Chapter 2
Literature review

2.1 Introduction

Mechanistic-empirical (ME) based pavement design has been widely accepted and recognized as a first tool supported by basic mechanics in pavement engineering. Pavement researchers in past studies have shown growing interest in ME based design. Before the idea of ME based design, earlier, pavement design was completely based on practical experience which was later on supported by experimental formulations. The need of sustainable design and recent advancement in computational efficiency has urged the necessity of more scientific methods for solving pavement engineering problems. This ME based design methodology may be universally accepted and provide a more precise prediction of structural response in asphalt pavement.

This chapter provides detailed review and critical conclusions of the existing scientific literature related to pavement design, material characterization, and finite element modelling of asphalt pavement and tire. Following the introduction, section 2.2 provides a concise overview of the prevailing pavement design methodologies. It highlights the main differences between two important codes from the design perspective of Indian roads, specifically the American AASHTO (2002) and IRC:37 (2018). Section 2.3 provides detailed discussion on pavement modelling considering past experiences, recent advancements, and current practices. In this section, detailed review of various pavement material modelling techniques like analytical, mathematical, experimental models based on laboratory and field test data, and numerical modelling has been presented to study asphalt pavement system. Since material characterization of unbound granular layers and

asphalt mixes are one of the main objectives of the present study so this section provides review of literature on experimental data collection using resilient modulus test for unbound granular materials (UGMs) and creep compliance test for asphalt mixes. Last section provides an extensive review of past literature on FE modelling of asphalt pavement system.

2.2 Review of pavement design

The early stage of asphalt pavement design was mainly based on empirical relations established from experimental data. Even in the recent era, the role of experiment and field observation is undeniable. Huang [16] noted that prior to the 1920s, the design of pavements relied entirely on observational data and experiential knowledge. In the initial phases of pavement design, crucial factors such as the impact of subgrade soil on the thickness of the pavement were overlooked. The post-World War II era saw the introduction of heavy vehicular loads, leading to the failure of pavements that were designed empirically. In an effort to understand the performance of pavement materials under varying load and environmental conditions, AASHTO carried out a comprehensive test in 1961 [19]. This rigorous testing and subsequent scientific research paved the way for the creation of a new set of design codes.

The surge in automobile usage, advancements in technology, and the expansion of highways have collectively led to substantial enhancements in design methodologies. The 20^{th} century witnessed the development of numerous design standards, each suited to the environmental conditions and locations of their origin. For instance, the AASHTO code, developed in the USA, has gained acceptance in various countries worldwide, while the IRC:37 [20], specific to asphalt concrete pavement design, is widely used in India.

2.2.1 Principles of asphalt pavement design

Asphalt concrete pavement mainly consists of asphalt layer at the top as wearing/surface and/or binder course laid over unbound/bound aggregate layers. These layers are firmly supported by soil subgrade layer at the bottom. The subgrade layer is generally characterized as top 500 mm compacted layer of natural soil. These layers are designed to endure repetitive cycles of vehicular load. To meet the transport requirement of the country, axle weight of the vehicles is increasing day by day, so use of more scientific data-based pavement design is the need of time for reliable and satisfactory pavement performance.

The pavement structure consists of various layers of different materials. The primary function of the pavement structure is to cater design traffic load for intended time period at an optimum economic cost. Furthermore, it should be serviceable for the vehicular loading under different environmental and field conditions. Thus, along with design traffic, the environmental conditions (temperature and moisture conditions) are also a crucial consideration in pavement design.

In the case of asphalt pavement, material failure occurs due to progressive deformation and accumulated strain. The ultimate strength of asphalt pavement is lower than the cement concrete pavement, even though, it is a most preferred construction method. The slow failure mechanism in asphalt pavement facilitates its repair and maintenance easily. The failure of asphalt pavement has been considered based on several criteria like excessive surface deformation of the asphalt layer, shear failure of the asphalt layer, shear failure of the granular layer, and surface cracking of the asphalt layer. Surface cracking of the asphalt layer, in particular, has been the subject of extensive research, with various types of cracks resulting from traffic loads or cyclic thermal loads being identified.

There are two primary methodologies for the design of asphalt pavement which is used worldwide. The first is the empirical method, which has its roots in the past, and the second is the mechanistic-empirical method, a more contemporary approach. The empirical approach to pavement design has its origin in 1929, when the principle of soil classification was introduced [16]. This principle was subsequently refined, leading to the development of the group index method, now referred to as the AASHTO method, for classifying soil. In this system, soils and mixtures of aggregate in granular subgrade are categorized from a-1 to a-7, with each category having distinct implications and uses in road construction.

The development of the California bearing ratio (CBR) test method considered the role of soil strength in determining pavement thickness. CBR is characterized as the ratio of the resistance of a specific subgrade material to penetration relative to the resistance of crushed rock to identical penetration.

2.2.2 Empirical methods

Empirical methods based on the California Bearing Ratio (CBR), typically establish a correlation between the necessary pavement thickness and the anticipated traffic loads. These loads are expressed as a quantity of Equivalent Standard Axles (ESA). This correlation is encapsulated in the AASHTO guidelines [19], where the ESA count is determined by the ratio of a single axle load to 80 kN, raised to the fourth power.

A similar principle has been employed in the IRC:37 guidelines [20] to formulate a chart. This chart enables designers to determine pavement thickness based on millions of standard axles (msa) load and CBR values. An example of such a chart is depicted in Figure 2.1.

Despite the longstanding use of the empirical design method, its limitations have become increasingly apparent in recent times. The empirical approach is heavily dependent on field construction investigations, yet it fails to offer a holistic understanding of the mechanical behaviour of pavements. Consequently, any formula derived from these investigations may be subject to scepticism. Furthermore, the empirical method's applicability is limited in several crucial aspects, including loading conditions, environmental factors, and material variations.

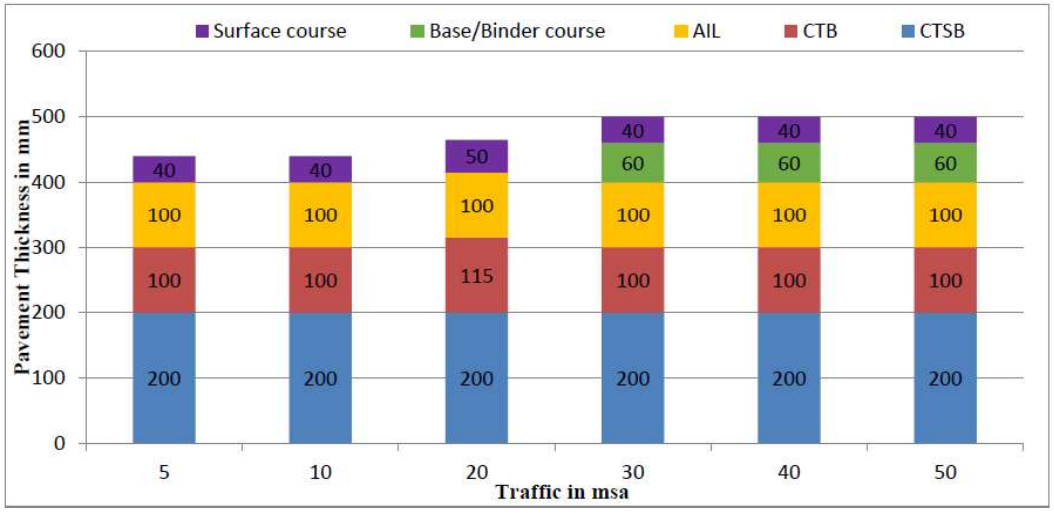


Figure 2.1. Pavement design chart (for 8% of effective CBR, IRC:37 [20]).

The demand for the application of recent advancements in knowledge and technology to establish a more scientific approach to pavement design is growing. This approach should not depend solely on laboratory findings but should also incorporate analytical solutions. The goal is to devise a method that can be readily adapted to new scenarios under different environmental conditions.

2.2.3 Mechanistic Empirical design

The Mechanistic-Empirical (ME) approach is a novel pavement design technique that merges the principles of mechanical science with empirical observations. This method calculates the primary response of the asphalt concrete pavement, including stress, strain,

and deformation, through the mechanical solution of the layered system. These obtained values are subsequently used as an input parameter to an empirical formula that correlates them with actual pavement performance. The ultimate objective is to create a more sustainable pavement system. The flowchart of the mechanistic empirical pavement design method is shown in Figure 2.2.

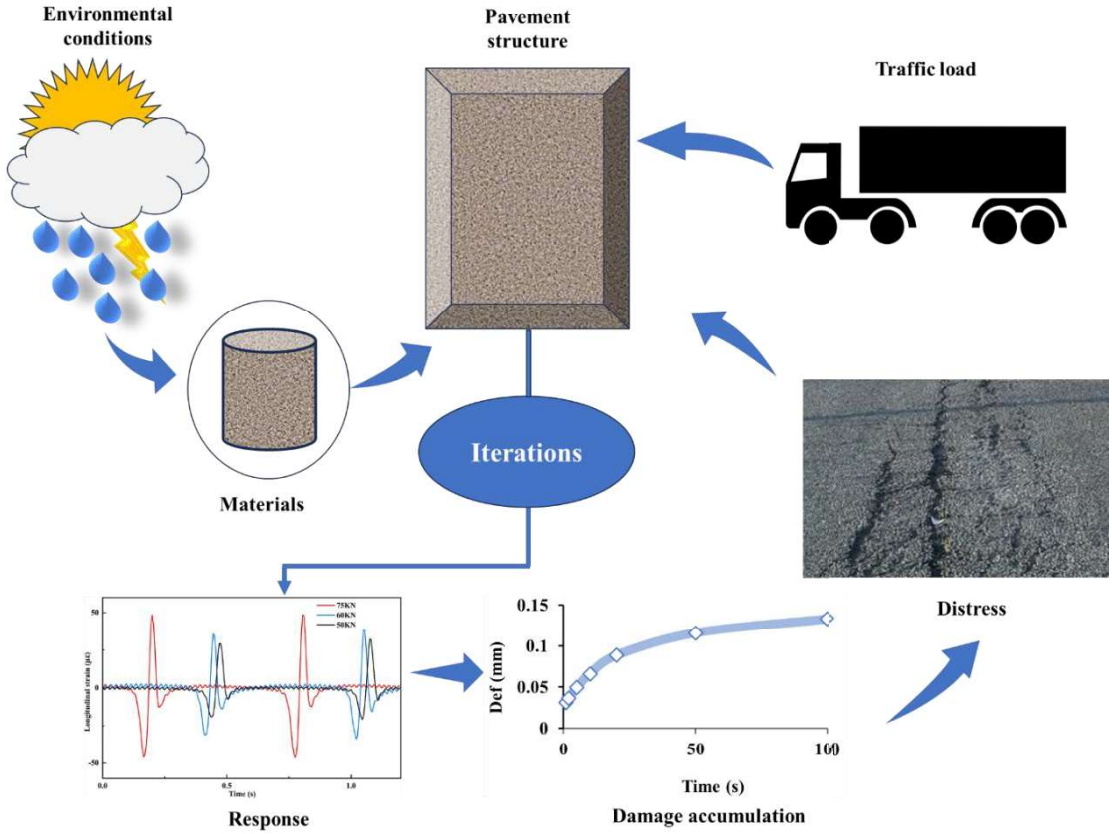


Figure 2.2. Flowchart of Mechanistic-Empirical pavement design method.

This approach employs a restricted use of mechanical principles such as continuity, equilibrium, and the principle of virtual work to a layered system. Materials are then modelled based on elastic, plastic, viscoelastic, and other suitable properties based on their response to applied load. The ME design process involves several steps: Initially, the layered pavement system is resolved using a chosen mechanistic model. The solution can be analytical (closed form) or numerical (like finite element). Materials can be modelled in various ways, including elastic, nonlinear elastic, resilient modulus, or

elastoplastic models such as von Mises, Tresca, Drucker-Prager, Mohr-Coulomb, and hardening or continuous yielding [14,15,21]. In the second step, the critical values calculated from the mechanical models are used as an input parameter to mechanistic-empirical formulas for the estimation of rutting and fatigue life of the pavement under repetitive cyclic loads and environmental conditions. Typically, uniaxial values such as the tensile strain at the bottom of the asphalt layer, vertical compressive strain at the top of the subgrade layer, vertical stress under the wheel load, and surface deflection under the wheel load are used for this final computation [16].

In both the empirical and ME methods, the pavement should be designed to ensure that various layers can resist a certain number of load cycles and maintain a serviceable condition. The serviceability of pavements is determined by limiting the pavement's critical distresses. Fatigue and rutting are two of the major distress modes in asphalt concrete pavement.

