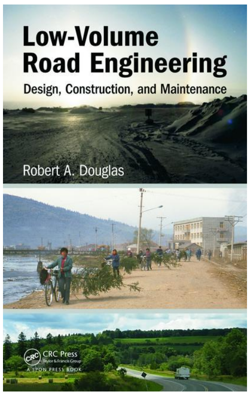

Low-Volume Road Engineering: Design, Construction, and Maintenance

Robert A. Douglas

Pavement design

Publication details
https://test.routledgehandbooks.com/doi/10.1201/b19036-10
Robert A. Douglas
Published online on: 08 Dec 2015

Chapter 7

Pavement design

7.1 INTRODUCTION

Pavement design is primarily a matter of selecting the thicknesses and materials of the layers that comprise the pavement. The designer selects the materials for the pavement layers, usually placing the strongest materials toward the top of the pavement where the stresses imposed by the wheel loads are greater. A top-down approach is taken, with each layer protecting the weaker layer below.

Beside purely mechanical considerations, other issues such as constructability, economics, sustainability, and the environment come into play in pavement design.

The discussion here will be limited to flexible pavements including asphalt, surface-treated, and aggregate-surfaced pavements.

7.2 INPUTS TO PAVEMENT DESIGN

The mechanical considerations for pavement design require a number of key inputs, including the traffic loading, the strength of the subgrade (see Chapter 5 for definitions of pavement components), strengths of the pavement layer materials, climatic effects, and the failure criterion for the design.

7.2.1 Traffic

The traffic is specified in terms of the numbers of axle passes and of the axle loading. To cope with the wide spectrum of vehicle configurations (numbers, placement, and spacing of axles) and axle loads, which can range from that of a light passenger vehicle to as much as 400 tonnes (440 tons) for heavy mining trucks, the concept of *equivalent single axle loads* (ESAL) is introduced. In the ESAL concept, the increment of pavement life used up by one pass of the axle is equated to the increment of pavement life used up by a number of passes of a “standard axle.” The standard axle load is

80 kN (18,000 lb). The pavement life used by the passage of axles is far from linear with the number of passes, rather, it is generally taken to be related to the fourth power of the axle load ratio:

$$\text{ESAL} \cong \left(\frac{W}{W_s}\right)^4 \tag{7.1}$$

where

ESAL is the number of equivalent single axle loads per pass of the axle
W is the axle load (kN or lb)
W_s is the standard axle loading (80 kN if working in SI units, 18,000 lb if working in U.S. Customary Units)

This is a simplistic approach to the determination of the design traffic volume, as there are many factors that affect the increment of damage caused by the passage of an axle or group of axles. The damage is related to the fatiguing stress reversal in the pavement layers with the approach and departure of each axle, so the individual axle loads, number of axles, and the spacing of the axles in a group are important factors. The index may be other than 4 for various pavement types, especially gravel-surfaced pavements. In addition, there is an interaction between the condition of the pavement and the damaging effect of the passage of an axle group. AASHTO [7.1] provides tables for ESAL as a function of numbers of axles in a group (single axle, tandem axles, triple axles) and the terminal present serviceability index,* PSI, a measure of pavement roughness.

Beyond these issues, just estimating the number of vehicles to use the road during its service life can be a challenge. There are a number of components, each often difficult to assess, in the traffic estimate, including the following:

- *Existing traffic*: The current number of vehicles using the road can vary from hour to hour during the day, day to day during the week, and season to season during the year. To take into account these variations for pavement design, the traffic volume is usually expressed as the annual average daily traffic, AADT. The AADT is the sum of vehicles passing a particular point on the road in both direction over 365 consecutive days, divided by 365.
- *Traffic growth*: As the use of the land surrounding the road, the population, and the wealth of the population all increase, the traffic volume can be expected to increase generally each year. A growth

* The U.S. "present serviceability index," *PSI*, is a subjective rating of pavement roughness, ranging from 0 ("very poor") to 5 ("very good"). The Canadian equivalent is "riding comfort index," *RCI*, ranging from 0 to 10. See Haas et al. [7.2].

factor is usually applied to the existing traffic volume, and the increase compounded annually over the service life of the road. Usually growth is observed, but a decline in volume is also conceivable.

- *Development growth*: The traffic on the road may increase due to some specific nearby development that takes place during the life of the road. For instance, a new agricultural processing plant may be constructed nearby, or a housing subdivision (estate) built, or a new quarry opened up, any of which will result in an increase in traffic using the road.
- *Attracted traffic*: Improving a road may make it a more attractive route than some other route currently being used, and if so, some of the traffic will divert to the new or improved road. On the other hand, conceivably some other route may become more attractive, resulting in diversion of traffic *away* from the road.
- *Generated traffic*: Improving a road may encourage vehicle operators to use it, where before they never used the road to ship goods or travel, or perhaps used some other mode (e.g., foot or bicycle paths, rail, etc.).

Taking these factors into account can be a challenge when carrying out pavement designs for public roads. However, estimating traffic for resource access roads, such as haul roads for forest or mining operations, can be easier. Usually the life of the facility, the amount of the resource (e.g., volume of timber, tonnage of ore) to be moved, and the capacity of the haul trucks are all known, making the estimation of the number of passes of the design vehicle a relatively simple matter.

7.2.2 Subgrade strength

The reason to build a pavement is to protect the weak subgrade; the weaker the subgrade, the more substantial the pavement will have to be. Subgrade strength can be measured with tests on laboratory specimens, or *in situ* (in place at the site). There are advantages and disadvantages to both. Laboratory testing permits the designer to control the parameters that affect soil behavior, but there is a risk of sample disturbance which will affect the results. *In situ* testing may give more direct estimates of the subgrade strength, but the cost of testing can be high, and access to the site is required.

