
COMPARATIVE EVALUATION OF AASHTO AND CBR FLEXIBLE PAVEMENT DESIGN METHODS USING LABORATORY- DERIVED SOIL PARAMETERS

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DOI: <https://doi.org/10.67203/sp.ICESID.2026>

ABSTRACT

Flexible pavement design plays a critical role in ensuring durable, safe and economical road infrastructure. This study compares pavement structures obtained using a CBR-based empirical method and the 1993 AASHTO flexible pavement design procedure using laboratory-derived properties of two lateritic soils. The non-marginal and marginal soils recorded CBR values of 42.79% and 13.03%, respectively. Design traffic was reconstructed from an AADT of 4,200 vehicles/day, 25% heavy vehicles, 4% annual growth, a 20-year design period, a directional distribution factor of 0.50, a lane distribution factor of 1.00 and an assumed vehicle damage factor of 3.0 ESAL/heavy vehicle, giving 17.12×10^6 design ESALs. For AASHTO design, 95% reliability, an overall standard deviation of 0.45, initial and terminal serviceability indices of 4.2 and 2.5, respectively, and CBR-derived resilient moduli were adopted. Required Structural Numbers were 3.55 and 4.68 for the non-marginal and marginal soils. The adopted AASHTO sections were 100 mm asphalt, 200 mm base and 200 mm sub-base for the non-marginal soil (500 mm total), and 130 mm asphalt, 230 mm base and 270 mm sub-base for the marginal soil (630 mm total). The CBR-based method yielded 413 mm and 450 mm, respectively. Thus, AASHTO sections were 21.1% and 40.0% thicker. The results demonstrate the effect of explicitly considering traffic, reliability, serviceability and subgrade stiffness, while confirming the continued usefulness of CBR-based design where data and resources are limited.

Keywords: flexible pavement; AASHTO; CBR; pavement thickness; ESAL; subgrade strength

1. INTRODUCTION

Road transportation remains a major means of moving people, goods and services in developing countries, and the performance of road networks depends substantially on the quality and durability of pavement structures. Flexible pavements are widely used because of their relatively low construction cost, ease of maintenance and adaptability to varying ground conditions. Their performance is influenced by traffic loading, subgrade strength, environmental conditions, drainage and material characteristics (Inyang et al., 2024; Nneka, 2025; Olowosulu et al., 2022; Owolabi et al., 2012). Inadequate design can result in rutting, cracking, potholes and excessive deformation, with associated maintenance costs and reduced service life.

Pavement design practice has developed from empirical procedures toward approaches that incorporate traffic, reliability and material-response considerations. The California Bearing Ratio (CBR) method uses subgrade bearing strength and empirical design relationships and is particularly attractive where detailed traffic, material and climatic information is limited (Chitade et al., 2024; Noka et al., 2025). The AASHTO procedure, in contrast, incorporates cumulative traffic expressed as ESALs, reliability, serviceability, subgrade resilient modulus and drainage considerations (Ahmed et al., 2023; El-Ashwah et al., 2022; Moharekpour et al., 2022; Prayudyanto et al., 2023; Ziar et al., 2023).

CBR-based procedures remain useful for preliminary and lower-data applications, but their empirical nature means that reliability and uncertainty are not represented explicitly. AASHTO therefore provides a more comprehensive framework for higher traffic levels, although it requires more design inputs and can produce thicker sections. A conventional flexible pavement consists of surfacing, base, sub-base and subgrade layers; the upper layers distribute wheel loads so that stresses transmitted to the subgrade remain within acceptable limits.

The aim of this study is to compare flexible pavement structures obtained using the CBR-based empirical method and the 1993 AASHTO design procedure for two lateritic subgrade soils. The specific objectives are to characterize the engineering properties of the soils, reconstruct design traffic, determine the AASHTO Structural Number and corresponding layer thicknesses, and compare the resulting sections in terms of total thickness and material quantity.

2. METHODOLOGY

2.1 Soil Sampling

Two lateritic soil samples representing distinct subgrade conditions were selected for the study. The samples were obtained from an existing borrow pit in Janguza Area, north of Kano City, Kano State, Nigeria. Bulk samples were collected at approximately 1.0 m below ground level to minimize the influence of organic matter and surficial contamination. The samples were packaged, labelled and transported to the geotechnical laboratory, where they were air-dried, pulverized without crushing individual particles and prepared for testing in accordance with relevant standard procedures (ASTM, 2008).

