differences between thicknesses would be a function of the design methodologies and material properties.

The thickness of the slab could be decreased by increasing the flexural strength of the concrete for example reducing the water to cement ratio or increasing the cement content. But this would make the system more brittle and there would be a higher cracking probability compared to the fiber-reinforced concrete system. Contribution of structural fibers to the mechanical and durability properties of concrete have been known for a very long time, and fiber reinforcement in concrete pavements is also becoming popular since its application is practical when compared to conventional reinforcement. However, the number of design methods that cover the use of fiber-reinforcement in concrete pavements is very limited and all the methods use a different approach. It causes the contribution of the fibers to differ according to the used approach. Also, it should be noted that the amount of contribution to be obtained using fibers can vary significantly according to fiber type, fiber amount, and concrete matrix properties. The contribution of fibers with different types (synthetic, steel and carbon) and amounts (0.2-1.0%) to thickness requirement was evaluated by examining how fiber contribution was taken into consideration in these methodologies. To see the effects of the concrete matrix on the effectiveness of fibers, two types of concrete (RCC and conventional PCC) were considered. The accepted methodology for the design can be summarized as follows.

1. The traffic, foundation, and road parameters obtained from IRC SP 46 (2013) have been used. Traffic spectrum, which is considered as a different way for each methodology and which affects the pavement thickness in the design process, has been converted to the desired format described in detail in the literature review (18-kip ESAL for AASHTO (1993), 6-hour axle load repetition for daytime and night time for IRC 58 (2011) , axle/1000 trucks for StreetPave v12 (2014)).
2. The material parameters for RCC and conventional PCC concrete pavements were taken from the previous studies found in the literature.
3. The pavement thickness was calculated according to each methodology for both plain and fiber reinforced concrete using these material and traffic parameters.
4. Finally, the results were compared.

During the design process, parameters related to foundation, road and traffic were kept constant to observe the effects of design methodologies and material properties.

The thickness design results of both plain and fiber reinforced RCC and conventional PCC pavements were examined under the following eight headings.

1. The results of thickness design for RCC and conventional PCC pavements were first evaluated separately in terms of plain and fiber-reinforced within each methodology.

2. To observe the causes of thickness similarities or differences according to methodologies, the calculated required thicknesses according to three methodologies were compared in terms of RCC and conventional PCC.
3. The effects of fibers on required thickness and the fiber-reinforced approaches were evaluated.
4. The effects of the concrete matrix on the effectiveness of fibers was examined in terms of fiber-reinforced RCC and conventional PCC.
5. The result of plain RCC and conventional PCC was evaluated in terms of temperature effect.
6. The effects of the presence of edge support and load transfer device such as dowel bars on the thicknesses calculated for RCC and conventional PCC were evaluated in terms of design methodologies. It was also examined which failure mode governed the design.
7. Shortcomings of methodologies also was examined.
8. Finally, Turkey's policy on concrete roads and recently published design procedures were evaluated briefly.

5.2. Conclusions

As a result of these comparisons, the conclusion part of the study was divided into 10 main sections as follows.

5.2.1. Thickness Design According to AASHTO (1993)

Result of the study were presented below.

- Dominant parameters and their effects: In the design, it was observed that the flexural strength of concrete (MOR) and load transfer coefficient (J) influenced the obtained thickness values. The higher the flexural strength, the lower the thickness obtained. Furthermore, the smallest load transfer coefficient has been chosen for “with dowel bar” and “with tied concrete shoulder” case for conventional PCC. The lowest thicknesses were obtained for this case.

- Slab edge support: Required thickness values could be reduced by up to ~5.5% for “without dowel bar” situation and ~8% for “with dowel bar” situation in the case of slab edge was supported.
- Use of dowel bars: The use of dowel bar is only applicable to conventional PCC, and the use of dowel bar provides a reduction in thickness of about 15%.
- Use of fibers: It was seen that the use of fiber can reduce the pavement thickness requirement by up to 35% (these values may increase or decrease depending on the materials).

5.2.2. Thickness Design According to IRC SP 46 (2013)

Result of the study were presented below.