2.2.3.1 Fatigue failure in asphalt pavement

Fatigue cracking is a series of interconnected cracks primarily caused by load associated stresses in the asphalt layer. While there is some debate on this topic, it is generally accepted that cracking starts at the bottom of the asphalt layer due to high tensile stress and strain in a specific location. The crack then propagates upward to the surface beneath the wheel, in the longitudinal direction of the road as shown in Figure 2.3. Under repeated traffic loading, the cracks interconnect and form polygons on the pavement surface. The pattern resembles alligator skin, hence the term "alligator cracking" is used [16]. It's worth noting that fatigue failure from testing is characterized as the ultimate number of cycles after which the specimen fails in tension. This can produce varying results, and the number is contingent on test parameters such as the loading mode, etc. [22].

Comprehensive review of various fatigue testing methodologies was done by past researcher [23] and proposed the optimal procedure for assessing fatigue in the asphalt layer of flexible pavement. The repeated flexure test emerged as the most effective. To investigate fatigue, diverse equipment was utilized to implement the basic flexure principle. A prominent example is the four-point bending (4PB) frame [24].

Figure 2.3. Fatigue cracks in asphalt concrete layer.

Some researchers have reported that the total fracture of the test specimen signifies the final failure in stress-controlled tests [24,25]. However, Rowe [26] suggested a different failure criterion to safeguard the pavement structure. According to his suggestion, failure is denoted by a 90% reduction in initial stiffness at the point where the largest crack appears in the specimen. This led to a revised concept of failure, which hinges on the initiation of the crack rather than the complete failure of the entire specimen.

The universally accepted method to define fatigue from different tests remains a contentious issue. Among the recently proposed concepts [27], failure was predicated based on the change in dissipated energy (ΔDE) between two successive loading cycles.

In 2013, Nega et al. [28] explored characterization methods for fatigue performance in western Australia.

2.2.3.2 Rutting failure in asphalt pavement

Another significant distress type in flexible pavement is rutting, which is characterized as surface depression in the wheel paths as illustrated in Figure 2.4. There might be an uplift along the sides of the rutting. It is generally accepted that rutting occurs due to the permanent deformation of different pavement layers or subgrade.

The subgrade is more susceptible to applied traffic load, and permanent deformation in this layer occurs due to compression, consolidation, shear failure, or any other type of movement. Plastic deformation of asphalt can also lead to rutting deformation, particularly where compaction has been insufficient or where asphalt is exposed to high temperatures. Excessive rutting can result in unserviceable conditions across the entire pavement [16].

Figure 2.4. Rutting in asphalt concrete layer [29].

The subgrade is considered to contribute significantly in the rutting failure of pavement. Researchers [30–33] have utilized accelerated pavement tests to demonstrate the

contribution of granular layers to surface rutting. It has been stated that granular layers account for 30% to 70% of rutting deformation at the pavement surface. Consequently, permanent deformation in the granular layers could play a vital role in pavement failure, which is recognized as rutting at the surface. This highlights the importance of efforts to comprehend unbound granular material (UGM) behaviour in response to traffic loading.

2.2.3.3 Structural response of asphalt pavement

As previously mentioned, the ME method requires the solution of the given geometry under vehicle loads in terms of stress and strain. Consider an element in Figure 2.5, it is demonstrated that the pressure load from the tire at the surface induces stress on each surface of the cubic element. For this element to achieve a state of equilibrium, the opposing faces of the cube must withstand stress of equal magnitude but in opposite directions. Stresses are categorized into two types, normal and shear stresses, represented as σ and τ respectively.

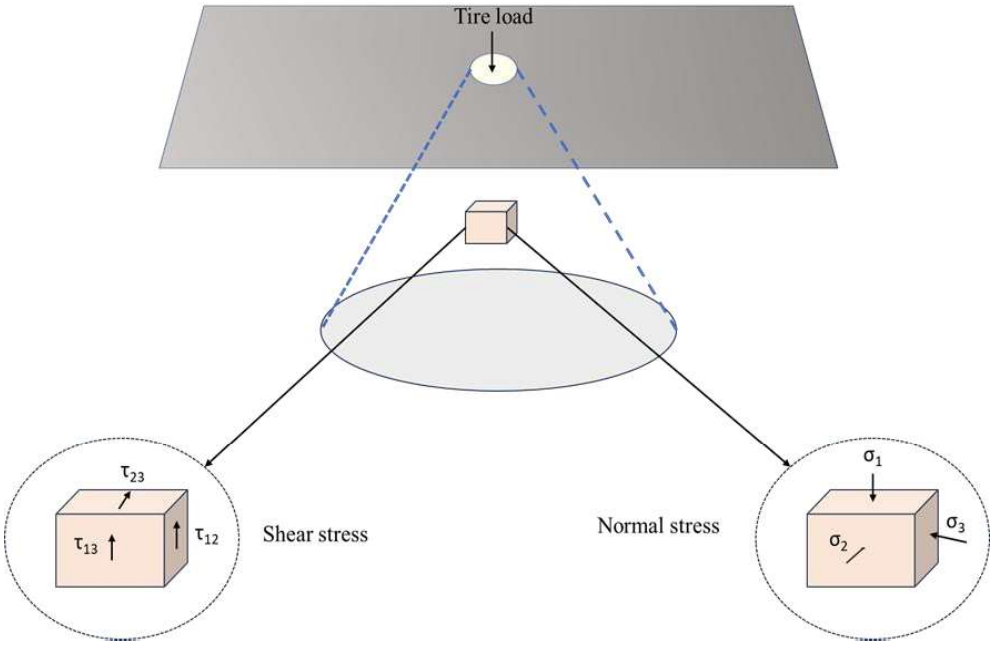


Figure 2.5. Induced shear and normal stresses under tire load.

These stresses induce deformation, and consequently strain, in the same direction as the stress in the element. Strain can also be divided into two types, normal and shear strain,

denoted as ε and γ respectively. In continuum mechanics, a relationship exists between stress and strain, which is determined by material behaviour. When materials are assumed to exhibit linear elastic behaviour, the stress-strain relationship is defined by Equation 2.1 and Equation 2.2.

$$\sigma = E\varepsilon \tag{2.1}$$

$$\tau = G\gamma \tag{2.2}$$

where E and G are elastic and shear modulus, respectively.

A straightforward method for evaluating the response of asphalt concrete pavement subjected to tire pressure involves viewing the system as a uniform half-space. This half-space is defined by a surface plane (where the tire load is applied) and extends infinitely in other directions (horizontally and vertically downward).

The Boussinesq theory can be utilized in this condition, where stresses and strains are determined under a concentrated load on the surface of an elastic half-space. This solution can be expanded to produce a circular area for representing the tire loading area. To improve the analysis, Burmister [18] proposed a solution for the layered system, where the half-space is composed of several layers on the surface connected to a semi-infinite space at the bottom. These two solutions are considered the preliminary analytical methods to compute the mechanistic response of layered pavement system.

Considering these points, it becomes crucial to explore material models for the various layers of asphalt concrete pavement. The choice of the material model directly influences the computed mechanistic response.

2.2.3.4 Response of asphalt concrete mix

Asphalt mixture in asphalt concrete layer consists of aggregates, fillers, fibers, modifiers, air voids, and binder extracted as residuals in refineries [34]. Though different constituents present in asphalt mixture exhibits substantially different properties, collectively asphalt mixture can be categorized as viscoelastic materials [35–37]. The behaviour of these mixes can be considered as linear viscoelastic (LVE) subjected to small strain (less than 100 μm/m) amplitudes [38,39]. The strain amplitude is defined as half the difference between the maximum and minimum strain in a cyclic loading. The concept of viscoelasticity is discussed herein.

(a) Low temperature effect

The materials whose stress-strain relationship is time and temperature dependent like asphalt, asphalt mixture, etc. is considered as viscoelastic materials [40]. The attributes of viscoelastic materials, along with their unique characteristics and testing methodologies, have been extensively discussed in the works of Ferry [41] and Tshoegl [42]. As discussed earlier, under static load application at low temperatures, asphalt mixtures can be considered as elastic material. Stress-strain response of elastic materials was first given by R. Hooke as shown in Eq. 2.3-2.4.

$$\gamma = \frac{\tau_c}{G} \qquad under\ shear \tag{2.3}$$

$$\varepsilon = \frac{\sigma_c}{E} \qquad under\ tension \tag{2.4}$$

where γ is shear strain, τ_c is shear stress, G is shear modulus, ε represents axial strain, σ_c is axial stress, and E is elastic modulus of the material. As per Hooke's law, deformation in the elastic solid is proportional to the applied stress and it is completely recoverable after unloading.

(b) High temperature effect

However, at very high temperatures, asphalt exhibit viscous like behaviour and rate of deformation in perfectly viscous material is proportional to the applied shear stress. According to Newton's law of viscosity, the rate of deformation in viscous material is given as per Eq. 2.5 and consecutively deformation can be related with shear stress as per Eq. 2.6.

$$\frac{d\gamma}{dt} = \frac{\tau_c}{\eta} \tag{2.5}$$

$$\gamma(t) = \frac{\tau_c t}{\eta} \tag{2.6}$$

here, t denotes time and η is coefficient of viscosity. The transformation of viscous flow is a process that cannot be reversed. The energy derived from an external force used to produce deformation in the viscous material is converted into thermal energy.

Material properties of a viscoelastic material, like asphalt concrete, can be represented as a combination of an elastic object and a viscous fluid. The earliest and the simplest model to represent behaviour of a viscoelastic material was put forth by Maxwell [43]. When a constant stress, either σ_c or τ_c, is applied initially, the deformation of the most basic viscoelastic material, as per Maxwell's model, is determined by the sum of the ideally elastic deformation and the ideally viscous deformation as per Eqs. 2.7 and 2.8.

$$under\ the\ shear, \gamma(t) = \frac{\tau_c}{G_g} + \frac{\tau_c t}{\eta} \tag{2.7}$$

$$under\ the\ tension, \varepsilon(t) = \frac{\sigma_c}{E_g} + \frac{\sigma_c t}{3\eta} \tag{2.8}$$

where, G_g and E_g are instantaneous shear and elastic moduli respectively. Immediately after the application of load, deformation in the material increases with time, however the

rate of deformation is almost constant after a few loads cycle. If the initial deformation ,$\gamma(t)$ and $\varepsilon(t)$ is applied in the viscoelastic material and then keeping it constant, then as per Maxwell, the stress in the material decreases with time as given by Eqs. 2.9-2.10.

$$under\ the\ shear, \tau(t) = G_g \gamma_c e^{-\left(\frac{t}{\lambda}\right)} \tag{2.9}$$

$$under\ the\ tension, \sigma(t) = E_g \varepsilon_c e^{-\left(\frac{t}{\lambda}\right)} \tag{2.10}$$

where, $\lambda = \eta/G_g$ is known as relaxation time of the material. Keping deformation in the material constant, stress gradually decreases as material relaxes. The value λ represents the time of essential loss in the initially applied stress.

(c) Relaxation time

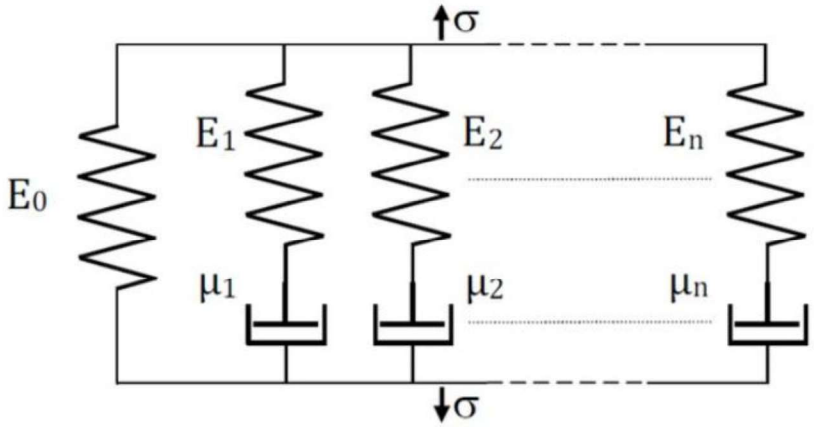
The principle of relaxation time, a key concept in the fields of physics and mechanics, was initially presented by Maxwell [43] in his seminal work, where he referred to it as the "time of relaxation" of the elastic force. When the duration of the load is small relative to the relaxation time (when $t/\lambda << 1$), the viscoelastic material exhibits behaviour similar to that of an elastic material and when $t/\lambda >> 1$, the material behaves more like a viscous material. The relaxation time concept encapsulates the notion of a system spontaneously returning to a state of equilibrium after being temporarily disturbed. It is defined as the ratio of viscosity of the material to elastic modulus. When applied rate of deformation in the asphalt mixture reduces to zero, these materials relax over characteristic period of time known as relaxation time.

