International Journal of Scientific Research & Engineering Trends
Volume 9, Issue 4, Jul-Aug-2023, ISSN (Online): 2395-566X

Analysis on Conditions on Asphalt Pavement Behaviour for Mechanistic Analysis

Kudikala Rajesh Kumar, Asst. Prof. Mudigonda Harish
Dept. of Civil Engineering
Holy Mary Institute of Technology & Science
Keesara, Bogaram, Telangana – 501301

Abstract- In the broadest sense, a pavement is any man-made surface designed to support the weight of moving vehicles, be they passenger cars or trucks carrying goods. In order for the subgrade (the most compressible section of the pavement structure) to support the weight of the wheels, the paving thickness and properties of the paving materials must be determined in order to construct a pavement. Pavement design is predicated on the principle of stress dissipation via the pavement layers to prevent failure of the subgrade soil. When a vehicle's weight is transferred through its wheels, the contact area between the tyres and the pavement is quite narrow. This causes a great deal of stress to be exerted on the pavement's surface. Since the stiffness of a pavement gradually decreases from top to bottom along a vertical portion, understanding how stress is distributed across a layered system of granular materials is crucial. Therefore, the design load in pavement is taken into account as the number of passes of a standard wheel load over a pavement section to be designed, as the wheel load by vehicle on a pavement at a certain design speed does not remain static on a fixed location. Therefore, the strength of the subgrade on which the pavement is to be built, as well as the projected loads on the pavement over the design period, are crucial to the design of the pavement. If the frequency of the loads is constant, a thinner pavement will be needed on weaker subgrade, and vice versa. The same is true for pavement, where a thicker section is needed for bigger load repetitions if the subgrade strength remains the same. Mechanistic design procedures are those that use models based on basic engineering mechanics to determine the stress level in a pavement and to foretell how the pavement will react and behave. However, empirical methods are those that rely on models derived from experience and observation of previous performance.

Keywords- Flexible pavement, Wearing course, Binder course, Subgrade, Moorum, Bitumen, Shear failure, Wheel load

I. INTRODUCTION

In this study, a mechanistic-empirical design technique is used to create a pavement plan that takes into account factors like stress, strain, and deflection. The principle of flexible pavement design is manifested in mechanistic or mechanistic empirical (ME) based design procedures that incorporate the treatment of life cycle costs and design reliability. This is because flexible pavement design focuses on structural aspects, such as material selection, strength characterization, and layer thickness determination. The research presented here synthesizes the theoretical study of pavement response from distress model with the experiences of several agencies.

The suggested method uses a hybrid of purely theoretical understanding and actual pavement reaction to derive the thickness of the pavement. Flexible Pavement Ingredients Typical pavement designs include layered systems, with harder materials used in high-stress areas and pliable ones used in low-stress regions. The typical flexible pavement's cross section is seen in Figure 1.

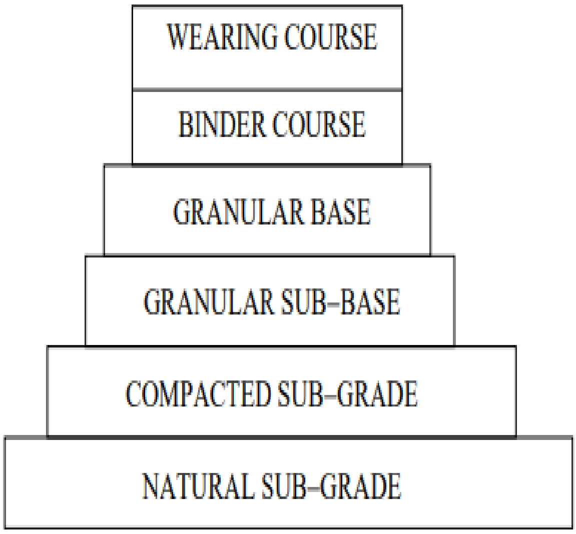


Fig 1. Sectional view of flexible pavement

1. Course with a wearing surface:
It is the final layer of flexible pavement and goes by several names, including wearing course and surface course. Typically, it is built with extensive grading. a combination of bitumen and other materials. It needs to be

sturdy, so it doesn't buckle under the weight of vehicles and provides a safe, nonslip surface for riders. To prevent water from degrading the pavement, the wearing course must be a waterproof layer. Common materials for the wearing course include premixed carpet, semi-dense bituminous, concrete, and bituminous concrete (BC).

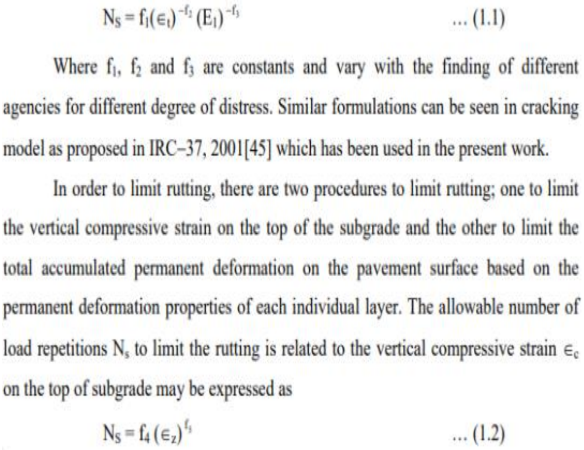
Course in Binders: The bituminous layer beneath the surface course is often known as the binder course or the bituminous foundation course. The binder course, which comprises bigger aggregates and less asphalt than the surface course, is not subject to the same stringent quality standards. Typically, a binder course is made out of bituminous macadam, also known as BM or DBM (densified bituminous macadam). Basic Education: The subsurface material underneath the Binder course is referred to as the base course. It might be made of sand, gravel, crushed rock, recycled asphalt, or a granular pavement like water-bound macadam (WBM) or wet-mix macadam (WMM). The granular base's compressive strength (CBR) should be between 80% and 100%.

Subbase Granular: The subbase course is a lower stratum of the foundation's foundation. Using two distinct granular materials helps keep costs down while still satisfying the need for a reliable absorption of stress inside the pavement's structure. Sub-base courses with more fines can act as a filter between the subgrade and the base course if the latter is open graded. The subbase material must meet the requirements of IRC: 37-2018 [45] with a CBR of at least 20% for design traffic up to 2 Msa and at least 30% for traffic over 2 Msa. Sand, moorum, gravel, laterite, brick metal, crushed concrete, crushed stone, or mixtures thereof may be used, so long as they fulfill the required grading and physical specifications. SUBGRADE The term "subgrade" refers to the layer of unprocessed or compacted soil directly under the sub-base layer. The term can also refer to the lowest layer of a paving system. Cut and filled subgrades alike benefit from proper compaction in order to maximize the subgrade's bearing capacity and minimize the necessary pavement thickness. Plan a layer of compacted subgrade with the necessary density and optimal moisture content at least on the upper 500 meters of the road's width if the subgrade is too soft.