- Dominant parameters and their effects: In the design, it was observed that the flexural strength of concrete (MOR) and stress ratio (which is the function of flexural strength and the equivalent stress caused by the axle load) influenced the obtained thickness values. The higher the flexural strength, the lower the thickness obtained. Also, stress ratios (0.45,0.5 or 0.6) used in the design determined according to the obtained toughness via fibers. The higher the stress ratio, the less the number of repetitions of the load required for cracking. In this study, since the traffic is constant, the expected axle load repetitions are constant. Therefore, an increase in the stress ratio provides a decrease in the required thickness. As a result, relatively low required thicknesses were found in designs using high stress ratio and high flexural strength.
- Slab edge support: Required thickness values could be reduced by up to ~17% in the case of slab edge was supported.
- Use of dowel bars: Since erosion is not a failure mode considered in the present design procedure the presence or absence of dowel bars at the joints is not regarded as a parameter affecting the design thickness. However, in the design guide IRC 46 (2013), the use of dowel bars is recommended for the following situations. If the thickness of the design is less than 200 mm, the use of dowel bars is not considered as reliable.

i. Thickness ≥ 200 mm in the design using fibers with low contribution to toughness,

ii. Thickness ≥ 150 mm in the design using fibers with high contribution to toughness

- Use of fibers: It was seen that the use of fiber could reduce the pavement thickness requirement by up to 28% (these values may increase or decrease depending on the materials). As expected, it was seen that the highest performance can be achieved with mixtures that do not decrease the flexural strength value and also contribute to the toughness. However, it was seen that the use of a fiber-reinforced concrete with poorly optimized mix yields the required pavement thickness values similar to that of plain concrete. (slab thickness requirement increased up to ~26% by using fibers)

5.2.3. Thickness Design According to StreetPave v12 (2014)

Result of the study were presented below.

- Dominant parameters and their effects: It should be noted that, the design for all mixtures in StreetPave were driven by the erosion analysis (as higher thickness values were found for erosion analysis). So, the small differences in thicknesses obtained from the erosion analysis resulted from the differences in the modulus of elasticity (E) of the concrete. However, if the failure mode were fatigue, the most important parameter affecting the design would be flexural strength of concrete (MOR).
- Slab edge support: Required thickness values could be reduced by up to ~20% for erosion analysis and ~10% for fatigue analysis in the case of slab edge was supported.
- Use of dowel bars: At the end of analysis the software proposes a required thickness for both doweled and undoweled case. In fact, StreetPave proposes to use 5 inches as the minimum undoweled pavement thickness and 6 inches as the minimum thickness for doweled concrete to ensure coverage over the dowel bars. It should be noted that, RCC aren't used dowel bars.
- Use of fibers: It should be noted that the fiber effect is not considered in the erosion calculations used in the StreetPave design approach. So, thickness values for fatigue failure were calculated by generated spreadsheet only to see the contribution of fibers to that failure mode. It was seen that the thickness requirement could be reduced by up to 23% by using fiber (for the traffic and material properties used in this study). As expected, it was seen that the highest performance can be achieved with mixtures

that do not decrease the flexural strength value and also contribute to the toughness. However, it was seen that the use of a fiber-reinforced concrete with poorly optimized mix yields the required pavement thickness values similar to that of plain concrete (slab thickness requirement increased up to ~7% by using fibers).

5.2.4. Evaluation of the Results in terms of 3 Design Methodologies

It is clear that each of the three design procedures investigated uses a different fundamental approach to the problem of predicting required thickness, but they all show similar sensitivity to key design inputs such as flexural strength, modulus of elasticity, the presence of edge support and dowel bar. Through this comparative analysis, AASHTO (1993), which is based wholly on 50+-year-old empirical data, was shown to produce thickness requirements that are significantly different than those required by the more recently developed mechanistic-empirical methods of StreetPave and IRC SP 46 (2013). However, obtaining higher thickness values for AASHTO was supported by a similar study conducted by Li et al (2011). Accordingly, Li et al. (2011) concluded that AASHTO (1993) provided reasonable structural designs for flexible pavements using a more realistic layer coefficient for flexible pavements, but that rigid pavements exhibited extremely thick slabs for the inputs given in the AASHTO (1993) guideline. The reason could be that AASHTO-1993 gave conservative design based on empirical equations.

On the other hand, it was seen that the lowest thickness values were obtained by IRC guidelines. The main reason for this is that the 28-day mean flexural strength was used in AASHTO (1993) guide, whereas the 90-day mean flexural strength was used in IRC methodologies. It should be noted that the strength of concrete increases with age and using 90-day flexural strength value decreased the required thickness.

It was seen that the required slab thicknesses calculated according to StreetPave were similar, although different types and amounts of fibers were used. This is because the StreetPave does not consider the contribution of fibers in the erosion analysis. Therefore, the thickness values given for the erosion analysis show only the effects of the fibers on the modulus of elasticity (E) of the concrete. However, it should be noted that the modulus of elasticity (E) of the concrete doesn’t show the contribution of the fibers to the behavior of the concrete, since the fibers contribute to the behavior of the concrete, especially in the post cracking region. If StreetPave had considered fiber contribution in the erosion analysis, the thickness values to be obtained would be close to those calculated by IRC guidelines.