Mix properties are significantly controlled by temperature and loading frequency. At high temperatures and under slow moving loads, AC mixes may exhibit a purely viscous flow, which ends up with load associated distresses such as pavement rutting. However, at low temperature and under fast moving load, AC mixes become progressively harder and

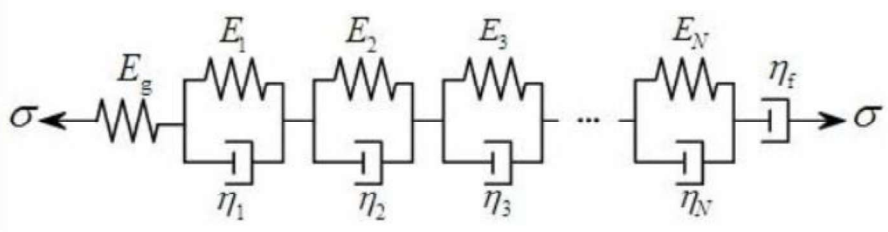
eventually brittle, which more likely behaves as an elastic solid and ultimately results in non-load associated distresses like low temperature cracking.

(d) Viscoelastic material models

Various models exist to represent such viscoelastic behaviour of the asphalt mixture [16]. The generalized Maxwell and generalized Kelvin models are two of the most recognized models employed to account for viscoelasticity (Figure 2.6). However, to better simulate the asphalt mixture behaviour, an assumption of the viscoelastoplasticity of asphalt layers is made [44].



(a)



(b)

Figure 2.6. (a) Generalized Maxwell model (b) Generalized Kelvin model [45]**.**

where, E_o, E_1, E_2… E_n are the elastic moduli and μ_1, μ_2….. μ_n are the coefficients of viscosity. The coefficients of viscosity in generalized Kelvin model have been represented by η.

2.2.3.5 Response of unbound granular materials

The base layer, subbase layer, and subgrade are typically modelled as a linear elastic model [46], nonlinear elastic model [47], or linear elastoplastic model [48]. Among nonlinear elastic models, a renowned stress-dependent model is the universal octahedral shear stress model [49], which accounts for the nonlinearity of UGM in a three-dimensional space. The plasticity of UGM has been modelled considering the classical plasticity criteria such as Drucker-Prager, Mohr-Coulomb, or Cam-Clay [50].

American Association of State Highways and Transportation Officials (AASHTO) provides specifications for using stress dependent nonlinear elastic properties of UGM. However, in India, pavement analysis and design as per IRC: 37 guidelines [20] are based on linear elastic properties of UGM which is derived empirically from soil CBR values. A comparison of these guidelines for material behaviour is shown in Table 2.1.

Table 2.1. Comparison of AASHTO and IRC:37 guidelines for various layer properties

Pavement layer	AASHTO guidelines	IRC:37 guidelines
Subgrade	Nonlinear stress dependent elastic behaviour modelled using resilient modulus and Poisson's ratio.	Elastic behaviour based on M_r and Poisson's ratio. Resilient modulus is evaluated based on soil CBR.
Subbase/Base (UGM)	Nonlinear elastic material properties based on repetitions of traffic load.	Elastic behaviour based on M_r and Poisson's ratio. Resilient modulus is evaluated based on soil CBR and layer thickness.

AC (Rutting)	Viscoelastic behaviour based on temperature, loading frequency, and dynamic modulus.	Elastic behaviour based on resilient modulus of asphalt concrete mix and Poisson's ratio.
AC (Fatigue)	Elastic behaviour based on elastic modulus.	Elastic behaviour based on resilient modulus of asphalt concrete mix and Poisson's ratio.

Apart from linear elastic properties of pavement materials as per the guidelines of IRC:37 [20], non-linear stress dependent properties of UGMs and linear viscoelastic properties of asphalt mix were also evaluated in this study. The method of material characterization has been reviewed in the following section.

2.3 Modelling of asphalt concrete pavement layers

In the field of pavement engineering, modelling serves as a tool to assess the performance of a layered structure under specified traffic loads. Essentially, the modelling technique is a method of converting a complex issue into a more manageable form to facilitate the derivation of potential solutions. This methodology is also employed in the engineering of asphalt pavements. The simplification process may involve the layer geometry, load, or material properties. The ultimate objective of modelling is to create a simulation that closely mirrors the actual problem. This can be accomplished by refining the simplifications made in earlier models. This section provides an overview of past scientific endeavours to model layered flexible pavement structures. Sub-section 2.3.1 discusses analytical closed-form solutions, sub-section 2.3.2 deals with laboratory and field experiments and measurements, and sub-section 2.3.3 examines various numerical modelling strategies.

2.3.1 Analytical models

Efforts to model the diverse behaviour of pavement layers were primarily undertaken in the 20th century. The theory proposed by Boussinesq served as a foundation for all subsequent solutions. Boussinesq's theory dealt with a semi-infinite homogeneous medium, assuming materials with linear elasticity. A concentrated load was applied on an axisymmetric coordinate, and the solution was derived using static equilibrium and kinematic equations. However, this solution is not directly applicable in pavement engineering due to the presence of different layers with varying elastic properties in an actual pavement system.

In Boussinesq's solution, the load was assumed concentrated at a point over the surface which do not simulate the real field condition. To better assess tire loading, Foster and Ahlvin [51] expanded the solution for a circular loading area by integrating Boussinesq's solution. They then depicted the layer's response in terms of horizontal and vertical stress and strain in charts.

To accommodate multilayer conditions in pavement, Burmister [18] explored solutions for two and three-layer elastic half-space, which significantly impacts solutions to pavement engineering problems. The solution also facilitated the evaluation of responses from multilayered systems. In this study, layers were assumed to be homogeneous, isotropic, and linearly elastic; the weight of the layers was ignored and loading was applied over a circular area as uniform pressure, and the layer interface was assumed continuous.

Huang [16] tried to utilize previous studies to multilayered systems. His book compiles previously published papers that present various charts for the responses of layered systems. These types of analytical solutions form the foundation of multilayer programs

such as KENLAYER [16] and CYRCLY [52], to evaluate pavement response under pre assumed shape of tire contact area. While the aforementioned studies consider the layers as linear elastic materials under static loading, other researchers have examined the effects of dynamic loading on asphalt materials and on UGM behaviour. Pavement engineers understand that UGM subjected to cyclic loads exhibit different behaviours with the change in the state of the stress applied. The most apparent reason for this is the compaction effect of loading cycles on UGM, which results in stiffer behaviour for a greater number of cycles.

2.3.2 Experimental models

This section provides a concise overview of the scientific efforts to model the behaviour of Unbound Granular Materials (UGM) and asphalt mixes in flexible pavement layers, based on laboratory experiments and field observations.

2.3.2.1 Material characterization of unbound granular layers

While UGM properties are diverse and complex, this section primarily focuses on the mechanical behaviour of UGM. Specifically, it reviews studies on elastic (both linear and stress dependent nonlinear) behaviour, aiming to offer preliminary insights into the final numerical simulation of layered systems.

Modelling is a process that holds link between experimental observation and analytical solution. Initially, failure mode in pavement layers is observed in the field. An experimental sample is then extracted from the pavement to simulate the same condition under controlled laboratory conditions, which yields scientific data. Finally, an analytical abstraction is made based on the laboratory data, enabling the researcher to derive a mechanical concept that governs the mechanical behaviour of pavement materials. This understanding enables future predictions of the material response to applied loading.

A review of the experimental models thus forms the initial stage for the analytical models developed in this dissertation. Some of these data are also used to verify the numerical simulation.

UGM's cyclic behaviour is not purely elastic, and some plastic deformation occurs in each cycle. The strain induced in each cycle comprises an elastic strain and a plastic strain. The elastic part of the strain is recoverable on unloading, while the plastic part is permanent. The resilient modulus is defined through the elastic strain in Equation 2.11.

$$M_r = \frac{\sigma_d}{\varepsilon_r} \tag{2.11}$$

where M_r is resilient modulus, σ_d is deviatoric stress, and ε_r is recoverable strain under repeated load.

(a) CBR based model

The resilient modulus is typically determined through the repeated load triaxial compression test. In India, IRC:37 [20] suggests Equation 2.12 and 2.13 to evaluate the resilient modulus of subgrade according to the existing California Bearing Ratio (CBR).

$$M_{rs} = 10 \times CBR \qquad for\ CBR\ \leq 5\% \tag{2.12}$$

$$M_{rs} = 17.6 \times (CBR)^{0.64} \qquad for\ CBR\ > 5\% \tag{2.13}$$

where M_{rs} is resilient modulus of subgrade soil (in MPa).

The influence of various factors on M_r has been studied through experimental investigations [53–55]. In these studies, a triaxial apparatus was used to determine the relationship between M_r and material properties. A study by Selig [56] indicated that there was a large lateral plastic strain in UGM during the first cycle of loading, while in the

following cycles the behaviour of UGM was observed as elastic. So, an initial conditioning period of 500 load repetitions were specified in triaxial compression testing to exclude plastic strains during first few cycles for evaluating M_r values. In the study carried out by Brown and Pell [57], the relationships between recoverable strain (elastic strain) and stress cycles, as well as permanent strain and stress cycles were investigated. The same study found that resilient strain was correlated to deviator stress.

One of the first studies to find that the nonlinearity of UGM is related to the state of stress was conducted by Boyce, Brown, and Pell [58]. In this experiment, the effects of aggregate size on anisotropic behaviour of UGM were investigated and it was concluded that the larger the aggregate, the more anisotropic behaviour in UGM is expected.

(b) Basic k-Θ model

The relation between material resilient modulus and stress state is known to pavement engineers. The resilient modulus of UGM increases due to an increase in confining pressure and these results in the nonlinear elastic behaviour of UGM. One of the first nonlinear models is known as k-Θ model, where M_r depends on bulk stress (Θ), as shown by Equation 2.14:

$$M_r = k_1 \theta^{k_2} \tag{2.14}$$

Here k_1 and k_2 are the material properties determined in the laboratory and θ is the bulk stress [59]. Later on, other researchers [60][61] also used this analogy to investigate the response of UGM under repeated loading. However, this model does not include the effect of deviator stress on UGM.

(c) Uzan model

This deviator stress has been considered in the model presented by Uzan [62]. It considers the effect of both confining as well as deviatoric stresses and thus it is capable of capturing modulus variations in UGM. The Uzan model is used to evaluate the stress hardening behaviour of UGM in axisymmetric finite element analysis. In this model, M_r is a function of bulk and deviator stress together as shown in Equation 2.15:

$$M_r = k_1 \left(\frac{\theta}{p_1}\right)^{k_2} \left(\frac{\sigma_d}{p_1}\right)^{k_3} \tag{2.15}$$

Here σ_d is the deviator stress and p_1 is the unit pressure. k_1, k_2, and k_3 are the material properties.

(d) Universal octahedral shear stress model

Witczak and Uzan [49] further expanded it to three-dimensional conditions by replacing the deviator stress term by octahedral shear stress (τ_{oct}). Since the universal model presented by octahedral shear stress considers variation of material properties in all three directions, this model is more suitable for 3-dimensional finite element analysis as given by Equation 2.16 - 2.17.

$$M_r = k_1 p_o \left(\frac{I_1}{p_0}\right)^{k_2} \left(\frac{\tau_{oct}}{p_o}\right)^{k_3} \tag{2.16}$$

$$\tau_{oct} = \frac{1}{3}\sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2} \tag{2.17}$$

Here I_1 is the first invariants of stress tensors, τ_{oct} is octahedral shear stress, p_o indicates the atmospheric pressure and k_1, k_2, and k_3 are the material properties evaluated using regression analysis.

(e) Lade and Nelson model

In an attempt to introduce an analytical base model which accounts for this nonlinearity, Lade and Nelson [63] derived an equation which correlated the M_r to mean normal stress and deviator stress. They used the concept of elastic energy and virtual work and indicated that the stiffness of the material should be a function of the first invariant of the stress tensor and the deviator stress as shown in Equation 2.18 – 2.19.

$$M_r = k_1 \left[\left(\frac{I_1}{p_1} \right)^2 + R \left(\frac{\tau_{oct}}{p_1} \right)^2 \right]^{k_2} \tag{2.18}$$

$$where\ R = 6 \frac{1+\nu}{1-2\nu} \tag{2.19}$$

where, ν is Poisson's ratio and all other parameters are same as stated earlier.