2. Earlier Pavement Engineering Projects:

Data-Driven Strategies: You might think of empirical techniques as a strategy for designing pavements by looking at how they've worked in the past. The first version of this system appeared in 1929 [24], and it involved assigning letter grades (A1–A8) to the subgrade and letter grades (B1–B3) to the subgrade that were not uniform. Subsequently (HRB 1945) [39], the approach was modified by classifying soils into one of seven groups, from A1 to A7, and assigning each of these classes a unique group index. When calculating the sub-base and overall pavement thickness without a strength test, Steele (1945) [67] suggested the use of such a soil categorization and group index. In 1929 [38] and Portar (1950) [58], the California Highway Department was the first to employ the empirical approach with a strength test. The California Bearing Ratio (CBR), the resistance to penetration of a subgrade soil relative to a standard crushed rock, was shown to be correlated with pavement thickness. After that, the U.S. Corps of Engineers (1940) [74] created a CBR- based methodology for designing roads and airport pavement. Kentucky [6] design curves were discovered, and they were based on the CBR technique but also accounted for the impact of comparable wheel loads. The Wyoming method [55] is a variation on the CBR design approach [33] that factors in the impact of precipitation, water table depth, frost action, and the current state of the road and traffic volumes. The Method of Limiting Shear Failure: It was hypothesized that the layer thickness may be determined using the bearing capacity technique, in which the tension created in different layers is kept below their respective bearing capacities. In 1946, Barber [7] used Terzaghi's [38, 68] bearing capacity approach to pavement design and first presented this method. The bearing capacity of pavements may be calculated using logarithmic spirals, as described by McLeod (1953) [52]. Yoder (1959) [78] compiled an overview of all of these techniques. The South African approach to designing pavement takes this into account as well [53]. Despite the fact that shear is associated with the rutting failure of pavement, it is not sufficient to design the pavement's structure just with shear in mind. 1.3.3 Restricting Potential Deflective Actions To ensure that vertical deflection does not go over a predetermined threshold, the limiting deflection technique is used to calculate the optimal thickness of pavements. Using a revised version of Boussinesq's equation (Boussinesq, 1885) [20], the Kansas State Highway Commission capped subgrade deflection at 2.54 millimeters in 1947.

The United States Navy (1953) used the two-layer hypothesis proposed by Burmister (1943) [21] to restrict surface deflection to 6.35 mm. Deflection appears to be an advantageous design parameter since it is simple to quantify in practice. Instead of deflections, high loads and strains are the root cause of pavement failures [77]. The deflection approach is still used successfully for overlay design [30, 46], specifically for determining the thickness of reinforcing an in-service pavement. It has also been explored in the Indian setting (Biswas et al., 1995) [8, 10, 13] how significant deflection is as a design criterion for Bituminous pavement. Transfer Function or Distress Model: Numerous correlations have been shown between stress levels on a pavement and its overall performance. The primary transform functions used in the current mechanistic empirical methods for flexible pavements relate (i) maximum wheel load tensile strain in the Bituminous layer eventually to fatigue cracking and (ii) wheel load compressive strain or stress at the top of the subgrade layer to rutting on the surface [26, 27]. These models are generated using statistical correlations between the reaction of the pavement and the observed behavior of test specimens in the laboratory or on the road itself. Most mechanistic empirical design procedures rely on transform functions; however, many of the available models fail to show satisfactory agreement. There was potential in employing the Rutting rate notion (Majidzadeh et al., 1976) [50]. The Shahin-McCullough (1972) [64] model of thermal cracking may also be used to assess the likelihood of thermal cracking in a newly constructed pavement. Fatigue cracking predictions typically make use of the cumulative damage concept first proposed by Miner (1945) [53]. The transform functions that connect the tensile strain in bituminous layers to the maximum permitted load repetitions are the primary differentiating factor among the available design techniques. The number of load repetitions (Nf) before fatigue cracking occurs is proportional to the tensile strain (t) at the base of the bituminous layer and the modulus of the bituminous mix (E1), according to the Asphalt Institute [4,5] and shell design [65] methodologies.



$$N_S = f_1(\epsilon_t)^{-f_2}\,(E_1)^{-f_3} \qquad \dots (1.1)$$

Where f_1, f_2 and f_3 are constants and vary with the finding of different agencies for different degree of distress. Similar formulations can be seen in cracking model as proposed in IRC–37, 2001[45] which has been used in the present work.

In order to limit rutting, there are two procedures to limit rutting; one to limit the vertical compressive strain on the top of the subgrade and the other to limit the total accumulated permanent deformation on the pavement surface based on the permanent deformation properties of each individual layer. The allowable number of load repetitions N_s to limit the rutting is related to the vertical compressive strain ϵ_c on the top of subgrade may be expressed as

$$N_S = f_4\,(\epsilon_z)^{f_5} \qquad \dots (1.2)$$

Mechanistic Empirical Methods: The input, such as wheel load, and the output, or pavement reaction, such as stress or strain, are the basis of the mechanistic-empirical (ME) design method [72]. Laboratory test or field performance data was utilized in conjunction with the response values to forecast distress. Because theoretical understanding of design has not been demonstrated enough to forecast pavement performance realistically, it is required to rely on observable performance.

The use of vertical compressive strain on the top of the subgrade as a failure criterion to limit permanent deformation was first proposed by Kerkhoven and Dormon (1953) [47], while the use of horizontal tensile strain at the base of the asphalt layer was suggested by Saal and Pell (1960) [63] to limit fatigue cracking. Dormon and Metcalf (1965) [29] were the first to introduce the aforementioned ideas for application in pavement design in the United States. Considering that plastic strains are proportional to elastic strains in paving materials, vertical compressive strain may be used to regulate permanent deformation. Therefore, the extent of permanent deformation on the pavement surface may be regulated by limiting the elastic stresses on the subgrade, which in turn limits the elastic strains in other components above the subgrade. Afterwards, both Shell Petroleum International (Claussen et al., 1977) [25] and the Asphalt Institute (Shook et al., 1982] [66] included these two criteria into their mechanistic and empirical design processes. The AUSTOROADS (1992) [56] and AASTHO (1993) [2] approaches represent a watershed moment in the development of mechanistic-empirical pavement design since they took into account many different factors in order to create a more durable, flexible pavement. Mechanistic- empirical approaches have the potential benefits of increasing design dependability, facilitating the prediction of distress types, and allowing extrapolation from limited field and laboratory data [42]. The most recent of these guidelines, published by the American Association of State Highway and Transportation Officials in 2002 [3], includes the idea of serviceability of pavement with improved probabilistic forecasting of traffic loads, material characterization, and design dependability.