Another reason why the thicknesses for plain and fiber-reinforced concretes were different from each other is that the methodologies consider different performance criteria. it was seen that each methodology provided the required thickness value for different critical failure modes as follows.

- IRC guidelines only calculates thickness for fatigue failure mode,
- StreetPave calculates thickness for both fatigue and erosion and gives a single value for critical situation (erosion for all mixtures in this study), and
- AASHTO gives a single thickness value calculated by empirical formula based on loss of serviceability concept, including distress modes such as fatigue and erosion.

5.2.5. FRC Pavements and FRC Pavement Design Approaches

As expected, it was seen that the highest performance can be achieved with mixtures that do not decrease the flexural strength value and contribute to the toughness. However, it was seen that the use of a fiber-reinforced concrete with poorly optimized mix yields the required pavement thickness values similar to that of plain concrete. Therefore, in order to achieve the desired contribution by using fiber in concrete road applications, the selection of fiber type and quantity, as well as the importance of the concrete matrix design are clear. It should be emphasized here that contribution of fibers to the pavement thickness requirement were investigated in the scope of the study presented here, and contributions of fibers to the durability properties of concrete pavements weren’t mentioned. More durable concrete pavements with lower thickness can be constructed by using the fibers in concrete pavement applications.

The FRC pavement design approaches which were used in the three methodologies were examined. Since the effect of fibers is considered with different approaches in the design methodologies used in the study, the contribution of the fibers varies according to the used approaches. It should be noted here that, although AASHTO (1993) doesn't consider the fibers in the design of concrete pavement, the contribution of the fibers was evaluated with the approach of Altoubat et al. (2008). In StreetPave v12 (2014), the contributions of fibers to the thickness requirement is taken into consideration in a similar way with the Altoubat et al. (2008). The only difference between these two approaches is the parameters that they use to increase the flexural strength. Instead of the equivalent flexural strength ratio value ($R_{e,3}$) used in Altoubat et al. (2008), the StreetPave uses the residual flexural strength ratio value. Therefore, in StreetPave the fibers seem to have contributed less by a percentage compared to the AASHTO (1993). This is because using the residual flexural strength ratio while increasing the flexural strength is more conservative than using the equivalent flexural strength ratio.

On the other hand, IRC SP 46 (2013) considers the effect of the fibers by increasing the stress ratio depending on the toughness of the fibers unlike the other two methods. In the other two methods, while the flexural strength is increased even if the fiber has low toughness, there are various conditions to consider the effect of the fiber in IRC SP 46 (2013) guideline. Therefore, for fibers are used in dose less than 0.3% by volume, the design is made considering the stress ratio of plain concrete (0.45). Also, for fibers are used in dose more than 0.3% by volume or FRC having low toughness (f_{e150k}<0.3 f_{ctk}), the limiting stress ratio should be taken as 0.5. Finally, FRC having high toughness (f_{e150k}>0.3 f_{ctk}), the limiting stress ratio should be taken as 0.6. This showed that the fibers should be used at a certain rate and meet the specified toughness requirement in order to take into account the effect of the fibers on IRC SP 46 (2013).

5.2.6. Thickness Design According to Fiber-Reinforced Conventional PCC and RCC

When the results given for RCC and PCC compared, it is clear that for both types of concretes a variety of contributions can be obtained by using different types of fibers in different amounts. Usage of appropriate type and amount of fibers in well-designed concrete mixture (whether RCC or PCC for this study) provides to usage of lower pavement thicknesses. Accordingly, maximum contributions were taken from the mixtures that significantly contributes to flexural strength (MOR) and toughness of the concrete. Result of the study were presented below.

- Required thickness of the concrete pavement could be reduced by up to 23% (LaHucik et al., 2017-RCC). However, despite the use of fibers, the required thickness of the concrete pavement increased by up to 8% (Öztürk, 2018-RCC) due to the poor performance of the fibers. (based on StreetPave v12 design software).
- Required thickness of the concrete pavement could be reduced by up to 28% (Mulheron, 2015-PCC). However, it was also seen that the required thickness of the concrete pavement increased by up to 26% (Öztürk, 2018-RCC) due to the poor performance of the fibers. (based on IRC SP 46 (2013) design guide)
- Required thickness of the concrete pavement could be reduced by up to 35% (Mulheron, 2015-PCC). However, it was also seen that the required thickness of the concrete pavement could be reduced up to 3% (Öztürk, 2018-RCC) unlike the other two methodologies.

It is obvious that the contribution of fibers varies depending on how the fibers are considered in the design methodology.

5.2.7. Results for Plain RCC and Conventional PCC in terms of Temperature Effect

The temperature differential between the top and bottom fibers of concrete pavements causes the concrete slab to curl, giving rise to stresses. Since IRC design guides are the only design methodologies that take the temperature effect into consideration on thickness design among the other design methodologies considered in this study, there was a need to make an evaluation. StreetPave and IRC design methodologies use the same fatigue equation.

