Structural Design of Pavements

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CT4860 STRUCTURAL DESIGN OF PAVEMENTS

PART IV
DESIGN OF CONCRETE PAVEMENTS

January 2006 ir. L.J.M. Houben

Preface

Cement concrete is used in many countries all over the world as a paving material for heavily loaded main highways, farm to market roads, public transport bus lanes, airport aprons etc.
Three types of concrete pavement are distinguished, which are:

- Plain (unreinforced) concrete pavement.
  Through transverse (and longitudinal) contraction joints the pavement is divided into a system of slabs of limited size. In the joints provisions are taken to ensure load transfer and evenness at long term.
  This pavement type is the most commonly used one, also in the Netherlands.
- Continuously reinforced concrete pavement.
  This pavement type has no transverse contraction joints. The continuous longitudinal reinforcement (reinforcement 0.6% to 0.75%) is controlling the crack pattern that develops due to hardening of the concrete and thermal shrinkage.
  This pavement type is widely used in e.g. Belgium and the USA, and since a few decades it is also applied (with a porous asphalt wearing course) on a number of motorway stretches in the Netherlands.
- Prestressed concrete pavement.
  In such a pavement such high compressive stresses are introduced that no cracks develop due to external loadings. This system allows the construction of rather thin concrete slabs of huge size.
  This pavement type never became very popular; only on Amsterdam Airport Schiphol this type of concrete pavement was constructed on the older aprons.

Because of its nature, concrete slabs will shrink, expand, curl and warp because of temperature movements.
These movements not only take place when the concrete is hardened but also during the hardening phase when shrinkage takes place.
All this means that much care should be given to the design and construction of concrete pavements especially since discontinuities (joints, edges and corners) are involved.
These lecture notes, which were carefully prepared by ir. L.J.M. Houben, are giving the necessary background information for the structural design of plain and reinforced concrete pavements. Emphasis is placed on the analytical design procedures but also some empirical design methods are presented.

Prof.dr.ir. A.A.A. Molenaar

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1. INTRODUCTION

In many countries concrete pavement structures are becoming increasingly popular due to competitive investment costs and lower maintenance costs (in comparison with asphalt pavements) and due to an increasing confidence in a good long term pavement behaviour (caused by more reliable design methods and better construction techniques).
This lecture note intends to explain the principles of the structural design of the two most widely applied types of concrete pavement, which are plain (unreinforced) concrete pavements and continuously reinforced concrete pavements.

In chapter 2 the materials usually applied in the various layers of (un)reinforced concrete pavements are described in general terms. Furthermore (indicative) values for the most important material properties are given.
Chapter 3 explains the usually applied theories for the calculation of the flexural tensile stresses (and the vertical displacements) within the concrete top layer due to the most important (external) loadings. These are not only the traffic loadings but also the temperature gradients and unequal subgrade settlements. Next the principles of the fatigue analysis, to take into account the repeated traffic loadings together with the temperature gradient loadings, are discussed. Furthermore the applicability of the finite element method with respect to the structural design of concrete pavements is reviewed in general terms.
In chapter 4 first some existing empirical concrete pavement design methods and the flowchart of some analytical design methods are briefly described, and a critical review about the applicability of these two types of design method is presented.
Finally, in chapter 5 the Dutch analytical structural design method for concrete pavements is discussed. The original method (developed in the eighties) as well as the (in the nineties) revised method, both only valid for plain concrete pavements, are briefly discussed. Emphasis is laid on the current method, released early 2005 as the software program VENCON2.0, that covers both plain and reinforced concrete pavements.

2. CONCRETE PAVEMENT STRUCTURE

2.1 General

The top layer of a concrete pavement structure consists of cement concrete that exhibits an elastic behaviour until the moment of failure (cracking). Cement concrete has a very high Young's modulus of elasticity, which results in a great load spreading in the top layer and hence in low stresses in the underlying substructure (base plus sub-base plus subgrade).
There are three possibilities to prevent uncontrolled cracking of the top layer due to shrinkage of the concrete, which occurs during the hardening process and in hardened concrete due to a decrease of temperature:

- in plain (unreinforced) concrete pavements every 3 to 6 m a transverse joint is made, and in wide pavements also longitudinal joints are made; this means that the pavement is divided into concrete slabs;
- in reinforced concrete pavements such an amount of longitudinal reinforcement (0.6 to 0.75%) is applied, that every 1.5 to 3 m a very narrow crack occurs;
- in prestressed concrete pavements by prestressing such compressive stresses are introduced that the (flexural) tensile stresses due to shrinkage, temperature and traffic loadings stay within acceptable values.

Prestressed concrete pavements are such a specialized (and costly) type of concrete pavement structure, that they are only applied at some airport-platforms (for instance at Amsterdam Airport Schiphol), where 'zero maintenance' is very important. In this lecture note no further attention will be given to prestressed concrete pavements.

The thickness of the (un)reinforced concrete top layer is 150 to 450 mm, dependent on the traffic loadings, the climate, the concrete quality, the type of concrete pavement and the properties of the substructure materials.
Considering the great load spreading in the concrete top layer, for reasons of strength a base is not (always) necessary. Nevertheless generally a base (with a high resistance to erosion) is applied to prevent as much as possible the loss of support of the concrete top layer, which could result in unevenness and/or early cracking of the concrete. Both cement-bound materials and unbound materials can be applied in the base. Generally the base thickness is 150 to 300 mm.

A sub-base can be necessary to realize an embankment, to prevent damage due to frost action (in cold climates) or to strengthen the concrete pavement structure. However, especially on weak subgrades an important reason for application of a sub-base is that in this way a stable platform is created to construct the overlying base and/or concrete top layer.

2.2 Subgrade

The Young's modulus of elasticity of the concrete top layer is much higher than that of the underlying layers, which means that the top layer takes the main part of the traffic loading. This implies that the bearing capacity of the subgrade has only a small effect on the stresses in the concrete layer due to a traffic loading.

Due to this small effect it is common use in the design of concrete pavement structures to simply schematize the subgrade as a Winkler-foundation, so into a system of independent vertical linear-elastic springs with stiffness k_o, the so-called ‘modulus of subgrade reaction’ (see 3.2).
The bearing capacity of the subgrade (modulus of subgrade reaction k_o) does have a great effect on the vertical displacements (deflections) of a concrete pavement structure due to a traffic loading.
Because of the characteristic behaviour of a concrete pavement structure and the high repair costs in case of failure, it is important to use in the design of the concrete pavement structure a relative low modulus of subgrade reaction, for instance the value that has a 95% probability of exceeding.

Besides the bearing capacity also the settlement behaviour of the subgrade is important.
Except the connection to bridges, founded on piles, equal settlements of the subgrade generally are not a problem.
However, by unequal subgrade settlements extra flexural stresses are introduced in the concrete pavement structure. The magnitude of these stresses is dependent on the wavelength and amplitude of the settlement pattern (related to the dimensions of the concrete pavement) and on the velocity of the settlement process (because of stress relaxation in the cement concrete) (see 3.5).

2.3 Sub-base

The thickness of the sub-base is dependent on the designed height level of the road surface, the frost penetration depth (in cold climates), the permeability and bearing capacity of the subgrade, the traffic loadings (especially during the construction of the road) and the properties of the sub-base material.
Generally an unbound granular material (like gravel, crusher run, blast furnace slags, sand etc.) is applied for the sub-base. The grading should meet the filter laws to the subgrade material and eventually to the unbound base material.

Similar to the subgrade, also the bearing capacity of the sub-base has a limited effect on the stresses in the concrete layer due to a traffic loading and a considerable effect on the deflections of the concrete pavement structure due to a traffic loading.
In the design of concrete pavement structures the effect of a sub-base generally is taken into account by means of a certain increase (dependent on the thickness and Young’s modulus of elasticity of the sub-base) of the modulus of subgrade reaction k_o (see 3.2).

In cold climates the sub-base may be within the frost penetration depth. In such cases the sub-base material should not be frost-susceptible.

In case of a good quality subgrade (sand or better) a sub-base in concrete pavement structures may not be necessary.

2.4 Base

2.4.1 General

For reasons of strength a base is not absolutely necessary in a concrete pavement structure because (similar to the subgrade and the sub-base) it has only a limited effect on the stresses in the concrete layer due to a traffic loading. However there are other reasons why in concrete pavement structures nowadays nearly always a base is applied, such as:

- below the concrete top layer there has to be a layer with a high resistance to erosion, to ensure a good support of the concrete layer and to prevent 'pumping' of fine material through joints and cracks;
- by applying a base the deflection of the concrete pavement structure due to a traffic loading is considerably reduced, which is very favourable with respect to the long term pavement behaviour (especially the evenness around joints in plain concrete pavements);
- the construction traffic and the construction equipment (slipformpaver) for the concrete layer require an even surface with sufficient bearing capacity; in general the sub-base and especially the subgrade can not fulfill these requirements.

At each side of the road the base has to be at least 0.5 m wider than the concrete layer, to give sufficient support to the slipformpaver.
To obtain an equal thickness of the concrete top layer, the surface of the base has to be rather even, for instance a maximum deviation of 15 mm over a distance of 3 m.
In cold climates the base is within the frost penetration depth and therefore has to be frost-resistant.

As base materials in concrete pavement structures especially unbound materials and cement-bound materials are applied.

2.4.2 Unbound base

An unbound base material has to fulfill the following requirements:

- a good permeability to remove as soon as possible (to the sub-base or subgrade) the rainwater that entered the concrete pavement structure;
- a good resistance to crushing to prevent frost-susceptibility and to prevent loss of support of the concrete top layer due to erosion;
- a good resistance to permanent deformation to prevent as much as possible loss of support of the concrete top layer due to the repeated traffic loadings.

Well graded crusher run, (high quality) gravel etc. can be used as unbound base materials in concrete pavement structures.
The thickness of the unbound base is dependent on the traffic loadings (also construction traffic), the bearing capacity of the underlying layers (sub-base, if any, and subgrade) and the properties of the base material. In general in road pavements the thickness of an unbound base is 200 to 300 mm.

Similar to the sub-base (see 2.3), in the design of concrete pavement structures the effect of an unbound base generally is taken into account by means of a further increase (dependent on the thickness and Young's modulus of elasticity of the base) of the 'modulus of subgrade reaction' of the subgrade plus the sub-base (see 3.2).

Generally an unbound base is mainly applied for lightly loaded concrete pavement structures. However in the USA high quality, permeable unbound base materials are also preferred for heavily loaded concrete pavement structures (high resistance against erosion, lower temperature stresses in the concrete top layer). In other countries a cement-bound base is used in heavily loaded concrete pavements for motorways, airport platforms, container yards, etc.

2.4.3 Cement-bound base

Like cement concrete also a cement-bound base material is subjected to shrinkage of the concrete, which occurs during the hardening process and in the hardened cement-bound base due to a decrease of temperature. Due to the friction with the underlying layer this shrinkage results in cracking. The more cement in the base material, the wider cracks will occur at greater mutual distance.
Without measures there will grow an uncontrolled crack pattern in the cement-bound base, with variable crack distances and crack widths. The major cracks give the risk of reflection cracking, which means the growth of cracks from the base into the concrete top layer.
There are some measures to prevent this reflection cracking:

1. Preventing the adhesion between the cement-bound base and the concrete top layer by the application of a 'frictionless' layer (plastic foil) on top of the base. For plain concrete pavements this measure has however the great disadvantage that only a very limited number of the joints actually will crack and these few joints then exhibit a very great width, with the risk of penetration of rainwater and subsequent erosion of the base, reduced driving comfort etc. In reinforced concrete pavements instead of the desired crack pattern (every 1.5 to 3 m a fine crack) only a few wide cracks at great mutual distance may easily grow with similar consequences as for plain concrete pavements. A 'frictionless' layer is therefore hardly ever applied between a cement-bound base and the concrete top layer.
2. Not preventing the adhesion between the cement-bound base and the concrete top layer, but controlling the cracking in the base by means of:
   a. distribution of the construction traffic for the concrete top layer, in such a way that there will grow a regular pattern of fine cracks in the cement-

bound base; of course the cement-bound base should not be totally destructed by the construction traffic

b. in case of unreinforced concrete pavements, by weakening the cement bound base at regular distances, so that the location of the cracks is fixed (similar to the joints in the unreinforced cement concrete); the cracks in the cement-bound base have to be exactly below the joints in the concrete top layer.

   In first instance there will be a substantial adhesion between the concrete top layer and a cement-bound base. However, due to different displacement behaviour (caused by the temperature variations and the traffic loadings) of the concrete layer and the base, during time this adhesion will disappear to a great extent. For reasons of safety therefore in general in the structural concrete pavement design it is assumed that there exists no adhesion between the concrete layer and the cement- bound base.

3. In reinforced concrete pavements in the Netherlands nowadays an asphalt layer is applied between the cement-bound base and the concrete top layer. Such an asphalt layer not only limits the risk for reflective cracking but also has a good resistance to erosion and yields over the whole contact area with the concrete top layer a rather constant friction which is very favorable to obtain a regular crack pattern in the concrete top layer.

In the design of concrete pavement structures the effect of a cement-bound base in general is also taken into account by means of an increase (dependent on the thickness and Young’s modulus of elasticity of the base) of the ‘modulus of subgrade reaction’ of the subgrade plus the sub-base (see 2.4.2 and 3.2). Sometimes a cement-bound base is taken into account by considering the cement concrete layer and the base as a layered slab (with or without friction between the layers) on elastic springs, with a stiffness equal to the ‘modulus of subgrade reaction’ of the sub-base plus the subgrade.

As material for a cement-bound base can be used:

- granular material (sand, gravel, crusher run, blast furnace slags, crushed concrete (obtained from demolition waste), asphalt granulate (obtained from asphalt road reconstruction) etc. stabilized by means of cement; the amount of cement depends on the grading of the granular material (the finer the more cement is needed) and the strength and erosion requirements
- lean concrete which is a mixture of gravel and/or crushed concrete, sand, water and cement.

Table 1 gives some properties of two cement-bound base materials, namely sandcement (sand stabilized with cement) and lean concrete.

The thickness of a cement-bound base for concrete road pavements generally is 150 to 250 mm. For very heavily loaded concrete pavements (for instance airport platforms) the cement-bound base thickness goes up to 600 mm.

Property	Sandcement	Lean concrete
Amount of cement: kg/m^3 % by mass	120 – 200 7 - 12	75 – 125 3 - 5
Granular materials (% by mass)	sand: 100%	sand: 25 – 45% gravel: 75 – 55%
Density (kg/m^3)	1800 – 1900	2200 – 2400
Mean flexural tensile strength (N/mm²) of cylinders taken from the pavement: after 7 days after 28 days	 0.5 – 1.0 1.0 – 2.0	 1.0 – 1.5 1.5 – 2.5
Mean compressive strength (N/mm²) of cylinders taken from the pavement: after 7 days after 28 days	 2.0 – 4.0 4.0 – 8.0	 3.0 – 6.0 5.0 – 10.0
Dynamic modulus of elasticity (N/mm²) of not cracked material: after 7 days after 28 days	 5000 – 8000 6000 - 12000	 10000 – 15000 15000 – 20000
Poisson's ratio ν	0.15 – 0.30	
Coefficient of linear thermal expansion α (°C^{-1})	$1\cdot10^{-5} – 1.2\cdot10^{-5}$	

Table 1. Indicative values for some properties of sandcement and lean concrete.

2.5 Concrete top layer

2.5.1 Mechanical properties of concrete

The top layer of a concrete pavement structure consists of cement concrete, which is a mixture of gravel and/or crusher run, sand, cement and water. The most important properties (with respect to the design of concrete pavement structures) of some (Dutch) concrete qualities are discussed below.

Various concrete qualities are applied in the top layer of concrete pavements. In the old Dutch Standard NEN 6720 (1), valid until July 1, 2004, the concrete quality was denoted as a B-value where the value represented the characteristic (95% probability of exceeding) cube compressive strength after 28 days for loading of short duration* ($f_{cc,k,o}$ in N/mm^2). In the new Standard NEN-EN 206-1 (2), or the Dutch application Standard NEN 8005 that is valid since July 1, 2004, the concrete quality is denoted as C-values where the last value represents the characteristic (95% probability of exceeding) cube compressive strength after 28 days for loading of short duration and the first

* loading of short duration: loading during a few minutes
loading of long duration: static loading during 10^3 to 10^6 hours, or
dynamic loading with about 2.10^6 load cycles

value represents the characteristic cylinder compressive stress at the same conditions (table 2).

Concrete quality		Characteristic (95% probability of exceeding) cube compressive strength after 28 days for loading of short duration, $f_{cc,k,o}$ (N/mm^2)
B-value	C-values	
B35	C28/35	35
B45	C35/45	45
B55	C45/55	55

Table 2. Dutch concrete qualities used in road construction.

Generally on heavily loaded plain concrete pavements, such as motorways and airport platforms, the concrete quality C35/45 is used. On lightly loaded plain concrete pavements (bicycle tracks, minor roads, etc.) mostly concrete quality C28/35 and sometimes C35/45 is applied. In reinforced concrete pavements mostly the concrete quality C28/35 is applied and sometimes the concrete quality C35/45.
The concrete quality C45/55 is used for precast elements such as concrete blocks, tiles and kerbs.

According to both the CEB-FIP Model Code 1990 (3) and the Eurocode 2 (4) the mean cube compressive strength after 28 days for loading of short duration ($f_{cc,m,o}$) is equal to:

$$f_{cc,m,o} = f_{cc,k,o} + 8 \quad (N/mm^2) \qquad (1)$$

For the structural design of concrete pavements the compressive strength is not very relevant. Much more important is the flexural tensile strength that is related to the tensile strength.
For elastically supported concrete pavements, the mean tensile strength after 28 days for loading of short duration ($f_{ct,m.o}$) is equal to:

$$f_{ct,m,o} = 0.9\ [1.05 + 0.05\ f_{cc,m,o}] = 0.9\ [1.05 + 0.05\ (f_{cc,k,o} + 8)] \quad (N/mm^2) \qquad (2)$$

The mean tensile strength after 28 days for loading of long duration ($f_{ct,m,\infty}$) is taken as 70% of the mean tensile strength for loading of short duration:

$$f_{ct,m,\infty} = 0.7 \cdot f_{ct,m,o} = 0.7 \cdot 0.9\ [1.05 + 0.05\ f_{cc,m,o}] =$$

$$= 0.7 \cdot 0.9\ [1.05 + 0.05\ (f_{cc,k,o} + 8)] \quad (N/mm^2) \qquad (3)$$

The value of the mean tensile strength, for loading of long duration, to be used in design calculations ($f_{ct,d,\infty}$) is obtained by means of equation 4:

$$f_{ct,d,\infty} = 1.4\ f_{ct,m,\infty}/\gamma_m = 1.4 \cdot 0.7 \cdot 0.9\ [1.05 + 0.05\ (f_{cc,k,o} + 8)]/1.2 \quad (N/mm^2) \qquad (4)$$

where: 1.4 = 'correction factor' for a better agreement of the calculated tensile strength with the values normally used in pavement engineering
γ_m = 1.2 is the material factor for concrete under tension

In the Dutch design method for concrete pavements (5) the mean tensile strength after 28 days for loading of short duration ($f_{ct,d,o}$) is used, and that is obtained by means of equation 5:

$$f_{ct,d,o} = f_{ct,d,\infty}/0.7 \approx 1.3\ [1.05 + 0.05\ (f_{cc,k,o} + 8)]/1.2 \quad (N/mm^2) \tag{5}$$

However, certainly for plain concrete pavements the flexural tensile strength is much more important than the tensile strength. According to both NEN 6720 (1) and the Eurocode 2 (4) the relation between the mean flexural tensile strength ($f_{ct,fl,o}$) and the tensile strength ($f_{ct,d,o}$) is defined as a function of the thickness h (in mm) of the concrete slab:

$$f_{ct,fl,o} = [(1600 - h)/1000)]\ f_{ct,d,o} =$$

$$= 1.3\ [(1600 - h)/1000)]\ [1.05 + 0.05\ (f_{cc,k,o} + 8)]/1.2 \quad (N/mm^2) \tag{6}$$

In the Dutch design method (5) the thickness design of both plain and reinforced concrete pavements is based on a fatigue analysis for the most critical points of the pavement (see 5.4.8). The following fatigue relationship is used (6):

$$\log N_i = \frac{12.903\ (0.995 - \sigma_{\max_i} / f_{ct,fl,o})}{1.000 - 0.7525\ \sigma_{\min_i} / f_{ct,fl,o}} \quad with\ 0.5 \le \sigma_{\max} / f_{ct,fl,o} \le 0.833 \tag{7}$$

where: N_i = allowable number of repetitions of wheel load P_i i.e. the traffic load stress σ_{vi} until failure when a temperature gradient stress σ_{ti} is present

σ_{mini} = minimum occurring flexural tensile stress (= σ_{ti})

σ_{maxi} = maximum occurring flexural tensile stress (= $\sigma_{vi} + \sigma_{ti}$)

$f_{ct,fl,o}$ = mean flexural tensile strength (N/mm^2) after 28 days for loading of short duration

Except the strength also the stiffness (i.e. Young's modulus of elasticity) of concrete is important for the structural design of concrete pavements. The Young's modulus of elasticity of concrete depends to some extent on its strength. According to NEN 6720 (1) the Young's modulus of elasticity E_c can be calculated with the equation:

$$E_c = 22250 + 250 \cdot f_{cc,k,o} \quad (N/mm^2) \quad with\ 15 \le f_{cc,k,o} \le 65 \tag{8}$$

where: $f_{cc,k,o}$ = characteristic cube compressive strength (N/mm^2) after 28 days for loading of short duration

For the three concrete qualities applied in pavement engineering, table 3 gives some strength and stiffness values. Besides some other properties are given such as the Poisson's ratio (that plays a role in the calculation of traffic load stresses, see 3.4) and the coefficient of linear thermal expansion (that plays a role in the calculation of temperature gradient stresses, see 3.3).