Pavement Planning Programs: The CHEV program (Warren and Dieckmann, 1963) [76] is the earliest software for pavement design, and its fundamental formulation is based on the Burmistered Layered Theory [21, 22]. In the DAMA software, the Asphalt Institute made certain adjustments so that non-linear elastic granular materials could also be taken into account (Hwang and Witczak, 1979 [44]). ELSYM5, for an elastic five-layer system under various wheel loads, was formulated after some tweaks were made to the original Burmister layered theory (Kopperman et al., 1986). [48]. Shell has created a tool called BISAR that takes into account both vertical and horizontal loads. [28] (DeJong et al., 1973). Finn et al. (1986) [32] created the PDMAP software to forecast fatigue cracking and rutting in Bituminous Pavements using the multi-layered theory with stress-dependent material characteristics. The finite element stress analysis software SAPIV [31] created at the University of California was found to be remarkably similar to the critical response produced using PDMAP.

The layering hypothesis has many flaws due to its central premise that layers are inherently the same in composition. Nonlinear materials, such as untreated granular bases and subbases, complicate the analysis of multilayer systems because of this assumption. Since the elastic modulus of these materials depends on stress and changes across the layer, the challenge of choosing a single location to represent the entire nonlinear layer instantly emerges. In most cases in pavement design, the most significant stress, strain, or deflection is all that's needed; therefore, picking a location close to the applied load makes sense. It can be challenging to apply the layered theory for evaluating nonlinear materials if the stresses, strains, or deflections at many locations, some near and some far from the load, are of interest. The finite element technique allows us to get beyond this obstacle. The finite element approach was initially used to analyze flexible pavements by Duncan et al. (1968) [31]. Later, ILLIPAVE (Raad and Figueroa, 1980) [60], a computer software company, adopted the technique. The substantial amount of processing time and data storage needed has prevented the application from seeing regular use in the design process. Considering serviceability performance and reliability concepts [61], the VESYS software was designed as a consequence of the AASHO road test. Nonetheless, a variety of regression equations were constructed for design application based on the responses acquired by ILLIPAVE (Thompson and Elliot, 1985) [70]; Gomez-Achecar and Thompson, 1986] [35]. MICHPAVE, a computer program created at Michigan State University (Harichandran et al., 1989) [36], also makes use of the nonlinear finite element approach. MnPAVE 2003 [54] is the most up-to-date program for designing pavements that can withstand heavy-duty axle loads, and it is based on a mechanistic empirical technique.

3. Aims of This Study:

The primary objective of this study is to compare and contrast various mechanistic-empirical methods of designing bituminous pavement with the aid of the concentration factor in a two-layer structure. In the current study, the concentration factor (n) is a function of the modulus ratio between the two layers (the paving layer and the subgrade soil). Stresses or strains on the top of the subgrade can be evaluated using the concentration factor of the current or projected pavement. Therefore, the relative durability of pavement and subgrade materials may be determined by using different concentration factor values. This work's definition of the concentration factor offers a novel way to think about the distribution of stress in a multi-layered mass system. Therefore, in this study, an effort is made to formulate the stress distribution in a layered system under both point and circular loads. The determined stresses can then be utilized for further pavement strain and deflection estimation. The primary goal of pavement design is to determine the optimal paving layer thickness above the subgrade to control the stresses and strains caused by wheel loads.

Accordingly, the purpose of this study is defined as Validating test findings and figuring out how thick the pavement has to be based on the amount of vertical compressive stress at the subgrade's surface. Validation of test findings and determination of pavement thickness based on vertical compressive strain acting on the top of the subgrade Validation of test findings and determination of full-depth asphalt pavement thickness based on radial tensile strain operating at the base of the bituminous layer Calculating the thickness of the pavement by measuring the elastic deflection at the pavement-subgrade contact and verifying the findings of this measurement Using data on deflections at the pavement-subgrade interface, predict the shape of the deflection bowl so that a design guideline for the pavement may be created.

II. STRESS BASED DESIGN OF PAVEMENT

A Stress-Based Design Approach to Pavement Fundamentals:

In the mechanistic design method, the road surface is viewed as an elastic multi-layer structure. It is believed that all materials behave similarly in all directions. Each of the layers, which extend indefinitely horizontally, has its own elastic modulus (E) and Poisson ratio (). Dorman [29] is often credited as the creator of the first mechanized technique for designing pavement. Two criteria for failure were proposed by him. Both rutting and failure due to fatigue can be caused by two types of strain: radial tensile strain (t) at the bituminous layer's base and vertical impressive strain (z) at the subgrade's top. A conventional two-wheel system is seen in Figure 2 being applied to a layered bituminous pavement construction.

In contrast, the current work takes a two-layer notion of pavement as the starting point for its design. A Granular or Asphalt layer sits atop a subgrade in a twolayer system. In this evaluation, just the base layer is taken into account. Tyres Granular substructure above a bituminous layer Subbasement granules z Subgrade.

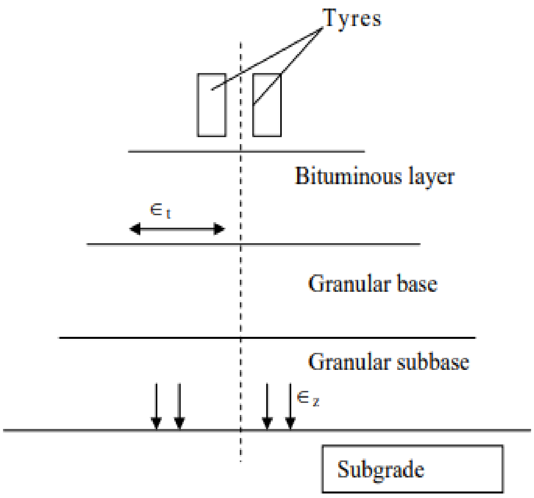


Fig 2. Critical strain location in bituminous pavement.

2. Theory of Stress Distribution in a Multi-Layer System of Elastic Materials:

To put it simply, pavement is a mass-layered structure that rests atop a subgrade of dirt. For the pavement surface to sustain the high stress generated by the application of wheel load, the stiffness of such a layered system of mass decreases with respect to depth. In order to satisfy the functional need of the pavement during its design era, the thickness and characteristics of the component paving layers must be determined. Subgrade strength, the intensity and frequency of wheel load application, environmental and climatic element, and other similar factors are key players in such design. The amount of vertical tension at the subgrade's surface is a major consideration in pavement engineering.

$$\sigma_z = K_B \frac{Q}{z^2} \qquad \dots(2.1)$$

$$\text{where, } K_B = \frac{3}{2\pi}\left[\frac{1}{1+(\frac{r}{z})^2}\right]^{5/2}$$

where, r = distance radially from point load

z = depth below the loaded surface.

Q = point load

K_B = Boussinesq's factor of vertical stress distribution.

σ_z = Vertical stress at depth z and at a radial distance r from the axis of loading.