Therefore, in order to see the effect of temperature, the thickness values for plain concrete mixtures calculated according to these methodologies were compared. Required thickness values evaluated in terms of with and without tied concrete shoulder for RCC and conventional PCC pavements. Result of the study were presented below.

- When the required thickness results were examined in terms of without tied concrete shoulder for StreetPave and IRC design methodologies, it was seen that required thickness of the concrete pavement increased up to 17% due to the extra stress caused by the temperature.
- When the required thickness results were examined in terms of with tied concrete shoulder for StreetPave and IRC design methodologies, it was seen that required

thickness of the concrete pavement increased up to 11% due to the extra stress caused by the temperature.

It is obvious that the temperature stresses should be considered in the design process for regions have an extreme type of climate such as India.

5.2.8. Results for Presence of Dowel Bar and Concrete Shoulder

Differences in the calculated thickness due to the presence of tied concrete shoulder and dowel bars were evaluated for RCC and conventional PCC in terms of three design methodologies and following results were obtained. Using concrete shoulder reduces the edge flexural stress which results in reduction of pavement thickness. Also, it is known that the use of load transfer devices such as dowel bars reduces the stresses due to edge load. It should be noted that dowel bars aren't used in RCC and the load transfer at joints is provided by aggregate-interlock.

Evaluation for conventional PCC pavement;

- If the slab edge was supported;
  - the required thickness values calculated according to AASHTO (1993) design guidelines could be reduced by up to ~ 8%,
  - the required thickness values calculated according to IRC design guidelines could be reduced by up to ~ 17%, and
  - the required thickness values calculated according to StreetPave could be reduced by up to ~ 20% for erosion analysis and ~ 10% for fatigue analysis.

- If the dowel bars were used;
  - the required thickness values calculated according to AASHTO (1993) design guidelines could be reduced by up to ~ 16%,
  - According to IRC design guidelines the presence or absence of dowel bars at the joints is not regarded as a parameter affecting the design thickness, and
  - the required thickness values calculated according to StreetPave could be reduced by up to ~ 22.

Evaluation for RCC pavement;

- If the slab edge was supported;
  - the required thickness values calculated according to AASHTO (1993) design guidelines could be reduced by up to ~ 5.5%,
  - the required thickness values calculated according to IRC design guidelines could be reduced by up to ~ 16%, and
  - the required thickness values calculated according to StreetPave could be reduced by up to ~ 20% for erosion analysis and ~ 10% for fatigue analysis.

- Dowel bars aren't used in RCC pavements.

AASHTO (1993) is the methodology that considers the effect of the presence of tied concrete shoulder on the required thickness in the least amount. For AASHTO (1993), the presence of tied concrete shoulder and dowel bars are only considered to make an adjustment of the load transfer coefficient (J) according to the table given in the guide.

For IRC design guidelines while the presence of tied concrete shoulder is considered within the stress equation, the presence of dowel bars at the joints is not regarded as a parameter affecting the design thickness. In the current design guidelines, erosion isn't considered as a failure mode and in the design guide there are some recommendations for the usage of dowel bars.

For StreetPave, edge support as existent in designs for tied concrete shoulder or curb and gutter or, alternatively, a widened lane condition. Also, at the end of analysis, the software proposes a required thickness for both doweled and undoweled case. It was seen that StreetPave considers the effect of presence of dowel bars on the required thickness more than other methodologies.

5.2.9. Shortcomings of the Methodologies

In most methodologies, the lack of consideration of the various main factors (such as vehicle loads, loss of support, thermal gradient or environmental conditions) that produce stress on the concrete pavement prevents accurate calculation of the required thickness. As a result of the comprehensive review of design methodologies following were found and evaluated separately for each design methodology.

Evaluation of AASHTO (1993);