Property	Concrete quality		
	C28/35 (B35)	C35/45 (B45)	C45/55 (B55)
Characteristic* cube compressive strength after 28 days for loading of short duration, $f_{cc,k,o}$ (N/mm^2)	35	45	55
Mean cube compressive strength after 28 days for loading of short duration, $f_{cc,m,o}$ (N/mm^2)	43	53	63
Mean tensile strength after 28 days for loading of short duration $f_{ct,d,o}$ (N/mm^2)	3.47	4.01	4.55
Mean flexural tensile strength after 28 days for loading of short duration, $f_{ct,fl,o}$ (N/mm^2):			
concrete thickness h = 180 mm	4.92	5.69	6.46
h = 210 mm	4.82	5.57	6.32
h = 240 mm	4.71	5.45	6.19
h = 270 mm	4.61	5.33	6.05
Young's modulus of elasticity, E_c (N/mm^2)	31000	33500	36000
Density (kg/m^3)	2300 - 2400		
Poisson's ratio ν	0.15 – 0.20		
Coefficient of linear thermal expansion α (°C^{-1})	$1{\cdot}10^{-5} – 1.2{\cdot}10^{-5}$		

* 95% probability of exceeding

Table 3. Mechanical properties of (Dutch) cement concrete for concrete pavement structures.

2.5.2 Reinforced concrete

Just after construction the reinforced concrete pavement is a long continuous strip in the longitudinal direction of the road (in the transverse direction the dimension of the pavement is limited to maximum 5 m through the construction of longitudinal joints). As a result of shrinkage of the concrete, during the hardening process and due to temperature decreases during or after the hardening process, longitudinal tensile forces develop in the concrete layer because of the friction with the underlying base layer. The primary goal of the (longitudinal) reinforcement in continuously reinforced concrete pavements is to control the crack pattern that develops due to the tensile forces. The longitudinal reinforcement should lead to a crack pattern of fine transverse cracks at mutual distances of 1.5 to 3 m.
The reinforcement is located in, or just above, the middle of the concrete layer. It thus is not a structural reinforcement but only a shrinkage reinforcement. In an indirect way the reinforcement however contributes to the strength of the pavement as the fine cracks result in a high load transfer (see 3.4.3).
The design of the reinforcement is explained in chapter 5 (i.e. section 5.4.10) where the current Dutch design method for concrete pavements (5) is discussed.

2.6 Layout of concrete pavement structures

2.6.1 Plain concrete pavements

In plain (unreinforced) concrete pavements joints have to be realized to prevent the concrete from uncontrolled cracking (occurring at the first cooling of the concrete after the construction or later at a decrease of temperature in the hardened concrete) due to friction with the underlying layer. One distinguishes:

- in the transverse direction: contraction joints, expansion joints (at the end of the concrete pavement, for instance in front of bridges) and construction joints (at the end of a daily production, in case of a breakdown of the construction equipment or in case of logistic problems with the supply of concrete);
- in the longitudinal direction: contraction joints and construction joints (between two lanes of concrete placement).

Through the transverse and longitudinal contraction joints a plain concrete pavement is divided into concrete slabs. To limit the temperature gradient stresses (see 3.3) the slabs should be more or less square with a maximum horizontal dimension smaller than about 5 m (on roads) and 7.5 m (on airports) respectively.

Figure 1 shows an example of an unreinforced concrete pavement for a two-lane industrial road.

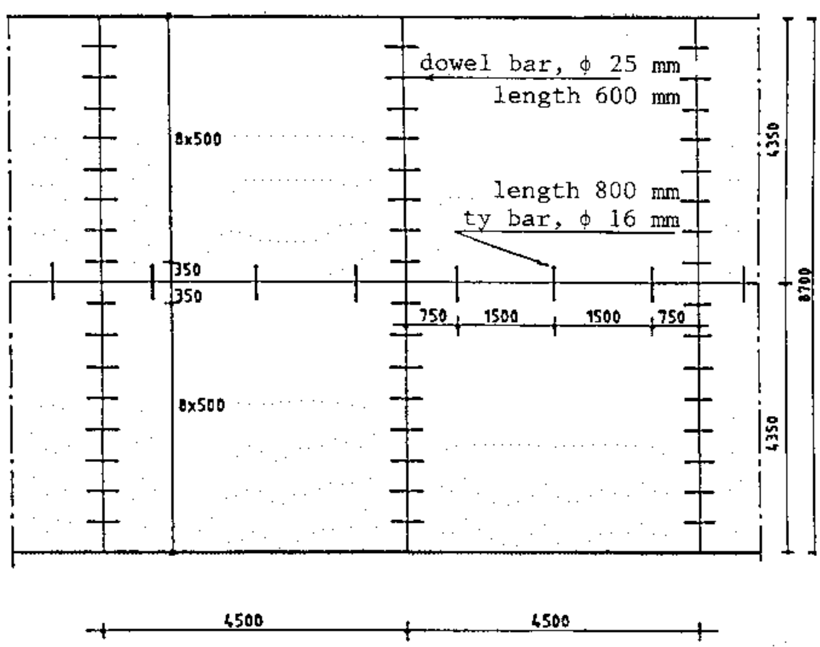


Figure 1. Concrete slab configuration, with dowel bars and ty bars, of a two-lane industrial road.

For a better load transfer, dowel bars are applied in the transverse contraction joints of heavier loaded concrete pavements at mid-height of the concrete slab. A dowel bar is a steel bar with a diameter Ø of about 10% of the concrete pavement thickness (normally Ø = 25 mm in road pavements and Ø = 32 mm in airport pavements) and a length of 500 to 600 mm. The distance between the dowel bars is 300 to 500 mm. The dowel bars should by no means obstruct the horizontal movements of the concrete slabs due to the variation of the absolute temperature and therefore they have a bituminous or plastic coating to prevent adhesion to the concrete (figure 2).

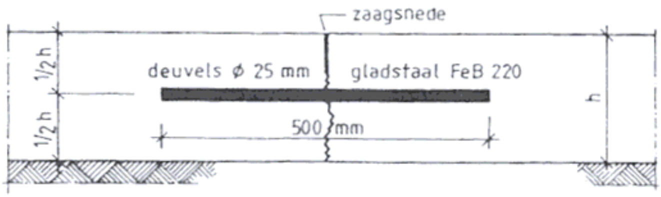


Figure 2. Transverse contraction joint with dowel bar (7).

In longitudinal contraction joints so-called ty bars are applied to prevent two adjacent rows of concrete slabs to float away from each other due to variation of the absolute temperature. The ty bars are located at mid-height, or even somewhat higher, of the concrete slab. The profiled steel ty bars have a diameter Ø = 16 mm and a length of at least 600 mm. At both ends the ty bars are fixed into the concrete, however the central one-third part of the ty bar has a coating (which prevents bond to the concrete) to distribute the occurring movements of the concrete slabs due to varying absolute temperatures over a sufficient length so that no flow of the ty bar steel occurs (figure 3). In longitudinal contraction joints normally 3 ty bars per concrete slab length are applied (see figure 1).

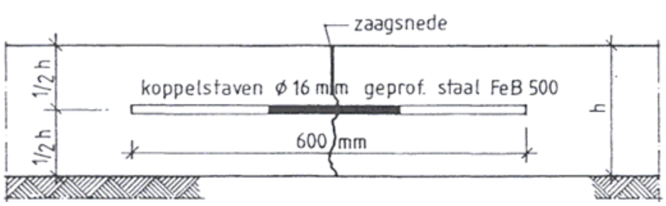


Figure 3. Longitudinal contraction joint with ty bar (7).

Contraction joints are made by sawing a 3 mm wide cut into the hardening concrete. This sawing has to be done as soon as possible and certainly within 24 hours after the construction of the concrete. The depth of the saw cut for longitudinal contraction joints should be 40 to 45% of the concrete thickness and for transverse contraction joints about 35% of the concrete thickness. By these saw cuts the concrete is weakened to such an extent that the inevitable cracks (due to shrinkage of the hardening concrete or a decrease of the absolute temperature of the hardened concrete) will appear below the saw cuts.
The joints may be left unfilled (which is usually done at minor roads) or they may be filled. In this latter case by further sawing the joints have to be

widened (e.g. to 8 mm) to a certain depth to enable filling of the joints (with a bituminous material or with special hollow plastic profiles) and to limit the strains in the joint-filling material at changing joint widths due to temperature variations.

On roads the thickness of plain concrete pavements varies from 180 mm (bicycle tracks) to 300 mm (motorways). On airports and other very heavily loaded pavements plain concrete pavement thicknesses up to 450 mm are applied.

2.6.2 Reinforced concrete pavements

A reinforced concrete pavement does not have transverse contraction joints. The shrinkage of the hardening concrete is constrained by the adhesion to the steel reinforcement and by the friction with the underlying layer, which results in a more or less regular pattern of fine transverse cracks. The amount of reinforcement should be such that the mean distance of the cracks is between 1.5 and 3 m. In this case the pavement remains more or less continuous and the crack width is so small (in general less than 0.4 mm, see 5.4.10) that there will be hardly any penetration of rainwater, and therefore the reinforcement will not be affected by corrosion. It takes some years to achieve the ultimate crack pattern.

The reinforcement exists of a primary longitudinal reinforcement (profiled steel, diameter 14 to 20 mm, amount of reinforcement 0.60% to 0.75%, see 5.4.10) and below it a secondary transverse reinforcement that also acts as support for the longitudinal reinforcement in the construction phase. The longitudinal reinforcement is about in the middle of the concrete thickness (see 5.4.10).

The load transfer in the very narrow cracks of a reinforced concrete pavement is very good (see 3.4.3).
The design of the thickness of a reinforced concrete pavement is normally done by assuming a plain concrete pavement instead. This is also done in the Dutch design method (5) that however takes into account some specific features of reinforced pavements. The required thickness of the reinforced concrete pavement is somewhat less (up to 20 mm) than would be necessary in the case of a plain concrete pavement. After the determination of the concrete thickness the required longitudinal reinforcement is calculated (see 5.4.10).

A reinforced concrete pavement is somewhat more flexible (with respect to the ability to follow unequal subgrade settlements) than a unreinforced concrete pavement, because the crack distance in a reinforced pavement is smaller than the transverse joint distance in a unreinforced pavement.

Generally spoken, a reinforced concrete pavement requires higher investment costs and lower maintenance costs than a unreinforced concrete pavement. In the Netherlands reinforced concrete pavements are therefore mainly applied

on motorways where it is government policy to apply 'silent roads' which means in practice a wearing course of porous asphalt ('ZOAB'). The fine cracks in the reinforced concrete pavement prevent ZOAB for reflective cracking.
Below the reinforced concrete pavement a 50 to 60 mm thick asphalt layer is applied. The reasons for this are:
- the asphalt layer has a very good resistance against erosion which is essential for a good structural behaviour of the concrete pavement;
- the asphalt layer provides a very uniform friction with the overlying reinforced concrete layer which is favorable for the development of a regular pattern of fine cracks in the concrete layer;
- the surface of the asphalt layer is very even which results in a constant thickness of the concrete later and that is also favorable for the development of a regular pattern of fine cracks in the concrete layer;
- in the case that a cement-bound base is applied the asphalt layer acts as an anti-reflective layer.

Figure 4 shows the top view of the reinforced concrete pavement on one carriageway of a motorway. The 12 m wide carriageway contains 2 traffic lanes and an emergency lane.

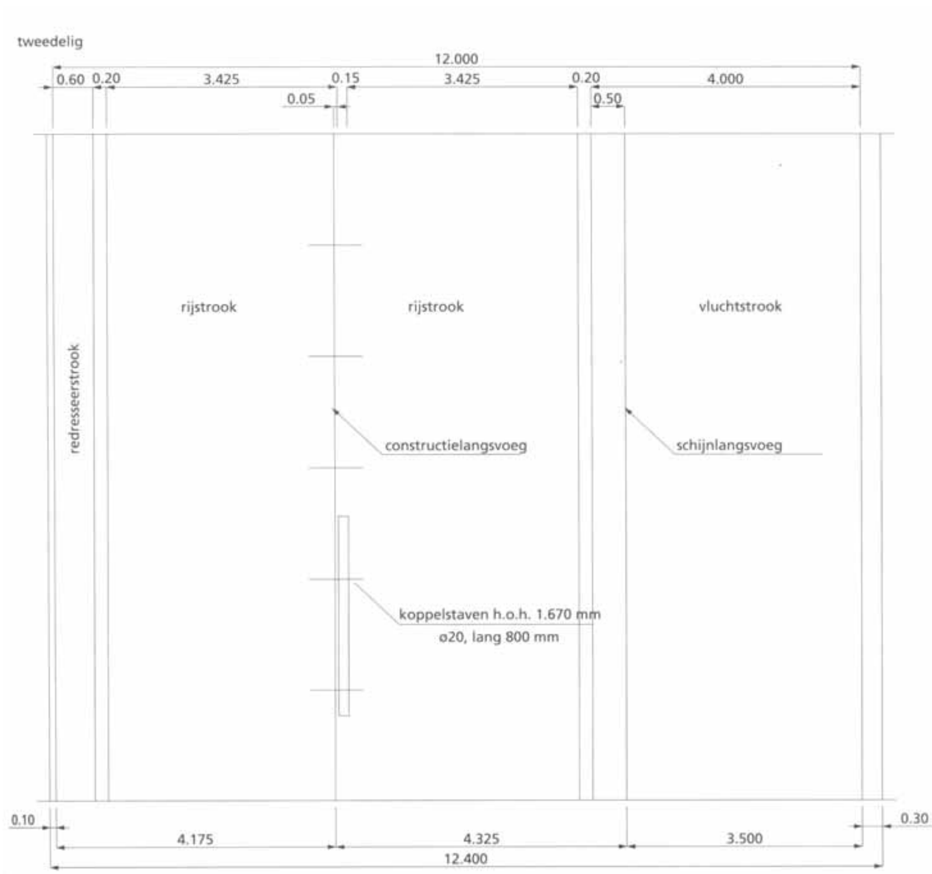


Figure 4. Top view of a carriageway of a motorway with a reinforced concrete pavement (8).

The pavement was constructed in two strips. One strip carries the left (fast) traffic lane and the second strip carries the right (slow) traffic lane plus the emergency lane.
The longitudinal construction joint between these two strips is located just left of the heavily loaded right traffic lane. In the construction joint ty bars (diameter Ø = 20 mm, length = 800 mm) are applied at a mutual distance of 1.67 m. Figure 5 shows in more detail the longitudinal construction joint with the longitudinal and transverse reinforcement and the ty bars.

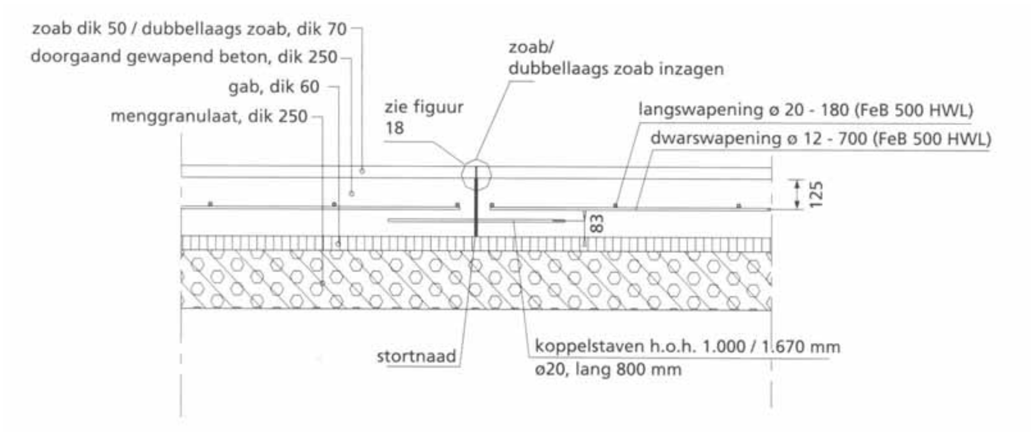


Figure 5. Longitudinal construction joint in reinforced concrete pavement (8).

The second concrete strip in figure 4 has a width of 7.825 m. This is far more than 5 m and therefore a longitudinal contraction joint has to be applied. In such a contraction joint in reinforced concrete pavements no ty bars are necessary (compare figure 3) as the present transverse reinforcement already fulfills the function of the ty bars.

3. STRESSES AND DISPLACEMENTS IN CONCRETE PAVEMENT STRUCTURES

3.1 General

In concrete pavement structures, especially the concrete top layer, stresses (and displacements) are introduced by:
- shrinkage during the hardening process of the concrete
- variation of the moisture content of the concrete
- a temperature change which is constant over the thickness of the concrete layer
- a temperature gradient in the concrete layer
- the traffic loadings
- unequal subgrade settlements.

The stresses and displacements due to the hardening shrinkage of the concrete and due to the regular temperature change are (practically spoken) eliminated by dividing the concrete layer into slabs (plain concrete pavements) or by the application of a continuous longitudinal reinforcement (reinforced concrete pavements).
Variation of the moisture content of the concrete (more specific drying out of the upper part of the concrete layer) is only relevant in very extreme climatic conditions (short period of heavy rainfall, followed by long hot dry periods). In moderate climates drying out of the concrete is not a serious problem, and therefore no further attention is paid to this phenomenon.
The (flexural tensile) stresses (and displacements) in concrete pavement structures due to traffic loadings, temperature gradients and unequal subgrade settlements are discussed in 3.3, 3.4 and 3.5 respectively.
Next in 3.6 in general terms the applicability of the finite element method for concrete pavement structures is discussed.
However, first in 3.2 some attention is paid to the amount of support that the substructure can give to the concrete top layer. This support, usually expressed as the modulus of substructure reaction, influences the flexural tensile stresses and especially the vertical displacements of the concrete top layer due to the above mentioned external loadings (traffic, temperature gradient, settlements).