In this situation, the pavement's job is to lessen the vertical stress on the subgrade so that it can sustain any kind of breakdown. However, it is observed that the strength or modulus of subgrade is proportional to the allowed vertical load on a subgrade [43]. Boussinesq [20] developed the fundamental equation for stress in ideal masses, assuming the material to be fully elastic, homogeneous, isotropic, and semi-infinite. The vertical tension at any depth below the soil surface caused by a point load at the surface is calculated using Boussinesq’s [20] formula.

The equation (2.1) shows that the vertical stress solely depends on the depth and radial distance, and not on the physical qualities of the transmission medium.Integrating this equation across a circular area, as shown in (2.1), will result in vertical stress on a vertical plane through the plate's center, as shown in (2.2).

$$d\sigma_z = \frac{3Q}{2\pi z^2}\left[\frac{1}{1+\left(\frac{r}{z}\right)^2}\right]^{5/2} dA \qquad \dots(2.2)$$

$$\sigma_z = q\left[1-\frac{z^3}{(a^2+z^2)^{3/2}}\right] \qquad \dots(2.3)$$

If the area where the tire meets the pavement is modeled as a circle, then the stress in the pavement system may be calculated using equation (2.3). These assumptions are ideal and not actual, yet they provide the basis for all equations that are derived using Boussinesq's[20] method. Because of its layered nature, the pavement in particular cannot be treated as homogenous and requires a tweak to Boussinesq's[20] method. In this paper, an effort is made to adapt Boussinesq's analysis such that it takes pavement elasticity into account while modeling a two-layer system. In order to calculate the stress under a uniform circular load, the following notion is introduced: (n) is a new parameter in equation (2.1) called the concentration factor[79].

$$\sigma_z = q\left[1-\left(\frac{z}{\sqrt{a^2+z^2}}\right)^n\right] \qquad \dots(2.4)$$

Modulus ratio (MR) describes how the value of parameter (n) shifts depending on the relative elasticity of the pavement and subgrade. Using an analytical method based on Huang's research[40] on the varying vertical stress imposed by a circular footing to a two-layered system, the value of the concentration factor (n) has been calculated. Such differences are depicted in Figure 3 with regards to modulus ratio and pavement depth.

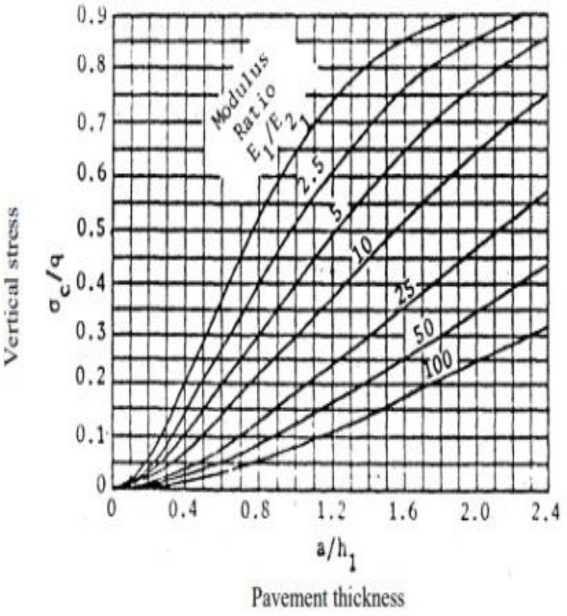
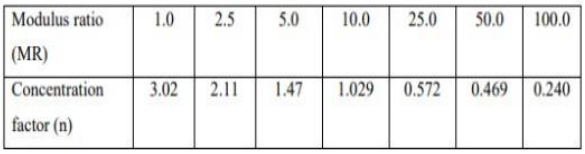


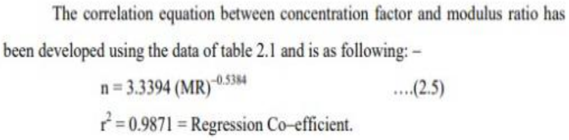
Fig 3. Vertical interface stress for two-layered system.

Figure *3* shows that for a given modulus ratio, the vertical stress approximately linearly varies with depth between (h1/a) = 1 and (h1/a) =. There is a clear nonlinearity in the fluctuation of vertical stress with respect to depth of pavement at low modulus ratio values, but this nonlinearity essentially disappears at high modulus ratio values. Figure *3* shows the vertical stress at depth (h1/a) = 1, which was used to calculate the concentration factor (n) for a certain modulus ratio by plugging the data into equation (2.4) with h1 = z. The resulting concentration factors (n) for various modulus ratios are tabulated in Table 1.

Table 1. Variation of Concentration factor with modulus ratio.



Modulus ratio (MR)	1.0	2.5	5.0	10.0	25.0	50.0	100.0
Concentration factor (n)	3.02	2.11	1.47	1.029	0.572	0.469	0.240



The correlation equation between concentration factor and modulus ratio has been developed using the data of table 2.1 and is as following: –

$$n = 3.3394\,(MR)^{-0.5384} \qquad \text{....(2.5)}$$

$r^2 = 0.9871$ = Regression Co–efficient.

3. Characterization of Wheel Load in the Design of Pavement:

In pavement design, it often becomes necessary to express multiple wheel loads in terms of equivalent single wheel load (ESWL)[41]. Such equivalence should be established in terms of a particular parameter such as stress, strain or deflection. So, an equivalent single wheel load is the load on single wheel or tyre that causes the same magnitude of pre-selected failure parameter at a given location within a pavement structure as that resulting from a set of multiple wheel loads at the same location within the pavement.
See Fig. 2.3 for an illustration of how the thickness of the pavement might influence the effectiveness of dual wheels. Wheel spacing determines the depth at which the stresses in the pavement from two wheels are equivalent to those from a single wheel. Near the pavement's surface, it is observed that the wheel load acts independently, while at increasing depths, the stresses overlap [79]. Center-to-center distance between two wheels carrying weight P is equal to regression coefficient, or = 0.9871. It's worth noting that for a homogeneous system of masses with a modulus ratio of 1, equation (2.4) becomes identical (to Boussinesq's equation) at n = 3.02. However, due to the nature of the correlation equation, a greater value of n may result from determining the concentration factor from eqn (2.5) for homogeneous mass. Equation (2.5) is important because it may be used to calculate the concentration factor (n) for a given modulus ratio and strength combination of subgrade and pavement materials.

4. Characterizing Wheel Load for Pavement Planning:

It is common practice in pavement engineering to convert multi-wheel loads to ESWL (equivalent single wheel load) to facilitate analysis and design [41]. This parity must be determined with respect to a selected parameter, such as stress, strain, or deflection. Therefore, a single wheel load is defined as the load on a single tyre that results in the same magnitude of a pre-selected failure parameter at a given location within a pavement structure as that resulting from a set of multiple wheel loads at the same location within the pavement. Stresses in the pavement are caused by the interaction of the two wheels at a depth of about S/2, although the influence of this stress overlap appears to be negligible with respect to pavement design at a depth of about 2Sd. The link between wheel depths and wheel spacing has been proven by theoretical analysis and stress testing in prototype pavement, therefore it is clear that this is the case.