Many of the common pavement design methods are based on the California bearing ratio (CBR). The CBR test compares the penetration resistance of a specimen of the subgrade soil to that of high quality compacted crushed rock [7.3]. In the test (Figure 7.1), a hardened steel cylinder 49.63 mm (1.95 in.) in diameter is pressed into a prepared specimen of the subgrade soil 152 mm (6.0 in.) in diameter by 178 mm (7.0 in.) height at a rate of 1.3 mm/min (0.05 in./min). The “CBR” of the specimen is the ratio

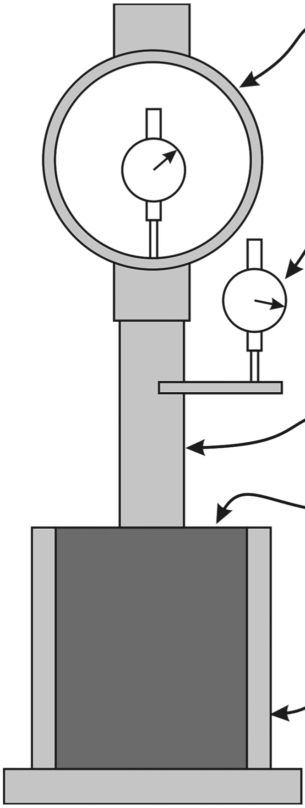

Figure 7.1 Schematic diagram of the CBR laboratory test apparatus. Proving ring or load cell measures the force applied to the loading cylinder, dial gauge or displacement transducer measures its penetration into the soil. (Not to scale.)

in percent of the pressure exerted on the specimen divided by a standard pressure at 2.5 mm (0.1 in.) and 5.0 mm (0.2 in.) penetration. The standard pressures are 6.9 MPa (1000 lb/in.^{2}) and 10.3 MPa (1500 lb/in.^{2}) for 2.5 and 5.0 mm penetration, respectively. The value of the CBR is usually the ratio calculated for 2.5 mm penetration. If the value is greater for 5.0 mm penetration, the test is run again, and if the value at 5.0 mm penetration is still greater than that at 2.5 mm penetration, the value at 5.0 mm penetration is taken as the final value of the CBR.

Care must be taken to obtain representative samples of the subgrade soil. The soil should be compacted to the same density, at the same moisture content, as the subgrade soil will have during the service life of the pavement.

Laboratory CBR tests are time consuming and expensive. The test with the same loading apparatus can be conducted in the field, but the cost is high for the little data obtained. For these reasons, more often designers carry out

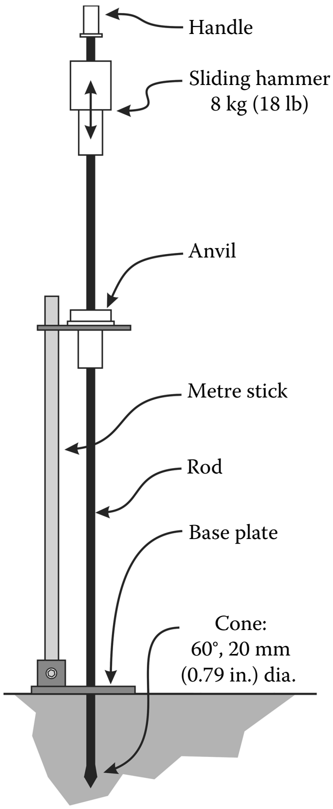

Figure 7.2 Schematic diagram of the manual DCP test apparatus. Sliding 8 kg (18 lb) hammer travels 575 mm (22.6 in.), striking the anvil. Penetration is measured by the anvil bracket's progress down the metre stick. (Not to scale.)

many replicates of some less expensive, rapid field test which is correlated to CBR. The test frequently chosen is the manual dynamic cone penetration (DCP) test (Figure 7.2). In the DCP test, a 20 mm (0.80 in.) diameter 60° cone is driven into the subgrade by an 8 kg (18 lb) hammer falling 575 mm (22.6 in.) [7.4]. The blow count in mm/blow has been correlated to CBR [7.4]. A graph of blow count versus depth is recorded for each test, and an appropriate subgrade CBR determined from the correlation. The test is rapid and relatively inexpensive to perform, but the manual version of the apparatus requires 2–3 operators. Where labor is inexpensive, the labor intensiveness of the test is not a problem; where labor *is* expensive, a mechanized version of the apparatus requiring just one operator can be used.

Other apparatus such as the Clegg hammer or light weight deflectometer can be used. The outputs of these are correlated to CBR and soil modulus, respectively.

In many pavement designs, subgrade resilient modulus M_r is the measure of the subgrade strength used. M_r is the slope of the stress–strain curve in a repeated-load triaxial test of the subgrade [7.5]. A design value of subgrade M_r can be correlated* to CBR. Common correlations are [7.1]:

$$M_r(\text{psi}) = 1500\ \text{CBR} \tag{7.2}$$

or

$$M_r(\text{MPa}) = 10\ \text{CBR} \tag{7.3}$$

Recognizing that there is a limit to the relation, NAASRA [7.6,7.7] proposed curvilinear relationships between M_r and CBR:

$$\text{For CBR} < 5 : M_r = 16.2\ \text{CBR}^{0.7} \tag{7.4}$$

$$\text{For } 5 < \text{CBR} < 15 : M_r = 22.4\ \text{CBR}^{0.5} \tag{7.5}$$

Preliminary values for subgrade strengths are provided in Table 7.1 [7.8].

7.2.3 Pavement layer material strengths

Values for the strengths of the pavement layers are either assumed or determined for pavement design. For finite element-based design methods, elastic moduli are usually used. The AASHTO design method defines a structural layer coefficient, a_i for each layer (see Table 7.2).

7.2.4 Climatic effects

Pavement design methods must take into account the effects of climate, particularly temperature and precipitation. Subgrade strength decreases markedly in wet periods and a large proportion of the pavement life can be used up when traffic is even a short, wet period or during the thaw in the spring of the year in temperate climates. Frozen pavements are obviously much stronger than those that are not and deteriorate little with traffic during periods of freezing temperatures. The more sophisticated pavement design methods take into account the state of the climate when traffic is applied.

* Which in turn is often correlated to DCP penetration/blow—thus resulting in design which relies on a correlation to a correlation!