3.2 Modulus of substructure reaction

One of the input parameters in the design of concrete pavement structures is the bearing capacity of the substructure (base plus sub-base plus subgrade). Generally the complete substructure is modeled as a dense liquid, which means that in the substructure no shear stresses can occur. The bearing capacity of the substructure thus is expressed as the 'modulus of substructure reaction' k, which is defined as (figure 6):

$$k = p/w \qquad (9)$$

where: k = modulus of substructure reaction (N/mm^3)

p = ground pressure (N/mm^2) on top of the substructure
w = vertical displacement (deflection) (mm) at the top of the substructure

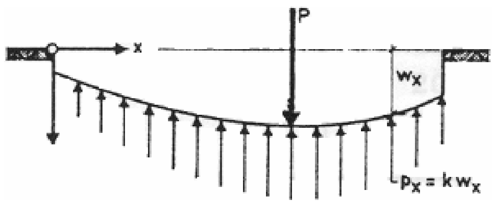


Figure 6. Definition of the ‘modulus of substructure reaction’ k.

In principal the modulus of substructure reaction has to be determined in situ by means of a plate bearing test, where the plate diameter and the magnitude of the load have to be in accordance with the stress conditions, actually present below the concrete top layer. A plate diameter of 760 mm (30 inch) is preferable because of the great load spreading in the stiff concrete top layer. In practice, however, mostly a plate diameter of 300 mm (12 inch) is used, because in this case the required load magnitude is much less. The modulus of substructure reaction for a 760 mm plate diameter then can be estimated by means of the following empirical relationship:

$$k_{760} = 0.4\ k_{300} \qquad (10)$$

For reasons of costs, plate bearing tests are not always done. Then the modulus of substructure reaction has to be determined in an indirect way, with an increasing possibility of inaccuracy.

Table 4 gives an indication of the value of various parameters, including the modulus of subgrade reaction k_o, for Dutch subgrades. These k_o-values are used in the current Dutch design method for concrete pavements (5).

Subgrade	Cone resistance q_c (N/mm^2)	CBR-value (%)	Dynamic modulus of elasticity E_o (N/mm^2)	Modulus of subgrade reaction k_o (N/mm^3)
Peat	0.1 - 0.3	1 - 2	25	0.016
Clay	0.2 - 2.5	3 - 8	40	0.023
Loam	1.0 - 3.0	5 - 10	75	0.036
Sand	3.0 - 25.0	8 - 18	100	0.045
Gravel-sand	10.0 - 30.0	15 - 40	150	0.061

Table 4. Indicative values of various parameters for Dutch subgrades.

When the CBR-value of the subgrade is known, then an indication of the k_o-value can also be obtained by means of figure 7.

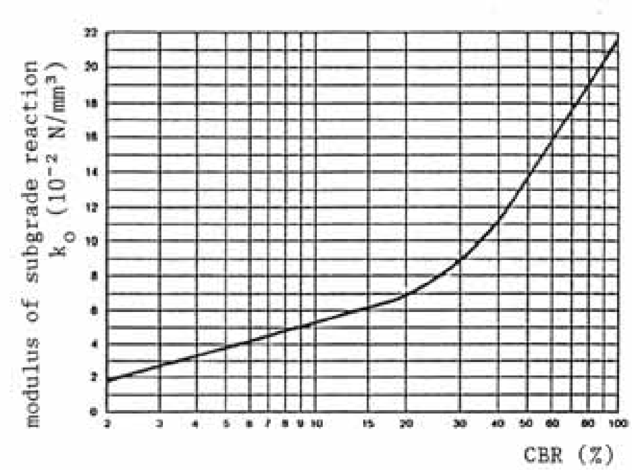


Figure 7. Indicative relationship between k_o and CBR for various types of subgrade.

In principal figure 7 also gives the possibility to estimate the k_o-value from the dynamic modulus of elasticity E_o (that can be derived for instance from Falling Weight Deflection measurements) by means of the following empirical relationship:

$$E_o = 10\ CBR \qquad (11)$$

where: E_o = dynamic modulus of elasticity (N/mm²) of the subgrade
CBR = CBR-value (%) of the subgrade

As already mentioned in chapter 2, generally a sub-base and/or a base are constructed over the subgrade. In the case of a very weak subgrade sometimes a lightweight fill material is applied between the subgrade and the sub-base. The effect of these layers can be estimated by means of figure 8 or equation 12.
The k-value at the top of a layer is found by means of figure 8 or equation 12 from the k-value at the top of the underlying layer and the thickness h_f (mm) and the dynamic modulus of elasticity E_f (N/mm²) of the layer under consideration. This procedure has to be repeated for each layer, so at the end the 'modulus of substructure reaction' k on top of the substructure, i.e. directly beneath the concrete top layer, is found.

Table 5 gives indicative values for the dynamic modulus of elasticity E_f and the minimum thickness of lightweight fill materials (5). If the lightweight fill material has an E_f-value that is lower than the E_o-value of the subgrade, in the calculations the lightweight fill material has to be considered as subgrade. This is especially the case if EPS (Expanded Polystyrene foam) is used as a lightweight fill material.
Table 6 gives indicative values of the dynamic modulus of elasticity E_f of various (sub-)base materials (5). Table 6 includes the lightweight foam concretes as they have an E_f-value that is substantially greater than the E_f-value of the lightweight materials mentioned in table 5 and the E_o-value of the subgrades mentioned in table 4.

Lightweight fill material	Density (kg/m^3)	Dynamic modulus of elasticity E_f (N/mm^2)	Minimum thickness (mm)
Argex (expanded clay particles) (fractions 4/10, 8/16) Flugsand (volcanic) Lava (volcanic)	-	100	250
EPS (Expanded Polystyrene foam): EPS60 EPS100 EPS150 EPS200 EPS250	15 20 25 30 35	10	250

Table 5. Indicative values of the dynamic modulus of elasticity E_f and the minimum thickness of lightweight fill materials.

(Sub-)base material	Density (kg/m^3)	Dynamic modulus of elasticity E_f (N/mm^2)
Unbound (sub-)base materials		
Sand	-	100
Masonry granulate	-	150 3)
LD steel slags	-	200
Natural stone aggregate	-	250
LD mix (steel slags plus blast furnace slags)	-	350
Hydraulic mix granulate	-	600
Mix granulate	-	400
Concrete granulate	-	800
Mixture of phosphor slags and blast furnace slags	-	400
Bound base materials		
AGRAC (asphalt granulate with cement)	-	3000 1)
AGREC (asphalt granulate with bitumen emulsion and cement)	-	1500 1)
AGREM (asphalt granulate with bitumen emulsion)	-	1200 1)
Mixture of phosphor slags and blast furnace slags	-	1000
Foam concrete	400	300 1)
	500	650 1)
	600	1200 1)
	700	1650 1)
	900	2200 1)
	1000	3700 1)
	1200	5800 1)
	1400	8400 1)
	1600	11500 1)
Sandcement	-	6000
Lean concrete	-	10000
Asphalt concrete	-	7500 2)

1) values for non-cracked material; for cracked or carved material take 75% of the value
2) value for asphalt temperature 20ºC and loading frequency 8 Hz
3) susceptible for erosion

Table 6. Indicative values of the dynamic modulus of elasticity E_f of (sub-)base materials.

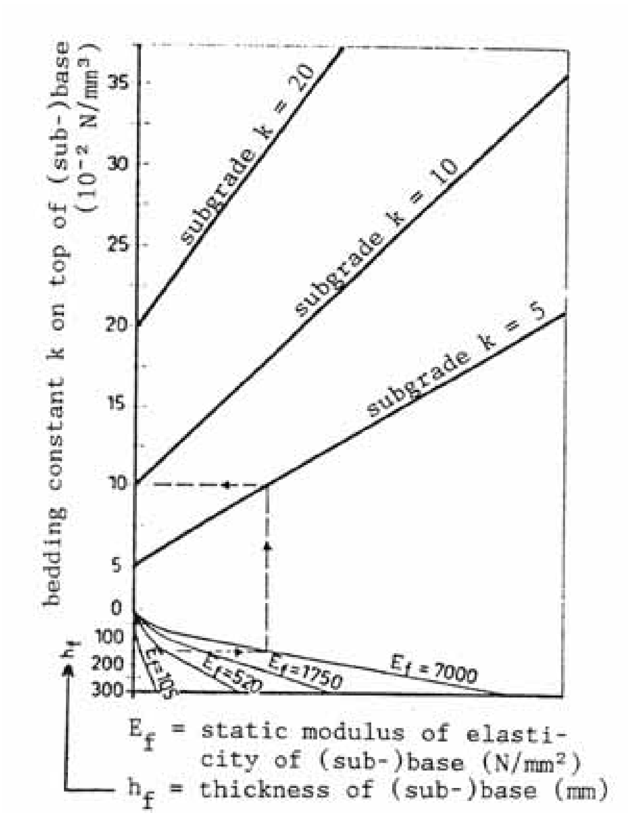


Figure 8. Nomograph for the determination of the k-value on top of a (sub-)base layer (1).

Figure 8 is the graphical representation of equation 12:

$$k= 2.7145.10^{-4} (C_1 + C_2.e^{C3} + C4.e^{C5}) \quad (12)$$

with: $C_1 = 30 + 3360.k_o$

$C_2 = 0.3778 (h_f – 43.2)$

$C_3 = 0.5654 \ln(k_o) + 0.4139 \ln(E_f)$

$C_4 = -283$

$C_5 = 0.5654 \ln(k_o)$

k_o = modulus of subgrade/substructure reaction at top of underlying layer (N/mm^3)

h_f = thickness of layer under consideration (mm)

E_f = dynamic modulus of elasticity of layer under consideration (N/mm^2)

k = modulus of substructure reaction at top of layer under consideration (N/mm^3)

The boundary conditions for equation 12 are:

- $h_f \geq 150$ mm (bound material) and $h_f \geq 200$ mm (unbound material)
- every layer under consideration has an E_f-value that is greater than the E_f-value of the underlying layer
- $\log k \leq 0.73688 \log(E_f) – 2.82055$
- $k \leq 0.16$ N/mm^3

Example

Sand subgrade: $k_o = 0.045$ N/mm^3 (i.e. $E_o = 100$ N/mm^2).
Sand sub-base, $h_f = 500$ mm, $E_f = 100$ N/mm²: at top of sub-base also k = 0.045 N/mm^3 (as $E_f = E_o$).
Lean concrete base, $h_f = 150$ mm, $E_f = 7500$ N/mm² (cracked): at top of base k = 0.112 N/mm^3.
In the structural design of concrete pavements for motorways the Dutch State Highway Authorities use a k-value of 0.105 N/mm^3 (9). This k-value is obtained for a 150 mm thick lean concrete base, with $E_f = 6000$ N/mm^2, on a sand sub-base and sand subgrade.

3.3 Stresses due to temperature variations

3.3.1 General

Temperature variations lead to stresses in the concrete top layer. These stresses can be distinguished into (figure 9):

1. stresses due to a temperature change ΔT which is constant over the thickness of the concrete layer
2. stresses due to a temperature gradient Δt which is constant over the thickness of the concrete layer
3. stresses due to an irregular temperature over the thickness of the concrete layer.

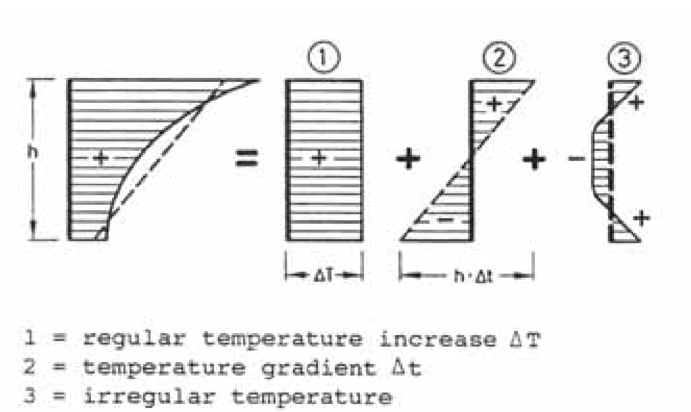


Figure 9. Temperature in the concrete top layer in case of heating at the surface.

A regular temperature increase or decrease ΔT leads to compressive and tensile stresses respectively in the concrete top layer due to friction over the underlying layer. However, for plain concrete pavements (that generally consist of slabs with both a length and a width smaller than 5 m (roads) or 7.5 m (airports), and for reinforced concrete pavements (with a 'slab' length equal to the crack distance of 1.5 to 3 m and a 'slab' width of maximum 5 m) these stresses are such small that they can be neglected.

The irregular temperature results in internal concrete stresses, which are only relevant for very thick concrete slabs. For normal concrete slab thicknesses they also can be neglected.

On the contrary, the temperature gradients Δt cause flexural stresses in the concrete pavement that are in the same order of magnitude as those caused by the traffic loadings, and thus cannot be neglected at all. Paragraph 3.3.2 deals with the magnitude and frequency of occurrence of temperature gradients. In 3.3.3 and 3.3.4 the calculation of the flexural stresses due to a temperature gradient will be discussed.

3.3.2 Temperature gradients

The temperature gradient Δt is defined as (figure 9):

$$\Delta t = \frac{T_t - T_b}{h} \qquad (13)$$

where: T_t = temperature (°C) at the top of the concrete layer
T_b = temperature (°C) at the bottom of the concrete layer
h = thickness (mm) of the concrete layer

In case of a negative temperature gradient the concrete slab curls. Due to the deadweight of the concrete slab there are flexural tensile stresses at the top of the slab. Generally a negative temperature gradient (that occurs during night) is not taken into consideration in the design of a concrete pavement, because:

- the negative gradient is smaller than the positive gradient (figure 10 and table 7)
- during night there is only a small amount of heavy traffic
- in the mostly dominating location of the concrete slab (in case of plain concrete pavements the centre of the longitudinal edge and in case of reinforced pavements along the transverse crack in the wheel track or in the centre of the slab width) the traffic loadings cause flexural compressive stresses at the top of the slab.

A positive temperature gradient (that occurs during day) causes warping of the concrete slab. Due to the deadweight of the concrete slab, in this case there are flexural tensile stresses at the bottom of the slab. These stresses are called 'warping stresses'.

Table 8 shows the standard temperature gradient frequency distribution that is included in the current Dutch method for the structural design of concrete pavements (5).

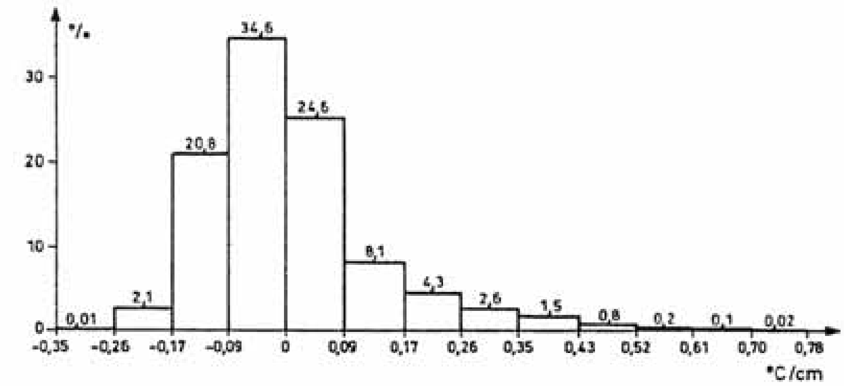


Figure 10. Mean distribution (%) of the mean temperature gradients, measured at a 230 mm thick concrete slab in Belgium from 1973 to 1977.

Temperature gradient class (°C/mm)	Hottest month analysis			1 year analysis
	h = 200	h = 250	h = 300	h =300
-0.04 – 0.00	56	54	54	59
0.00 – 0.04	17	23	26	31
0.04 – 0.08	21	23	20	10
0.08 – 0.12	6	-	-	-

Table 7. Calculated temperature gradient frequency distributions (%) for a concrete slab (thickness h mm) in New Delhi, India.

Temperature gradient class (ºC/mm)	Average temperature gradient Δt (ºC/mm)	Frequency distribution (%)
0.000 – 0.005	0.0025	59
0.005 – 0.015	0.01	22
0.015 – 0.025	0.02	7.5
0.025 – 0.035	0.03	5.5
0.035 – 0.045	0.04	4.5
0.045 – 0.055	0.05	1.0
0.055 – 0.065	0.06	0.5

Table 8. Standard temperature gradient frequency distribution in Dutch design method for concrete pavements.

3.3.3 Temperature gradient stresses (Eisenmann theory)

Eisenmann has developed a theory for the calculation of warping stresses in concrete slabs (10). He has introduced the term 'critical slab length', which is defined as that slab length where a concrete slab, equally heated at the surface, only touches the substructure at the four corners and in the centre of the slab. The critical slab length can be calculated by means of the equation:

long slab: (L/W > 1.2 or L/W < 0.8): $l_{crit} = 200\ h\ \sqrt{E\ \alpha\ \Delta t}$ (14a)

square slab: (0.8 ≤ L/W ≤ 1.2): $l_{crit} = 228\ h\ \sqrt{E\ \alpha\ \Delta t}$ (14b)

where: l_{crit} = critical slab length (mm)
h = thickness (mm) of the concrete slab
E = Young's modulus of elasticity (N/mm²) of concrete
α = coefficient of linear thermal expansion (°C^{-1})
Δt = positive temperature gradient (°C/mm)
L = slab length (mm)
W = slab width (mm)

Furthermore Eisenmann takes into account that near the edges the concrete slab is supported over a certain distance, the 'support length' C. This means that the span L' of the concrete slab is always less than the slab length L:

$$L' = L - \frac{2}{3} C \tag{15}$$

The support length C is approximately:

$$C = 4.5 \sqrt{\frac{h}{k\ \Delta t}} \qquad \text{if } C << L \tag{16}$$

where: C = support length (mm)
h = thickness (mm) of the concrete slab
k = modulus of substructure reaction (N/mm^3)
Δt = positive temperature gradient (°C/mm)

Depending on the ratio between the slab span L' and the critical slab length l_{crit} Eisenmann distinguishes three cases (figure 11):

1. When the slab span L' is far greater than the critical slab length l_{crit}, then the central part of the concrete slab is resting on the substructure. In this central part the (normal) warping stress σ_t (the flexural tensile stress at the bottom in the centre of the slab in the longitudinal direction) is:

$$\sigma_t = \frac{1}{1-\upsilon} \frac{h\,\Delta t}{2} \alpha\ E \qquad \text{if } L' > 1.1\ l_{crit} \tag{17}$$

where: υ = Poisson's ratio of concrete
h = thickness (mm) of the concrete slab
Δt = positive temperature gradient (°C/mm)
α = coefficient of linear thermal expansion (°C^{-1})
E = Young's modulus of elasticity (N/mm^2) of concrete

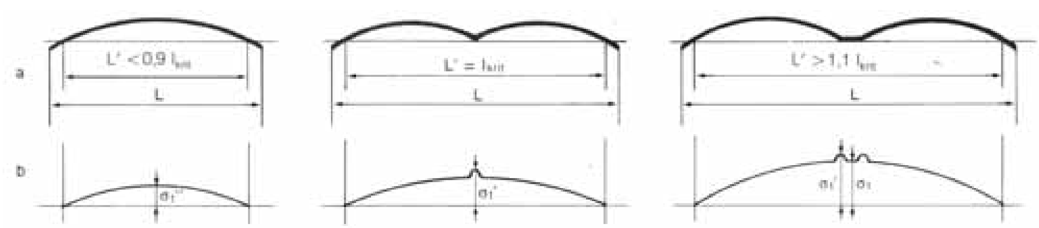
2. Directly besides the central part of the concrete slab, which is resting on the substructure, there is an increased warping of the slab, resulting in (disturbed) warping stresses $\sigma_t^{'}$ that are some 20% greater than the normal warping stress σ_t:

$$\sigma_t^{'} = 1.2\ \sigma_t \tag{18}$$

The disturbed warping stress $\sigma_t^{'}$ also occurs in the centre of a concrete slab where L' = l_{crit}.