2.2 Laboratory Testing

Particle-size distribution and Atterberg limits were determined using standard geotechnical procedures. Standard Proctor compaction testing was used to determine maximum dry density (MDD) and optimum moisture content (OMC), while CBR testing was conducted on prepared specimens to assess subgrade bearing capacity. The measured CBR values were used directly in the CBR-based design and as the basis for estimating resilient modulus for the AASHTO analysis.

The retained dataset does not contain the complete particle-size distribution required for formal AASHTO or USCS group assignment. Accordingly, the soils are additionally described using the CBR-based subgrade strength classes in the Federal Ministry of Works guidance. The non-marginal soil (CBR = 42.79%) falls within S6 (CBR ≥ 30%), while the marginal soil (CBR = 13.03%) falls within S4 (CBR 8–14%). These classes are used for engineering interpretation and are not substitutes for formal soil classification.

2.3 Traffic Analysis and Design ESAL

Cumulative design traffic was expressed as 80 kN (18-kip) Equivalent Single Axle Loads (ESALs). In the absence of an axle-load survey, a vehicle damage factor (VDF) of 3.0 ESAL per heavy vehicle was adopted as an explicit assumption. The traffic growth factor was calculated as $GF = [(1 + r)^n - 1]/r$, where r is annual growth and n is the design period. For $r = 0.04$ and $n = 20$ years, $GF = 29.778$. Design ESAL was calculated as $W18 = AADT \times PHV \times 365 \times GF \times DD \times LDF \times VDF$, using $AADT = 4,200$ vehicles/day, $PHV = 0.25$, $DD = 0.50$, $LDF = 1.00$ and $VDF = 3.0$, giving $W18 = 17.12 \times 10^6$ ESALs. The assumed VDF should be replaced by project-specific axle-load data for detailed design.

2.4 AASHTO 1993 Flexible Pavement Design Procedure

The required Structural Number (SN) was determined using the flexible pavement design equation in the AASHTO Guide for Design of Pavement Structures (AASHTO, 1993):

$$\log_{10}(W18) = ZRSo + 9.36\log_{10}(SN + 1) - 0.20 + \{\log_{10}[\Delta PSI/(4.2 - 1.5)]\}/\{0.40 + 1094/(SN + 1)^{5.19}\} + 2.32\log_{10}(MR) - 8.07$$

where $W18$ is cumulative 18-kip ESALs; ZR is the standard normal deviate corresponding to the selected reliability; So is the overall standard deviation; ΔPSI is allowable serviceability loss; and MR is subgrade resilient modulus in psi. Reliability of 95% ($ZR = -1.645$), $So = 0.45$, initial serviceability $p_0 = 4.2$ and terminal serviceability $p_t = 2.5$ were adopted, giving $\Delta PSI = 1.7$.

Because resilient modulus was not measured directly, it was estimated from CBR using $MR (\text{psi}) = 2555(\text{CBR})^{0.64}$. The resulting moduli were approximately 28,278 psi for the non-marginal soil and 13,212 psi for the marginal soil. Substitution into the AASHTO equation at $W18 = 17.12 \times 10^6$ gave required SN values of 3.55 and 4.68. Layer contribution was checked using $SN = a_1D_1 + a_2m_2D_2$

+ a3m3D3, with a1 = 0.44 for asphalt concrete, a2 = 0.14 for granular base, a3 = 0.11 for granular sub-base and m2 = m3 = 1.0. Layer thicknesses D are expressed in inches in the SN equation.

2.5 CBR-Based Pavement Design Procedure

The CBR-based pavement sections were interpreted using the empirical pavement design approach in Federal Ministry of Works, Highway Manual Part 1: Design, Volume III: Pavement and Materials Design (2013), which incorporates CBR-based pavement catalogues and guidance derived in part from TRL Overseas Road Note 31. The design CBR values of 42.79% and 13.03% were used with the design traffic level to select the corresponding pavement thicknesses. The specific catalogue source is stated to improve reproducibility.