Table 7.1 Recommended M_r values for subgrades

				Resilient modulus M_r for subgrade conditions (MPa)			*Resilient modulus M_r for subgrade conditions (lb/in.^{2})*		
Description	*MTO classification*	*Drainage characteristics*	*Susceptibility to frost action*	*Good*	*Fair*	*Poor*	*Good*	*Fair*	*Poor*
Rock, rock fill, shattered rock, boulders/cobbles	Boulders/ cobbles	Excellent	None	90	80	70	13,100	11,600	10,200
Well-graded gravels and sands suitable as granular borrow	GW, SW	Excellent	Negligible	80	70	50	11,600	10,200	7,300
Poorly graded gravels and sands	GP, SP	Excellent to fair	Negligible to slight	70	50	35	10,200	7,300	5,100
Silty gravels and sands	GM, SM	Fair to semi-impervious	Slight to moderate	50	35	30	7,300	5,100	4,400
Clayey gravels and sands	GC, SC	Practically impervious	Negligible to slight	40	30	25	5,800	4,400	3,600
Silts and sandy silts	ML, MI	Typically poor	Severe	30	25	18	4,400	3,600	2,600
Low plasticity clays and compressible silts	CL, MH	Practically impervious	Slight to severe	35	20	15	5,100	2,900	2,200
Medium to high plasticity clays	CI, CH	Semi-impervious to impervious	Negligible to severe	30	20	15	4,400	2,900	2,200

Source: Reproduced with permission from Hajek, J.J., Smith, K.L., Rao, S.P., and Darter, M.I., Adaptation and verification of AASHTO pavement design guide for Ontario conditions, Ministry of Transportation of Ontario, Pavements and Foundation Section, Prepared by ERES Consultants, Applied Research Associates, Inc., Toronto, Ontario, Canada, 2008 © Queen's Printer for Ontario, 2015.

If a pavement design method requires that only a single value of subgrade strength be declared, the value must be very carefully selected, to represent the most damaging season in the design life of the pavement during which there is traffic.

7.2.5 Failure criterion and reliability

Pavement design methods must have an expressed or inherent definition of "failure." This can be a maximum rut depth, or a maximum strain, or the terminal state of the pavement (e.g., measured by pavement roughness).

The more sophisticated pavement design methods also recognize the notion of design reliability. Because of the variability of subgrade strength and pavement material strengths along the road, there will be places where the design is inadequate. A design reliability in percent reflects the designer's tolerance to pavement failure. In the context of low-volume roads, lower reliabilities than those expected for high-volume highways are tolerated. The higher the reliability, the thicker the pavement. A reliability of 100% implies that the road is overbuilt for most of its length, which is economically unacceptable.

7.3 OTHER CONSIDERATIONS

Beside the purely mechanical, other considerations need to be addressed in pavement design.

The most economical design is desired but that may not necessarily imply that the least expensive pavement be constructed. There is an interaction between the design of the pavement and its maintenance. A pavement with a low construction cost could ultimately lead to expensive maintenance and rehabilitation, so the *whole life* cost of the pavement must be taken into account. In addition, there is an interaction between pavement design and vehicle operating costs. A pavement constructed at minimal cost may result in elevated vehicle operating costs, including fuel consumption and vehicle maintenance.

Pavements must be constructable. They must not require expensive materials, unavailable construction equipment, or uncommon construction methods. They must not be overly complicated to construct, or logistical problems will result.

There is a growing awareness of sustainability. Pavements are constructed of nonrenewable materials, and the supply of aggregates is not unlimited. Designs which permit the recycling of materials during the life of the pavement are becoming more common. Designers are also beginning to accept the use of marginal but still adequate materials, rather than specifying the best material in every case even when not warranted.

Designers are becoming more conscious of the impact of road and pavement construction on the environment, and more environmentally friendly solutions such as warm asphalt, which requires less energy to produce and results in the production of less pollution, are being implemented more often.

7.4 TYPES OF PAVEMENT DESIGN METHODS

Pavement designs fall into a handful of classes, depending upon their sophistication.

The "design" may simply be based on precedent: "this works, that does not—we'll do it this way." This is the simplest approach and is often implemented. Many road authorities have a standard suite of pavement designs based on road classification. Once the designs are calibrated, the approach works well, unless and until new conditions are encountered.

Pavement designs may be presented in simple design catalogs—tables of pavement layer design thicknesses which are functions of traffic classes and subgrade strength classes. Traffic is expressed in ESAL, and subgrade strength may be expressed in CBR classes. Inherent in this approach are assumptions about material strengths and the failure criterion.

Various CBR charts exist. These are graphs of pavement thickness plotted against either traffic in ESAL with the subgrade CBR as a parameter or plotted against subgrade CBR with traffic in ESAL as a parameter. It is assumed that the pavement has a single layer of high-quality crushed rock. Other materials can be accommodated with the concept of equivalent base thickness, where the thickness of a given layer is adjusted according to its strength compared to that of crushed rock base material.

Mechanistic-empirical pavement design methods combine some consideration of the mechanics of pavement behavior with empirical data or experience. Various design equations and charts present the results.

Selected examples of the types of pavement design methods are given below.

7.5 SELECTED PAVEMENT DESIGN METHODS

7.5.1 AASHTO methods

Low-volume road pavement design methods can be found in the *1993 AASHTO Guide for Design of Pavement Structures* [7.1]. Their origins can be traced back to the venerable AASHO Road Test [7.1,7.9,7.10], where, starting in 1958 at a site west of Chicago, near Ottawa, Illinois, 126 Army trucks of varying configuration trafficked six pavement test loops with 836 individual pavement designs for 19 h daily for 2 years. The methods include design catalogs and empirical methods.