3. When the slab span L' is far smaller than the critical slab length l_{crit} then the (reduced) warping stress $\sigma_t^{''}$ can be calculated by means of the equation:

$$\sigma_t^{''} = \left(\frac{L^{'}}{0.9\ l_{crit}}\right)^2 \sigma_t \quad \text{if } L^{'} < 0.9\ l_{crit} \tag{19}$$



L = slab length a: vertical deformations
L' = slab span b: flexural tensile stresses

Figure 11. Vertical deformations and warping stresses in the longitudinal section through the slab centre as a function of the slab span for positive temperature gradients.

The equations 17 to 19 apply to the warping stresses in the longitudinal direction in or nearby the centre of the concrete slab.
For calculation of the warping stresses in the transverse direction in or nearby the centre of the concrete slab, in the equations 17 to 19 the slab length L has to be replaced by the slab width W and the slab span L' in the longitudinal direction has to be replaced by the slab span W' in the transverse direction.

Due to the uni-axial stress condition, Eisenmann takes the temperature stress in the centre of a slab edge as 85% of the warping stress (in the slab centre) in the same direction.

At the slab edge outside the centre of it, the temperature stress calculated for the slab edge can be reduced by means of the factor R that is equal to:

longitudinal edge: $$R = \frac{4\,x\,(L^{'} - x)}{L^{'2}} \tag{20a}$$

transverse edge: $R = \dfrac{4\,y\,(W^{'} - y)}{W^{'2}}$ (20b)

where: L' = slab span (mm) in the longitudinal direction
W' = slab span (mm) in the transverse direction
X = distance (mm) of the point under consideration at the longitudinal edge to the nearest transverse edge minus 1/3 of the support length C
y = distance (mm) of the point under consideration at the transverse edge to the nearest longitudinal edge minus 1/3 of the support length C

The equations 17 to 19 make clear why nowadays relatively small concrete slabs are applied in plain concrete pavements, with horizontal dimensions smaller than about 5 m (roads) or 7.5 m (airports) respectively. For these slab dimensions the span of the slabs both in the longitudinal (L') and the transverse (W') direction is much smaller than the critical slab length l_{crit}, resulting in reduced warping stresses $\sigma_t^{"}$ (equation 19).

3.3.4 Temperature gradient stresses (Dutch method)

In the Netherlands originally VNC (Cement Industry Association) has developed an analytical design method for plain concrete pavements (11,12). Recently the method has been upgraded, and extended with continuously reinforced concrete pavements, by CROW (5).
One has realized that the most critical point of the pavement structure is somewhere at an edge of the plain concrete slab or at a crack of the reinforced concrete slab. At the edges or cracks there is by definition a uniaxial stress situation in the concrete slab (only stresses parallel to the edge or crack and no stress perpendicular to the edge or crack). For the calculation of the temperature gradient stresses this means that only a concrete beam (with unit width) along the edge or crack needs to be taken into account and not an entire plain concrete slab or a whole part of the reinforced concrete pavement between two transverse cracks.

In the case of a small positive temperature gradient Δt the warping (upward displacement) of the concrete slab along the edge or crack is smaller than the compression (downward displacement) of the substructure (characterized by the modulus of substructure reaction k) due to the deadweight of the concrete slab. This implies that the concrete slab remains fully supported (figure 12 left). The flexural tensile stress σ_t at the bottom of the concrete slab in the center of a slab edge or crack can then be calculated by means of the equation:

$$\sigma_t = \frac{h \cdot \Delta t}{2}\, \alpha\, E \qquad (21)$$

where: h = thickness (mm) of the concrete slab
Δt = small positive temperature gradient (°C/mm)
α = coefficient of linear thermal expansion (°C^{-1})
E = Young's modulus of elasticity (N/mm²) of concrete

At positive temperature gradients greater than a certain limit value Δt_l the concrete slab at the edge or crack looses contact with the substructure and is only supported at its ends over the support length C, see equation 16 (figure 12 right). In this case the concrete beam at the edge or crack is loaded by its deadweight and the flexural tensile stress σ_t at the bottom of the concrete slab in the center of a slab edge or crack can be calculated by means of the equations (in which a volume weight of 24 kN/m³ has been assumed for the concrete):

longitudinal edge: $\sigma_t = 1.8 * 10^{-5} \quad L'^2 / h$ (22a)

transverse edge: $\sigma_t = 1.8 * 10^{-5} \quad W'^2 / h$ (22b)

where: L' = slab span (mm) in longitudinal direction (equation 15)
W' = slab span (mm) in transverse direction
h = thickness (mm) of the concrete slab

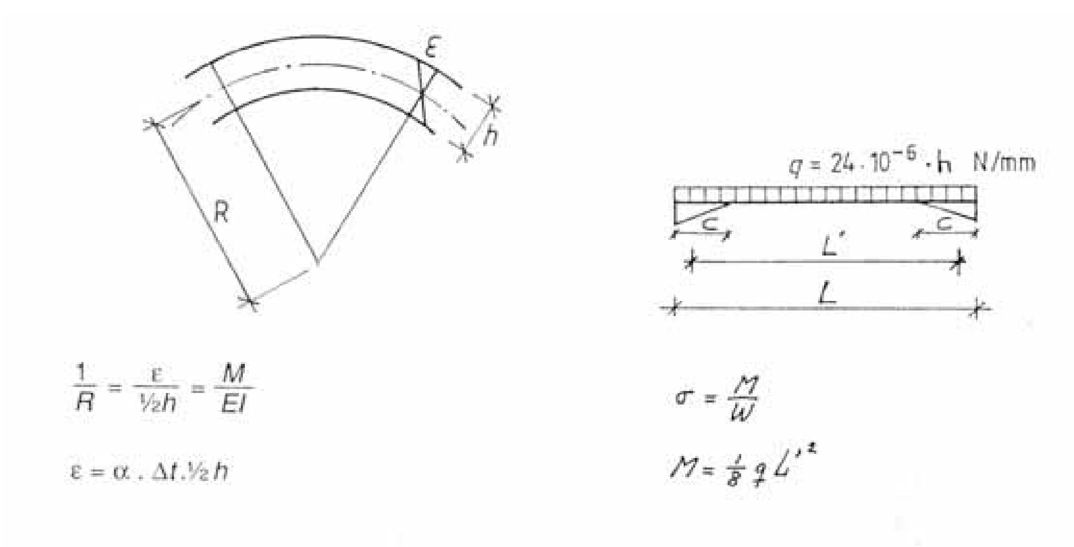


Figure 12. Basic equations of the Dutch method for the calculation of the flexural tensile stress at the bottom of the concrete slab in the centre of the edge (plain concrete pavement) or the crack (reinforced concrete pavement) due to a positive temperature gradient Δt that is smaller (left) or greater (right) than the limit temperature gradient Δt_l.

The value of the limit temperature gradient Δt_l follows for the center of a longitudinal edge (plain concrete pavement) from equalizing the equations 21 and 22a and for the center of a transverse edge or transverse crack

(reinforced concrete pavement) from equalizing the equations 21 and 22b. The determination of Δt_l is an iterative calculation.
It is however not necessary to calculate the limit temperature gradient Δt_l. The actual temperature gradient stress σ_t in the centre of the longitudinal edge is the smallest value resulting from the equations 21 and 22a, and similarly the actual temperature gradient stress σ_t in the centre of the transverse edge or crack is the smallest value resulting from the equations 21 and 22b.
To obtain the temperature gradient stresses along an edge or crack of the concrete slab equation 20 can be used.

The temperature gradient stresses calculated by means of this Dutch method are in good agreement with finite element calculation results (12,13,14).

Example

In this calculation example the temperature gradient stresses according to the Dutch method are calculated in a plain concrete road pavement at 2 possibly critical positions of the concrete slab, i.e. the center of the longitudinal edge and the transverse edge in the wheeltrack.
The starting points for the calculation are:
- modulus of substructure reaction k = 0.105 N/mm^3
- concrete slabs with dimensions L = 4.5 m, W = 3.75 m and h = 210 mm
- concrete quality C28/35 (B35):
  - Young's modulus of elasticity E = 31000 N/mm^2
  - Poisson's ratio υ = 0.15
  - coefficient of linear thermal expansion $\alpha = 10^{-5}\ ^\circ C^{-1}$
- positive temperature gradient Δt = 0.06, 0.05, 0.04, 0.03, 0.02, 0.01 and 0.0025 °C/mm
- the truck width is equal to 2.25 m, which means that a wheel track is at a distance of 0.75 m from the longitudinal edges of the concrete slabs.

The calculated temperature gradient stresses are shown in table 9.

Positive temperature gradient Δt (°C/mm)	Support length C (mm) eq. 16	longitudinal edge				transverse edge					
		centre				centre				wheel track	
		slab span L' (mm) eq. 15	limit temperature gradient Δt_l (°C/mm) eq. 21 + 22a	temperature gradient stress σ_t (N/mm^2)		slab span W' (mm) eq. 15	limit temperature gradient Δt_l (°C/mm) eq. 21 + 22b	temperature gradient stress σ_t (N/mm^2)		reduction factor R eq. 20b	temperature gradient stress σ_t (N/mm^2)
				eq. 22a	eq. 21			eq. 22b	eq. 21		
0.06	822	3952		1.34		3202		0.88		0.51	0.45
0.05	900	3900		1.30		3150		0.85		0.49	0.42
0.04	1006	3829		1.26		3079		0.81		0.47	0.38
0.03	1162	3725	0.038		0.98	2975	0.023	0.76		0.43	0.32
0.02	1423	3551			0.65	2801			0.65	0.35	0.23
0.01	2012	3158			0.33	2408			0.33	0.13	0.04
0.0025	4025	1817			0.08	1067			0.08	0	0

Table 9. Example of the calculation of temperature gradient stresses (Dutch method) in a plain concrete road pavement.

3.4 Stresses and displacements due to traffic loadings

3.4.1 Introduction

In 1926 Westergaard published his original equations for the calculation of the maximum flexural tensile stress and the maximum vertical displacement due to a single wheel load in the interior, at the edge or at the corner of a single concrete slab. In latter years Westergaard himself as well as other researchers published several 'modified' Westergaard-equations. Until today this Westergaard-theory is widely used all over the world to calculate the stresses and displacements in concrete pavement structures due to traffic loadings. The Westergaard-theory is described in 3.4.2.
In reality a concrete pavement structure does not exist of a single concrete slab, but of a number of concrete slabs with longitudinal and transverse joints (plain concrete pavements) or longitudinal joints and transverse cracks (reinforced concrete pavements). The load transfer in these joints and cracks, and the consequences for the stresses and displacements of the concrete pavement structure, are discussed in 3.4.3.

3.4.2 Single concrete slab (Westergaard-theory)

Westergaard developed a theory for the maximum stress (flexural tensile stress) and the maximum vertical displacement (deflection) due to a single wheel load, located in the interior (middle), along the edge or in a corner of a single (concrete) slab on an elastic foundation (springs with a stiffness equal to the modulus of substructure reaction k).
The fully supported slab is assumed to be such large, that the edges and corners don't have any significant influence on the maximum stress and deflection.
In the cases that the single wheel load is in the interior or along the edge or crack of the concrete slab, both the flexural tensile stress and deflection are maximum at the bottom of the concrete slab in the load centre. In case of a single wheel load in the corner of the slab, the deflection is maximum exactly in the corner while the flexural tensile stress is maximum at some distance of the corner at the top of the concrete slab (figure 13).

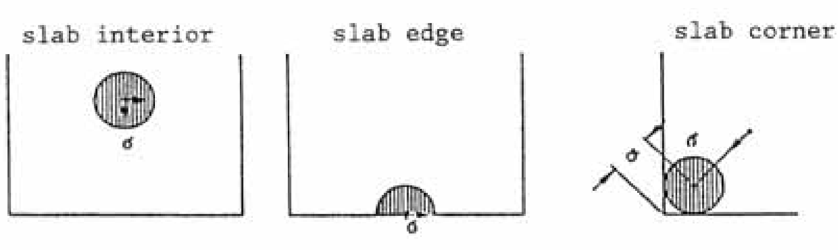


Figure 13. Loading positions in Westergaard's theory.

There exist a lot of (modified) Westergaard-equations to calculate the maximum flexural tensile stress σ and the maximum deflection w for interior, edge and corner loading. The most important equations are (15,16,17):

interior loading

ordinary theory, circular loading area (15,17)

$$\sigma = \frac{3\,P\,(1+\upsilon)}{2\pi\,h^2}\quad \left\{ln\left(\frac{2\,l}{a}\right) + 0.5 - \gamma\right\} \tag{23}$$

$$w = \frac{P}{8\,k\,l^2}\left[1 + \frac{1}{2\,\pi}\left\{l\,n\left(\frac{a}{2\,l}\right) + \gamma - 1.25\right\}\left(\frac{a}{l}\right)^2\right] \tag{24}$$

edge loading

ordinary theory, semi-circular loading area (15,17)

$$\sigma = \frac{0.529\,P}{h^2}\quad (1 + 0.54\,\upsilon)\quad \left\{\log\left(\frac{E\,h^3}{k\,a_2^4}\right) - 0.71\right\} \tag{25}$$

$$w = \frac{P}{\sqrt{6}\,k\,l^2}\quad (1 + 0.4\,\upsilon) \tag{26}$$

new theory, circular loading area (16,17)

$$\sigma = \frac{3\,(1+\upsilon)\,P}{\pi\,(3+\upsilon)\,h^2}\quad \left\{l\,n\left(\frac{E\,h^3}{100\,k\,a^4}\right) + 1.84 - \frac{4}{3}\,\upsilon + \frac{1-\upsilon}{2} + 1.18\,(1 + 2\,\upsilon)\,\frac{a}{l}\right\} \tag{27}$$

$$w = \frac{\sqrt{2 + 1.2\,\upsilon}\quad P}{\sqrt{E\,h^3\,k}}\left\{1 - (0.76 + 0.4\,\upsilon)\,\frac{a}{l}\right\} \tag{28}$$

new theory, semi-circular loading area (16,17)

$$\sigma = \frac{3\,(1+\upsilon)\,P}{\pi\,(3+\upsilon)\,h^2}\quad \left\{l\,n\left(\frac{E\,h^3}{100\,k\,a_2^4}\right) + 3.84 - \frac{4}{3}\,\upsilon + 0.5\,(1 + 2\,\upsilon)\,\frac{a_2}{l}\right\} \tag{29}$$

$$w = \frac{\sqrt{2 + 1.2\,\upsilon}\quad P}{\sqrt{E\,h^3\,k}}\left\{1 - (0.323 + 0.17\,\upsilon)\,\frac{a_2}{l}\right\} \tag{30}$$

Extensive finite element calculations have strongly indicated that the ordinary Westergaard-equations 25 and 26 for edge loading are not correct. On the contrary, the new Westergaard-equations for edge loading, equations 27 to 30, are in good agreement with the finite element method (see 3.6). The differences between a circular loading area and a semi-circular loading area are marginal.

Corner loading

Circular loading area (15,17)

$$\sigma = \frac{3P}{h^2}\left\{1-\left(\frac{a_1}{l}\right)^{0.6}\right\} \tag{31}$$

$$w = \frac{P}{k\,l^2}\left\{1.1-0.88\left(\frac{a_1}{l}\right)\right\} \tag{32}$$

distance from corner to point of maximum stress:

$$x_1 = 2\sqrt{a_1\,l} \tag{33}$$

In the equations 23 to 33 is:

σ = flexural tensile stress (N/mm²)
w = deflection (mm)
P = single wheel load (N)
p = contact pressure (N/mm²)
a = $\sqrt{\frac{P}{\pi\,p}}$ = radius (mm) of circular loading area
a_2 = $\sqrt{\frac{2\,P}{\pi\,p}}$ = radius (mm) of semi-circular loading area
E = Young's modulus of elasticity (N/mm²) of concrete
υ = Poisson's ratio of concrete
h = thickness (mm) of concrete layer
k = modulus of substructure reaction (N/mm^3)
l = $\sqrt[4]{\frac{E\,h^3}{12(1-\upsilon^2)k}}$ = radius (mm) of relative stiffness ofconcrete layer
γ = Euler's constant (= 0.5772156649)
a_1 = a $\sqrt{2}$ = distance (mm) from corner to centre of corner loading
x_1 = distance (mm) from corner to point of maximum flexural tensile stress due to corner loading

Example

Edge loading (equations 27 and 28)

P = 50 kN = 50000 N
p = 0.7 N/mm²
a = $\sqrt{50000/\pi \cdot 0.7} = 150$ mm
k = 0.105 N/mm^3
υ = 0.15 (concrete quality C28/35 (B35))
E = 31000 N/mm² (concrete quality C28/35 (B35))
h = 180 mm: l = 619 mm, σ = 3.21 N/mm², w = 0.429 mm
210 mm: l = 695 mm, σ = 2.52 N/mm², w = 0.350 mm
240 mm: l = 768 mm, σ = 2.04 N/mm², w = 0.292 mm

Pickett and Ray have transformed the Westergaard-equations into influence charts (18). These charts also allow the determination of the flexural tensile stress and the deflection due to complex load systems, such as dual wheel tyres, tandem and triple axles, and airplane gears.
In the figures 14 and 15 the influence charts of Pickett and Ray for the bending moment in the slab interior and along the slab edge respectively are shown. In these influence charts the wheel load contact area has to be drawn on scale; to this end the radius of relative stiffness (l) of the concrete top layer is drawn as a reference. The flexural tensile stress σ at the bottom of the concrete layer due to a wheel load P is found from an influence chart for the bending moment M by means of the equation:

$$\sigma = \frac{M}{\frac{1}{6}h^2} = \frac{6\ p\ l^2\ N}{10000\ h^2} \quad \left(N/mm^2\right) \tag{34}$$

where: p = contact pressure (N/mm^2) of wheel load P
l = radius (mm) of relative stiffness of concrete layer
h = thickness (mm) of concrete layer
N = number of blocks at the chart, covered by the contact area of wheel load P

In the figures 16 and 17 the influence charts of Pickett and Ray for the deflection in the slab interior and along the slab edge respectively are shown. Also in these charts the wheel load contact area has to be drawn on scale. The deflection w of the concrete layer due to a wheel load P is found from an influence chart for the deflection by means of the equation:

$$w = \frac{0.0005\ p\ l^4\ N}{D} = \frac{0.0005\ p\ N}{k} \quad (mm) \tag{35}$$

where: p = contact pressure (N/mm^2) of wheel load P
l = radius (mm) of relative stiffness of concrete layer
k = modulus of substructure reaction (N/mm^3)
D = $\frac{E\,h^3}{12\left(1-\upsilon^2\right)}$ = bending stiffness (N/mm) of concrete layer
N = number of blocks at the chart, covered by the contact area of wheel load P