Table 1: AASHTO Design Input Parameters

Parameter	Symbol	Adopted value	Basis/remark
Design traffic	W18	17.12×10^6 ESAL	Traffic reconstruction
Reliability	R	95%	Higher-order road design assumption
Standard normal deviate	ZR	-1.645	Corresponding to 95% reliability
Overall standard deviation	So	0.45	Typical flexible pavement value
Initial serviceability	p0	4.2	AASHTO flexible pavement value
Terminal serviceability	pt	2.5	Major road design value
Serviceability loss	Δ PSI	1.7	p0 – pt
Layer coefficients	a1, a2, a3	0.44, 0.14, 0.11	Asphalt, granular base, granular sub-base
Drainage coefficients	m2, m3	1.0, 1.0	Assumed good/normal drainage
FMW/ORN31 Subgrade Strength Class	S6 (CBR $\geq$ 30%)	S4 (CBR 8–14%)	

3. RESULTS AND DISCUSSION

3.1 Engineering Properties of the Soils

Table 2: Engineering Properties of the Lateritic Soils

Property	Non-Marginal Soil	Marginal Soil
Liquid Limit (%)	20.0	27.0
Plastic Limit (%)	14.58	17.15
Plasticity Index (%)	5.42	9.85
MDD (Mg/m ³)	2.03	2.00
OMC (%)	11.1	10.0
CBR (%)	42.79	13.03

The engineering properties of the two lateritic soils show clear differences in subgrade quality. The non-marginal soil recorded a liquid limit of 20.0%, plastic limit of 14.58% and plasticity index of 5.42%, compared with 27.0%, 17.15% and 9.85%, respectively, for the marginal soil. The lower

plasticity of the non-marginal soil indicates lower susceptibility to moisture-induced volume changes, whereas the marginal soil is more likely to experience deformation under moisture fluctuations.

The compaction characteristics were similar. The non-marginal soil achieved MDD of 2.03 Mg/m³ at OMC of 11.1%, while the marginal soil achieved 2.00 Mg/m³ at 10.0%. The slightly higher density of the non-marginal soil indicates a somewhat denser and potentially more supportive soil matrix.

The largest difference was in CBR: 42.79% for the non-marginal soil and 13.03% for the marginal soil. Under the FMW/ORN31 grouping these correspond approximately to S6 and S4. Formal AASHTO or USCS classification is not reported because the complete particle-size distribution data were unavailable.

3.2 Flexible Pavement Design Using the CBR Method

The CBR-based pavement sections are presented in Table 3. For the non-marginal soil, with CBR of 42.79%, the required structure comprised 113 mm surfacing, 150 mm base and 150 mm sub-base, giving a total thickness of 413 mm. For the marginal soil, with CBR of 13.03%, the section comprised 100 mm surfacing, 150 mm base and 200 mm sub-base, giving a total thickness of 450 mm.

The marginal soil therefore required an additional 37 mm of total pavement thickness compared with the non-marginal soil, mainly because of the additional 50 mm of sub-base. This confirms the empirical CBR principle that lower subgrade strength generally requires thicker pavement layers to reduce stresses transmitted to the subgrade.

Table 3: Flexible Pavement Design Thickness Using the CBR Method

Layer	Thickness (mm)	
	Non-Marginal Soil	Marginal Soil
Surfacing	113	100
Base	150	150
Sub-base	150	200
Total	413	450

3.3 Flexible Pavement Design Using the AASHTO Method

The AASHTO pavement structures are presented in Table 4. Using $W_{18} = 17.12 \times 10^6$ ESALs, $R = 95\%$, $S_o = 0.45$, $\Delta PSI = 1.7$ and the CBR-derived resilient moduli, the required SN values were 3.55 for the non-marginal soil and 4.68 for the marginal soil.

Table 4: Flexible Pavement Design Thickness Using the AASHTO Method

Layer	Thickness (mm)	
	Non-Marginal Soil	Marginal Soil
Structural Number (SN)	3.55 required 3.70 provided	4.68 required 4.69 provided
Asphalt Surface	100	130
Base	200	230
Sub-base	200	270
Total Thickness	500	630

For the non-marginal soil, 100 mm asphalt surface, 200 mm granular base and 200 mm granular sub-base provide $SN = 3.70$, exceeding the required 3.55. The total thickness is 500 mm. For the marginal soil, 130 mm asphalt surface, 230 mm granular base and 270 mm granular sub-base provide $SN = 4.69$, exceeding the required 4.68. The total thickness is 630 mm.

The lower CBR-derived resilient modulus of the marginal soil is therefore reflected in a substantially larger required Structural Number and thicker pavement section.