(f) Thompson and Robnett model

Unlike the aggregate materials in base and subbase layer, soil in subgrade layer behaviour to cyclic loading is different. Resilient modulus of aggregate materials was found to increase with increasing stress level as discussed in previous sections however, modulus of fine-grained soil decreases in proportion to the increasing stress level thus showing stress softening behaviour. The constitutive relations were primarily established between M_r and σ_d. The resilient behaviour of fine-grained soil is most commonly modelled using the bilinear model also known as Thompson and Robnett model [64]. The model used in the present study for finite element analyses is given below:

$$M_r = K_1 + K_3(K_2 - \sigma_d) \qquad when\ \sigma_d \leq K_2 \tag{2.20}$$

$$M_r = K_1 - K_4(\sigma_d - K_2) \qquad when\ \sigma_d \geq K_2 \tag{2.21}$$

where K_1, K_2, K_3, and K_4 are model parameters obtained from regression analyses of the repeated load triaxial test data and σ_d denotes the deviatoric stress. The parameter K_1 in the bilinear model is referred as breakpoint modulus and considered as a characteristic soil property.

(g) Review of other models

Research is still ongoing to propose new models to account for the nonlinearity of UGM. A relatively recent approach was introduced by Hjelmstad and Taciroglu [11], where M_r depends on both first and second invariants of the strain tensor instead of the stress tensor.

In the study carried out by Fahey and Carter [65], an experimental nonlinear equation was introduced in which the shear modulus of sand was corelated to induced shear stress. To examine the accuracy of nonlinear models in the prediction of mechanical responses of UGM, Gonzalez, Saleh, and Ali [66] carried out a series of simulations and field measurements. The conclusion confirms the validity of nonlinear models. In 2009, Lee, Kim, and Kang [67] presented a new nonlinear model which was based on an experimental method. This model linked the resilient modulus to induced stresses and the time history of the stresses.

The difference between various proposed nonlinear models was investigated by Attia and Abdelrahman [68] through experimental tests. Nine different constitutive models were investigated including Uzan (2-D and 3-D), Witczak (5 parameters) and K-θ model.

Recently, Araya et al. [69,70] studied triaxial tests and carried out an ABAQUS simulation. They introduced a new test, termed RL-CBR, and established a correlation between the California Bearing Ratio (CBR) and the stress-dependent M_r.

The effect of aggregate shape and characteristics on M_r was studied by Mishra and Tutumluer [71]. In this research, different nonlinear models were used, and experimental results were generated and compared against field data.

2.3.2.2 UGMs response to applied loading

While the experimental investigations reviewed above, provide researchers with input data, it is necessary to use this data in a full numerical simulation. The purpose of any constitutive model presented either experimentally or analytically is to be used for a numerical simulation. In this way, a constitutive model can contribute to the final design of a pavement system. The aforementioned experimental studies relate to the nonlinear elastic behaviour of UGM. Understanding the material response to applied loading to simulate actual field condition and pavement analyses has always been core of the research for pavement engineers. A few of the studies were reviewed establishing relation between pavement response and applied loading.

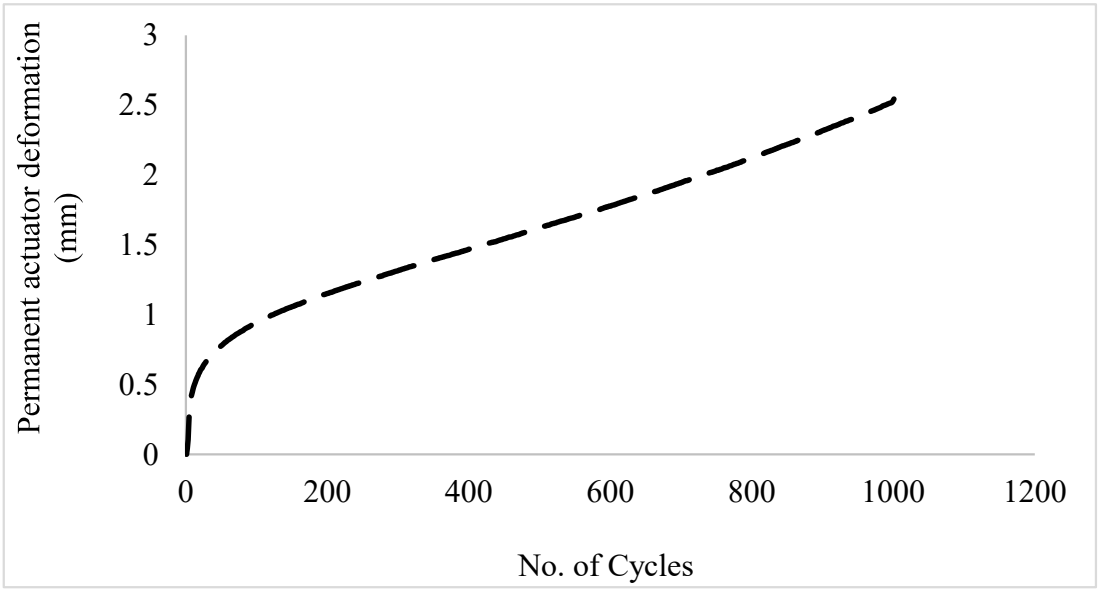


Figure 2.7. Accumulated permanent deformation v/s number of load cycles.

Deformation and accumulated strain in the pavement with the passes of cyclic load were studied. When Unbound Granular Materials (UGM) undergo cyclic loading, initially, the

slope of accumulated strain vs load cycle is steeper as it mainly consists of plastic strain and recovery is lesser. However, with the passes of loading cycle, this slope gets flatter as most of the strain is recovered. The pattern of accumulated permanent deformation against the number of load cycles can be depicted as shown in Figure 2.7 [72]. The scientific community has shown increasing interest in establishing a correlation that encapsulates the permanent strain of UGM as a function of the cycle number. Key equations in this area include the logarithmic function [73,74], a hyperbolic function focusing on final deformation [75,76], and the log-log equation proposed [77].

2.3.2.3 Material characterization of asphalt concrete mixture

Direct/indirect empirical approaches in the current pavement design procedures may result in premature pavement failure or over-designed pavements [12]. However, these empirical approaches are still being practiced in many developing countries around the world. In modern days, a lot of developing countries such as India are identifying an urgent need to shift their design practices from empirical to semi-mechanistic/mechanistic pavement design suited to their local requirements. In such cases, different material characterization tests will be required. One of the important aspects that need to be incorporated is the time and temperature dependency of the pavement materials. Whereas, for the characterization of time/temperature dependency of binding materials, Dynamic shear rheometer (DSR) tests are often used, however, for mixtures either no such tests are carried out or they are scarcely used. In the authors' opinion, the characterization of temperature dependency of the mixture is important for countries like India where wide temperature variation across the region and seasons is observed. Depending on material choices and their production processes, they exhibit different mechanical characteristics resulting in different performances.

As discussed in the previous paragraph, understanding the mix behaviour is important for reaching the expected lifetime. Therefore, uncertainties related to the mix design phase will result in a significant reduction in its lifetime. To minimise such risks, road agencies in India mandate the testing of these mix samples under controlled laboratory conditions, for example, the Marshall stability test, resilient modulus test, etc. However, tests measuring the viscoelastic characterization of the mixture are not posed as a requirement. It is well known that depending upon loading and ambient (temperature) conditions, asphaltic materials such as BC can exhibit viscoelastic/visco-elasto-plastic behaviour [78]. Hence, many researchers in the past have studied the viscoelastic behaviour of binder [79][80] and bituminous mixes [81][82][83][84][85][86] for the analysis of flexible pavement. It is important to measure the properties of BC at a wide range of frequencies and temperatures to obtain viscoelastic behaviour. At high temperatures or under slow-moving loads, it may exhibit a purely viscous flow reflecting its propensity towards rutting. However, at relatively low temperatures and fast-moving loads, it becomes progressively harder and eventually brittle, which makes it vulnerable to non-load-associated distresses like low-temperature cracking. Moreover, it is susceptible to fatigue-related problems at normal temperatures as most of the vehicular load is applied at these temperatures [87].

(a) Time-temperature dependency of asphalt mixes using creep compliance test

State-of-the-art equipment such as dynamic modulus test, bending beam rheometer test, and creep compliance test is recommended to be used in developed nations. However, in developing nations, often limited resources are available and therefore conducting different types of tests is not always possible. In such cases, a simple test is required to obtain the desired behaviour.

The characterization of viscoelastic properties is often done by measuring creep behaviour using creep compliance test as shown in Figure 2.8 [88]. The creep-related tests are popular because these tests make it possible to determine and separate the time-independent (elastic strain) and time-dependent (viscoelastic) components of the strain response [89] in a simplified way. In addition, parameters obtained from creep tests at low temperatures (m-value) are used to predict thermal cracking development and propagation, and those at high temperatures are used to predict rutting in asphalt layer [90].

Creep compliance test facility is readily available in most of the research institutions in India. Creep compliance test measures material deformation with time at different temperatures which is defined as time-dependent strain per unit stress [91]. In addition to temperature, creep compliance varies with the constituent's properties and their proportion in the bituminous mix. Compacted mix properties like air void also affect creep compliance results, hence, affecting generalized Kelvin model (GKM) parameters.

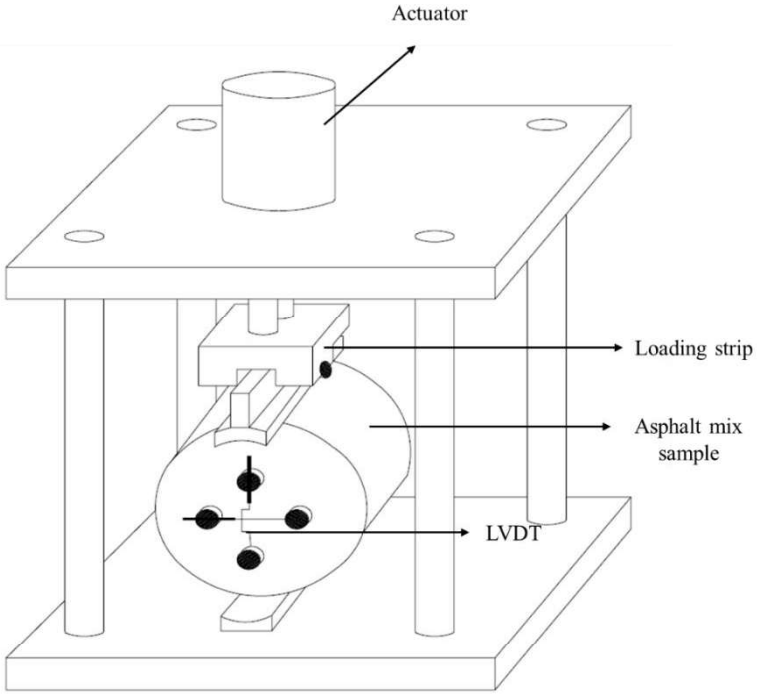


Figure 2.8. Creep compliance test set up [92].

Once creep compliance is obtained, viscoelastic behaviour is obtained through the time-temperature superposition principle [88].

(b) Current practices and modelling tools

Even though, as highlighted in the previous section, viscoelastic characteristics are important to be included in the design of pavements, most of the present mechanistic design standards [93][10][95] assume the BC layer as elastic during multilayer pavement analysis. Some of the modern-day tools (ABAQUS, ANSYS, 3-D Move) allow for the inclusion of viscoelastic characteristics for pavement design. However, in India, generally, elastic modulus (E) and Poisson ratio (υ) are used to characterize materials as linear elastic and subsequently used as input parameters in different design tools [96]. However, researchers [97] concluded that the elastic theory cannot represent permanent deformation or delayed recovery, which is an important characteristic of BC materials. BC shows elastic behaviour at low temperatures only and thus the use of elastic theory to explain material's response is limited to low-temperature analysis [98]. For intermediate and high temperatures, elastic theory often underestimates these responses [98].

As explained in the previous paragraph, since asphalt mixes exhibits viscoelastic behaviour, controlling and conducting experiments for all different types of combinations to represent field conditions is nearly impossible. In such cases, analytical and computational tools are readily used. Increasing computational capabilities have allowed pavement researchers to study complex material behaviour using tools like finite element (FE), discrete element (DE), etc. [78]. To include viscoelastic characteristics in such tools, often a generalised Kelvin viscoelastic model (GKM) or Prony series model [99][97] is used.