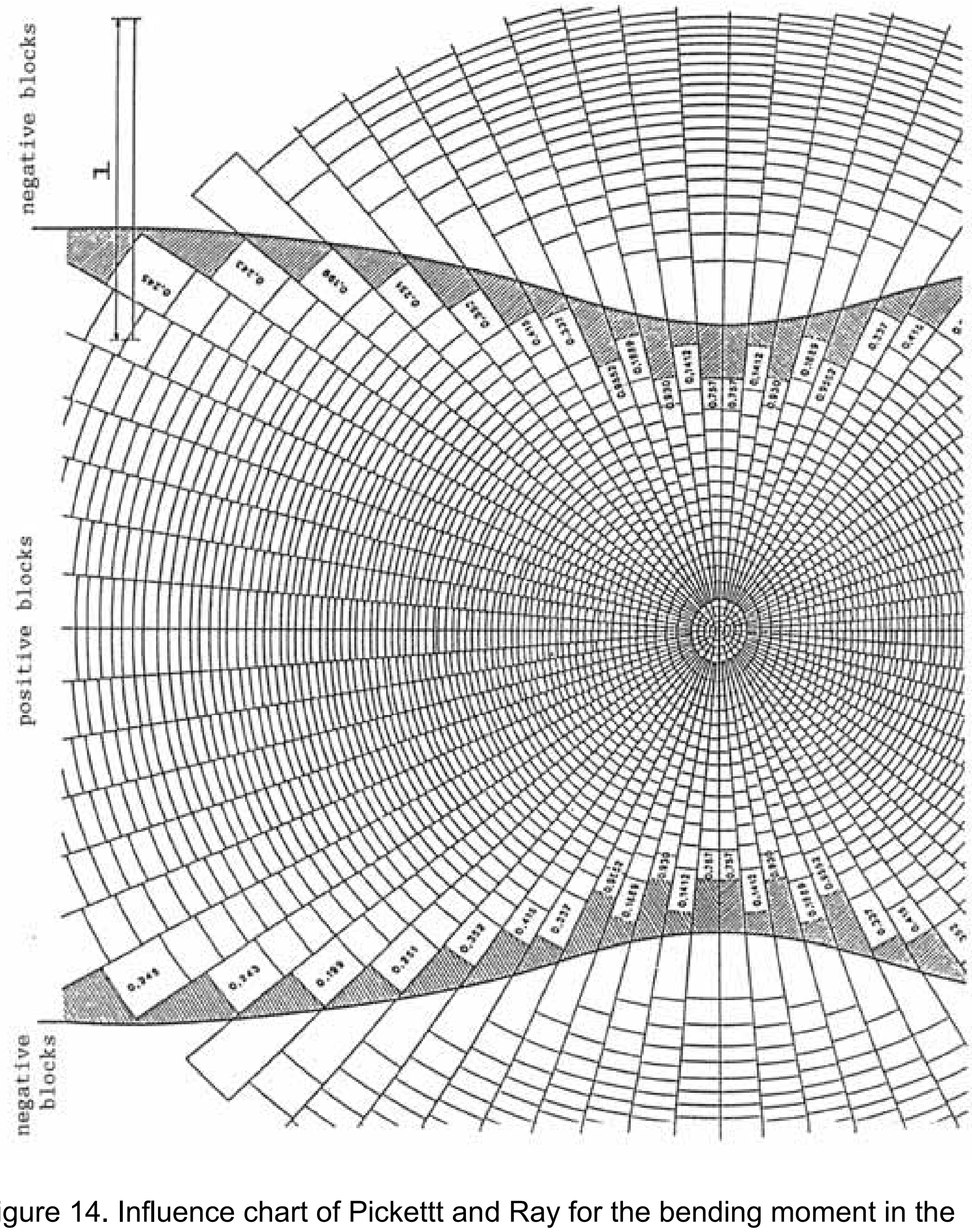


Figure 14. Influence chart of Pickettt and Ray for the bending moment in the slab interior.

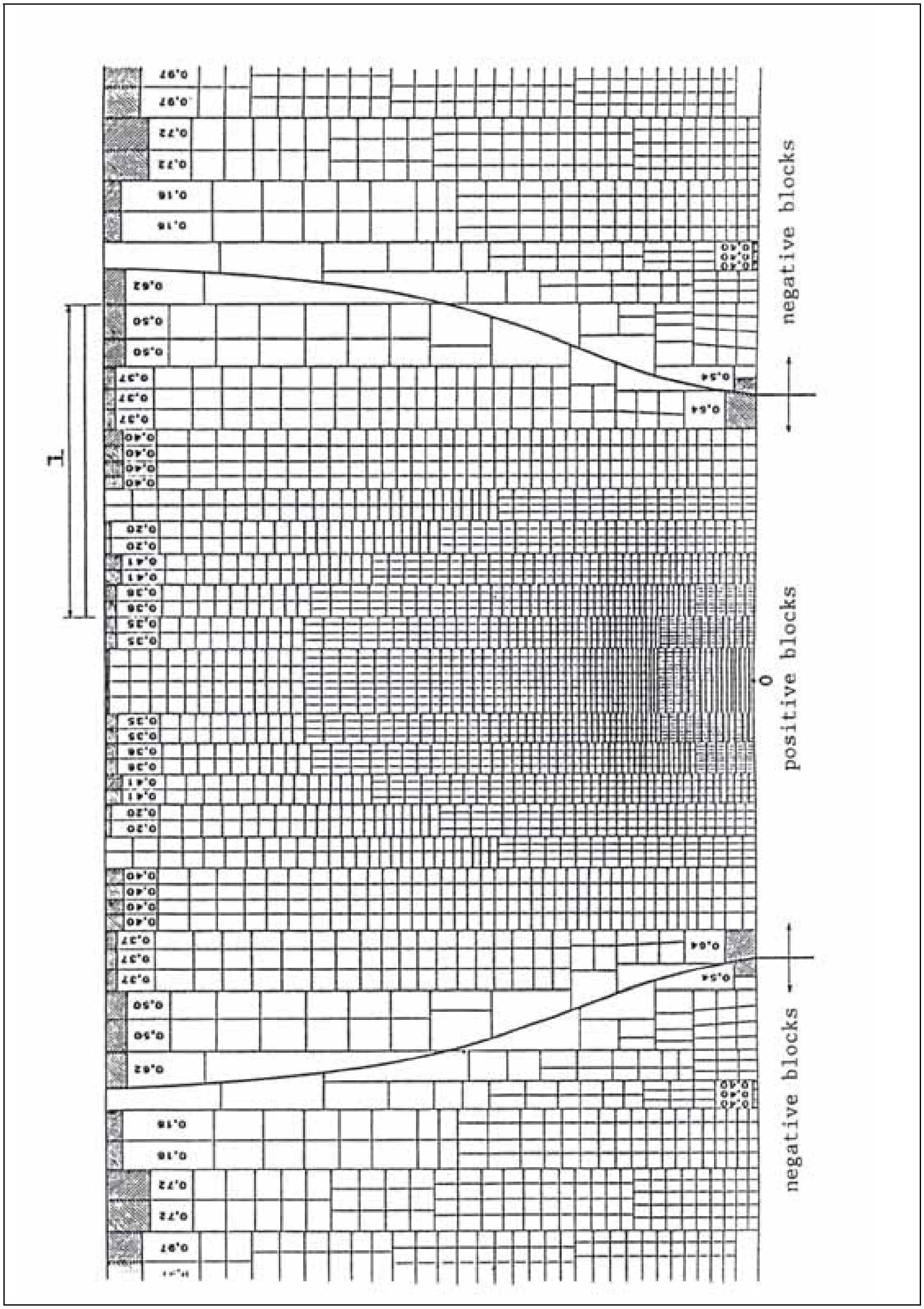


Figure 15. Influence chart of Pickett and Ray for the bending moment along the slab edge.

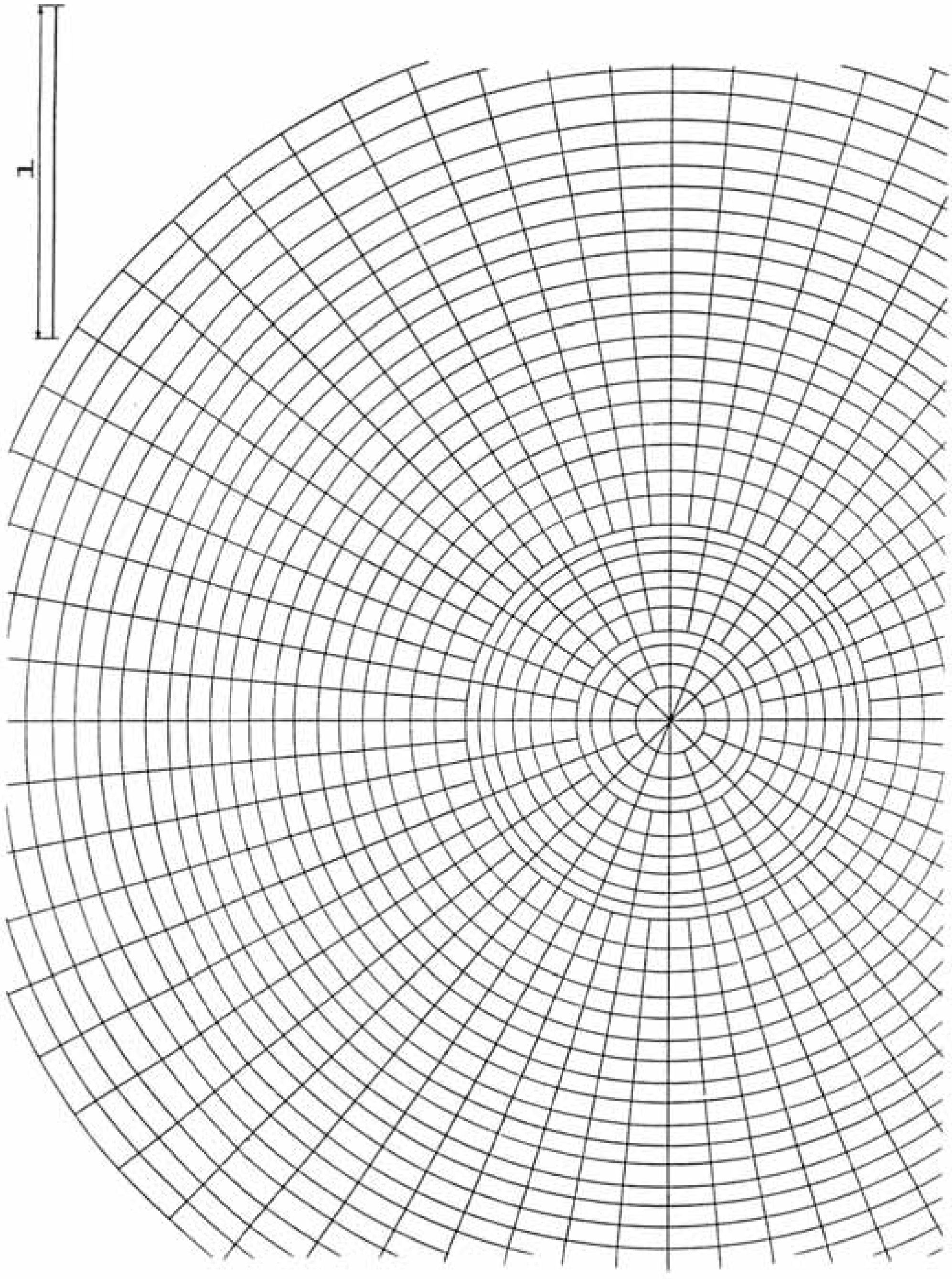


Figure 16. Influence chart of Pickett and Ray for the deflection in the slab interior.

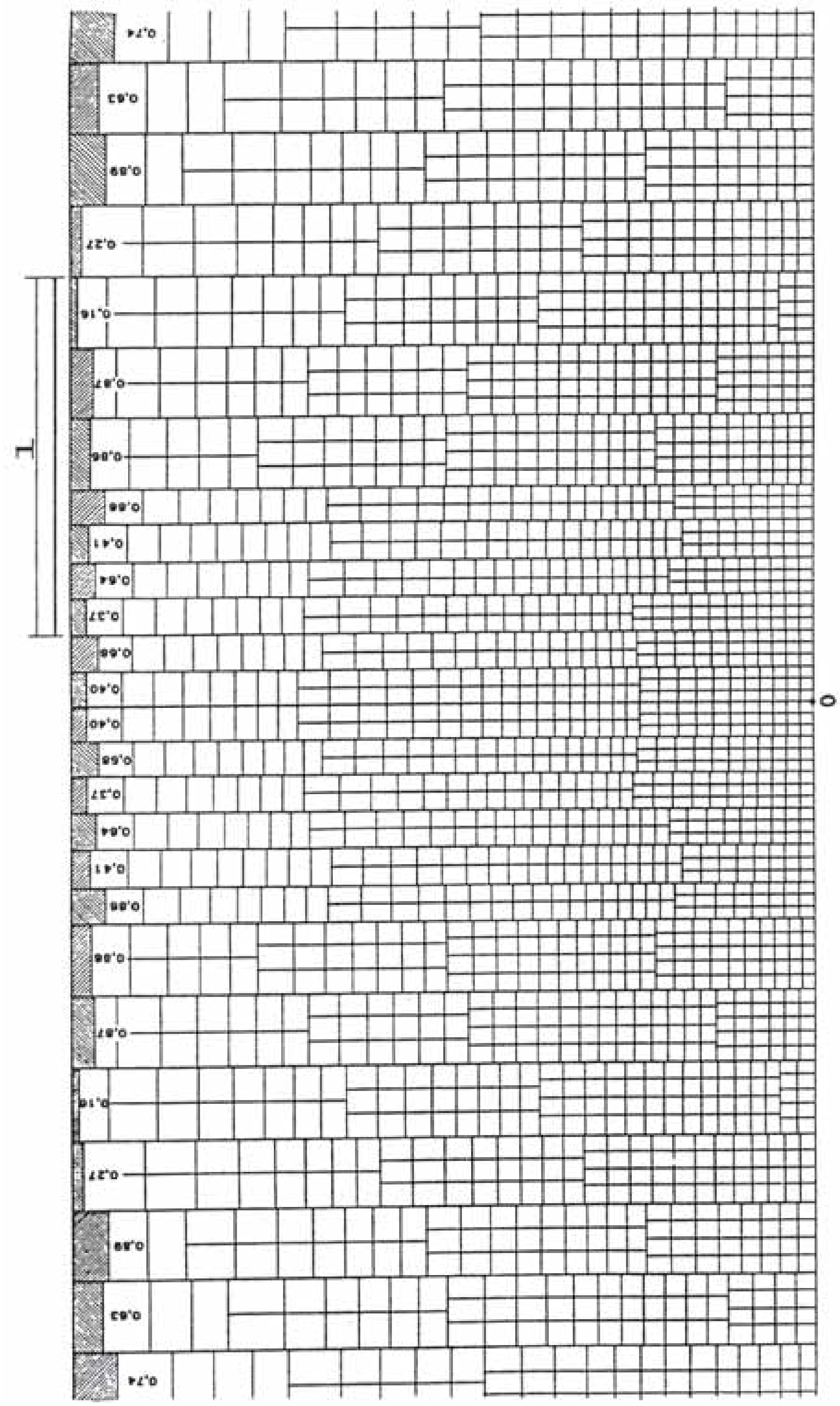


Figure 17. Influence chart of Pickett and Ray for the deflection along the slab edge.

3.4.3 Load transfer in joints/cracks

In 3.4.2 equations and charts have been given for the calculation of flexural tensile stresses and deflections due to traffic loadings in a single concrete slab. However, in reality a concrete pavement consists of a number of concrete slabs with joints (plain concrete pavements) or cracks (reinforced concrete pavements) between them. The load transfer in these joints and cracks is dependent on the joint or crack width (which depends on the slab length), the amount of traffic and the type of joint or crack construction, which means: aggregate interlock, reinforcement, ty bars and dowel bars.
First the last mentioned four influence factors will be discussed separately and then the total load transfer in the joint or crack (i.e. the joint or crack efficiency).

Aggregate interlock is the phenomenon that both rough sides of a joint or crack stick into each other. The load transfer due to aggregate interlock is dependent on:

a. the joint or crack width: the greater this width, the lower the load transfer
b. the effective thickness of the concrete layer (= total thickness minus the depth of the saw cut for a contraction joint): the greater this thickness, the higher the load transfer
c. the rate of support of the concrete layer by the substructure: the higher the modulus of substructure reaction k, the smaller the deflections w of the slab edges due to traffic loadings P (see equations 47 and 49), the lower the shear forces at the joint or crack sides, the lower the polishing of the concrete, and thus the higher the load transfer
d. the magnitude of the traffic loadings: the deflections of the slab edges are proportional to the magnitude P of the traffic loadings (see equations 28 and 30), and therefore the smaller the traffic loadings, the higher the load transfer (similar to c)
e. the aggregate shape in the concrete mixture: the more rough (the higher the angle of internal friction) the aggregate, the higher the load transfer.

Both the amount and the diameter of ty bars in longitudinal joints of plain concrete pavements are such small, that they will not have any significant direct influence on the load transfer. However, indirectly they have a considerable effect because due to the ty bars the joint width is very limited and therefore the load transfer by means of aggregate interlock will be maintained.
For the reinforcement in reinforced concrete pavements the same applies as for ty bars.

The amount and diameter of dowel bars in contraction joints of plain concrete pavements are such that they yield a considerable load transfer. Load transfer by means of dowel bars mainly occurs by shear forces rather than by bending forces. The stiffness S_d of one single dowel bar is defined as (18):

$$S_d = \frac{2 \quad E_d \quad I_d}{\frac{1 + (1 + \beta \, \nu)^2}{\beta^3} + \frac{\nu^3}{6}} \qquad (36)$$

where: S_d = dowel stiffness (N/mm)
E_d = Young's modulus of elasticity (N/mm^2) of dowel steel
d_d = diameter (mm) of dowel bar
I_d = $\pi/64 \; d_d^4$ = moment of inertia (mm^4) of dowel bar
V = joint width (mm)
E = Young's modulus of elasticity (N/mm^2) of concrete
h = thickness (mm) of concrete layer
d = ½ h - ½ d_d = thickness (mm) of concrete above the dowel bar
K = $\frac{E}{d}$ = modulus of dowel support (N/mm^3)
β = $\sqrt[4]{\frac{K \, d_d}{4 \, E_d \; I_d}}$ = relative stiffness (mm^{-1}) of a dowel bar embedded in concrete

In a contraction joint, however, there is not one single bar but a number of dowel bars. The efficiency η of all these dowel bars together is defined as (figure 18):

$$\eta = 100 \, \frac{w_{u,\max}}{1/2 \left(w_{l,\max} + w_{u,\max}\right)} \quad = \quad \frac{2 \, w_{u,\max}}{w_{l,\max} + w_{u,\max}} \qquad (37)$$

where: η = dowel efficiency (%)
$w_{l,max}$ = maximum deflection (mm) at the joint edge of the loaded concrete slab
$w_{u,max}$ = maximum deflection (mm) at the joint edge of the unloaded concrete slab

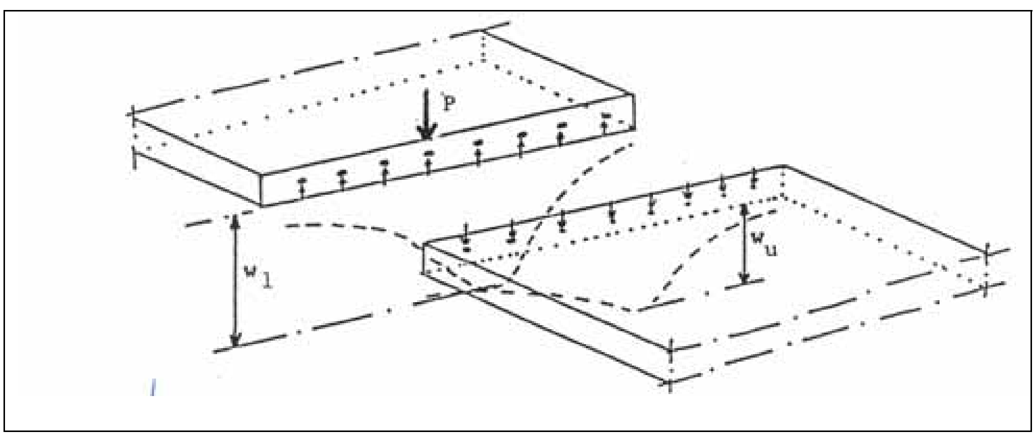
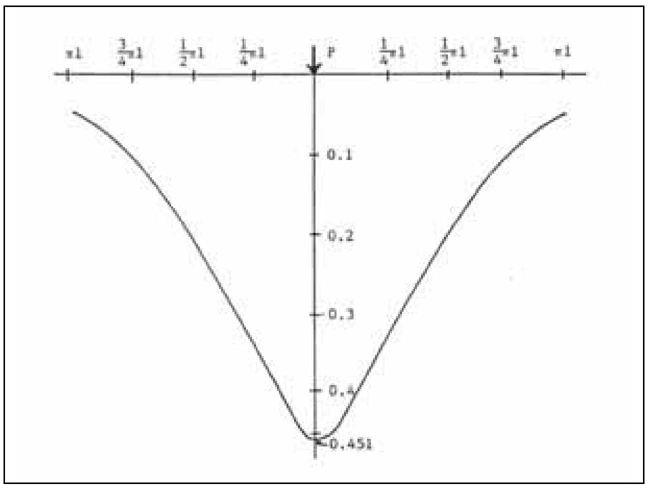


Figure 18. Deflection at the contraction joint edges of the loaded and an adjacent unloaded concrete slab.