- AASHTO (1993) design procedure is strictly empirical and evaluates the performance in terms of loss of serviceability caused by 18-kip single axle load applications. Also, the type of hazard is not distinguished as individual distresses like fatigue or erosion. The 1993 AASHTO guide designs pavements to a single performance criterion, the present serviceability index (PSI),
- ESAL was used to characterize the traffic loading and the equivalency factors developed at the AASHO Road Test are highly doubtful to be applicable to today's traffic stream (combination of axle load, traffic levels and types of axles). The AASHO Road Test pavements carried approximately 1 million axle loads, while today, interstate pavements are designed for 50 to 200 million or more axle load applications. The original empirical pavement design models may not produce realistic designs. On the other hand, In PCA (1984), it was stated that fatigue cracking becomes more critical under single axle loads and erosion becomes more critical under multi axle loads such as tandem and tridem. In view of this situation, it is not reasonable to use ESAL for rigid pavement design. Since to convert a multi axle load into an equivalent single axle load changes the failure mode from erosion at joints to fatigue at mid slab.
- Pavements at the AASHO Road Test site were constructed over a single silty-clay (AASHTO A-6) subgrade. The effect of this single subgrade was "built into" the empirical design models.
- The AASHO Road Test design equations were based on results of traffic test over a two-year period. Long term effects of temperature and moisture on the loss of serviceability were not considered. No field verification was performed for the original models being extended over time based on empirical methods.
- Since the AASHO Road Test design equations are based on traffic test results over a two-year period, the effects of long-term temperature and moisture on loss of serviceability were not considered.
- AASHTO (1993) doesn't have provisions for the use of fibers.

Evaluation of IRC SP 46 (2013);

- IRC SP 46 (2013) evaluates the performance through the cumulative fatigue damage analysis (considering bottom-up and top-down cracking) based on Miner's hypothesis. Considering the fact that tandem, tridem and multi-axle vehicles constitute a large percentage of the total commercial vehicles on the roadways in India, it is clear that erosion is of crucial importance in designs and that the current design approach needs to be developed accordingly.
- IRC SP 46 (2013) doesn’t employ any reliability approach to estimate the pavement performance.
- For IRC design guidelines while the presence of tied concrete shoulder is considered within the stress equation, the presence of dowel bars at the joints is not regarded as a parameter affecting the design thickness since erosion isn’t considered as a failure mode.
- Although drainage is an important factor in pavement performance especially in terms of erosion, it has been overlooked in current design methodologies.

Evaluation of StreetPave v12 (2014);

- Fibers can help the load transfer in joints by crack control and micro-dowel action. However, StreetPave doesn’t consider this contribution of fibers. Therefore, in terms of erosion analysis the software should be revised by considering load transfer action of fibers in joints.
- The curling (caused by temperature changes) and warping (caused by differences in moisture content) stresses are ignored.
- StreetPave does not consider climate.
- StreetPave v12 (2014) (also PCA (1984)), evaluates the performance with the cumulative fatigue considering only bottom-up cracking. Therefore, StreetPave offers a limited and incomplete analysis of bottom-up cracking and faulting only. However, modern concrete pavement performance predictions, including each of these potential failure modes and both bottom-up and top-down cracking.

5.2.10. Concrete Pavements in Turkey

The application of concrete pavements that have been practiced for many years in developed countries is relatively new in Turkey. The studies on the “Technical Specification for Concrete Roads” and “Rigid Superstructures Design Guide” are still going on. The highways in Turkey is compatible with the AASHTO (1993) design criteria. AASHTO (1993) project design criteria were also taken as basis in the project planning guide for rigid superstructures in 2019. However, with the developing technological accumulation, especially in the USA, important steps are taken in the transition from empirical design method (AASHTO 93) to mechanistic design method in order to create more realistic and sustainable designs by making important studies on road superstructure design. Concrete road is a new spreading pavement type in our country. Therefore, more mechanical knowledge and experience about these roads needs to be increased in a short time.

On the other hand, in the last decade, Roller Compacted Concrete (RCC) road applications that were launched with the efforts of Turkey Cement Manufacturers' Association have been widely used in many of our provinces due to its advantages and the studies are going on successfully. It is stated that the general design approach of the guide is in compliance with the “StreetPave” software, which was prepared by the American Concrete Pavement Association in 2014 and made available internationally. On the other hand, calculations and evaluations have been made by considering the engineering characteristics of road construction sector, traffic composition and road construction materials of our country.

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APPENDIX A: THICKNESS DESIGN EXAMPLE FOR PCC CONTROL ACCORDING TO THREE DESIGN METHODOLOGIES

Table A.1. Bottom-up cracking fatigue analysis for single axle (PCC Control) (h=22) – IRC 58 (2011).

Expected Repetition	Flexural Stress (Mpa)	Flexural Strength (Mpa)	Stress Ratio	Allowable Repetition	Fatigue Damage
1	2	3	4=2/3	5	6=1/5
60365	3.3	5.55	0.55	136031	0.44
57971	3.2		0.53	259499	0.22
60764	3.1		0.51	580864	0.10
43170	3.0		0.49	1696520	0.03
9911	28		0.46	8449068	0.00
5388	2.7		0.44	infinity	0.00
8714	2.6		0.42	infinity	0.00
8814	2.5		0.40	infinity	0.00
8814	2.3		0.38	infinity	0.00
10809	2.2		0.36	infinity	0.00
10809	2.1		0.34	infinity	0.00
47062	2.0		0.33	infinity	0.00
Total Fatigue Damage					0.798

Table A.2. Bottom-up cracking fatigue analysis for tandem axle (PCC Control) (h=22) – IRC 58 (2011).