Table 7.2 Recommended layer coefficient, a_i values for pavement materials

Pavement material	*Layer coefficient, a_i*
Bituminous bound materials	
New and recycled hot mix asphalt	0.42
Existing hot mix asphalt	0.14–0.28
Cold mix asphalt	0.28–0.38
Recycled asphalt pavement (RAP) Granular A blend stabilized with expanded asphalt cement	0.20–0.25
Existing cold mix asphalt	0.11–0.24
Bituminous-treated Granular A (about 3% asphalt cement)	0.31
Unbound materials	
New Granular A	0.14
Existing Granular A	0.10–0.14
New Granular B Type I	0.09
Existing Granular B Type I	0.05–0.09
New Granular B Type II	0.09–0.14
Existing Granular B Type II	0.06–0.14

Source: Reproduced with permission from Hajek, J.J., Smith, K.L., Rao, S.P., and Darter, M.I., Adaptation and verification of AASHTO pavement design guide for Ontario conditions, Ministry of Transportation of Ontario, Pavements and Foundation Section, Prepared by ERES Consultants, Applied Research Associates, Inc., Toronto, Ontario, Canada, 2008 © Queen's Printer for Ontario, 2015.

Note: Granular A, B, and O are MTO material designations: see Tables 6.4 and 6.5, and MTO [7.12].

For single layer aggregate-surfaced roads, the Guide [7.1] presents a catalog of layer thicknesses in inches. These are a function of the traffic levels shown in Table 7.3, five levels of the relative quality of the subgrade soil, and the location of the road's site in one of six U.S. climatic regions. Note that it is assumed the resilient modulus of the aggregate pavement layer is 30,000 lb/in.^{2}. The failure criterion is an overall design serviceability loss ΔPSI of 3.0.

Table 7.3 Traffic levels for AASHTO low-volume aggregate-surfaced road pavement design catalog

Designation	*Traffic volume (ESAL)*
High	60,000–100,000
Medium	30,000–60,000
Low	10,000–30,000

Source: AASHTO, *AASHTO Guide or Design of Pavement Structures*, American Association of State Highway and Transportation Officials (AASHTO), Washington, DC, 1993.

In a second, more general method accommodating multiple pavement layers, the Guide [7.1] defines a structural number, *SN*, a measure of overall pavement strength:

$$SN = a_1 D_1 + a_2 D_2 + a_3 D_3 \tag{7.6}$$

where

a_1, a_2, and a_3 are layer coefficients (Table 7.2) for the surface, base, and subbase layers, respectively, reflecting their material strengths; and

D_1, D_2, and D_3 are the thicknesses of the surface, base, and subbase layers, respectively.

Catalogs of *SN* values are provided, one each for reliabilities of 50% and 75%. These are also a function of traffic level (Table 7.4), relative quality of subgrade soil, and location within one of six U.S. climatic zones. The failure criterion is a terminal present serviceability index PSI_t of 1.5. With the *SN* value in hand, and known values for the layer coefficients a_i, designers task is to adjust the pavement thicknesses D_i to satisfy Equation 7.6.

A third, nomograph-based design method for single-layer pavements is presented in the Guide [7.1]. The nomograph takes into account the following parameters:

- Allowable rut depth
- Subgrade resilient modulus
- Resilient modulus of the aggregate layer
- Design traffic volume in ESAL

The amount of damage in each of four seasons (winter—frozen, spring thaw—saturated, spring/fall (autumn)—wet, and summer—dry) is calculated and checked against two criteria: rutting, and loss of serviceability.

Table 7.4 Traffic levels for AASHTO flexible pavement design catalog

Designation	*Traffic volume (ESAL)*
High	700,000–1,000,000
Medium	400,000–600,000
Low	50,000–300,000

Source: AASHTO, *AASHTO Guide or Design of Pavement Structures*, American Association of State Highway and Transportation Officials (AASHTO), Washington, DC, 1993.

The more general flexible pavement design method the Guide [7.1] presents can also be used for low-volume road pavement design. This is also based on a nomograph which takes into account the following parameters:

- Reliability
- Overall standard deviation of the data
- Estimated total ESAL over the design life of the pavement
- Subgrade resilient modulus
- Design serviceability loss

The output of the nomograph is the structural number, *SN*, which in turn is used in Equation 7.6. The nomograph is based on the following Equation [7.1]:

$$\log_{10} W_{18} = Z_r s_o + 9.36 \log_{10}(SN+1) - 0.20 + \frac{\log_{10}[\Delta PSI/(4.2-1.5)]}{0.40 + \left(1094/(SN+1)^{5.19}\right)} + 2.32 \log_{10} M_r - 8.07 \tag{7.7}$$

where

W_{18} is the estimated traffic (ESAL)
Z_r is standard normal deviate (determines reliability—see p. I-63 of the Guide [7.1])
s_o is the overall standard deviation of the input data
SN is the structural number (inches)
ΔPSI is the loss in serviceability over the life of the road
M_r is the resilient modulus of the subgrade (lb/in.^{2})

Note that the equation is valid only for U.S. Customary Units

The equation can be readily programmed into a spreadsheet. Because the equation is written with the design traffic as a function of the structural number *SN*, whereas usually the traffic is known and the structural number is to be determined, an iterative approach can be adopted. A trial structural number is assumed, and Equation 7.7 solved to calculate the design traffic volume. Based on a comparison of the calculated traffic volume and the known design traffic volume, a new structural number is input to Equation 7.7, a new traffic volume calculated, and the cycle continued until convergence on the design structural number, *SN*. Design then proceeds to Equation 7.6 and the pavement layer thicknesses determined.

The Ministry of Transportation of Ontario (MTO) has published pavement design catalogs for "King's Highways and Freeways" and "Secondary Highways" in the so-called Blue Book [7.11]. Ontario-specific values for input to the AASHTO design equation (Equation 7.7) are given in Hajek et al. [7.8], including a_i values (Table 7.2).

7.5.2 British pavement design for tropical and subtropical countries

A pavement design catalog for bitumen-surfaced roads is available in *Overseas Road Note 31* [7.4]. Table 7.5 lists the types of pavements included in the design catalog. The catalog covers traffic volumes from 300,000 to 30 million ESAL.

7.5.3 Australian design chart

Australian and New Zealand practice uses either a mechanistic-empirical design approach or a design chart. The design chart is appropriate for low-volume roads consisting of a high quality compacted crushed rock base topped with a thin, nonstructural bituminous surface of surface treatment or asphalt 50 mm or less in thickness. An early version of the chart appears in Austroads [7.13]. It has the thickness of the granular base plotted against the design traffic, ranging from 10^5 ESALs to 10^8 ESALs. Curves are shown for subgrade CBR values ranging from 2 to >30, and the minimum base thickness is 100 mm. No indication of the strength of the base material or the failure criterion is given.