The dowel efficiency η appears to be:

$$\eta = \frac{1}{1 + \dfrac{k \ l^2}{0.902 \ S_d \ \gamma}} \qquad (38)$$

where: η = dowel efficiency (%)
k = modulus of substructure reaction (N/mm^3)
l = radius (mm) of relative stiffness of concrete layer
S_d = dowel stiffness (N/mm) (eq. 36)
γ = factor depending on the deflection curve along the slab edge (see figure 19)



$$\gamma = \frac{\text{sum of ordinates of curve at location of dowels}}{\text{centre ordinate (= 0.451)}}$$

Figure 19. Normalized deflection curve of the slab edge.

It appears from equation 38 that the dowel efficiency η is greater when:
- the modulus of substructure reaction k is smaller
- the radius of relative stiffness l of the concrete layer is smaller (i.e. the concrete layer is thinner)
- the dowel stiffness S_d is greater (i.e. the dowel diameter is greater)
- the factor γ is bigger (i.e. the number of dowels is greater).

Similar to the dowel efficiency η, Teller and Sutherland have defined the total load transfer in a joint or crack as follows (19):

$$W = 100 \frac{2w_u}{w_l + w_u} \qquad (39)$$

where: W = joint/crack efficiency (%) related to deflections
w_l = deflection (mm) at the joint/crack edge of the loaded concrete

slab

w_u = deflection (mm) at the joint/crack edge of the unloaded concrete slab

In the case of full load transfer the deflections w_l and w_u at both edges of the joint/crack are equal, which means that the joint/crack efficiency W = 100%. When there is no load transfer at all, the deflection w_u of the edge of the unloaded slab is equal to 0, which means that the joint/crack efficiency W = 0%.

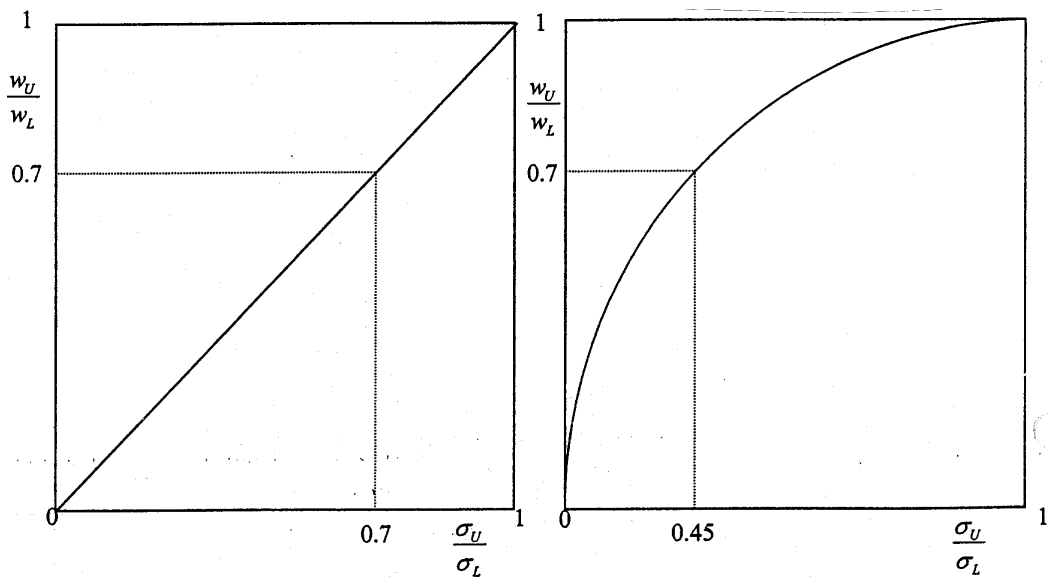


Figure 20. Models for transfer of deflections vs. transfer of flexural tensile stresses in joints.

Analytical concrete pavement design methods, such as the Dutch design method, are primarily based on a correlation between occurring and allowable flexural tensile stresses in the concrete pavement. This means that the load transfer across joints/cracks should be defined in terms of transfer of flexural tensile stresses.
Based on measurements at concrete pavements at airports, Barenberg (20) has developed a non-linear relationship between transfer of deflections and transfer of flexural tensile stresses (figure 20-right). From this relationship it follows that the transfer of flexural tensile stresses in a joint/crack is less than the transfer of deflections.
In the current Dutch design method (5), however, a linear relationship between the transfer of flexural tensile stresses and the transfer of deflections in a joint/crack is assumed (figure 20-left). The load transfer in joints/cracks can then be incorporated in the design of concrete pavement structures by means of a reduction of the actual wheel load P_{act} to the wheel load P (to be used in equations 23 to 32) according to:

$$P = \left(1 - 1/2\, W/100\right) P_{act} = \left(1 - \frac{W}{200}\right) P_{act} \qquad (40)$$

In the Dutch design method it is assumed that the joint efficiency W of a free edge of a plain or reinforced concrete pavement (at the outer side of the carriageway) is not 0% but 20% (in the case that below the concrete pavement a unbound base is applied) or 35% (in the case that a bound base is applied) or 70% (in the case that a widened bound base is applied). These values resulted from calculations for concrete pavements on a Pasternak-foundation. A Pasternak-foundation is a more realistic foundation model consisting of coupled vertical springs that allow the transfer of shear stresses (see figure 27). The Winkler-foundation, used in the Dutch design method, consists of uncoupled vertical springs without transfer of shear stresses (see figure 6).

In the current Dutch design method the following values of the joint efficiency W are used for longitudinal joints in plain or reinforced concrete pavements:

- non-profiled construction joints without ty bars in plain concrete pavements on unbound base: W = 20%
- non-profiled construction joints without ty bars in plain concrete pavements on bound base: W = 35%
- contraction joints with ty bars in plain concrete pavements: W = 70%
- profiled construction joints in plain concrete pavements: W is calculated by means of equation 60a
- joint in reinforced concrete pavement (where always an asphalt layer is applied below the concrete pavement): W = 35%

For transverse joints/cracks the following values of the joint efficiency W are used in the Dutch design method:

- cracks in reinforced concrete pavement: W = 90%
- profiled construction joints or contraction joints, both without dowel bars, in plain concrete pavements: the joint efficiency W at long term is calculated by means of the equation:

$$W = \{5.\log(k.l^2) – 0.0025.L – 25\}.\log N_{eq} – 20 \log(k.l^2) + 0.01.L + 180 \qquad (41a)$$

- profiled construction joints or contraction joints, both with dowel bars, in plain concrete pavements: the joint efficiency W at long term is calculated by means of the equation:

$$W = \{2.5.\log(k.l^2) - 17.5\}.\log N_{eq} - 10\log(k.l^2) + 160 \qquad (41b)$$

In the equations 41a and 41b is:

W = joint efficiency (%) at the end of the pavement life
L = slab length (mm)
k = modulus of substructure reaction (N/mm^3)
l = radius (mm) of relative stiffness of concrete layer (see paragraph 3.4.2)
N_{eq} = total number of equivalent 100 kN standard axle loads in the centre of the wheel track during the pavement life, calculated with a 4th power, i.e. the load equivalency factor $l_{eq} = (L/100)^4$ with axle load L in kN

Example

L = 4.5 m = 4500 mm
k = 0.105 N/mm^3
ν = 0.15 (concrete quality C28/35 (B35))
E = 31000 N/mm^2 (concrete quality C28/35 (B35))
h = 210 mm
l = 695 mm
transverse joint without dowel bars (equation 41a):
$N_{eq} = 10^5$: W = 67.3%
$N_{eq} = 10^6$: W = 54.5%
$N_{eq} = 10^7$: W = 41.8%
transverse joint with dowel bars (equation 41b):
$N_{eq} = 10^5$: W = 84.3%
$N_{eq} = 10^6$: W = 78.5%
$N_{eq} = 10^7$: W = 72.8%
This example clearly shows on one hand the beneficial effect of the application of dowel bars and on the other hand the detrimental effect of the number of heavy axle loadings on the joint efficiency W in transverse joints.

3.5 Stresses due to unequal settlements

Generally concrete pavement structures will not be applied in areas where substantial unequal settlements of the subgrade can be expected. These unequal settlements will result in a low riding quality that only can be improved by means of very expensive maintenance measures.
When nevertheless a concrete pavement structure is designed for a subgrade showing unequal settlements, one has to realize that these settlements introduce extra 'settlement flexural stresses' in the concrete slab because, due to the deadweight, the slab tries to follow as much as possible the settlement profile (figure 21).

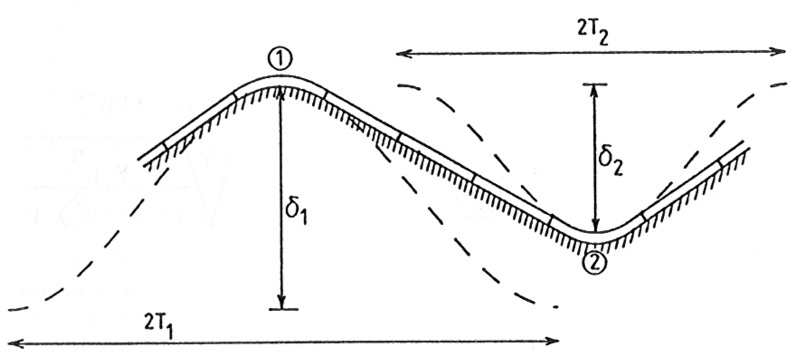


Figure 21. Settlement profile.

Assuming a sinusoidal settlement profile (in figure 26: $\delta_1 = \delta_2 = \delta_m$, $T_1 = T_2 = T$), the maximum settlement stress σ_s in the most heavily (by unequal settlements) loaded concrete slab (cross section 2 in figure 21, flexural tensile stress at the bottom of the concrete slab) can be calculated using the theory of elasticity (21):

$$\sigma_s = \frac{\pi^2 \; h \; E'}{4\left(1-\upsilon^2\right)} \; \frac{\delta_m}{T^2} \; \alpha \tag{42}$$

where: σ_s = flexural tensile stress (N/mm^2)
- h = thickness (mm) of the concrete slab
- υ = Poisson's ratio of concrete
- E' = fictitious modulus of elasticity (N/mm^2) of concrete, taking into account the stress relaxation because of the static settlement loading (E' ≈ E_c/3)
- δ_m = difference (mm) of level of highest and lowest point (i.e. two times the amplitude) of the sinusoidal settlement profile
- T = halve the wave length (mm) of the sinusoidal settlement profile
- α = factor, dependent on the wave length T, the slab length L and the ratio of the modulus of substructure reaction k and the radius of relative stiffness l:

$$\alpha = 1 - F \cos\left(\frac{\pi \; L}{2 \; T}\right) \tag{43}$$

where: F = factor, dependent on the ratio of the slab length L and the radius of relative stiffness l (figure 22)

Unequal settlements usually are greatest in the longitudinal direction of a road.
For the normally applied concrete qualities, concrete slab thicknesses and moduli of substructure reaction the reduction factor α is (nearly) 1 in case of a slab length L of more than about 4 m, which is usually the case for plain concrete pavements.
For reinforced concrete pavements (slab length = distance between the cracks = 1.5 to 3 m) the factor α is smaller than 1. This leads to a reduction of the settlement flexural stresses. The smaller the ratio of the modulus of substructure reaction k and the radius of relative stiffness l of the concrete slab and the smaller the slab length L, the smaller the factor α and thus the smaller the settlement stresses.

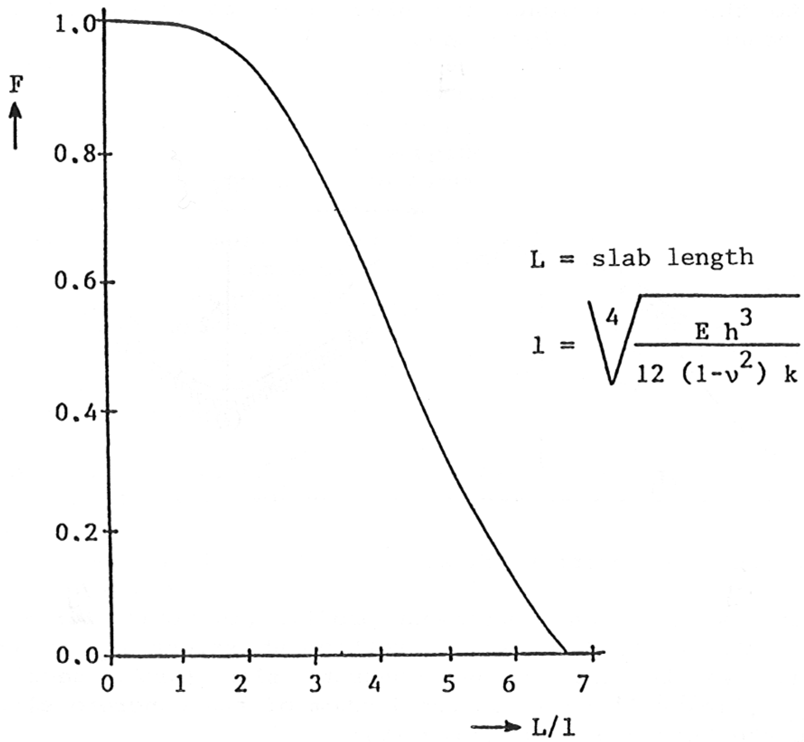


Figure 22. Factor F as a function of the slab length L and the radius of relative stiffness l of the concrete slab.

3.6 Fatigue analysis

A plain concrete pavement is subjected to alternating loadings caused both by traffic loadings (which successively are present and absent) and by an alternating temperature gradient (as a result of variations in air temperature and sun radiation).

The frequency of the temperature gradient changes is much, much lower than the frequency of the traffic loadings. The loading pattern on the concrete pavement thus can be represented by a long wave (wavelength: hours) of the temperature gradient flexural stress σ_t and on top of that a short wave (wavelength: parts of a second for normal driving traffic) of the traffic load flexural stress σ_v. This loading pattern is schematically shown in figure 23.

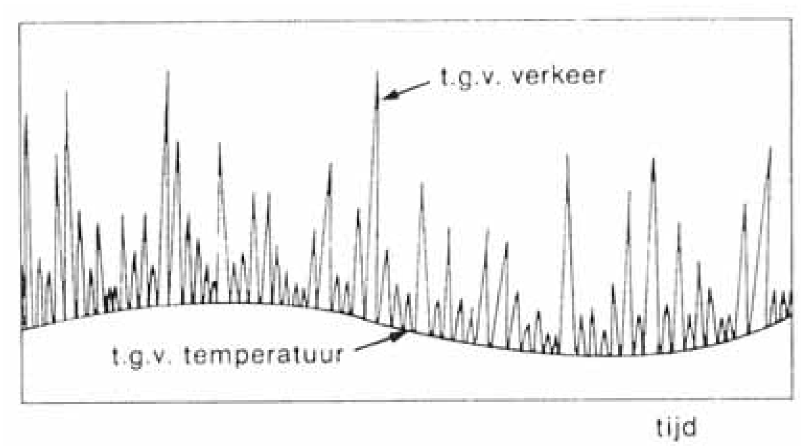


Figure 23. Schematic representation of the loading pattern on a concrete pavement.

In analytical design methods for concrete pavements the main design criterion is to prevent the pavement for cracking. Therefore a strength analysis is performed and because of the great number of load repetitions this means a concrete fatigue analysis. Such an analysis is performed for the critical locations of the pavement, which are:

- for plain concrete pavements:
  - the centre of the longitudinal edges (free edge and/or longitudinal joint)
  - the centre of the wheel track at the transverse joint
- for reinforced concrete pavements:
  - the centre of the wheel track at the transverse crack

In every critical location of the concrete slab the temperature gradient stress σ_{ti} has to be calculated for every positive temperature gradient Δt_i and also the traffic load stress σ_{vi} has to be calculated for every wheel load P_i , taking into account the load transfer (if any).
Next a fatigue damage analysis is carried out for every location by calculating the allowable number of load repetitions N_i, for each combination of wheel load P_i and temperature gradient Δt_i, by means of the appropriate concrete fatigue relationship (e.g. equation 7).

The design criterion (i.e. cracking occurs) is the cumulative damage law of Palmgren-Miner:

$$\sum_i \frac{n_i}{N_i} = 1.0 \qquad (44)$$

where: n_i = occurring number of repetitions of wheel load P_i i.e. the traffic load stress σ_{vi} during the pavement life when a temperature gradient stress σ_{ti} is present

N_i = allowable number of repetitions of wheel load P_i i.e. the traffic load stress σ_{vi} until failure when a temperature gradient stress σ_{ti} is present

<u>Example</u>

In this section an example of a fatigue analysis for a plain concrete pavement is given.

The starting point is a 2-lane road on which during the desired pavement life of 30 years at working days on average 200 trucks per lane are expected. On average a truck has 3 axles while the number of working days per year is taken as 275. The cumulative number of truck axle load repetitions in 30 years thus is equal to 30 * 275 * 200 * 3 = 4,950,000 per lane.
It is assumed that all the truck axles are single axles with super single (extra wide) tires (one tyre at either side of the axle) and that in all cases the tyre pressure is 0.7 N/mm^2. The following wheel load frequency distribution is used for the trucks (per lane in 30 years):
P_{act} = 60 kN: 3% = 148,500
P_{act} = 50 kN: 12% = 594,000
P_{act} = 40 kN: 20% = 990,000
P_{act} = 30 kN: 30% = 1,485,000
P_{act} = 20 kN: 35% = 1,732,000

For the determination of the load transfer in the transverse joints (equation 41) the traffic loading has to be expressed as the total number (N_{eq}) of equivalent 50 kN wheel loads (= 100 kN axle loads) during the pavement life, calculated with the equation:

$$N_{eq} = \sum_i \left(P_i / 50 \right)^4 n_i \qquad (45)$$

where: P_i = wheel load (kN)
n_i = number of repetitions of wheel load P_i during the pavement life

The temperature gradient frequency distribution of table 8 is applied.

In this example the following plain concrete pavement structure (that was also taken in the calculation examples in the sections 3.3.3 and 3.4.3) is analysed:

a. substructure (containing a cement-bound base) with a modulus of substructure reaction k = 0.105 N/mm^2
b. concrete slabs with dimensions: length L = 4.5 m, width W = 3.75 m and thickness h = 210 mm; in the middle of the road there is a longitudinal contraction joint with ty bars and the transverse contraction joints contain dowel bars
c. concrete quality C28/35 (B35) (see table 3):
   dynamic modulus of elasticity E = 31,000 N/mm^2
   Poisson's ratio υ = 0.15
   coefficient of linear thermal expansion $\alpha = 10^{-5}$ °C^{-1}
   mean flexural tensile strength $f_{ct,fl,o}$ = 4.82 N/mm^2 (see table 3 for h = 210 mm)

The fatigue relationship currently applied in the Dutch design method (equation 7) will be used.