Expected Repetition	Flexural Stress (Mpa)	Flexural Strength (Mpa)	Stress Ratio	Allowable Repetition	Fatigue Damage
1	2	3	4=2/3	5	6=1/5
80376	2.9	5.55	0.47	3978449	0.02
58203	2.8		0.45	28754511	0.00
20122	2.7		0.44	infinity	0.00
13858	2.6		0.42	infinity	0.00
14911	2.4		0.40	infinity	0.00
6984	2.3		0.38	infinity	0.00
21618	2.2		0.36	infinity	0.00
28769	2.1		0.34	infinity	0.00
34922	2.0		0.33	infinity	0.00
35476	1.9		0.31	infinity	0.00
49334	1.8		0.29	infinity	0.00
189743	1.7		0.28	infinity	0.00
Total Fatigue Damage					0.022

Table A.3. Top-down cracking fatigue analysis for single axle (PCC Control) (h=22) – IRC 58 (2011).

Expected Repetition	Flexural Stress (Mpa)	Flexural Strength (Mpa)	Stress Ratio	Allowable Repetition	Fatigue Damage
1	2	3	4=2/3	5	6=1/5
49801	3.0	5.55	0.49	1664478	0.03
47826	2.9		0.47	4553856	0.01
50131	2.8		0.46	19645521	0.00
35615	2.7		0.44	infinity	0.00
8177	2.6		0.43	infinity	0.00
4445	2.5		0.42	infinity	0.00
7189	2.4		0.40	infinity	0.00
7271	2.4		0.39	infinity	0.00
7271	2.3		0.37	infinity	0.00
8918	2.2		0.36	infinity	0.00
8918	2.1		0.34	infinity	0.00
38826	2.1		0.34	infinity	0.00
Total Fatigue Damage					0.043

Table A.4. Top-down cracking fatigue analysis for tandem axle (PCC Control) (h=22) – IRC 58 (2011).

Expected Repetition	Flexural Stress (Mpa)	Flexural Strength (Mpa)	Stress Ratio	Allowable Repetition	Fatigue Damage
1	2	3	4=2/3	5	6=1/5
66310	3.0	5,55	0.49	1108254	0.06
48018	2.9		0.48	2648913	0.02
16600	2.8		0.46	8721941	0.00
11433	2.7		0.45	58020041	0.00
12302	2.7		0.44	infinity	0.00
5762	2.6		0.42	infinity	0.00
17835	2.5		0.41	infinity	0.00
23734	2.4		0.39	infinity	0.00
28811	2.3		0.38	infinity	0.00
29268	2.2		0.37	infinity	0.00
40701	2.1		0.35	infinity	0.00
156538	2.1		0.34	infinity	0.00
Total Fatigue Damage					0.080

Table A.5. Top-down cracking fatigue analysis for tridem axle (PCC Control) (h=22) – IRC 58 (2011).

Expected Repetition	Flexural Stress (Mpa)	Flexural Strength (Mpa)	Stress Ratio	Allowable Repetition	Fatigue Damage
1	2	3	4=2/3	5	6=1/5
14350	2.9	5.55	0.47	3762971	0.00
13308	2.8		0.46	14646258	0.00
9439	2.7		0.45	infinity	000
19536	2.6		0.43	infinity	0.00
27741	2.5		0.42	infinity	0.00
32954	2.5		0.40	infinity	0.00
42722	2.4		0.39	infinity	0.00
36439	2.3		0.38	infinity	0.00
12485	2.2		0.36	infinity	0.00
8671	2.1		0.35	infinity	0.00
8506	2.0		0.33	infinity	0.00
48237	2.0		0.33	infinity	0.00
Total Fatigue Damage					0.005

Table A.6. Cumulative fatigue damage analysis results for PCC Control – IRC 58 (2011).

Slab thickness (cm)	Cumulative Fatigue Damage from Bottom-Up Cracking Analysis (Single+Tandem Axles)	Cumulative Fatigue Damage from Top-Down Cracking Analysis (Single+Tandem+Tridem Axles)	Total Cumulative Fatigue Damage (Top-down+Bottom-Up)
22	0.82	0.13	0.948

Table A.7. Thickness design parameters for PCC Control – AASHTO (1993).