A similar chart is presented in ARRB [7.14], an unsealed roads manual which includes a New Zealand supplement. The chart is similar in form to that in Austroads [7.13], with base layer thickness plotted against traffic for values of subgrade CBR from 3 to >30, but the range of traffic is lower, running from 10^3 to 5×10^5 ESAL.

More recently, the same chart appears in Hoque and Jameson [7.15]. The thickness of the granular base is given on the chart as Equation 7.8:

$$t = 0.475 \times \left[219 - 211 \log \mathrm{CBR} + 58\left(\log \mathrm{CBR}\right)^2\right] \times \log 14 V_t \tag{7.8}$$

where

t is the granular base layer thickness, with a minimum of 100 mm (mm)
CBR is the subgrade CBR
V_t is the traffic volume (ESAL)

The equation is only valid for SI units. Note that the equation is incorrect as published in the reference.

7.5.4 Comparison of methods

While it is tempting to compare the results of different pavement design methods for the same inputs, the comparisons may not always be fruitful for the following reasons:

- The methods may be applicable to specific sites or conditions

Table 7.5 Pavement designs in overseas road note 31 pavement design catalog

Surface	*Base*	*Pavement designs available for traffic level*							
Surface treated	Granular	T1	T2	T3	T4	T5	T6		
Surface treated	Composite (unbound and cemented)	T1	T2	T3	T4	T5	T6	T7	
Semistructural	Granular			T3	T4	T5	T6		
Semistructural	Composite (unbound and cemented)			T3	T4	T5	T6	T7	T8
Structural	Granular						T6	T7	T8
Structural	Composite (unbound and cemented)						T6	T7	T8
Semistructural	Bituminous				T4	T5	T6	T7	T8
Surface treated	Cemented	T1	T2	T3	T4	T5	T6		
Traffic levels	**Million ESAL**								
T1	<0.3								
T2	0.3–0.7								
T3	0.7–1.5								
T4	1.5–3.0								
T5	3.0–6.0								
T6	6.0–10								
T7	10–17								
T8	17–30								

Source: TRL, A guide to the structural design of bitumen-surfaced roads in tropical and subtropical countries, *Overseas Road Note 31*, 4th edn., Transportation Research Laboratory (TRL), Crowthorne, U.K., 75pp. http://www.transport-links.org/transport_links/filearea/publications/1_716_Microsoft%20Word%20-%20Overseas%20Road%20Note%2031%20edit1.pdf, visited January 15, 2015, 1993.

- The range of validity of the inputs to the design methods may be different
- There may be different inherent assumptions about input parameters
- Assumptions may not be explicit
- A parameter included in one method may not be included in another
- Failure criteria may be different

For these reasons, it should not be surprising that using the same inputs with two different pavement design methods does not necessarily yield the same resultants.

Designers are advised to adopt a method which suits their conditions and calibrate that method against existing pavements that have been successful and others that have failed in service.

7.6 U.S. MECHANISTIC-EMPIRICAL DESIGN METHOD

There has recently been a desire to develop a pavement design method which is based more on the mechanisms of pavement behavior, rather than on empiricism. Efforts have been made over the past decade in the United States to develop a mechanistic-empirical design method, culminating in the *AASHTO Mechanistic-Empirical Pavement Design Guide* (MEPDG) and associated software [7.16,7.17].

Moving from the previous empirical method described in Section 7.5.1 to the MEPDG and associated software presents the following advantages [7.17]:

- The evaluation of a broader range of vehicle loadings, material properties, and climatic effects
- Improved characterization of the existing pavement layers
- Improved reliability of pavement performance predictions

However, there is an increase in the quantity and complexity of the input data, and it is expected that implementing the MEPDG will increase the time required for pavement design and evaluation.

The mechanistic-empirical design method can be carried out at one of three levels [7.17]:

- *Level 1*: inputs are measured values, including site-specific traffic. This level requires the greatest parameter knowledge, and the greatest time, resources, and cost to obtain.
- *Level 2*: inputs are calculated from correlations with other, more easily obtained data.
- *Level 3*: inputs are based on default values, regional values, or expert opinion.

The software is arranged in modules which the designer uses to analyze a trial pavement design. The pavement structure, performance criteria, input data such as traffic, climate, layer properties, and calibration factors are input for the trial design, and the modeled behavior checked against the performance criteria. If all are acceptable, the pavement design is tentatively accepted, subject to an analysis of other factors such as whole-life costs and constructability.

The MEPDG approach, while being much more realistic than the AASHTO empirical approach presented in the 1993 Guide [7.1] seems unduly complicated and onerous for application at this time to LVR. However, it is still in development and bears monitoring over the next few years.

7.7 HEAVY-DUTY PAVEMENT DESIGN FOR HAUL ROADS

The designs presented in the previous sections have all applied to pavements designed for conventional axle loads, circa 8–10 tonnes (18,000–22,000 lb). But what about pavements designed for the very heavy hauling in forestry, mining, oil sands, or oil and gas operations? The axle loads for the vehicles on these roads can range up to hundreds of tons, "running off the chart" for conventional pavement design methods. Designers of these roads must look to other, completely different pavement design methods. It takes a completely different approach; the pavements can be extremely thick.

7.7.1 "Pavements" versus "Embankments"

When does a pavement become so thick it starts to look like an embankment? It depends on the circumstances.

Pavements are typically thin structures, where the live loading of the vehicles is the primary concern. Embankments are typically thick structures where the primary concern is the self-weight dead load of the embankment materials themselves. At what thickness does a "pavement" start to look and behave more like an "embankment"? It is dependent on the wheel load of the vehicle.