Prony series is a mathematical representation used to model the viscoelastic behavior of materials, essentially describing how a material deforms over time when subjected to stress, by expressing its relaxation modulus as a sum of exponential terms. It is often used in engineering fields like materials science and mechanics to analyze the behavior of materials like rubber or asphalt mixes under constant/cyclic loading conditions. The primary application of a Prony series is to represent the viscoelastic properties of materials. A Prony series is expressed as a sum of exponential terms, where each term includes a coefficient representing the amplitude and an exponent related to the relaxation time of the material. This method is used to determine the parameters of the Prony series by analyzing experimental data like stress-strain curves, allowing for the fitting of a model to the observed behavior of a material. These data are used as an input parameter in FE simulations of material response. Furthermore, Zhang et al. [100] showed that the generalized Maxwell model is well suited for the prediction of bituminous concrete deformation under repeated load.

The generalized Kelvin model (GKM) and the generalized Maxwell model (GMM) are the two widely used models for characterizing linear viscoelastic properties of bituminous mix [101]. These models best fit the relaxation modulus and creep compliance as a series of decaying exponentials also called as Dirichlet or Prony series [101]. Viscoelastic functions represented using the Prony series can be mathematically converted from the time domain to the frequency domain [101].

(c) The generalized Maxwell model

The GMM for a viscoelastic solid consists of a spring and several Maxwell elements assembled in parallel. Maxwell elements are a combination of spring and dashpot representing elastic and viscous components of the mix. The GMM is convenient for

analysing the relaxation behaviour of linear viscoelastic materials. The relaxation modulus of GMM has the form of the Prony series as shown below:

$$E(t) = E_{\infty} + \sum_{k=1}^{n} E_k \exp\left(-\frac{t}{\rho_k}\right) \tag{2.22}$$

Where E_{∞} is the equilibrium modulus, ρ_k, and E_k are the relaxation time and the stiffness of the k^{th} Maxwell element respectively. The complex modulus of GMM in the frequency domain is given by Eq. 23 [102].

$$E^*(\omega) = E_{\infty} + \sum_{k=1}^{n} \frac{i\omega\rho_k E_k}{1 + i\omega\rho_k} \tag{2.23}$$

where ω is the angular frequency.

(d) The generalized Kelvin model

The GKM consisting of a spring and several Voigt elements connected in series is convenient for analysing the creep behaviour of viscoelastic material [101]. The complex compliance (D^*) and creep compliance (D(t)) of GKM is given by [102] as follows:

$$D^*(\omega) = D_o + \sum_{k=1}^{n} \frac{D_k}{1 + i\omega\tau_k} \tag{2.24}$$

$$D(t) = D_o + \sum_{k=1}^{n} D_k \left[1 - \exp\left(-\frac{t}{\tau_k}\right)\right] \tag{2.25}$$

where D_o is instantaneous creep compliance, τ_k, and D_k are the retardation time and creep compliance of the k^{th} Voigt element respectively. Model parameters τ_k and D_k are determined by a series of constraint optimization processes using the least square

approach [103]. The sum of squares of the difference between predicted and experimental values are minimised to obtain these constant parameters.

2.3.3 Numerical models

The advancement in computer technology has amplified the curiosity of pavement researchers in the numerical representation of physical problems. Such numerical methods can be conveniently constructed and modified to tackle a range of problems.

This segment provides a comprehensive overview of the numerical representation of asphalt pavement. Numerical modelling can be scrutinized from several perspectives, including the type of analysis, which could be static or dynamic, the geometrical attributes of the model considering the model dimensions (axisymmetric, plane-strain, or three-dimensional), and lastly, the techniques it utilizes to resolve the problem.

In addition to the Finite Element Method (FEM), there have been alternative methods employed to analyse pavement structure. These methods include the generalized finite element, discrete element methods, and artificial intelligence [104–108].

This section reviews some of the significant studies investigating methods other than FEM and the later section provides a thorough review of the application of FEM in the simulation of pavement layers.

2.3.3.1 Generalized finite element method

The Generalized Finite Element Method (GFEM) presents an alternative strategy by using local approximations in the form of enrichment functions to enhance the solution. Moreover, GFEM can tackle issues related to mesh design, which often pose significant challenges when solving particularly complex 3D problems, especially in the presence of discontinuities. GFEM was utilized to study near-surface cracking [104]. GFEM has the

potential to utilize computational efficiency of modern-day computers for crack modelling within the same framework as FEM. In GFEM, an enrichment function is utilized to enhance the capacity of the shape-function of elements. Figure 2.9 demonstrates the concept of this shape function.

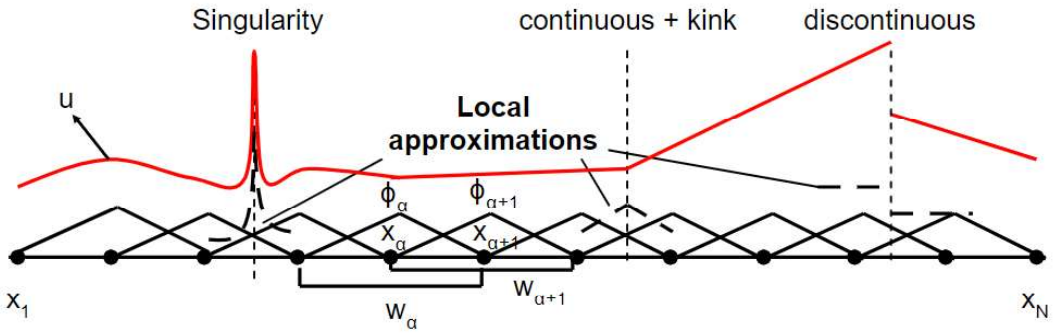


Figure 2.9. One dimensional representation of shape functions and local approximations [104].

This function improves the standard shape function of FEM with the ability to estimate displacement on irregularities. These irregularities could be discontinuous points, kink points, or singular points. Although the GFEM equips the FEM with the ability to simulate cracks, a drawback is the escalation in computation costs. The integration of this method with FEM in dynamic loading is challenging due to the interaction between surfaces.

In 2007, Saltan and Sezgin [109] attempted to merge the concept of neural networks with the Finite Element Method (FEM). In their study, an Artificial Neural Network (ANN) was utilized to simulate the behaviour of Unbound Granular Materials (UGM) used for the subbase. The ANN was trained using experimental data, and FEM was subsequently used as a tool for back calculation.

The principles used in ANN model to simulate material behaviours are shown in Figure 2.10. In this model, a solver composed of multiple nodes across different layers is trained

using a set of available data. This data set is input into the nodes. The ANN then attempts to predict the outputs, which are basically compared with the actual outputs. The weights of the layers are adjusted accordingly. This process is repeated until the ANN achieves satisfactory accuracy.

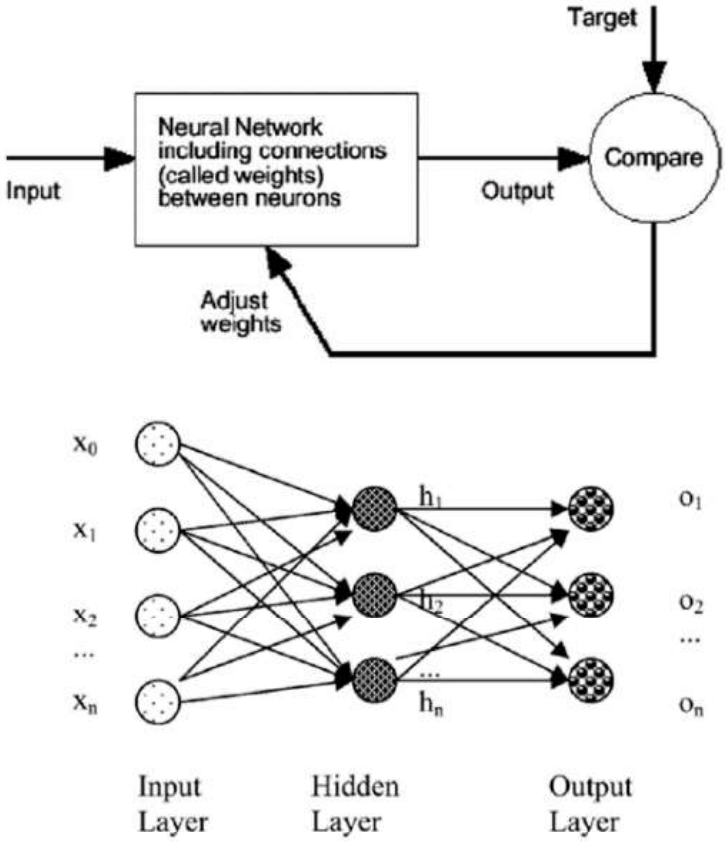


Figure 2.10. The process involved in ANN solver [109].

The predicted values can then be integrated into an FEM for back calculation of the data. This method has the advantage of being linked to an experimental package, thereby offering a higher possibility of validation. However, providing experimental data can be challenging when a complex analysis, such as dynamic analysis, is considered.

2.3.3.2 Discrete element method

Another numerical approach primarily used to address cracks and discontinuity in pavement layers is the Discrete Element Method (DEM). In this method, the element size

is reduced to the size of grains or particles, and the interaction between particles is modelled through a spring, dashpot-spring, slider, or a combination of these. Figure 2.11 depicts the concept of DEM.

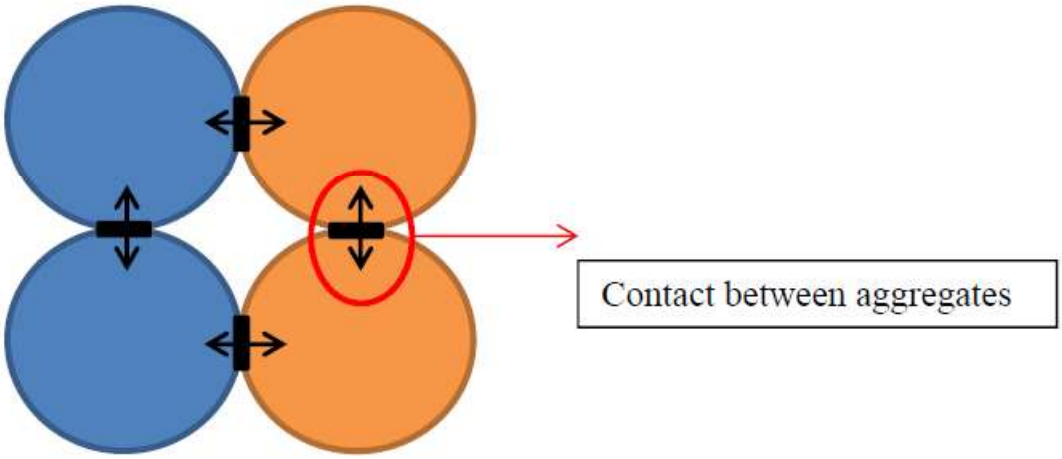


Figure 2.11. Principles involved in DEM.

DEM was employed to simulate asphalt aggregates and their interaction [110]. In this research, four numerical samples were modelled using DEM, and the effect of element size was examined. The researchers concluded that the element size had an insignificant impact on the simulation of creep stiffness for Asphalt Concrete (AC), but the calculation of permanent deformation and cracks required a fine element size.

All the aforementioned methods are either based on FEM or work closely with FEM. The FEM simulation is one of the most widely accepted methods for numerical simulation because it allows for complex analysis, such as a dynamically interactive layered system, along with complex material properties. Therefore, FEM was chosen as the primary approach to achieve the objectives of this research.

2.4 Finite element modelling of asphalt concrete pavement system

Analytical solutions are good for a rough estimate of pavement response, however, when dealing with contact modelling at tire pavement interface where large deformation of tire

body and dynamic loading is involved, it is challenging to find analytical solutions [54]. Therefore, to deal with such complex models, numerical solution techniques gradually evolved. Usually, the finite element method is widely employed to deal with tire pavement system. Estimation of pavement response with higher accuracy and reliability needs a good representation of the tire and the pavement model in such models. The Finite Element Method (FEM) is an engineering technique that utilizes computer technology to address complex engineering problems. From a mathematical perspective, it is a numerical method used to approximate solutions to a set of differential equations with specified boundary values. FEM utilizes procedures from variational calculus to minimize a specific error function. In this method, a physical problem's medium is divided into smaller domains, known as elements, and the partial differential equation (like an equilibrium equation) is solved within each of these elements.

The interconnection of the elements must meet certain criteria (typically continuity), and the solution should align with the defined loading and boundary conditions. Once the problem is solved within the element, a general solution for the entire domain can be projected. In the field of pavement engineering, FEM has been extensively employed in recent times to model pavement structures. This section provides a review of the application of FEM in simulating flexible pavement.