Parameter	**Value**
Design Life (year)	30
Number of ESALs (W_{18})	620673155
Reliability (R) (%)	85
Normal Standard Deviation (Z_R)	-1.037
Total Standard Deviation (S_0)	0.35
Initial Serviceability (P_I)	4.5
Terminal Serviceability (P_T)	2.5
Loss of Serviceability	2
Drainage Coefficient (C_d)	1
Load Transfer Coefficient (J)	3.2
Required Minimum Thickness (in)	15.15
Design Thickness (cm)	38.354
Flexural Strength (MPa)	5.55
Modulus of Elasticity (MPa)	31357
Modulus of Subgrade Reaction (MPa/m)	285

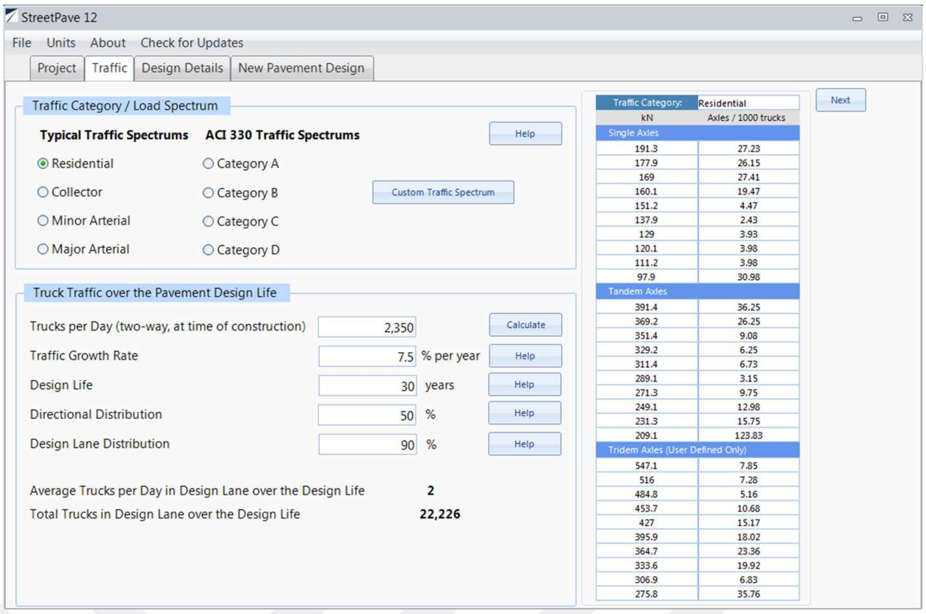

Figure A.1. Determination of traffic parameters-StreetPave v12 (2014).

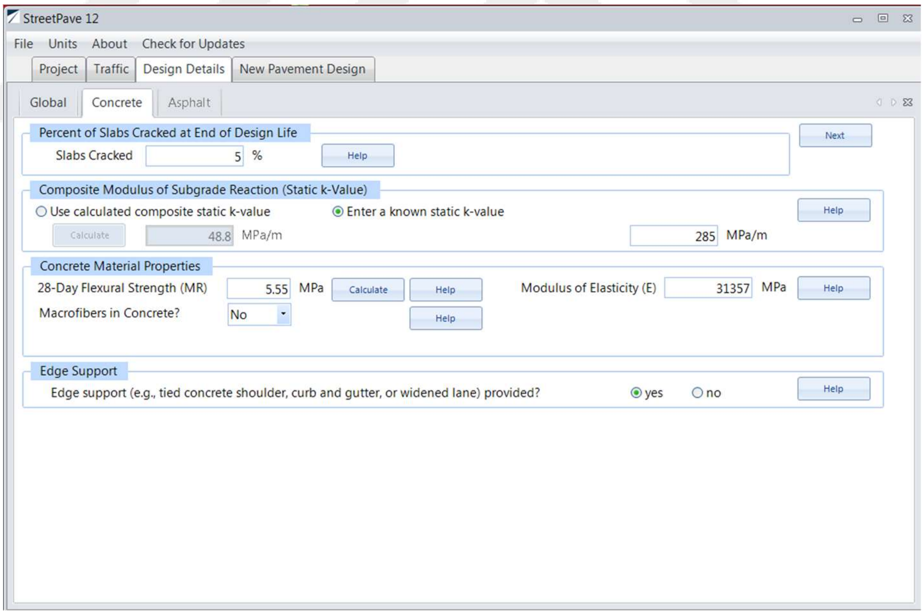

Figure A.2. Determination of design details-StreetPave v12 (2014).