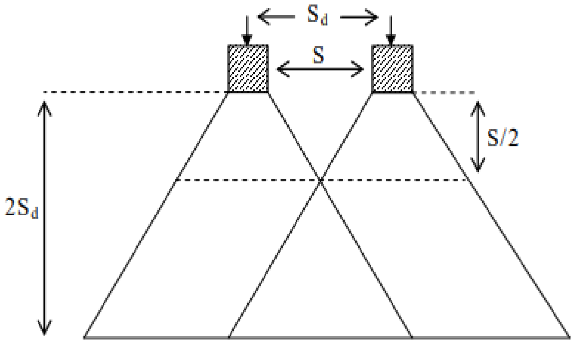


Fig 4. Influence of multiple wheels on stress distribution.

Taking into account the configuration of the rear axle, it is clear that the total of four wheels is really made up of two sets of dual wheel assembly. According to the axle and wheel arrangement of a typical Indian vehicle [46] and as seen in Figure 5, when such dual wheels are replaced by similar single wheel loads, such loads are found to be spread at a distance of 1.278m.

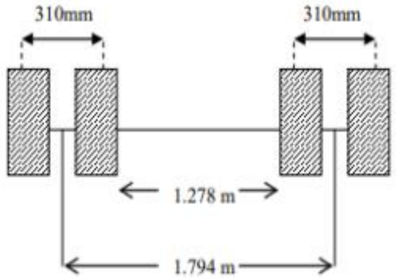


Fig 5. Rear axle and wheel configuration of a standard truck.

In this study, we use the following method to calculate the stress dispersion angle caused by a wheel load in a two-layer system. Given that the stress distribution is linear from (z/a) = 1.0 to (z/a) =, as shown by Burmister [23] and Huang's theory [40], Figure *6* depicts the stress distribution pattern in a two-layered system. The stress at (z/a) = 1.0 has been employed in the analysis, and it is readily available from Figure *3* for a variety of modulus ratios.

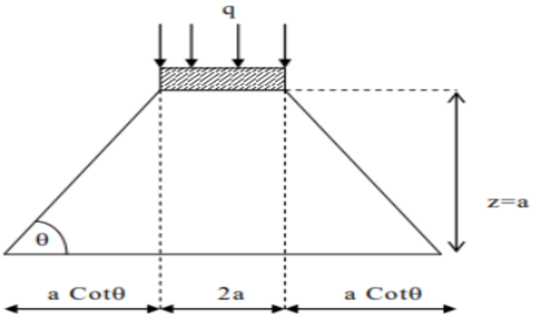


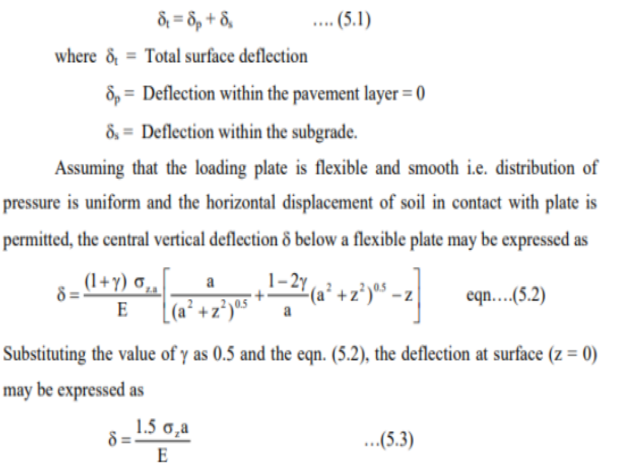
Fig 6. Dispersion of stress in layered mass by circular area.

III. PAVEMENT DESIGN BASED ON ELASTIC DEFLECTION

1. Elastic deflection theory applied to the design of pavement:

It's worth noting that this study defines a two-layer scheme for the pavement construction. As a result, the amount of deflection in the pavement structure has a direct bearing on the pavement's performance. Pavement performance may be measured in a number of ways; some use surface deflection ([14, 16, 22]) while others use deflection at the pavement-subgrade contact ([49, 77]).

Bituminous pavement is thought to undergo elastic deformation when subjected to a load, which is the basis for the deflection technique of evaluating pavement [14, 18]. While it is not possible to treat all pavements the same, the following solutions can be used when the modulus of elasticity of the pavement to that of the subgrade is somewhat near to 1. In deflection-based design, it is assumed that the deflection at the surface is caused purely by the subgrade and not by the pavement structure above it. In this concept, pavement design entails determining how thick of a paving material must be laid on the subgrade to endure a given number of wheel load repetitions before failure.



$$\delta_t = \delta_p + \delta_s \qquad \text{.... (5.1)}$$

where δ_t = Total surface deflection

δ_p = Deflection within the pavement layer = 0

δ_s = Deflection within the subgrade.

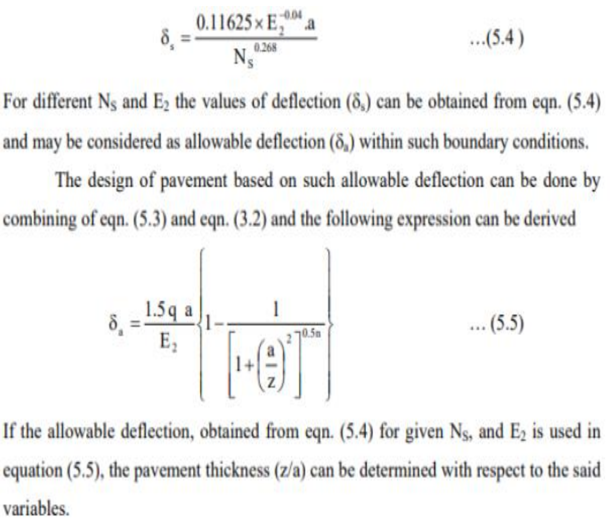
Assuming that the loading plate is flexible and smooth i.e. distribution of pressure is uniform and the horizontal displacement of soil in contact with plate is permitted, the central vertical deflection δ below a flexible plate may be expressed as

$$\delta = \frac{(1+\gamma)\ \sigma_{z,a}}{E}\left[\frac{a}{(a^2+z^2)^{0.5}} + \frac{1-2\gamma}{a}(a^2+z^2)^{0.5} - z\right] \qquad \text{eqn....(5.2)}$$

Substituting the value of γ as 0.5 and the eqn. (5.2), the deflection at surface (z = 0) may be expressed as

$$\delta = \frac{1.5\ \sigma_z a}{E} \qquad \text{...(5.3)}$$

The elastic modulus (E) of the mass, the form and quality of the loading plate, and the deflection at each point are all shown to be dependent in eqn. (5.3). The two-layer system's deflection at the pavement subgrade contact may be calculated using the same equation (5.3). Using equation (2.8) to determine the value of z for a range of design loads and subgrade strengths (E2), we may deduce the following connection between S, NS, and E2:



$$\delta_s = \frac{0.11625 \times E_2^{-0.04} .a}{N_S^{\ 0.268}} \qquad \text{...(5.4)}$$

For different N_S and E_2 the values of deflection (δ_s) can be obtained from eqn. (5.4) and may be considered as allowable deflection (δ_a) within such boundary conditions.