Fatigue analysis for centre of longitudinal edge

Because of the cement-bound base, according to section 3.4.3 the free edge (at the road verge) has a joint efficiency W = 35% while the longitudinal contraction joint with ty bars has a joint efficiency W = 70%. This means that the free edge is most critical. According to equation 40 the wheel load P to be used in the Westergaard equation for traffic load stresses σ_v (equation 27) at the free edge is equal to $0.825.P_{act}$.

The temperature gradient stresses σ_t are taken from the calculation example in paragraph 3.3.4 (table 9). They thus are calculated according to the Dutch method (section 3.3.4).

In table 10 the fatigue damage analysis for the centre of the free edge is shown. It has been assumed that 10% of the wheel loadings (= 0.1 * 4,950,000 = 495,000 wheel load repetitions in 30 years) drive directly over the edge.

temperature gradient Δt (°C/mm)	temperature stress σ_t (N/mm²)	wheel load 0.825 P_{act} (kN)	traffic load stress σ_v (N/mm²)	minimum stress $\sigma_{min} = \sigma_t$ (N/mm²)	maximum stress $\sigma_{max} = \sigma_t + \sigma_v$ (N/mm²)	allowable number of load repetitions N	occurring number of load repetitions n	fatigue damage n/N
0.06	1.34	49.5	2.50	1.34	3.84	1721	74	0.0430
		41.25	2.19		3.53	19287	297	0.0154
		33.0	1.86		3.20	252556	495	0.0020
		24.75	1.50		2.84	4178412	743	0.0002
		16.5	1.10		2.44	94421427	866	0.0000
0.05	1.30	49.5	2.50	1.30	3.80	2212	149	0.0674
		41.25	2.19		3.49	24324	594	0.0244
		33.0	1.86		3.16	312159	990	0.0032
		24.75	1.50		2.80	5052204	1485	0.0003
		16.5	1.10		2.40	~	1733	0
0.04	1.26	49.5	2.50	1.26	3.76	2832	668	0.2359
		41.25	2.19		3.45	30566	2673	0.0875
		33.0	1.86		3.12	384559	4455	0.0116
		24.75	1.50		2.76	6090714	6683	0.0011
		16.5	1.10		2.36	~	7796	0
0.03	0.98	49.5	2.50	0.98	3.48	14419	817	0.0567
		41.25	2.19		3.17	137625	3267	0.0237
		33.0	1.86		2.84	1519393	5445	0.0036
		24.75	1.50		2.48	20866955	8168	0.0004
		16.5	1.10		2.08	~	9529	0
0.02	0.65	49.5	2.50	0.65	3.15	80101	1114	0.0139
		41.25	2.19		2.84	671775	4455	0.0066
		33.0	1.86		2.51	6462411	7425	0.0011
		24.75	1.50		2.15	~	11138	0
		16.5	1.10		1.75	~	12994	0
0.01	0.33	49.5	2.50	0.33	2.83	353602	3267	0.0092
		41.25	2.19		2.52	2651262	13068	0.0049
		33.0	1.86		2.19	~	21780	0
		24.75	1.50		1.83	~	32670	0
		16.5	1.10		1.43	~	38115	0
0.0025	0.08	49.5	2.50	0.08	2.58	1016071	8762	0.0086
		41.25	2.19		2.27	~	35046	0
		33.0	1.86		1.94	~	58410	0
		24.75	1.50		1.58	~	87615	0
		16.5	1.10		1.18	~	102218	0
						Total	495000	0.6197

Table 10. Fatigue damage analysis for the centre of the longitudinal free edge of the concrete slab.

Table 10 clearly shows that in the centre of the longitudinal free edge the fatigue damage n/N is mainly caused by the two heaviest wheel load classes P_{act} = 60 kN and P_{act} = 50 kN. The fatigue damage due to the two following wheel load classes P_{act} = 40 kN and P_{act} = 30 kN is very small, while the lowest wheel load class P_{act} = 20 kN (and thus also luxury cars with wheel loadings of 2 to 5 kN) does not cause any fatigue damage at all.
The total fatigue damage in the centre of the longitudinal free edge amounts 0.62. This is smaller than the Palmgren-Miner criterion (Σ n/N = 1.0), which means that the chosen plain concrete pavement is (a little bit) over-designed with respect to the strength criterion for the centre of the longitudinal edges.

Fatigue analysis for centre of wheel track at the transverse joint

For the determination of the load transfer (joint efficiency W) at the transverse joint at the long term, first the total number (N_{eq}) of equivalent 50 kN wheel loads in the centre of the wheel track during the pavement life has to be determined by means of equation 45. Assuming that 50% of the wheel loadings (= 0.5 * 4,950,000 = 2,475,000 wheel load repetitions in 30 years) drives exactly in the centre of the wheel track, Neq is equal to:

$$N_{eq} = 0.5 * 2{,}475{,}000 * \{(60/50)^4 * 0.03 + (50/50)^4 * 0.12 + (40/50)^4 * 0.20 + \\ + (30/50)^4 * 0.30 * + (20/50)^4 * 0.35\} = 386{,}060$$

It follows then from equation 41b for the transverse joints with dowel bars: joint efficiency W = 80.9%.
According to equation 40 the wheel load P to be used in the Westergaard equation for traffic load stresses σ_v (equation 27) at the transverse joint is equal to $0.5955.P_{act}$, say $P = 0.60.P_{act}$.

The temperature gradient stresses σ_t are again taken from the calculation example in paragraph 3.3.4 (table 9). They thus are calculated according to the Dutch method (section 3.3.4).

Table 11 shows that the fatigue damage at the transverse joint in the centre of the wheel track is extremely small and that all the damage is caused by the combination of the heaviest wheel load group P_{act} = 60 kN and the greatest temperature gradient Δt = 0.06 ºC/mm.

Conclusion

In this calculation example the strength criterion for the centre of the longitudinal free edge is the dominant design criterion for the plain concrete pavement (and this is usually the case). At that location the occurring combination of flexural tensile stresses and number of repetitions thereof (high temperature gradient stresses σ_t, limited number of load repetitions n but high traffic load stresses σ_v due to the limited load transfer) causes more fatigue damage than the occurring combination of stresses and repetitions thereof in the wheel track at the transverse joint (low temperature gradient

temperature gradient Δt (°C/mm)	temperature stress σ_t (N/mm^2)	wheel load 0.60 P_{act} (kN)	traffic load stress σ_v (N/mm^2)	minimum stress $\sigma_{min} = \sigma_t$ (N/mm^2)	maximum stress $\sigma_{max} = \sigma_t + \sigma_v$ (N/mm²)	allowable number of load repetitions N	occurring number of load repetitions n	fatigue damage n/N
0.06	0.45	36	1.98	0.45	2.43	6486615	371	0.0001
		30	1.73		2.18	~	1485	0
		24	1.47		1.92	~	2475	0
		18	1.18		1.63	~	3713	0
		12	0.86		1.31	~	4331	0
0.05	0.42	36	1.98	0.42	2.40	~	743	0
		30	1.73		2.15	~	2970	0
		24	1.47		1.89	~	4950	0
		18	1.18		1.60	~	7425	0
		12	0.86		1.28	~	8663	0
0.04	0.38	36	1.98	0.38	2.36	~	3341	0
		30	1.73		2.11	~	13365	0
		24	1.47		1.85	~	22275	0
		18	1.18		1.56	~	33413	0
		12	0.86		1.24	~	38981	0
0.03	0.32	36	1.98	0.32	2.30	~	4084	0
		30	1.73		2.05	~	16335	0
		24	1.47		1.79	~	27225	0
		18	1.18		1.50	~	40838	0
		12	0.86		1.18	~	47644	0
0.02	0.23	36	1.98	0.23	2.21	~	5569	0
		30	1.73		1.96	~	22275	0
		24	1.47		1.70	~	37125	0
		18	1.18		1.41	~	55688	0
		12	0.86		1.09	~	64969	0
0.01	0.04	36	1.98	0.04	2.02	~	16335	0
		30	1.73		1.77	~	65340	0
		24	1.47		1.51	~	108900	0
		18	1.18		1.22	~	163350	0
		12	0.86		0.90	~	190575	0
0.0025	0	36	1.98	0	1.98	~	43808	0
		30	1.73		1.73	~	175230	0
		24	1.47		1.47	~	292050	0
		18	1.18		1.18	~	438075	0
		12	0.86		0.86		511088	0
						Total	2475000	0.0001

Table 11. Fatigue damage analysis for the centre of the wheel track at the transverse joints between the concrete slabs.

stresses σ_t, great number of load repetitions n but limited traffic load stresses σ_v because of the great load transfer in the transverse joints).
The total fatique damage in the centre of the longitudinal free edge is 0.62, which means that the plain concrete pavement is hardly over-designed.

3.7 Finite element method

The finite element method enables the most accurate modeling of the real situation with respect to the external loadings, the geometry of the discontinuous concrete pavement, the material characteristics, and the interaction between the various layers of the pavement structure. So from finite element calculations one can expect more detailed and more realistic data about stresses and displacements within a concrete pavement structure then can be obtained by means of the analytical methods such as from Eisenmann (3.3.3), the Dutch method (3.3.4) and Westergaard (3.4.2).

For instance, on the basis of numerous ILLI-SLAB (17) and KOLA (22,23,24,25) finite element calculations it was shown that the original Westergaard-equations for edge loading (equations 25 and 26) are incorrect (figure 24), and that the most correct equations are the 'new' Westergaard equations for a (semi-)circular loading area (equations 27 to 30) (figure 25).

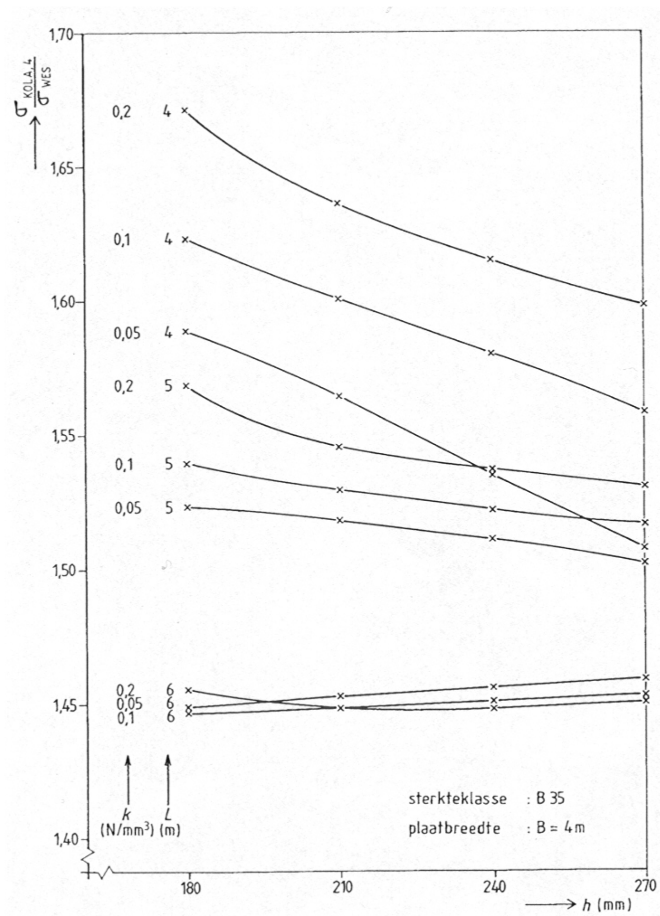


Figure 24. Ratio of the flexural tensile stress $\sigma_{KOLA,4}$ (calculated by means of KOLA) and the flexural tensile stress σ_{WES} (calculated by means of the original Westergaard-equation 25) in the centre of the longitudinal edge of a concrete slab (concrete quality C28/35 (B35), width W = 4 m) as a function of the modulus of substructure reaction k, the slab length L and the slab thickness h.

Some examples of phenomena or concrete pavement structure types that cannot be handled by analytical methods (such as the Westergaard, Eisenmann and the Dutch method) but instead can be analyzed very well by means of the finite element method are:

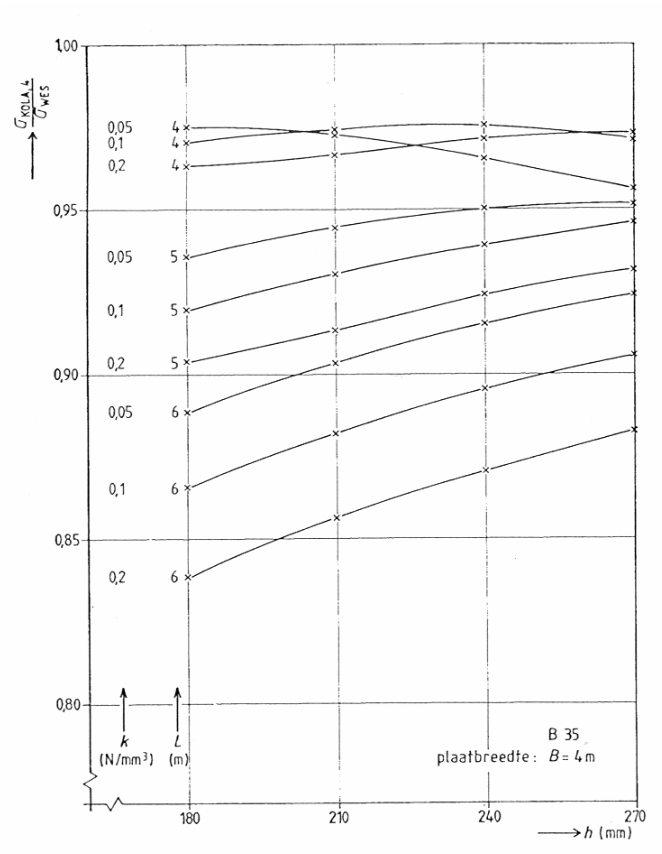


Figure 25. Ratio of the flexural tensile stress $\sigma_{KOLA,4}$ (calculated by means of KOLA) and the flexural tensile stress σ_{WES} (calculated by means of the new Westergaard-equation 29) in the centre of the longitudinal edge of a concrete slab (concrete quality C28/35 (B35), width W = 4 m) as a function of the modulus of substructure reaction k, the slab length L and the slab thickness h.

1. Combined effect of temperature gradient and traffic loadings

At concrete pavement structures the variable traffic loadings are moving, fast or slow, in some position over the concrete slab that is already subjected to some temperature gradient. Due to the alternating warping of the concrete slab as a consequence of the in time varying temperature gradient, especially at the long term full adhesion between the concrete slab and the underlying substructure is very questionable. This means that due to the present temperature gradient the concrete slab will be partly supported, in case of a positive gradient only along the edges and in case of a negative gradient only in the central area of the slab.
It is obvious that the flexural stresses induced by the traffic loadings in such a partly supported slab will be greater than those in a fully supported slab (as was assumed by Westergaard).

The combined effect of a temperature gradient and traffic loadings can be analyzed very well by means of the finite element method (figure 26) (26,27).

Although the increase of the flexural stresses is not very dramatically (5 to 15%), there is a substantial reduction of the concrete pavement life because of the concrete fatigue behaviour (equation 7).

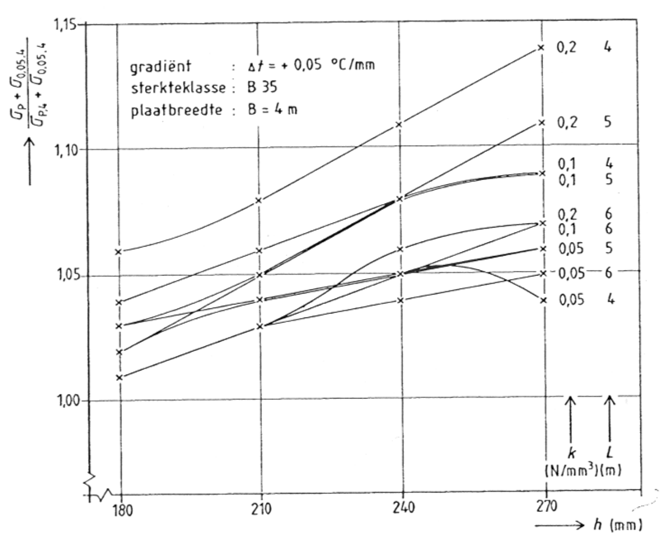


Figure 26. Ratio of the flexural tensile stresses $\sigma_{p+0.05,4} / \left(\sigma_{p,4} + \sigma_{0.05,4}\right)$ below a single wheel load P in the centre of the longitudinal edge of a concrete slab (concrete quality C28/35 (B35), width W = 4 m) as a function of the modulus of substructure reaction k, the slab length L and the slab thickness h; temperature gradient = 0.05 °C/mm.

2. <u>Small concrete slabs</u>

Besides the restriction of full contact with the substructure there is another major restriction of the Westergaard-analysis. This restriction is that the analysis is only valid for a 'large' slab, which means that the edges and corners of the slab are so far away that they don't have any significant influence on the maximal flexural stresses and deflections due to the single wheel load. This restriction mostly is expressed as a minimum required ratio of the slab dimension and the radius of relative stiffness l of the slab for the Westergaard-analysis to be valid.
In the literature no unique value for this minimum ratio can be found. There is some doubt about the validity of the Westergaard-analysis for the usually applied (in situ) plain concrete slabs, but it is for sure that the Westergaard-analysis does not apply to the frequently cracked reinforced concrete pavements and to small precast concrete slabs and tiles.

The structural behaviour of small concrete slabs can be analyzed very well by means of a finite element program, and one can try to find a slab dimension related adjustment factor to the Westergaard-analysis.

3. Material behaviour

Another reason for the application of a finite element method is that this method allows a proper modelling of the actual behaviour of the materials, applied in concrete pavement structures.
An example is the stress-dependent resilient and permanent deformation behaviour of unbound base and sub-base materials. Especially a proper modeling of the actual permanent deformation behaviour of these unbound materials is very important, as the permanent deformation determines the unevenness of the pavement at long term as well as the rate of loss of support around the edges of the concrete slabs.
On the other hand, the frequently used cement-bound base is certainly able to take shear stresses in contrast to the usual substructure schematization into a dense liquid (modulus of substructure reaction k, see figure 6). The shear stress transfer in the base can be schematized more realistic by a so-called Pasternak-foundation, in which the linear-elastic vertical springs, with the stiffness k, are coupled and which is characterized by 2 parameters i.e. k and G (figure 27).
In principle a finite element program can handle such more complex material behaviour.

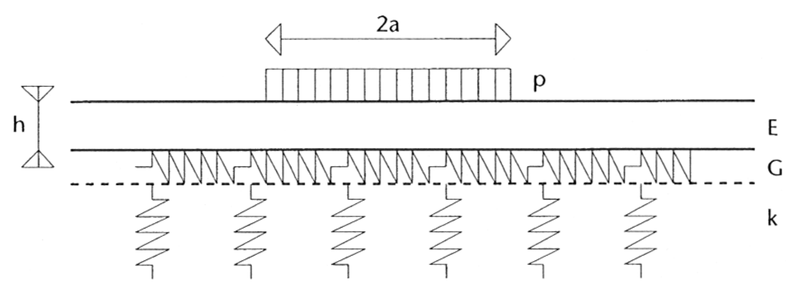


Figure 27. Schematization of a concrete slab on a Pasternak foundation.

4. Discontinuities

A last reason for the application of the finite element method is that this method allows a detailed investigation of the behaviour of concrete pavements around sometimes called “details”, like joints in unreinforced concrete pavements. In these pavement’s discontinuities the load transfer due to aggregate interlock, dowel bars or reinforcement steel can be simulated quite well in the finite element method by means of an appropriate set of springs (22).