2.4.1 Finite element modelling of asphalt pavement

Section 2.3.1 gives an overview of the basic concepts on computing the response of layered systems, assuming linear elasticity. A numerical simulation was formulated based on analytical solutions. These analytical solutions (originally proposed by Boussinesq [17] and Burmister [18]) served as the foundation for various pavement software. One such program was BISIAR [111]. As per Kim's [112] study, this program was created by

researchers at Shell to predict the behaviour of layered systems, assuming linear elasticity. It utilizes Burmister's theory and is capable of solving multi-axle loading.

In 1977, CIRCLY [113] computer program was launched for use in pavement design. This program employs the linear elastic theory for a layered semi-infinite half-space. CIRCLY considers that the stress-strain response of the material is linear elastic and the medium is infinite in horizontal direction. The vertical dimension is confined by a horizontal stress-free surface at the top and an infinite depth in the downward direction. The software can account for the anisotropy of UGM. The ratio of anisotropy is assumed constant as 0.5 and cannot be modified, and the Poisson ratio in all directions is the same. Tire pressure is modelled as a uniformly distributed load on a circular contact surface. Since the materials are assumed to behave linearly elastically, the superposition principle is valid in all steps of the analysis.

Another software extensively used by pavement engineers is KENLAYER [16]. KENLAYER operates under the same assumptions as CIRCLY, and the linear elastic theory is used to calculate pavement responses in different layers. KENLAYER cannot model anisotropy, but modelling nonlinearity through iterative calculations is possible.

Hadi and Symons [114] used the same approach to study the number of allowable repetitions on a layered flexible pavement. In this study, the upper layer material was modelled as an isotropic material, while the under-layer material used for the subgrade was modelled as orthotropic. MSC/NASTRAN and STRAND programs were used for the FE model and the results of the simulation were compared with CIRCLY. The allowable number of repetitions then was evaluated based on the guide provided by Austroads. It was concluded that the number of allowable repetitions calculated using CIRCLY was lower than from the FE model.

Another interesting comparison of this kind of simulation was presented by Ullidtz [115]. This investigation compared the field data measurement and calculated responses of different pavement programs: BISAR, CAPA3D, CIRCLY, KENLAYER, MICHPAVE, NOAH, SYSTUS, and VEROAD. The field data was obtained from full-scale pavement projects by CEDEX in Spain, DTU in Denmark, and LAVOC in Switzerland. The study concluded that the accuracy of prediction of linear elastic theory from the behaviour of UGM in pavement is an important question.

Wardle, Youdale, and Rodway [116] reviewed mechanistic pavement design, using CIRCLY to evaluate the vertical strain in layered pavement under four 20-ton wheels. A method was introduced for a more accurate estimation of the response of UGM layers. In this method, the UGM layer should be divided into sublayers, which can simulate the nonlinearity of UGM.

Tutumluer, Little, and Kim [117] studied the cross-anisotropic properties of materials, using the finite element program GT-PAVE and linear elastic program CIRCLY for modelling purposes. In later studies [47,118], KENLAYER and CIRCLY were used to evaluate the results of a linear elastic FE model developed in ABAQUS program.

KENLAYER and HDM-4 programs were compared by past researchers [119]. The programs were used to predict flexible pavement performance, and according to the results, KENLAYER was the best for performance analysis, while HDM-4 was the most appropriate tool for strategic analysis.

Ghadimi et al. [120] investigated the effects of geometrical parameters in the numerical modelling of flexible pavement systems. In the first step of this study, a layered flexible pavement was modelled through CIRCLY, KENLAYER, and ABAQUS [121]. Then the results were compared and concluded that results obtained from FEM showed a stiffer

behaviour of the layered system, and the surface deflection calculated by CIRCLY and KENLAYER was greater than that calculated by FEM.

Ghadimi et al. [122] proposed a novel technique for determining the nonlinearity of granular layers. This method, influenced by the work of Wardle [116], diverged from the traditional approach of segmenting the layers into sublayers. Instead, the analysis was segmented into sub-stages, with materials in each stage presumed to be linear elastic. In this study, the granular materials used for the base were modelled based on Uzan's model [62] of material behaviour assumptions, while the subgrade materials were modelled according to Thompson and Robnett's [64] approach. The outcomes of the analysis were subsequently compared with the computed results from CIRCLY and KENLAYER.

2.4.1.1 Stress-strain relation in 3-dimensional condition

When employing FEM to model flexible pavement layers, the initial consideration is the geometrical dimension. To date, three types of geometry have been considered for FE modelling: two-dimensional plane-strain, reduced three-dimensional axisymmetric, and full three-dimensional modelling. It's important to note that the dimensions of the model significantly influence the computational time. When considering the elastic theory in a three-dimensional medium, the correlation between stress and strain is expressed as follows in Equations 2.26 to 2.28 [123]:

$$\varepsilon_{xx} = \frac{1}{E}\left[\sigma_{xx} - \nu\left(\sigma_{yy} + \sigma_{zz}\right)\right] \tag{2.26}$$

$$\varepsilon_{yy} = \frac{1}{E}\left[\sigma_{yy} - \nu(\sigma_{xx} + \sigma_{zz})\right] \tag{2.27}$$

$$\varepsilon_{zz} = \frac{1}{E}\left[\sigma_{zz} - \nu\left(\sigma_{xx} + \sigma_{yy}\right)\right] \tag{2.28}$$

where, σ is normal stress, ε is normal strain, E represents modulus of elasticity of the material, and ν is Poisson's ratio.

While 3-D (three-dimensional) modelling is recognized as the most comprehensive simulation of a real-world problem, it requires significant amount of computational time. This necessitates the modelling of only a restricted area near the tire contact in 3-D. This limitation becomes more pronounced when taking into account the actual dimensions of a road. The real road possesses a substantial longitudinal dimension and width compared to the load-bearing area of a single tire. The segment chosen for simulation is constrained by boundary conditions (roller, fixed, etc.), which may impose additional limitations, and may influence the outcomes.

2.4.1.2 Stress-strain relation in plane strain condition

The geometric characteristics of a pavement can be represented under the assumption of plane strain, a typical simplification in geomechanics. In the context of plane strain modelling, it is postulated that one component of strain (for instance, ε_z) is zero due to the model's extended dimension along that axis (i.e., z). This assumption considerably simplifies the system of equations for the problem being modelled. Nevertheless, a significant limitation of this 2-D plane strain modelling is the assumption of the loading tire exerting a continuous strip pressure on the pavement surface. This is clearly not precise as the actual loading area is an elliptical region [124]. The stress-strain relationship for the plane strain condition is expressed in Equation 2.29 to Equation 2.31 [123].

$$\sigma_{xx} = \frac{(1-\nu)E}{(1+\nu)(1-\nu^2)}\left(\varepsilon_{xx} + \frac{\nu}{1-\nu}\varepsilon_{yy}\right) \tag{2.29}$$

$$\sigma_{yy} = \frac{(1-\nu)E}{(1+\nu)(1-\nu^2)}\left(\varepsilon_{yy} + \frac{\nu}{1-\nu}\varepsilon_{xx}\right) \tag{2.30}$$

$$\sigma_{zz} = \frac{E}{1+\nu}\varepsilon_{xy} \tag{2.31}$$

2.4.1.3 Axisymmetric formulations

The axisymmetric formulation, a common geometry used for pavement simulation, assumes that the model is symmetric along a vertical axis in cylindrical coordinates. This method simulates a 3-D model using a 2-D formulation. The stress-strain relationship in cylindrical coordinates is demonstrated in Equation 2.32 and 2.33.

$$\varepsilon_r = \frac{1}{E}(\sigma_r - \nu\sigma_z) \tag{2.32}$$

$$\varepsilon_z = \frac{1}{E}(\sigma_z - \nu\sigma_r) \tag{2.33}$$

The computational time required is similar to that of a 2-D plane strain. However, a significant limitation in pavement modelling is that only one tire can be modelled and the contact area must be circular. This makes the simulation of dual tires or multiple axles impossible in this formulation. Furthermore, interface shear, cracks, and shoulder conditions are overlooked [124].

One of the earliest significant studies [125] on the impact of the geometrical size of an FE model for pavement simulation explored the range of boundary conditions for axisymmetric problems, assuming the material to be nonlinear. The researchers sought to identify the appropriate domain by comparing the results of FEM with linear elastic layered solutions. They concluded that a boundary condition as far as 12 times the loading radius horizontally and 18 times the loading radius in depth could yield acceptable results. However, a 50-times R (loading radius) was suggested for more precise results.

The initial studies [124] on various geometrical parameters such as aspect ratio, element size, and model dimensions were carried out. Three types of 2-D plane strain, 2-D axisymmetric, and a 3-D simulation were built, and BISAR was used to obtain layered elastic results. It was found that the 3-D and axisymmetric FEM models produced similar results and were capable of modelling vehicular loading. Later on, other researchers [126] also studied the effects of the geometrical dimensions of FE models in 3-D, axisymmetric, and two-dimensional analysis. They found that a 3-D formulation could yield more accurate results, but the computation time was significantly increased. The researchers proposed a modification to the 2-D analysis that could improve the accuracy of the results to an acceptable level.

Holanda et al. [127] explored a new technique using objective-oriented programming. This technique was applied in both axisymmetric and three-dimensional models. By comparing the results with the available analytical solution, the authors concluded that the location of the model's boundaries had a significant effect on deformation. However, it can be addressed by considering sufficiently distant boundaries. A series of studies [47,118,128] used the ABAQUS FEM program to construct axisymmetric and 3-D models. Different nonlinear models were implemented and the results were compared. The differences were noticeable according to their conclusion.

2.4.1.4 Geometric considerations in finite element modelling

Three major factors regarding the geometry of models are the formulation of the medium (2-D plane strain, 2-D axisymmetric, or 3-D), the dimensions of the modelled area, and the mesh density. Ghadimi et al. [129] examined different models assuming plane strain, axisymmetric, and 3-D formulations. They reported the effect of each formulation on the results and compared the results with linear elastic programs CIRCLY and KENLAYER. According to their conclusions, the axisymmetric and 3-D models yielded an acceptable

agreement with the analytical solution; however, the difference between the plane strain results was not within the accepted range. The aforementioned literature considers the problem of medium formulation. It is also crucial to correctly select the dimensions of the model. The dimensions of the modelled area can depend on the type of analysis (static or dynamic) and the type of pavement response required (whether it is strains, stresses, or deformation).

In a study conducted by Uddin and Pan [130], a numerical simulation was carried out to model discontinuities and cracks in the pavement. The model was constructed using 3-D FEM formulation for a dynamic analysis and the results were checked against the results of the falling weight deflectometer (FWD). The pavement model assumed a block shape with the boundary conditions (BC) with rollers on the sides and fixed at the bottom. In this study, one axis of symmetry was assumed through the loading centre and therefore half of the actual experimental area was numerically modelled. The model had dimensions of 26.6 m in the X direction, 9.15 m in the Y direction, and a thickness of 12.2 m for the subgrade layer.

Another study by Mallela and George [131] carried out a numerical simulation of FWD load on the pavement. In this model, only a quarter of the actual experimental geometry was modelled. Several studies [47,118,128] utilized a dimension of 20-R (loading radius) in the horizontal axisymmetric model and 140-R in the vertical direction (3 m x 21 m). Ghadimi et al. [132] employed 55-R horizontal and 167-R vertical section in the axisymmetric model. It is clear that the overall geometrical sizes employed in recent studies [47,118,124,128–131] exceed those used in earlier FEM simulations [59,83,125].

The increased size of the models necessitates a greater number of elements and more computational resources. A coarse mesh (large element size and fewer element numbers)

results in less accurate outcomes, while a fine mesh (smaller element size and a larger number of elements) improves accuracy but at the expense of computational time. Here, the final geometrical aspects of a numerical model, which include mesh size and element type, have their impacts.

2.4.1.5 Effect of element types in finite element modelling

Elements can be considered either first order (linear) or second order (quadratic). In the first order elements, linear interpolation is used to estimate the values between Gauss points, while in quadratic (second order) elements, the interpolation has a higher polynomial order. Past researchers [124] examined the effect of interpolation order and found that the higher order element results in a more accurate result. However, the computation time for quadratic elements was also higher.

The impact of element interpolation studied by other researchers [120] demonstrated that the computational time for higher order elements (eight-node axisymmetric elements or 20-node brick elements) was longer than for linear elements. However, to achieve the same accuracy using linear elements requires a finer mesh and a larger element number, which subsequently increases the computational time. As a conclusion, using a higher order element is more efficient than a finer mesh of linear elements if the computation time is acceptable.