The design of pavement based on such allowable deflection can be done by combining of eqn. (5.3) and eqn. (3.2) and the following expression can be derived

$$\delta_a = \frac{1.5q\ a}{E_2}\left\{1 - \frac{1}{\left[1+\left(\frac{a}{z}\right)^2\right]^{0.5n}}\right\} \qquad \text{... (5.5)}$$

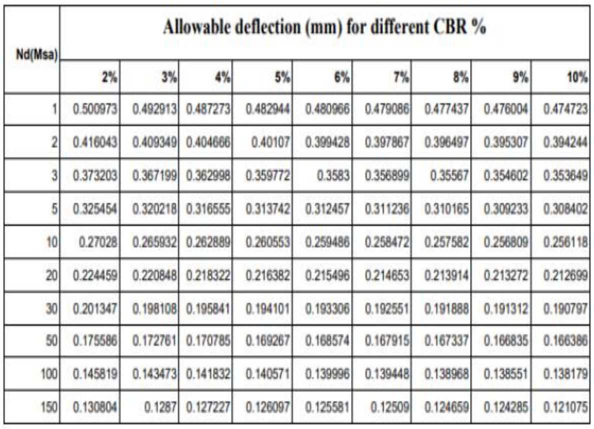
If the allowable deflection, obtained from eqn. (5.4) for given N_S, and E_2 is used in equation (5.5), the pavement thickness (z/a) can be determined with respect to the said variables.

The Use of Deflection-Based Criteria in Validating a Pavement Design Model First, using eqn (5.4), obtain the value of permissible deflection, which will be used later to calculate the pavement thickness.

IV. RESULTS

In this study, we employ the identical NS and E2 intervals as in IRC: 37-2018 [45]. The prior calculation of 215.8 mm for (a) in this paper has been utilized here as well. Table *2* displays the results of this calculation for the permissible deflection of various NS and E2.

Table 2. Summary of Allowable deflection for different design loads and subgrade strength.



Nd(Msa)	Allowable deflection (mm) for different CBR %								
	2%	3%	4%	5%	6%	7%	8%	9%	10%
1	0.500973	0.492913	0.487273	0.482944	0.480966	0.479086	0.477437	0.476004	0.474723
2	0.416043	0.409349	0.404666	0.40107	0.399428	0.397867	0.396497	0.395307	0.394244
3	0.373203	0.367199	0.362998	0.359772	0.3583	0.356899	0.35567	0.354602	0.353649
5	0.325454	0.320218	0.316555	0.313742	0.312457	0.311236	0.310165	0.309233	0.308402
10	0.27028	0.265932	0.262889	0.260553	0.259486	0.258472	0.257582	0.256809	0.256118
20	0.224459	0.220848	0.218322	0.216382	0.215496	0.214653	0.213914	0.213272	0.212699
30	0.201347	0.198108	0.195841	0.194101	0.193306	0.192551	0.191888	0.191312	0.190797
50	0.175586	0.172761	0.170785	0.169267	0.168574	0.167915	0.167337	0.166835	0.166386
100	0.145819	0.143473	0.141832	0.140571	0.139996	0.139448	0.138968	0.138551	0.138179
150	0.130804	0.1287	0.127227	0.126097	0.125581	0.12509	0.124659	0.124285	0.121075

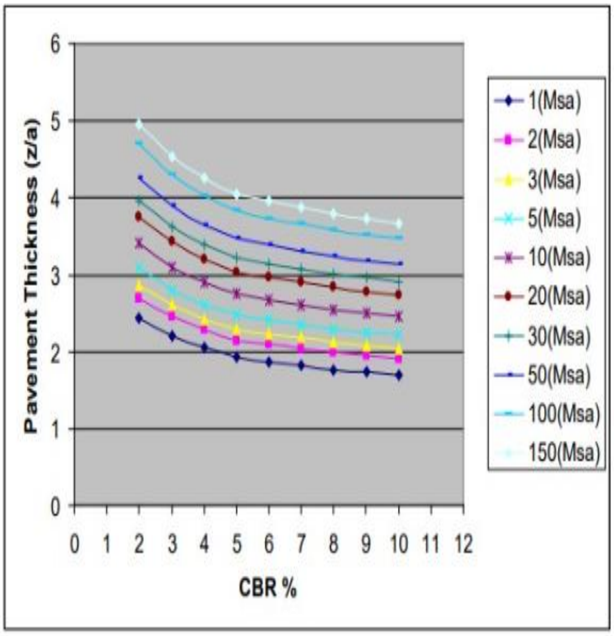


Fig 7. Variation of pavement thickness (deflection based) with design load and subgrade strength.

The subgrade has the lowest elastic modulus of any layer in a road, therefore limiting its deflection is a primary goal of the pavement layer that sits over it. Therefore, based on the stipulated design load (NS) and the subgrade strength (E2), the pavement thickness should be established proportional to the permissible deflection inside the subgrade. It's also worth noting that the same procedure applies for figuring out the concentration factor (n) to plug into eqn (5.5). In addition, the values of 'q' and 'a' in this work's other design models are consistent with one another. In eqn. (5.5), the ranges of NS and E2 are the same as in IRC: 37-2018. Table 3–Table *9* and Figure *7* display the pavement thickness (z/a) calculated using eqn. (5.5) for a variety of NS and E2 values for the subgrade. Pavement thickness estimates based on stress and those based on deflection are compared in Figure *7*–Figure *8*.

Table 3. Pavement thickenss for different design loads for a subgrade CBR 2% (Deflection based criteria).

N_S (Msa)	E_2 (kg/cm^2)	n	a (cm)	q (kg/cm^2)	Deflec–tion (mm)	(z/a)
1	200	0.732	21.58	5.6	0.500973	2.439238
2	200	0.732	21.58	5.6	0.416043	2.70179
3	200	0.732	21.58	5.6	0.373203	2.865999
5	200	0.732	21.58	5.6	0.325454	3.084959
10	200	0.732	21.58	5.6	0.27028	3.405357
20	200	0.732	21.58	5.6	0.224459	3.755117
30	200	0.732	21.58	5.6	0.201347	3.974524
50	200	0.732	21.58	5.6	0.175586	4.267723
100	200	0.732	21.58	5.6	0.145819	4.697817
150	200	0.732	21.58	5.6	0.130804	4.967949

Table 4. Pavement thickness for different design loads for a subgrade CBR 3% (Deflection based criteria).