The vertical stress due to the wheel load can be modeled using the Boussinesq equation [7.18] and compared to the vertical stress imposed by the weight of saturated soil above a point in the subgrade. Where the two curves for the two stress distributions cross, the depth where a pavement becomes more like an embankment can be identified (Figures 7.3 and 7.4). There is a smooth, gradual transition from pavement to embankment, but the depth where the curves cross can serve as an indicator of the change. Note in Figure 7.4 that the depth where designing for pavement behavior is at least as important as designing for embankment behavior is as much as

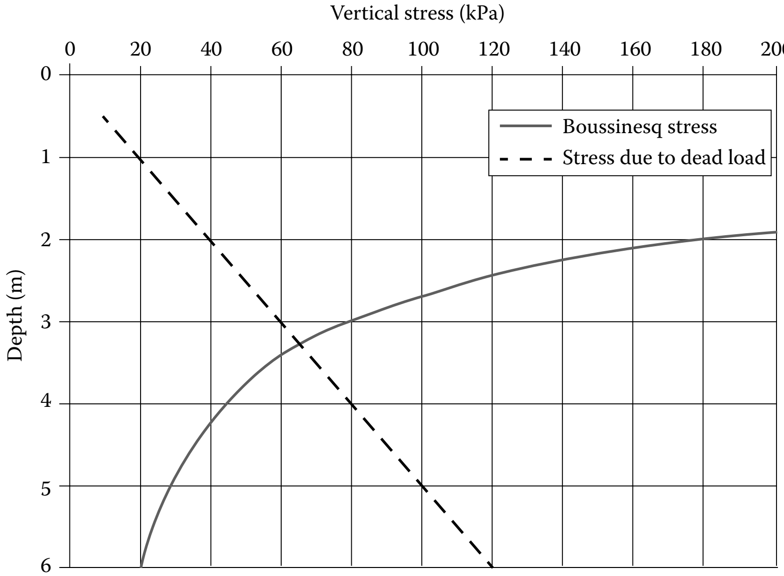

Figure 7.3 Vertical stresses due to live load and dead load, against depth. Live load stresses are for a 300 tonne (660,000 lb) axle with dual tires, dead load stresses are for a soil unit weight of 20 kN/m³ (130 lb/ft³).

3.3 m (11 ft) for a 300 tonne (330 ton) axle load. The structure should be designed as a pavement and also checked for adequacy as an embankment (e.g., base bearing capacity, settlement, and slope stability).

7.7.2 Designs from ports and airfields literature

Haul vehicles serving port facilities and large commercial aircraft impose heavy wheel loads on the pavements designed for them, and the pavement design methods for them can be used to design pavements for haul roads, sometimes with some necessary adaptations. Examples of such pavement design methods are presented in Knapton [7.19,7.20], AI [7.21], and Argue [7.22].

7.7.3 CBR chart

Atkinson [7.23] referenced in Tannant and Regensburg [7.24] presents a design chart with base cover thickness plotted against subgrade CBR for a range of *wheel* loads from 12 to 80 tonnes (26,000 to 180,000 lb). The failure criterion is not stated. A procedure is detailed in Tannant and Regensburg [7.24] which facilitates the design of multiple-layer granular pavement designs.

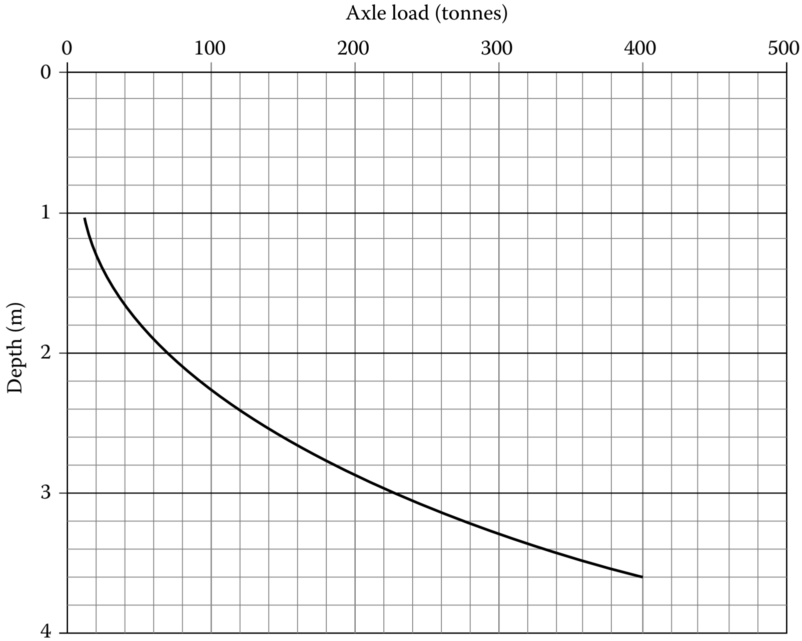

Figure 7.4 Depth where vertical stress due to live load equals vertical stress due to dead load, as a function of axle load.

7.7.4 Design based on "critical strain"

Atkinson's CBR chart [7.23,7.24], although catering to axle loads as great as 160 tonnes (350,000 lb), may still not go far enough for some mining, oil sands, or oil and gas haul road designs. Ultra-heavy dump truck gross vehicle masses as heavy as 600 tonnes (1.3 million lb), with loaded back axles imposing 400 tonnes (880,000 lb) are not uncommon (Figure 7.5). Design based on the *critical strain limit* can be used to design pavements for ultra-heavy haul trucks.