2.4.1.6 Interactions between layers

Advancements in FE based software enable the modelling of layer interactions through the use of the interface element. Researchers [133] used this capability to study the interaction effect in pavement layers composed of hot mix asphalt (HMA) laid over joint concrete pavement (JCP). In their study, the interface constitutive model was Mohr-Coulomb frictional behaviour. The main purpose of this study was to investigate crack

development in asphalt and concrete layers and the effects of different interface parameters were studied with regard to the developed cracks. Later on, same frictional behaviour of interface elements was used to study the effect of the interaction between soil and the AC layer [134]. They used the ABAQUS program in a FE simulation of dynamic loading. In this study, the asphalt layer laid on a Portland cement concrete (PCC) layer. The study investigated the influence of the different properties of the interface elements on the strain induced in pavement layers, and reported a significant difference in the final results.

2.4.1.7 Boundary conditions

It is well-known that boundary conditions influence the results of FE simulations. It should be mentioned that the effect of boundary conditions for static analysis is different from dynamic analysis. In static analysis, setting boundary conditions as rollers in the sides and fixed at the bottom can be accepted. If boundary element is far enough from the loading area, the error produced can be negligible. Researchers have proposed different criteria for the boundary conditions in static and dynamic analysis. In dynamic analysis, the rollers and fixed boundary may produce reflective waves which can induce another type of error (resonance phenomenon) which is not vanished easily by setting distant boundary condition. One of the possible solutions for this problem is the use of infinite elements. This method has been used by researchers to avoid the effect of reflected waves on FE simulations [135–138].

The ABAQUS program [139], which was utilized in this research as the FEM software, offers a variety of elements. This capability allows the simulation to be both accurate and time-efficient.

2.4.2 Finite element modelling of load/tire

The next aspect of modelling discussed in this dissertation pertains to the loading and boundary conditions in asphalt pavement system. Three key aspects are involved in simulating loads and boundary conditions. First, the loading can be modelled either dynamically or statically. Static loading simplifies the analysis, thereby reducing computation time, but pavement loading is inherently dynamic. The second factor is the interaction effect between boundary conditions and layers. Layers can be assumed to be fully bonded, or the interaction between layers can be considered. The third crucial factor in modelling is the tire pressure on the asphalt surface. The geometry of the tire print used to apply pressure, along with the pressure distribution on the tire-pavement contact area, can influence the modelling results.

2.4.2.1 Static loading condition

Several multilayer programs introduced for pavement engineering purposes use static loading conditions [18,111], [52]. Static loading cannot account for the time dependency of materials and inertial forces. The major advantage of static loading is its simplicity and reduction in computation time. Static loading overlooks the forces induced by mass and material damping. This type of loading is particularly beneficial for investigations into geometry or elastic materials, as the loading and unloading path is exactly the same for elastic materials.

Early studies [140–142] used static loading as a uniformly distributed pressure on the asphalt concrete surface. Static loading has also been used by researchers to investigate other aspects of FEM [124,126,127]. When static load is applied, the simulation results can be used in existing design codes, while dynamic load leads to direct calculation of the

deformation induced in pavement. Therefore, static loading continues to be of interest to pavement scientists [47,118,128,143,144].

2.4.2.2 Dynamic loading condition

There are also studies that investigated the dynamic behaviour of materials but did not include dynamic analysis [145–150]. These studies considered the change in material properties during the dynamic loading through experimentation, and then applied the results in a constitutive model. However, the effect of mass and damping force was not considered in the final analysis.

The first study [151] to model dynamic loading in the ABAQUS FEM program modelled a moving load considering uniformly distributed pressure variation as a trapezoidal shape. A function correlating the loading speed to loading cycle was then used and an analysis was conducted for different speeds. According to this study, increasing the speed of loading resulted in a decrease in surface deflection. Most interestingly, it was found that static loading led to more deflection than dynamic loading.

The dynamic analysis of asphalt pavement system was carried using new constitutive model known as the 'distributed state concept' to model the distress induced in pavement during dynamic loading [13,152]. In this concept, there is a reduced dissipation of energy in each cycle, leading to stiffer material behaviour in each subsequent cycle. Later on, researchers [48] simulated dynamic load as a triangular pulse over a 0.1 s period. The researchers elucidated that the outcomes of surface deflection might be anticipated to be less than 50% of the value computed from static loading. This was due to the absorption of energy introduced to the entire pavement system through damping and mass inertia, which are absent in the case of static loading. Beskou and Theodorakopoulos [153] conducted a comprehensive review of the previous approaches to the simulation of

dynamic loading, where the dynamic analysis was categorized according to representative models for AC foundations. Effect of varying asphalt layer thicknesses was studied considering dynamic analysis and concluded that static loading resulted in a smaller surface deflection [132].

In the aforementioned studies, the interaction between soil and pavement has not been fully addressed. In the field of pavement engineering, there are few studies which discusses the effect of the interaction between soil and pavement.

One of the earliest studies [154] carried out a nonlinear analysis of moving loads. The effect of the soil-pavement interaction was taken into account using the coupled boundary element method (BEM) and FEM. They presented a convolution integral of the interactive force transferred between layers. The iterative method was then used to calculate the stress and strain in each step. In the coupled FEM-BEM analysis, pavement layers were modelled through the FEM formulation and the whole soil body was modelled through BEM. The dynamic elastic and dynamic elastoplastic solutions were then compared. Two major conclusions were that the dynamic analysis resulted in less deflection (as described in studies mentioned earlier), and the elastic soil medium led to less deflection than the elastoplastic soil medium.

2.4.2.3 Shape of tire contact area

The final aspect of FEM for load simulation is the modelling methodology for loading pressure and tire contact area. Barksdale [73] conducted an investigation into the dynamic loading of traffic on a layered pavement structure. This study examined the shape and duration of stresses in different layers. An axisymmetric FE model employing linear elastic material properties was subsequently analysed. It was concluded that the stress pulses near the pavement surface closely resembled a half-sinusoidal form, and a

triangular form in the subgrade layer. The speed of the moving load was then correlated to the duration of stress pulses and the results were presented.

The concept of simulating tire load on pavement through repeated periodic pressure was recommended in early studies [73,82,155]. In FEM analysis, the tire pressure is transferred to the pavement layers through distributed pressure on the contact area between the tire and pavement. Since, tire is a complex structure to model accurately, past researchers assumed shape of the suitable contact area in their studies. Several shapes can be assumed for this purpose. The first one is a circular area, which was frequently employed by early researchers [18,113,141,142,156,157]. The similar shape is considered in IITPAVE analysis also. Huang [16] outlined a method to convert the circular shape of a loading area into a rectangular shape. A rectangular shape has an advantage in 3-D FEM modelling because brick elements are used in mesh generation. Using a circular area in full 3-D modelling may cause some mesh generation difficulties especially when brick elements are used. Figure 2.12 illustrates the relationship between rectangular and circular shape [16].

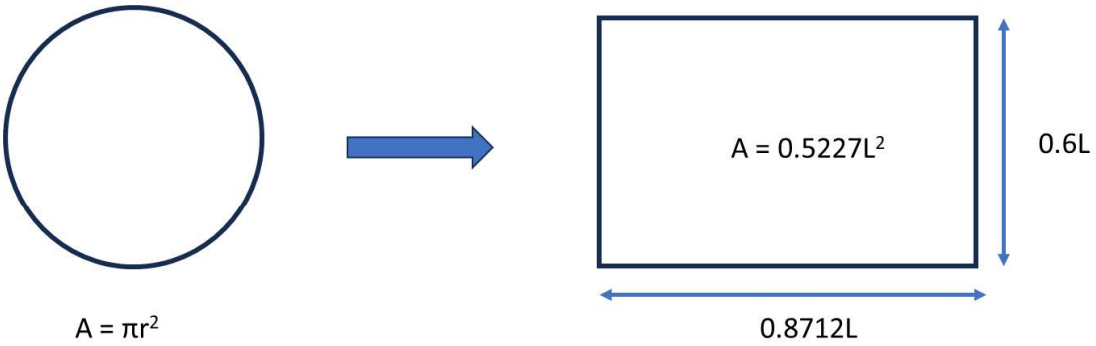


Figure 2.12. Conversion of circular contact area to rectangular [16].

Past researchers [98] studied tire pressure and the contact area in asphalt pavement using a 3-D FE model. The results were verified against laboratory and field data. The anisotropic behaviour of the asphalt layer was found to have a significant effect on the

results. In this study, the contact surface was modelled using rectangular strips with distributed pressures. The study by Fang et al. [158] also modelled tire load using strips considering time dependent pressure distribution. Later on, effect of different loading assumptions on the FE analysis of asphalt pavement were carried out using DIANA program [159]. A rectangular contact area was assumed and time dependent and time independent variation of pressure were considered. It was found that time dependent pressure resulted in decreased deformation at points near contact area, compared to results of a uniform distribution of load. Hadi and Bodhinayake [160] conducted a dynamic FE analysis using rectangular contact area to study the effect of nonlinear properties of unbound layers. Both static and cyclic loading conditions were considered. Cyclic loading was assumed to be a triangular periodic function considering Barksdale's [73] assumption. The study concluded that the nonlinear material assumption under cyclic loading yielded close results to data measured in field. Later on, researchers [132] considering same rectangular shape of contact area analyse pavement system with time dependent variation of distributed stress. The pressure variation was a triangular periodic function.

According to the aforementioned literature, it can be concluded that a circular area can be used as a representative contact area if axisymmetric modelling is used, and in the case of 3-D FEM, a rectangular area can be used without significant error.

2.4.2.4 FE modelling of solid tire

The current design method of AC pavement assumes a circular contact area and uniform distribution of vehicle load, which could significantly differ from the actual field conditions [161,162]. The assumption is used because it simplifies the calculation in pavement analysis. However, it is difficult to explain the behaviour of distresses in AC

pavement gradually developing from the surface under heavy vehicles using a simple uniform loading in FE analysis [161], [163–165]. Therefore, it is necessary to consider nonuniform contact stress distribution in the FE model

Figure 2.13. The CAD model of a solid tire [166].

A 3-dimensional solid tire was modelled to simulate the complex stress distribution over the loading area. It is noted that solid tire in India is generally used in forklift trucks and construction equipment. Solid tires are more popular in heavy-duty vehicles due to their ability to withstand excessive loads and operational capability in harsh environments [167]. It is also easy to evaluate the material properties of solid tire as it consists of single rubber material. The CAD model of a solid tire is shown in Figure 2.13. For modelling tire rubber, the suitable hyper-elastic material model was obtained using relevant tensile test data and a curve fitting approach [168]. Hyper-elastic models are generally used to calculate elastomer's properties that respond elastically when subjected to large deformations [169]. These material models are expressed in the form of polynomial functions.

2.4.2.5 FE modelling of pneumatic tire

The complexities of a real tire that contains the components such as tire tread, sidewalls, belts, plies, and inner-liners as shown in Figure 2.14, make the modelling of such a structure a very challenging task [170]. Large deformation, incompressibility, viscoelasticity, material nonlinearity, and its rolling/skidding contact with the pavement surface further make the numerical analysis more complicated.

Zorowski [171] was among the first to utilise a FE modelling technique to analyse the dynamic behaviour of a tire. Following his work, Ridha [172] proposed a linear FE model to relate the deformed shape of the tire as the result of shrinkage to its original mould shape. In these studies, due to the symmetry of the tire geometry, 2D tire models were usually generated. However, applications of the 2D tire model are limited to central symmetrical loading/boundary conditions such as pressure inflation and rim mounting.

Farroni et al. [173] presented a 3D mathematical-physical tire model to predict both the steady-state and transient behaviours of a tire. The coupling of multibody dynamics of the physical model can improve its capability further. Implementation of thermal, wear, and grip models can be taken into account for further analysis. Padovan et al. [174] developed a finite element scheme enabling the simulation of rolling tires. Padovan used the concept of moving Lagrangian to 3-dimensional isoparametric elements. The 3-D elements were used to model the tire tread, however, the inner components of the tire like a carcass, belt, and ply have been omitted from the study which can affect results considerably.