3.4 Comparative Analysis

A comparison of the two design methods shows that AASHTO produces thicker pavement sections for both soil conditions. For the non-marginal soil, thickness increases from 413 mm under

CBR to 500 mm under AASHTO, an increase of 87 mm (21.1%). For the marginal soil, thickness increases from 450 mm to 630 mm, an increase of 180 mm (40.0%).

Table 5: *Comparison of Pavement Thickness and Material Quantity*

Soil	CBR total (mm)	AASHTO total (mm)	Difference (mm)	Increase (%)	Extra volume (m ³ /m ²)
Non-marginal	413	500	87	21.1	0.087
Marginal	450	630	180	40.0	0.180

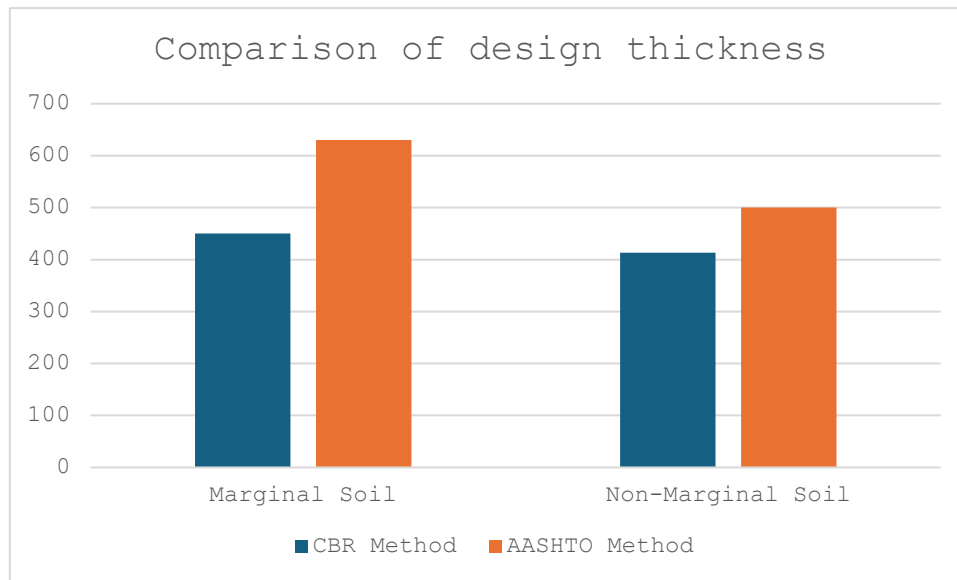


Figure 1: *Comparison of AASHTO and CBR Design Methods*

Figure 1 shows that the AASHTO sections are thicker for both soils, with the difference more pronounced for the weaker marginal subgrade. The thickness difference corresponds to an additional material quantity of approximately 0.087 m³/m² for the non-marginal soil and 0.180 m³/m² for the marginal soil. This provides a material-quantity proxy for economic comparison, although monetary cost and life-cycle cost cannot be established without project-specific unit rates and maintenance assumptions.

The greater AASHTO thickness should not be interpreted as proof of superior performance from thickness alone. Its principal distinction is the explicit inclusion of reliability, uncertainty, serviceability and resilient modulus in the design framework. Conversely, the CBR method remains attractive where data, time and analytical resources are limited. For detailed projects, the assumed VDF should be replaced with measured axle-load data and resilient modulus should preferably be measured directly or calibrated locally.

4. CONCLUSION

This study demonstrates that pavement design methodology materially affects the resulting flexible pavement structure. For lateritic soils with CBR values of 42.79% and 13.03%, the CBR-based method produced total pavement thicknesses of 413 mm and 450 mm, respectively, whereas the AASHTO 1993 analysis produced adopted sections of 500 mm and 630 mm. The AASHTO sections were therefore 21.1% and 40.0% thicker.

The difference is associated with the explicit consideration of cumulative ESALs, reliability, serviceability and resilient modulus in the AASHTO framework. The CBR method remains useful for preliminary and lower-data applications because of its simplicity and relatively low data requirements, but its reduced material quantity should not be interpreted automatically as lower life-cycle cost.

The analysis is limited by the assumed VDF, the use of CBR-derived rather than directly measured resilient modulus, incomplete particle-size distribution data and the absence of project-

specific drainage, seasonal and unit-cost information. Future work should incorporate axle-load surveys, direct resilient-modulus testing, complete AASHTO/USCS classification data, locally calibrated environmental and drainage parameters, and project-specific life-cycle cost analysis.

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