The premise of the critical strain method is that the vertical strain anywhere in the pavement should not exceed a critical limit, which is a function of the number of passes of the design axle [7.24]:

$$\varepsilon_{critical} = \frac{80{,}000}{N^{0.27}} \tag{7.9}$$

where

$\varepsilon_{critical}$ is the critical strain (micro-strain)
N is the number of passes of the design axle load

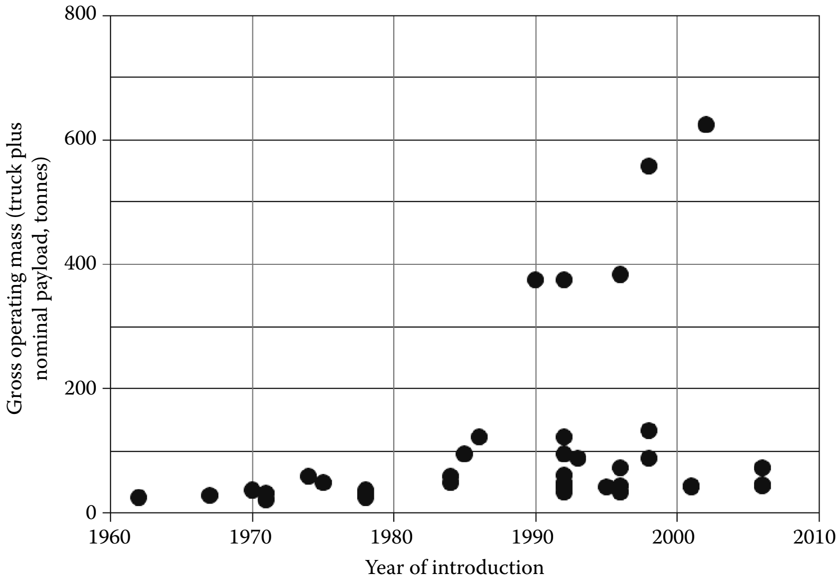

Figure 7.5 Rated operating mass of one manufacturer's off-road mining trucks, by model year of introduction. (From Caterpillar, *Caterpillar Performance Handbook*, 29th edn., Caterpillar Inc., Peoria, IL, pp. 25-70 and 25-71.)

Unfortunately, no design criterion is stated. Morgan et al. [7.26 cited in 7.24] determined that the critical vertical strain at the top of the subgrade should be about 1500 με. Thompson and Visser [7.27, 7.28 cited in 7.24] advocated a maximum strain limit of 2000 με at the top of the road surface. Knapton [7.29 cited in 7.24] developed an equation for heavy loads on docks at container ports; Equation 7.9 is a modified form for roads.

The following steps are carried out in the method:

1. The critical strain limit is calculated (Equation 7.9), usually lying between 1500 and 2000 με.
2. The resilient modulus for each pavement layer is set.
3. Based on experience, initial pavement layer thicknesses are set for a trial pavement design.
4. The vertical strains are calculated at particular points in the pavement cross section.
   a. Finite element analysis or elastic layer analysis is used.
   b. Typical locations where strains are calculated include the mid-depth, top and bottom faces of the pavement layers, and the top of the subgrade, at locations beneath the center of a tire, the center of a dual-tire wheel, the center of an axle, on the centerline of the

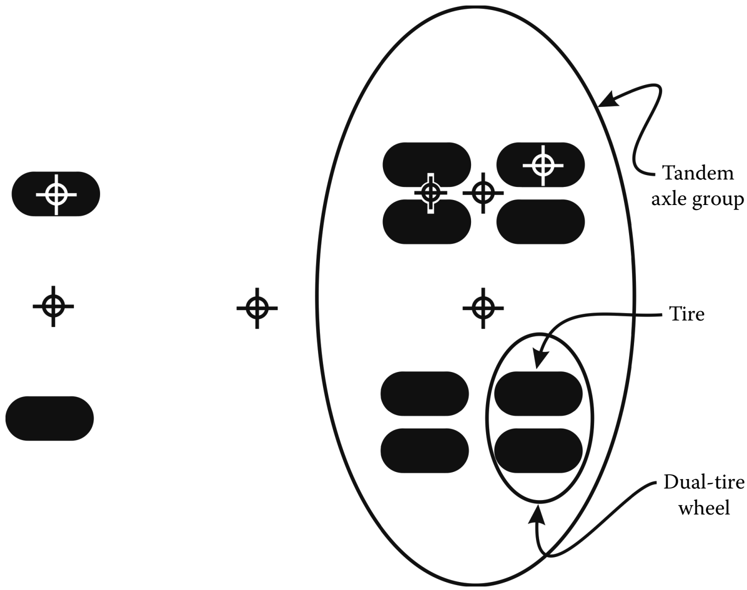

Figure 7.6 Plan view of typical locations where vertical strains are calculated in the critical strain method, for a tandem axle, dual wheel, 3 axle dump truck. Strains are computed at various depths below the locations indicated by symbol. (Not to scale.)

dual-tire wheels between axles in an axle group, at the center of an axle group, and at the center of the vehicle (Figure 7.6).

5. The calculated vertical strains are compared to the critical strain.
   a. If any is greater than the critical strain, the design is unacceptable, and one or more layer thicknesses should be increased, and the process repeated, beginning with Step 3.
   b. If any is much less than the critical strain, the design is acceptable, but inefficient. The layer(s) above the lightly strained layer should be made thinner, and the process repeated beginning with Step 3.
   c. If all calculated strains are equal to or just less than the critical strain, the design is acceptable and efficient.

Care should be taken with the elastic analysis carried out in Step 4 (Douglas et al. [7.30]). Granular materials are incapable of sustaining tensile strains, yet the finite element or layered elastic analysis may erroneously compute significant tensile strains, under-estimating the calculated vertical strains. A “no-tension” finite element or layered elastic analysis should be performed.

REFERENCES

7.1 AASHTO. 1993. *AASHTO Guide or Design of Pavement Structures*. Washington, DC: American Association of State Highway and Transportation Officials (AASHTO).

7.2 Haas, R., Hudson, W.R., and Zaniewski, J. 1994. *Modern Pavement Management*. Malabar, FL: Krieger Publishing Company, 583pp.

7.3 Bowles, J.E. 1992. *Engineering Properties of Soils and the Measurement*. Toronto, Ontario, Canada: McGraw-Hill Inc., 241pp.

7.4 TRL. 1993. A guide to the structural design of bitumen-surfaced roads in tropical and subtropical countries, *Overseas Road Note 31*, 4th edn. Crowthorne, U.K.: Transportation Research Laboratory (TRL), 75pp. http://www.transport-links.org/transport_links/filearea/publications/1_716_Microsoft%20Word%20-%20Overseas%20Road%20Note%2031%20edit1.pdf, visited January 15, 2015.