N_S (Msa)	E_2 (kg/cm^2)	n	a (cm)	q (kg/cm^2)	Deflec–tion (mm)	(z/a)
1	300	0.9107	21.58	5.6	0.492913	2.206035
2	300	0.9107	21.58	5.6	0.409349	2.450288
3	300	0.9107	21.58	5.6	0.367199	2.602762
5	300	0.9107	21.58	5.6	0.320218	2.805798
10	300	0.9107	21.58	5.6	0.265932	3.102441
20	300	0.9107	21.58	5.6	0.220848	3.425796
30	300	0.9107	21.58	5.6	0.198108	3.628441
50	300	0.9107	21.58	5.6	0.172761	3.899051
100	300	0.9107	21.58	5.6	0.143473	4.295693
150	300	0.9107	21.58	5.6	0.1287	4.54466

Table 5. Pavement thickness for different design loads for a subgrade CBR 4% (Deflection based criteria).

N_S (Msa)	E_2 (kg/cm^2)	n	a (cm)	q (kg/cm^2)	Deflec–tion (mm)	(z/a)
1	400	1.063	21.58	5.6	0.487273	2.046987
2	400	1.063	21.58	5.6	0.404666	2.279685
3	400	1.063	21.58	5.6	0.362998	2.424681
5	400	1.063	21.58	5.6	0.316555	2.617509
10	400	1.063	21.58	5.6	0.262889	2.898828
20	400	1.063	21.58	5.6	0.218322	3.205057
30	400	1.063	21.58	5.6	0.195841	3.396795
50	400	1.063	21.58	5.6	0.170785	3.652672
100	400	1.063	21.58	5.6	0.141832	4.027445
150	400	1.063	21.58	5.6	0.127227	4.26255

Table 6. Pavement thickness for different design loads for a subgrade CBR 5% (Deflection based criteria).

N_S (Msa)	E_2 (kg/cm^2)	n	a (cm)	q (kg/cm^2)	Deflec–tion (mm)	(z/a)
1	500	1.199	21.58	5.6	0.482944	1.927016
2	500	1.199	21.58	5.6	0.40107	2.1517
3	500	1.199	21.58	5.6	0.359772	2.291448
5	500	1.199	21.58	5.6	0.313742	2.477059
10	500	1.199	21.58	5.6	0.260553	2.747466
20	500	1.199	21.58	5.6	0.216382	3.041424
30	500	1.199	21.58	5.6	0.194101	3.225318
50	500	1.199	21.58	5.6	0.169267	3.470573
100	500	1.199	21.58	5.6	0.140571	3.829535
150	500	1.199	21.58	5.6	0.126097	4.0546

Table 7. Pavement thickness for different design loads for a subgrade CBR 6% (Deflection based criteria).

N_S (Msa)	E_2 (kg/cm^2)	n	a (cm)	q (kg/cm^2)	Deflec-tion (mm)	(z/a)
1	554	1.266	21.58	5.6	0.487273	1.871465
2	554	1.266	21.58	5.6	0.404666	2.092655
3	554	1.266	21.58	5.6	0.362998	2.230093
5	554	1.266	21.58	5.6	0.316555	2.412509
10	554	1.266	21.58	5.6	0.262889	2.678056
20	554	1.266	21.58	5.6	0.218322	2.966523
30	554	1.266	21.58	5.6	0.195841	3.146897
50	554	1.266	21.58	5.6	0.170785	3.387379
100	554	1.266	21.58	5.6	0.141832	3.739221
150	554	1.266	21.58	5.6	0.127227	3.959759

Table 8: Pavement thickness for different design loads for a subgrade CBR 7% (Deflection based criteria)

N_S (Msa)	E_2 (kg/cm^2)	n	a (cm)	q (kg/cm^2)	Deflec-tion (mm)	(z/a)
1	611	1.336	21.58	5.6	0.479086	1.820668
2	611	1.336	21.58	5.6	0.397867	2.03889
3	611	1.336	21.58	5.6	0.356899	2.174342
5	611	1.336	21.58	5.6	0.311236	2.353988
10	611	1.336	21.58	5.6	0.258472	2.615296
20	611	1.336	21.58	5.6	0.214653	2.898945
30	611	1.336	21.58	5.6	0.192551	3.076221
50	611	1.336	21.58	5.6	0.167915	3.31249
100	611	1.336	21.58	5.6	0.139448	3.658037
150	611	1.336	21.58	5.6	0.12509	3.874564

Table 9. Pavement thickness for different design loads for a subgrade CBR 8% (Deflection based criteria).

N_S (Msa)	E_2 (kg/cm^2)	n	a (cm)	q (kg/cm^2)	Deflec-tion (mm)	(z/a)
1	666	1.3919	21.58	5.6	0.477437	1.769583
2	666	1.3919	21.58	5.6	0.396497	1.984733
3	666	1.3919	21.58	5.6	0.35567	2.118133
5	666	1.3919	21.58	5.6	0.310165	2.294927
10	666	1.3919	21.58	5.6	0.257582	2.551876
20	666	1.3919	21.58	5.6	0.213914	2.830582
30	666	1.3919	21.58	5.6	0.191888	3.004683
50	666	1.3919	21.58	5.6	0.167337	3.236639
100	666	1.3919	21.58	5.6	0.138968	3.575747
150	666	1.3919	21.58	5.6	0.124659	3.788175

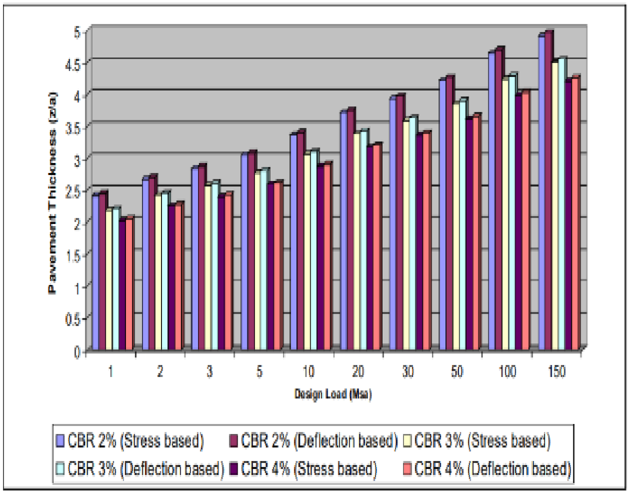


Fig 8. Variation of pavement thickness between stress based and deflection-based criteria (CBR 2%, CBR 3% and CBR 4%).

V. CONCLUSIONS

The assumption of a two-layer flexible pavement is fundamental to the mechanistic-empirical approach to pavement design that makes use of the concentration factor. The stress distribution at the pavement-subgrade interface in a two-layer pavement system with a variable modulus ratio informed the formulation of the concentration factor. The resulting concentration factor was utilized throughout the project to calculate the vertical tension at the pavement surface contact.