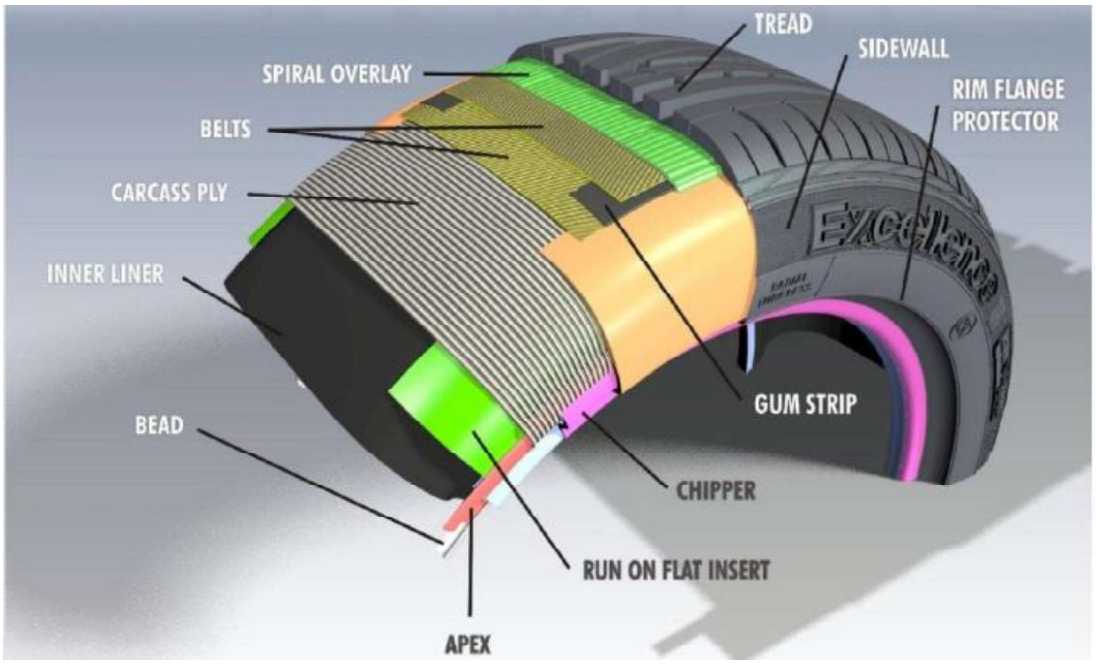


Figure 2.14. Cross sectional geometry of a pneumatic tire (Goodyear) [175].

Recently, Srirangam [176] has investigated many complicated aspects of a tire simulation e.g., a tire in rolling/sliding/braking status. In this analysis procedure, finite element techniques such as generating a 3D tire model from the corresponding 2D schemes, the inclusion of the complex tire tread patterns in a virtual way, the use of the surface elements to simulate reinforcements, and the advanced contact modelling capabilities have enabled the analysts to study the mechanical behaviour of the tire during various operating and loading conditions in an easier and more accurate way.

2.4.3 Finite element modelling of tire-pavement interaction

The finite element method (FEM) is the most widely used theoretical approach intended to provide approximate solutions to engineering problems. The popularity of FEM lies in its high computational efficiency in solving complex problems. It facilitates modelling the tire-pavement interaction (see reference framework as shown in Figure 2.15) for evaluating pavement response with great accuracy.

In the early years, internal codes such as NOSAP [177] and AGGIE [178] were used to model constitutive behaviours of rubber-like materials, but they were far from adequate in resolving the complexities associated with the tire-pavement interaction mechanism. The continuing increase in computing power and advances in numerical techniques have enabled the FE method to simulate complex phenomena such as frictional contact at the tire pavement interface. Many popular available FE tools such as ANSYS, ADINA, and ABAQUS have been widely used by both tire and pavement researchers [179].

Tires rolling/sliding over a dry smooth pavement have been modelled in past studies [180–183]. However, the primary focuses of these studies were on the mechanical behaviour of tires and little attention was paid to pavement response analyses. Cho et al. [184], developed a numerical–analytical solution for estimating the energy loss of a passenger car tire rolling on a flooded smooth pavement due to skid resistance. Based on NASA hydroplaning equation, Ong et al. [185] and Fwa et al. [186] performed the simulation of a locked wheel on the flooded pavement. The outcome of the analysis indicates, that with increasing sliding velocity; hydroplaning risk increases, and normal force between tire and pavement decreases thus skid resistance decreases. Tang et al. [187] proposed a finite element model for estimating wet skid resistance at different rainfall intensities. The effect of tire operating conditions, tire tread design, and pavement geometric design were mainly considered in the proposed model, and concluded that to ensure greater safety against skidding, a higher cross slope shall be provided. Additionally, an increase in rain intensity decreases skid resistance significantly.

Wang et al. [188] developed a model for simulating tire-pavement interaction to analyse the contact stresses and forces during vehicle manoeuvring. For investigating the structural response of the layered inelastic pavement, Wollny et al. [189] proposed a tire-pavement interaction model based on FE analysis.

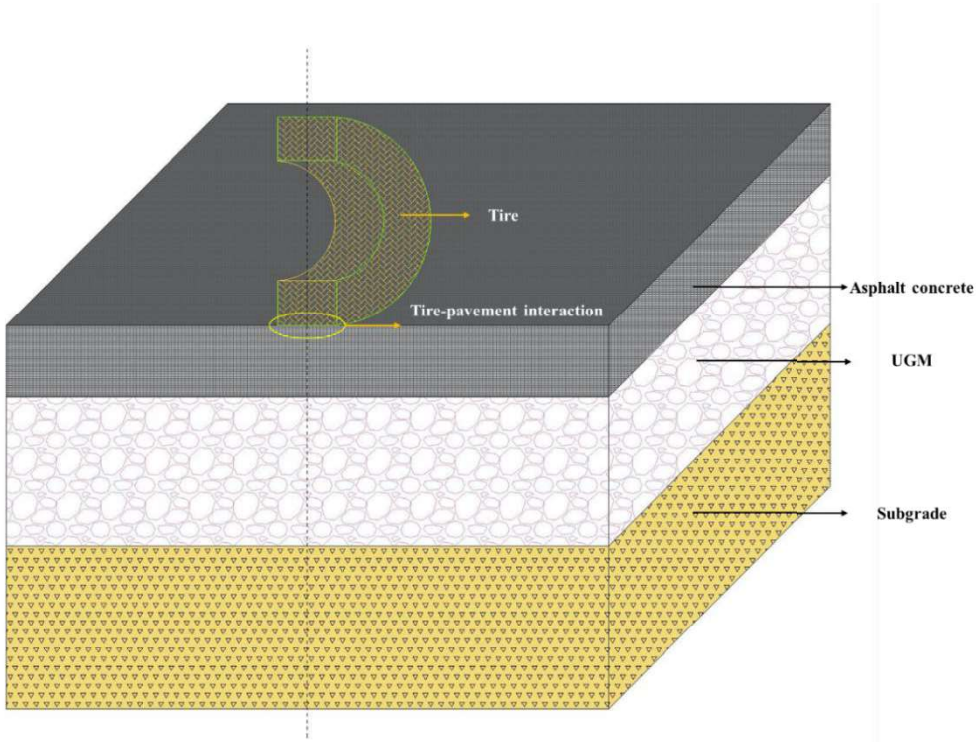


Figure 2.15. Finite element-based framework of tire-pavement interaction model.

However, most of the proposed models simulated the pavement sub-model considering it as a rigid flat plate that neglected the pavement surface characteristics. Anupam [190] proposed a finite element model for simulating asphalt concrete pavement as a grooved surface to analyse skid resistance. However, it simulates the tire sub-model as a non-rotating body which is a major limitation of this model. Additionally, few other studies investigated the pavement responses i.e., the stresses and strains developed within pavement materials under static or dynamic loads [191–193]. Although complex constitutive pavement material models were adopted in these models, the tire loading was idealized and represented by a uniform load patch. Such an unrealistic loading condition inevitably renders the capabilities of these models to accurately assess skid resistance at tire-pavement interface.

2.5 Problem statement and research gap

Asphalt concrete pavement is a complex system of multi-layer structure. The dynamic interaction of tire-pavement system and various layers, environmental conditions, and complex material behaviour makes pavement analysis further complicated. In India, mechanistic empirical design approach is practiced as per the guidelines of IRC:37 for the design of asphalt pavement. Linear elastic theory is used to analyse distresses in the pavement considering circular shape of tire contact area. These assumptions simplify the analyses at the cost of accuracy and may either underestimate or overestimate material strength in various layers. Past studies have shown that, materials in asphalt layer exhibits viscoelastic response to the traffic loading. The pavement response considering local temperature variations and loading conditions has been presented in many studies in past. However, viscoelastic properties of asphalt mixes under high temperature conditions for a tropical country like India still needs to be explored. Previous studies as discussed earlier found material stiffness in unbound layer nonlinear stress dependent. So, it is important to consider realistic material response to cyclic loading in pavement analysis. Other than this complex behaviour of materials, loading is also nonuniform at tire-pavement interface. Researchers found FE based method a good choice for pavement analyses considering complex material behaviour of pavement layers and nonuniform loading conditions. Based on the above literature review, few gaps in studies were identifies as follows:

- Previous studies have majorly focused on the viscoelastic material characterization of binders using DSR tests; however, limited research is available exploring asphalt mix behaviour subjected to environmental conditions specially in India. The characterization of time-temperature dependency of asphalt mixes are essential for countries like India where huge

temperature variation across the region and season is observed. The asphalt mix behaviour significantly affects the pavement performance and any uncertainty during the mix design phase may result in significant reduction in the design life of the pavement. The AASHTO in its mechanistic empirical pavement design guide (MEPDG) also sets provisions of using dynamic modulus test data as an input parameter for level 1 analysis. However, tests measuring viscoelastic behaviour of asphalt mixes are not posed as mandatory requirements in India due to lack of test facility, experimental data, and complexities involved in pavement analysis using commercial tools.

- Currently the design practices in India considers material properties of UGMs based on resilient modulus values obtained empirically from CBR test data. However, a number of studies reported that, CBR based elastic properties of UGMs may overestimate the material performance against axle loading. As discussed previously, stress dependent nonlinear elastic behaviour, more closely simulates the load response of these materials. Triaxial compression test facility for obtaining stress dependent M_r values is now available in few of the research laboratories in the country. The effect of incorporating stress dependent M_r values for better reliability in pavement analysis and design need to be explored for wide range of materials.
- Other than material properties, loading is considered as another important input parameters in pavement analysis and design. IITPAVE (tool used for the design of asphalt pavement in India) considers circular shape of tire contact area, thus uniform contact stress distribution at tire-pavement interface. This is a major simplification which makes the analysis easy but approximate. However, contact stress distribution is nonuniform as reported in previous

studies and consideration of actual tire for loading is needed to simulate realistic field conditions.

- To use these complex material behaviour and realistic tire loading as input parameters, one needs advanced computational tool like FEM for the pavement analysis. FEM also facilitates modelling of layer interactions and boundary conditions which makes pavement analysis further reliable.

This study attempts to address the aforementioned research gaps by suitably characterizing different materials used in various layers by conducting creep compliance and repeated load triaxial compression tests. FE model of realistic solid tire was developed to simulate nonuniform contact stress distribution of tire-pavement interface.

2.6 Summary of chapter

This chapter provides detailed review of numerical modelling techniques and their role in asphalt pavement design. It discusses two primary design strategies, one based on empirical equations and experiential knowledge, and the other based on mechanistic analysis. The chapter also highlights the ongoing advancements in the field aimed at refining the accuracy of modelling in the mechanistic design of flexible pavements.

Modelling, as a scientific instrument for mechanistic analysis, is divided into three main categories: analytical, experimental, and numerical. These categories are interconnected and not entirely distinct. The analytical method aims to formulate mathematical differential equations that forms the geometry of pavement structures. Experimental modelling aims to generate equations that closely represents material behaviour, while numerical simulation integrates the developed experimental models into differential equations to solve the entire problem.

One of the most commonly employed methods of numerical simulation is the FE analysis, which has been considered in this study. The use of this method in the analysis of asphalt pavement layers is examined. Three primary considerations in modelling through FEM are the geometrical idealization of the problem, the simulation of the pavement materials, and the simulation of tire loading.

The problem can be modelled as plane-strain, axisymmetric, or 3-D. The material modelling can be linear elastic, nonlinear elastic, nonlinear elastoplastic, or more advanced material modelling to simulate more closely with real field behaviour. Lastly, the simulation of the loading tire can be static, dynamic, or can consider the dynamic interaction between the soil and the Asphalt Concrete layer.

A review of the literature reveals a gap in complete three-dimensional modelling that considers nonlinear elastic material behaviour of UGM and viscoelastic behaviour of asphalt mixes in Indian conditions. Also, there is a need to simulate nonuniform contact stress distribution at tire-pavement contact considering 3-D tire geometry for pavement analyses to simulate more closely field like situation.

Based on the above literature studies and research objectives as highlighted in chapter 1, the research methodology was devised and presented in chapter 3. It will give a brief idea to accomplish set objectives for the present study. The order of experimental plan, material modelling, and numerical analysis method has been shown in the form of flowchart for a quick review of the methodology adopted. The last section of chapter 3 discusses FE modelling of tire-pavement system to be used as a tool for evaluating structural response of the pavement. It highlights the FE modelling of asphalt pavement, tire body for simulating nonuniform contact stress distribution at tire-pavement interface, element types, boundary conditions, and validation of FE model.