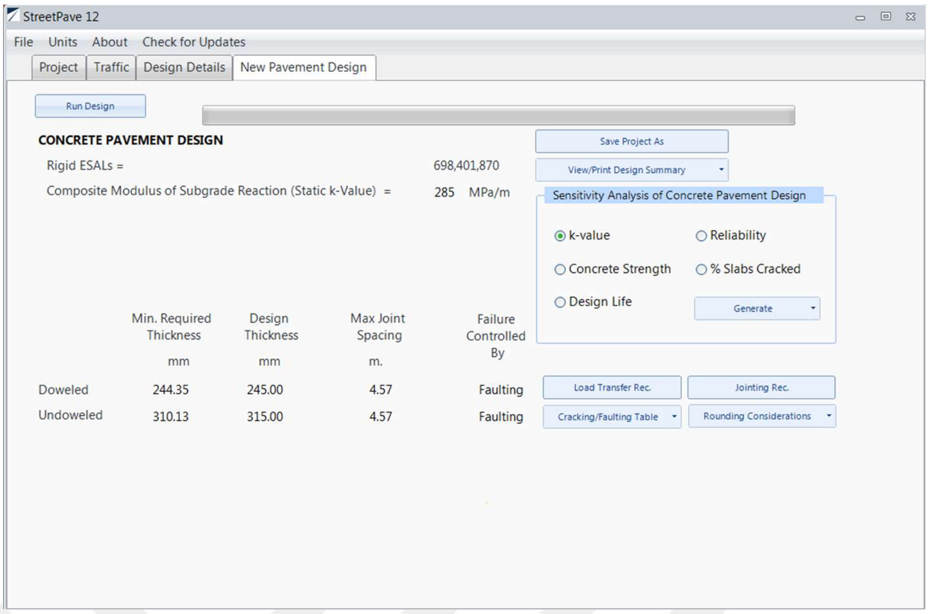

Figure A.3. Thickness design results (with concrete shoulder)-StreetPave v12 (2014).

Fatigue and Erosion Analysis

			Cracking Analysis			Faulting Analysis		
Axle Load, kN	Axles per 1000 Trucks	Expected Repetitions	Stress Ratio	Allowable Repetitions	Fatigue Consumed %	Power	Allowable Repetitions	Erosion Consumed %
Single Axles								
191.3	27.23	1,087,518	0.349	unlimited	0	13.955	4,753,202	22.88
177.9	26.15	1,044,384	0.326	unlimited	0	12.069	12,732,556	8.2
169	27.41	1,094,706	0.31	unlimited	0	10.891	35,400,170	3.09
160.1	19.47	777,597	0.295	unlimited	0	9.775	351,390,022	0.22
151.2	4.47	178,524	0.28	unlimited	0	8.718	unlimited	0
137.9	2.43	97,050	0.256	unlimited	0	7.252	unlimited	0
129	3.93	156,957	0.241	unlimited	0	6.346	unlimited	0
120.1	3.98	158,954	0.225	unlimited	0	5.501	unlimited	0
111.2	3.98	158,954	0.209	unlimited	0	4.716	unlimited	0
97.9	30.98	1,237,286	0.186	unlimited	0	3.655	unlimited	0.01
Tandem Axles								
391.4	36.25	1,447,760	0.297	unlimited	0	14.378	4,016,893	36.04
369.2	26.25	1,048,378	0.281	unlimited	0	12.793	8,226,112	12.74
351.4	9.08	362,639	0.269	unlimited	0	11.59	18,126,421	2
329.2	6.25	249,614	0.253	unlimited	0	10.171	107,345,694	0.23
311.4	6.73	268,784	0.24	unlimited	0	9.101	unlimited	0
289.1	3.15	125,805	0.224	unlimited	0	7.844	unlimited	0
271.3	9.75	389,398	0.211	unlimited	0	6.908	unlimited	0
249.1	12.98	518,398	0.194	unlimited	0	5.824	unlimited	0
231.3	15.75	629,027	0.181	unlimited	0	5.021	unlimited	0.01
209.1	123.83	4,945,549	0.165	unlimited	0	4.104	unlimited	0.05
Tridem Axles								
547.1	7.85	313,515	0.287	unlimited	0	14.577	3,727,269	8.41
516	7.28	290,750	0.272	unlimited	0	12.967	7,503,069	3.88
484.8	5.16	206,081	0.256	unlimited	0	11.446	20,422,553	1.01
453.7	10.68	426,540	0.241	unlimited	0	10.025	151,839,859	0.28
427	15.17	605,863	0.227	unlimited	0	8.88	unlimited	0.01
395.9	18.02	719,687	0.212	unlimited	0	7.633	unlimited	0.01
364.7	23.36	932,957	0.196	unlimited	0	6.478	unlimited	0.01
333.6	19.92	795,569	0.18	unlimited	0	5.42	unlimited	0.01
306.9	6.83	272,778	0.167	unlimited	0	4.587	unlimited	0
275.8	35.76	1,428,191	0.151	unlimited	0	3.705	unlimited	0.01
				Total Fatigue Used %:	0		Total Erosion Used %:	99.12

Close

Figure A.4. Results of fatigue and erosion analysis-StreetPave v12 (2014).