Both pavement and subgrade failures were modeled as linear elastic processes. Vertical compressive strain for pavement design has been calculated using the resulting tension at the subgrade's surface. Because vertical strain is created predominantly by vertical stress and the influence of horizontal stress is very minimal, the simplified approach adopted for measurement of vertical compressive strain in the work is valid in highway pavement. The top-of-subgrade vertical compressive strain measured using this method has been deemed a failure parameter that correlates with rutting on pavement.

In addition to cracking and fatigue, radial tensile strain at the base of the Bituminous or Asphalt layer has been evaluated as a failure criterion. The radial tensile strain was calculated using yet another simplified method that took into account the preponderance of vertical stress over horizontal stress. Since plastic strains in paving materials are related to elastic strains, regulating elastic strains on subgrade will likewise control plastic strains in other component layer above the subgrade.

Findings from a prevalent distress model have suggested that the failure stress or strain in a pavement with regard to normal axle load repetitions should be used as failure criterion. In order to determine the optimal pavement thickness, the failure stress and strain have been correlated with mechanically generated pavement compositions of varied modulus ratios. Stress-based parameters for determining pavement thickness have proven useful in the construction of low- traffic highways, when the frequency of axle loads is modest. Axle load repetitions are lower in the present experiment compared to the data from Kentucky, while subgrade CBR is greater in the present work. The linear elastic failure of pavement as analyzed here shows patterns of agreement with the findings of other empirical methodologies.

The primary shift in formulation that has been integrated into this study is the use of axle load rather than dual wheel load to account for stress overlapping. This work's vertical compressive strain-based pavement thickness looks to be pretty similar to previous research in this area. When compared to the findings of the IRC: 37-2018 model, it is clear that the current model comes rather near to those numbers. The assumption established in the model of the current work is mostly responsible for the observable changes in pavement thickness throughout design load ranges for varying subgrade strengths. Compared to the IRC: 37-2018, which is based on a non-linear elastic failure model, the proposed model requires a thicker pavement due to its incorporation of the linear behavior of paving materials. It is worth noting that IRC: 37-2018 values for CBR 2% to CBR 5% are quite close to the thickness of pavement up to 50 Msa. The results begin to diverge with increasing CBR and load.

It has also been discovered that if proper boundary conditions are maintained for comparison analysis, the pavement thickness with full depth asphalt layer determined on the basis of radial tensile strain would converge. The rutting and fatigue equation utilized in this study is based on experience in Indian field condition, as recommended in IRC: 37-2018, hence it is important to note that the current model is based on mechanistic empirical design of bituminous road in the Indian context. In this context, comparing the results of the current work to those of A. Das and B.B. Pandey [14] makes more sense. However, the results converge with AASTHO[1], highlighting the importance of the many assumptions made in each methodology, and highlighting the need to develop a suitable correlation factor for use in predicting pavement reaction using the current approach.

Elastic deflection at the surface of the subgrade is also taken into account in the design of the pavement. It is discovered that the thickness of the pavement calculated using deflection-based criteria is very consistent with the thickness calculated using stress-based criteria. The similarity in results may be due to the fact that both approaches satisfy the linear-elastic failure assumptions.

It is important to note, however, that in deflection-based design, the amount of permissible deflection in subgrade is particular to the subgrade strength and load repetitions being used. Allowable deflection is a useful notion for determining overlay thickness while reinforcing a pavement. The same rationale has been used to the design of a pavement's deflection bowl when subjected to an axle load, and the resulting product compares favorably to findings from relevant study using international software and field research.

REFERENCES

1. Plati, C.; Loizos, A.; Gkyrtis, K. Assessment of modern roadways using non-destructive geophysical surveying techniques. Surv. Geophys. 2020, 41, 395–430. [CrossRef]
2. Zhao, S.; Al-Qadi, I.L.; Wang, S. Prediction of thin asphalt concrete overlay thickness and density using nonlinear optimization of GPR data. NDT E Int. 2018, 100, 20–30. [CrossRef]
3. Marecos, V.; Fontul, S.; Antunes, M.L.; Solla, M. Evaluation of a highway pavement using non-destructive tests: Falling Weight Deflectometer and Ground Penetrating Radar. Constr. Build. Mater. 2017, 154, 1164–1172. [CrossRef]
4. Gkyrtis, K.; Loizos, A.; Plati, C. Integrating pavement sensing data for pavement condition evaluation. Sensors 2021, 21, 3104. [CrossRef] [PubMed]
5. Rada, G.R.; Perera, R.; Prabhakar, V. Relating Ride Quality and Structural Adequacy for Pavement Rehabilitation/Design Decisions; Report No. FHWAHRT-12-035; Federal Highway Administration: Washington, DC, USA, 2012.
6. Ruiz, A.; Guevara, J. Sustainable Decision-Making in Road Development: Analysis of Road Preservation Policies. Sustainability 2020, 12, 872. [CrossRef]
7. De Bortoli, A.; Féraille, A.; Leurent, F. towards Road Sustainability—Part I: Principles and Holistic Assessment Method for Pavement Maintenance Policies. Sustainability 2022, 14, 1513. [CrossRef]
8. Pereira, P.; Pais, J. Main flexible pavement and mix design methods in Europe and challenges for the development of an European method. J. Traffic Transp. Eng. 2017, 4, 316–346. [CrossRef]
9. Tosti, F.; Bianchini Ciampoli, L.; D'Amico, F.; Alani, A.M.; Benedetto, A. An experimental-based model for the assessment of the mechanical properties of road pavements using ground-penetrating radar. Constr. Build. Mater. 2018, 165, 966–974. [CrossRef]
10. Kavussi, A.; Abbasghorbani, M.; Moghadas-Nejad, F.; Bamdad Ziksari, A. A new method to determine maintenance and repair activities at network level pavement management using falling weight deflectometer. J. Civ. Eng. Manag. 2017, 23, 338–346. [CrossRef]
11. Plati, C.; Gkyrtis, K.; Loizos, A. Integrating non-destructive testing data to produce asphalt pavement critical strains. Nondestruct. Test. Eval. 2021, 36, 546–570. [CrossRef]

12. Crook, A.L.; Montgomery, S.R.; Guthrie, W.S. Use of falling weight deflectometer data for network-level flexible pavement management. Transp. Res. Rec. 2012, 2304, 75–85. [CrossRef]
13. Ahmed, A.; Erlingsson, S. Viscoelastic response modeling of a pavement under moving load. Transp. Res. Procedia 2016, 14, 748–757. [CrossRef]
14. Shakiba, M.; Gamez, A.; Al-Qadi, I.L.; Little, D.N. Introducing realistic tire-pavement contact stresses into Pavement Analysis using Nonlinear Damage Approach (PANDA). Int. J. Pavement Eng. 2017, 18, 1027–1038. [CrossRef]
15. Chabot, A.; Chupin, O.; Deloffre, L.; Duhamel, D. ViscoRoute 2.0: A tool for the simulation of moving load effects on asphalt pavement. Road Mater. Pavement Des. 2010, 11, 227–250. [CrossRef]