7.5 Lay, M.G. 2009. *Handbook of Road Technology*, 4th edn. Abingdon, U.K.: Spon Press, 933pp.

7.6 NAASRA. 1979. *Interim Guide to Pavement Thickness Design*. Sydney, New South Wales, Australia: National Association of Australian State Road Authorities (NAASRA).

7.7 Austroads. 2009. *Review of Relationship to Predict Subgrade Modulus from CBR (California Bearing Ratio)*. Sydney, New South Wales, Australia: Austroads Inc., 23pp.

7.8 Hajek, J.J., Smith, K.L., Rao, S.P., and Darter, M.I. 2008. Adaptation and verification of AASHTO pavement design guide for Ontario conditions. Toronto, Ontario, Canada: Ministry of Transportation of Ontario, Pavements and Foundation Section. Prepared by ERES Consultants, Applied Research Associates, Inc.

7.9 Smith, K.D., Zimmerman, K.A., and Finn, F.N. 2004. *The AASHO Road Test, Living Legacy for Highway Pavements*. TR News 232: May–June. Washington, DC: US National Academies, Transportation Research Board, pp. 14–24.

7.10 Swift, E. 2011. *The Big Roads: The Untold Story of the Engineers, Visionaries, and Trailblazers Who Created America's Superhighways*. Boston, MA: Houghton Mifflin Harcourt, pp. 198–199.

7.11 MTO. 1990. *Pavement Design and Rehabilitation Manual*. MTO STO-90-01. Toronto, Ontario, Canada: Ministry of Transportation of Ontario (MTO), 277pp.

7.12 MTO. 2004. Material specifications for aggregates—Base, subbase, select subgrade, and backfill material. Ontario Provincial Standard Specification OPSS 1010. Toronto, Ontario, Canada: Ministry of Transportation of Ontario (MTO), 18pp. http://www.raqsb.mto.gov.on.ca/techpubs/ops.nsf/d37f5a16d8174ffa85256d130066857f/702bdff1eaaf4d9e85257b5c006080ac/$FILE/OPSS.MUNI%201010%20Nov%2013.pdf, visited January 13, 2015.

7.13 Austroads. 1992. *Pavement Design, a Guide to the Structural Design of Road Pavements*. Austroads Publication No. AP-17/92. Surrey Hills, New South Wales Australia: Austroads.

7.14 ARRB. 2000. *Unsealed Roads Manual, Guidelines to Good Practice* (Giummarra, G., ed.). Vermont South, Victoria, Australia: ARRB Transport Research Ltd., p. 3.2.

7.15 Hoque, Z. and Jameson, G. 2006. *Pavement Design for Light Traffic: A Supplement to the Austroads Pavement Design Guide*. Sydney, New South Wales, Australia: Austroads Inc., p. 15.

7.16 FHWA. Get to know the new M-E pavement design guide. Focus, July 2006. Washington, DC: US Department of Transportation, Federal Highway Administration (FHWA), 7pp.

7.17 Pierce, L.M. and McGovern, G. 2014. *Implementation of the AASHTO Mechanistic-Empirical Pavement Design Guide and Software*, NCHRP Synthesis 457. Washington, DC: US Transportation Research Board, 71pp.

7.18 Lambe, T.W. and Whitman, R.V. 1969. *Soil Mechanics*, SI Version. Toronto, Ontario, Canada: John Wiley & Sons, Inc., 553pp.

7.19 Knapton. 1996. *The Structural Design of Heavy-Duty Pavements for Ports and Other Industries*, 3rd edn. Leicester, U.K.: Precast Concrete Federation Limited, 45pp.

7.20 Knapton. 2008. *Heavy-Duty Pavements, the Structural Design of Heavy-Duty Pavements for Ports and Other Industries*, 4th edn. Leicester, U.K.: British Precast Concrete Paving and Kerb Association, Precast Concrete Federation Limited, 87pp.

7.21 AI. 1986. *Thickness Design, Asphalt Pavements for Heavy Wheel Loads*, Manual Series 23, 1st edn. College Park, MD: The Asphalt Institute (AI), 69pp.

7.22 Argue, G.H. 2005. *Canadian Airfield Pavement Engineering Reference*. Ottawa, Ontario, Canada: Glen Argue. (May possibly be available from glen.argue@sympatico.ca.)

7.23 Atkinson, T. 1992. Design and layout of haul roads. *Chapter 13.4—SME Mining Engineering Handbook*, 2nd edn. Littleton, CO: Society of Mining, Metallurgy, and Exploration, Inc., pp. 1334–1342.

7.24 Tannant, D.D. and Regensburg, B. 2001. *Guidelines for Mine Haul Road Design*. Edmonton, Canada: School of Mining and Petroleum Engineering, Department of Civil and Environmental Engineering, University of Alberta, 111pp. https://circle.ubc.ca/bitstream/id/90332/Haul_Road_Design_Guidelines.pdf Visited January 17, 2015.

7.25 Caterpillar. undated. *Caterpillar Performance Handbook*, 29th edn. Peoria, IL: Caterpillar Inc., pp. 25-70, 25-71.

7.26 Morgan, J.R., Tucker, J.S., and McInnes, D.B. 1994. A mechanistic design approach for unsealed mine haul roads. *Pavement Design and Performance in Road Construction* 1412:69–81.

7.27 Thompson, R.J. and Visser, A.T. 1997. An introduction to the integrated design of surface mine haul roads. *International Journal of Surface Mining, Reclamation and Environment* 11: 115–120.

7.28 Thompson, R.J. and Visser A.T. 1997. A mechanistic structural design procedure for surface mine haul roads. *International Journal of Surface Mining, Reclamation and Environment* 11: 121–128.

7.29 Knapton, J. 1988. *The Structural Design of Heavy-Duty Pavements for Ports and Other Industries*. London, U.K.: British Ports Federation.

7.30 Douglas, R.A., Laprade, R., and Lawrence, K. 2015. High-strength geotextiles in ultra-heavy-load haul roads. In *Proceedings 11th International Conference on Low-Volume Roads*. Washington, DC: US Transportation Research Board, 13pp.