It can be concluded that the finite element method is a powerful tool for an in-depth analysis of the structural behaviour of concrete pavements.

4. DESIGN METHODS FOR CONCRETE PAVEMENT STRUCTURES

4.1 General

In this chapter first some empirical design methods for plain and reinforced concrete pavement structures are briefly discussed and then some remarks with respect to the applicability of this kind of design methods are made.
Then analytical design methods, based on an analysis of the flexural stresses in the concrete top layer due to external loadings, are briefly discussed and reviewed in general terms.
The current analytical Dutch method (5), and its development during the last decades, for the structural design of plain and reinforced concrete pavements is described more extensively in chapter 5.

4.2 Empirical design methods

Empirical design methods are (mainly) based on the analysis of the actual long term behaviour of in service concrete (test) pavements. This implies that an empirical design method can only be developed in regions/countries where concrete pavement structures are already used during a long time and on a large scale.
Examples of empirical concrete pavement design methods are:
1. the AASHTO-method (USA)
2. the PCA-method (USA)
3. the BDS-method (Great Britain)
4. the BRD-method (Germany)

4.2.1 The AASHTO-method

The AASHTO-method (28) for the design of concrete pavement structures is based on the results of the AASHO Road Test that was carried out in the USA in the years 1958 to 1960. The design criterion is serviceability (rideability), which is mainly determined by the pavement's longitudinal (un)evenness.
The input parameters for this design method, that covers both plain and reinforced concrete pavements, are:
1. traffic
   - the reliability factor (R)
   - the overall standard deviation (S_o)
   - the cumulative number of equivalent 80 kN (18 kip) single axle loads on the design traffic lane during the design life; the load equivalency factor is dependent on the axle load, the axle configuration, the thickness of the concrete slab (D) and the terminal serviceability index (p_t)
   - the design serviceability loss ($\Delta PSI = p_i - p_t$, where p_i = initial serviceability index, p_t = terminal serviceability index at the end of the design life) means that serviceability (rideability), mainly determined by

the longitudinal (un)evenness of the pavement's surface, is taken as the design criterion

2. concrete slab
   - the mean flexural tensile strength of the concrete, determined after 28 days by means of a 4-point bending test (S'_c)
   - the (mean) Young's modulus of elasticity of the concrete (E_c)
   - the load transfer (J), dependent on the type of concrete pavement structure, the type of joint construction (whether of not dowel bars), the type of reinforcement, the slab thickness, the modulus of substructure reaction, etc.
3. substructure
   - the drainage coefficient (C_d)
   - the effective modulus of substructure reaction (k), where the seasonal variation (for instance due to frost/thaw-action) of the resilient modulus of the substructure layers and the potential loss of support of the concrete slab are taken into account.

On the basis of these input parameters, the mean concrete slab thickness D (in half-inches) is found by means of the two segments of figure 28.

4.2.2 The PCA-method

The PCA-method (29) is mainly based on the AASHO Road Test, and additional finite element calculations and tests. The design criterion is the cumulative 'internal damage' due to concrete fatigue and pavement erosion.
The input parameters for application of this design method for both plain and reinforced concrete pavements are:

1. traffic
   - for every axle load class the number of load repetitions (both for single axles and dual axles and triple axles) on the design traffic lane during the design life
   - the load safety factor (LSF)
2. concrete slab
   - the type of concrete slab (plain/reinforced, whether or not dowel bars, type of reinforcement, etc.)
   - the length of the concrete slab
   - the mean flexural tensile strength of the concrete, determined after 28 days by means of a 4-point bending test (MR)
3. substructure
   - the mean modulus of substructure reaction (k).

By means of a great number of tables and charts both the concrete fatigue damage and the pavement erosion damage for an initial concrete pavement structure are analyzed. Depending on the result, next an iteration process starts to determine the pavement structure that is optimal with respect to fatigue and erosion damage.

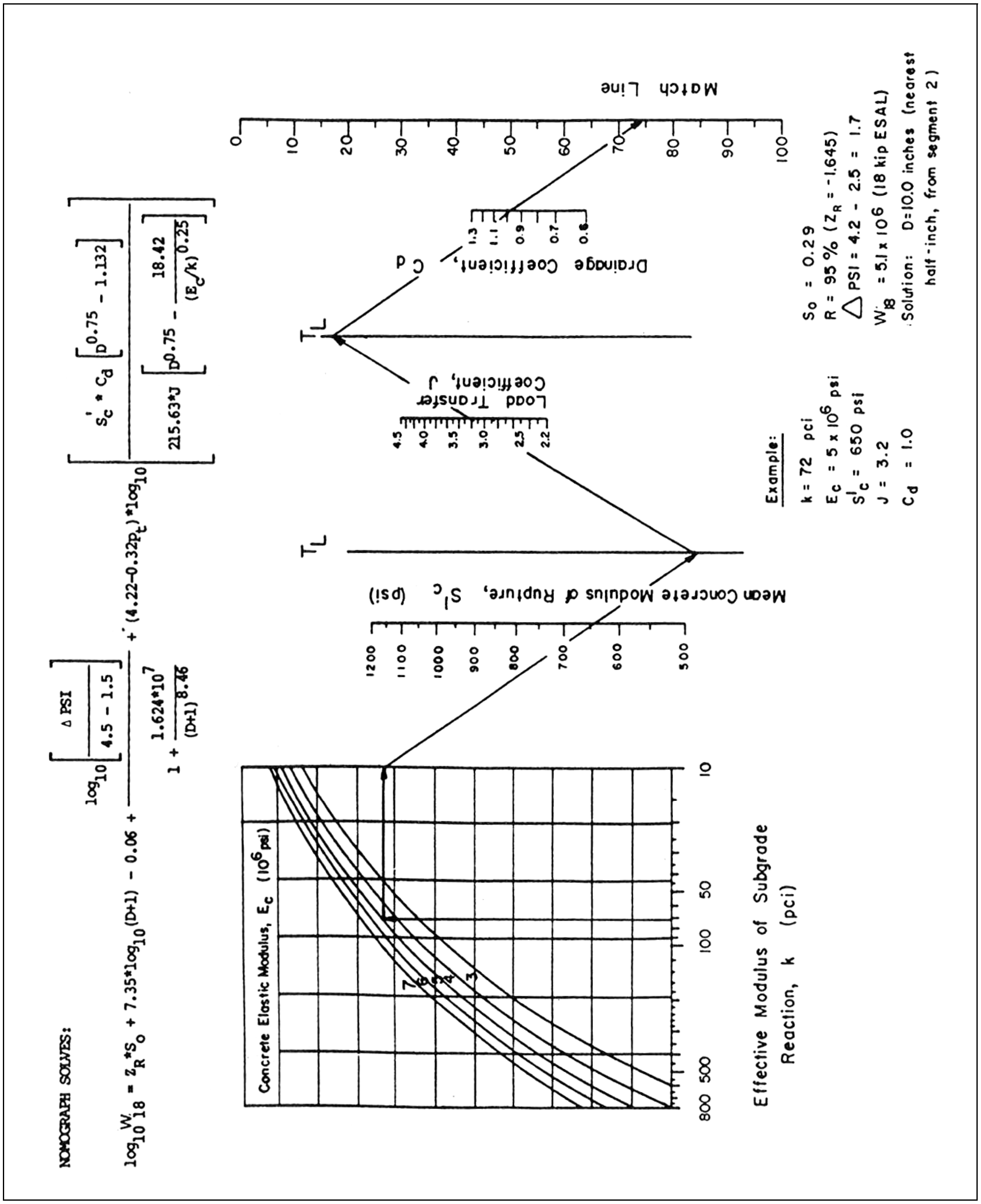


Figure 28. AASHTO design chart for concrete pavement structures, based on using mean values for each input variable (segment 1) (28).

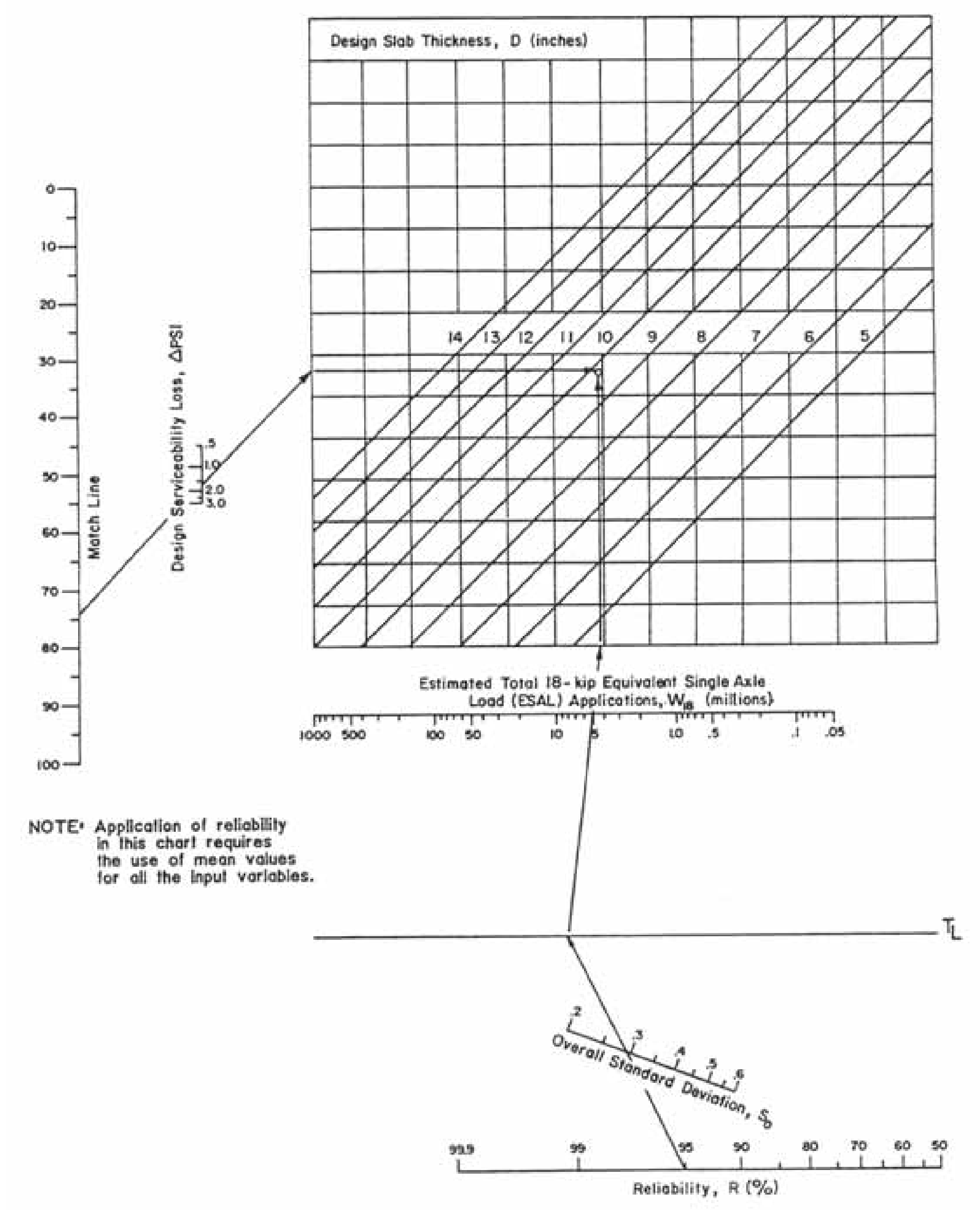


Figure 28. AASHTO design chart for concrete pavement structures, based on using mean values for each input variable (segment 2) (28).

4.2.3 The BDS-method

The British design method (<u>30</u>) for both plain and reinforced concrete pavements is mainly based on the behaviour of test pavements, realized in Great Britain. This method requires the following input parameters:

1. <u>traffic</u>
   - the number of commercial vehicles (unloaden weight more than 1.5 tons) per day (cv/d) in one direction in the first year
   - the number of lanes per direction
   - the design life (taken as 40 years)
2. <u>concrete slab</u>
   - the concrete width besides the design traffic lane
3. <u>substructure</u>
   - CBR-value of the subgrade.

As an example, the design procedure for plain concrete pavements is described below:

a. determine the thickness of the 'capping layer' (=lower sub-base, CBR-value at least 15%) on the basis of the subgrade CBR-value by means of figure 29
b. determine cv/d and read in figure 30 whether this value has to be corrected in case of a two-lane road (the design chart, figure 31, is developed for multi-lane motorways)
c. determine the thickness of the concrete slab by means of figure 31
d. determine by means of figure 32 whether this slab thickness has to be increased
e. determine the maximum slab length (= joint spacing) by means of the notes in figure 31.

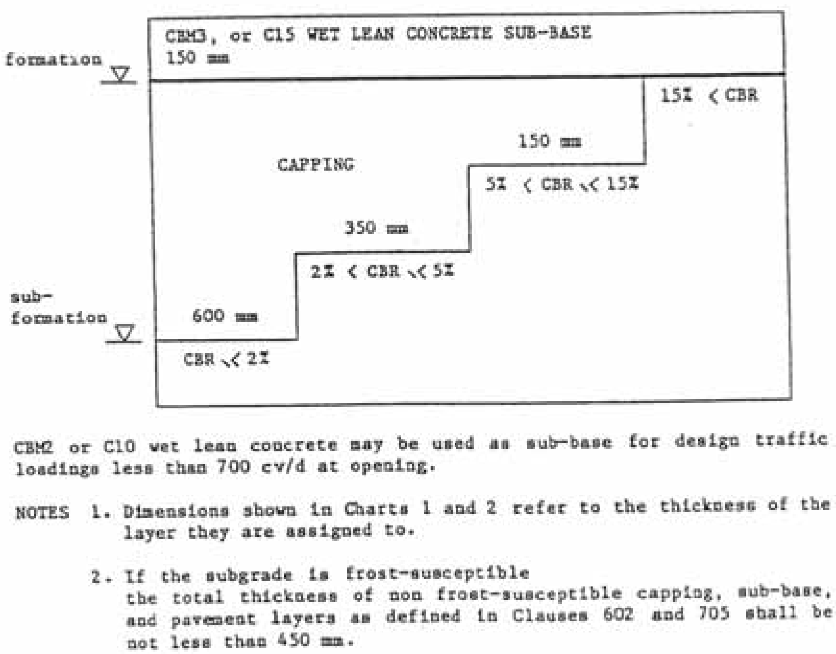


Figure 29. Thickness of the 'capping layer' as a function of the subgrade CBR value (<u>30</u>).

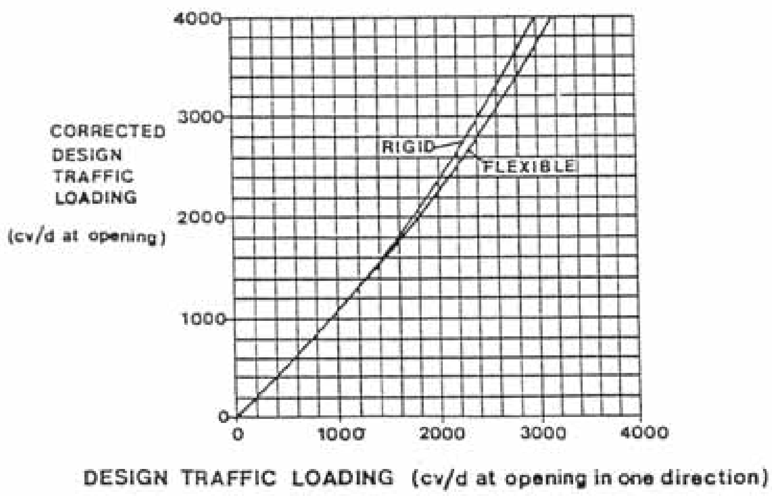


Figure 30. Correction of the design traffic loading for two-lane roads (30).

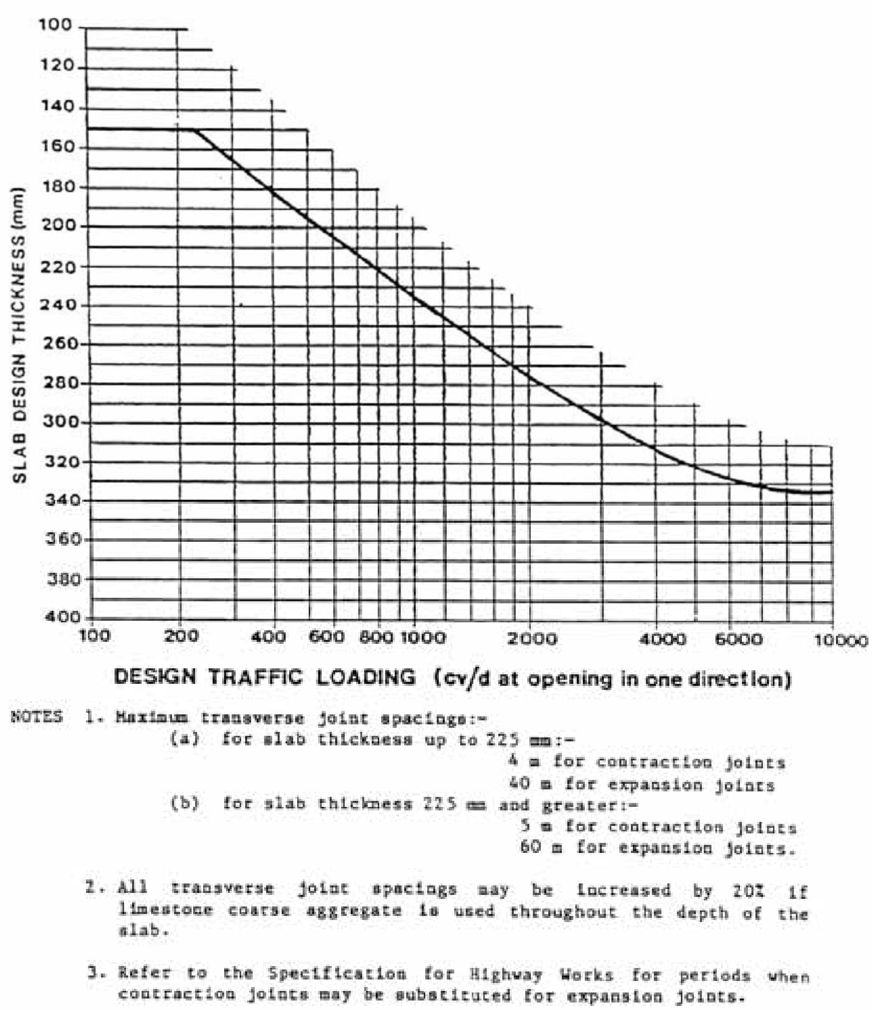


Figure 31. Design thickness for plain concrete slabs (30).

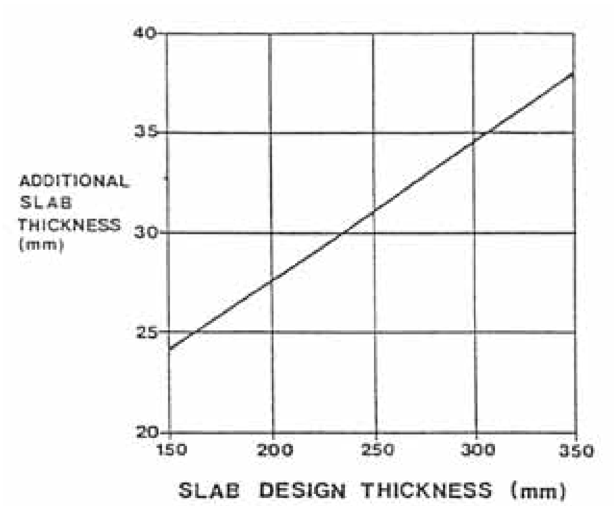


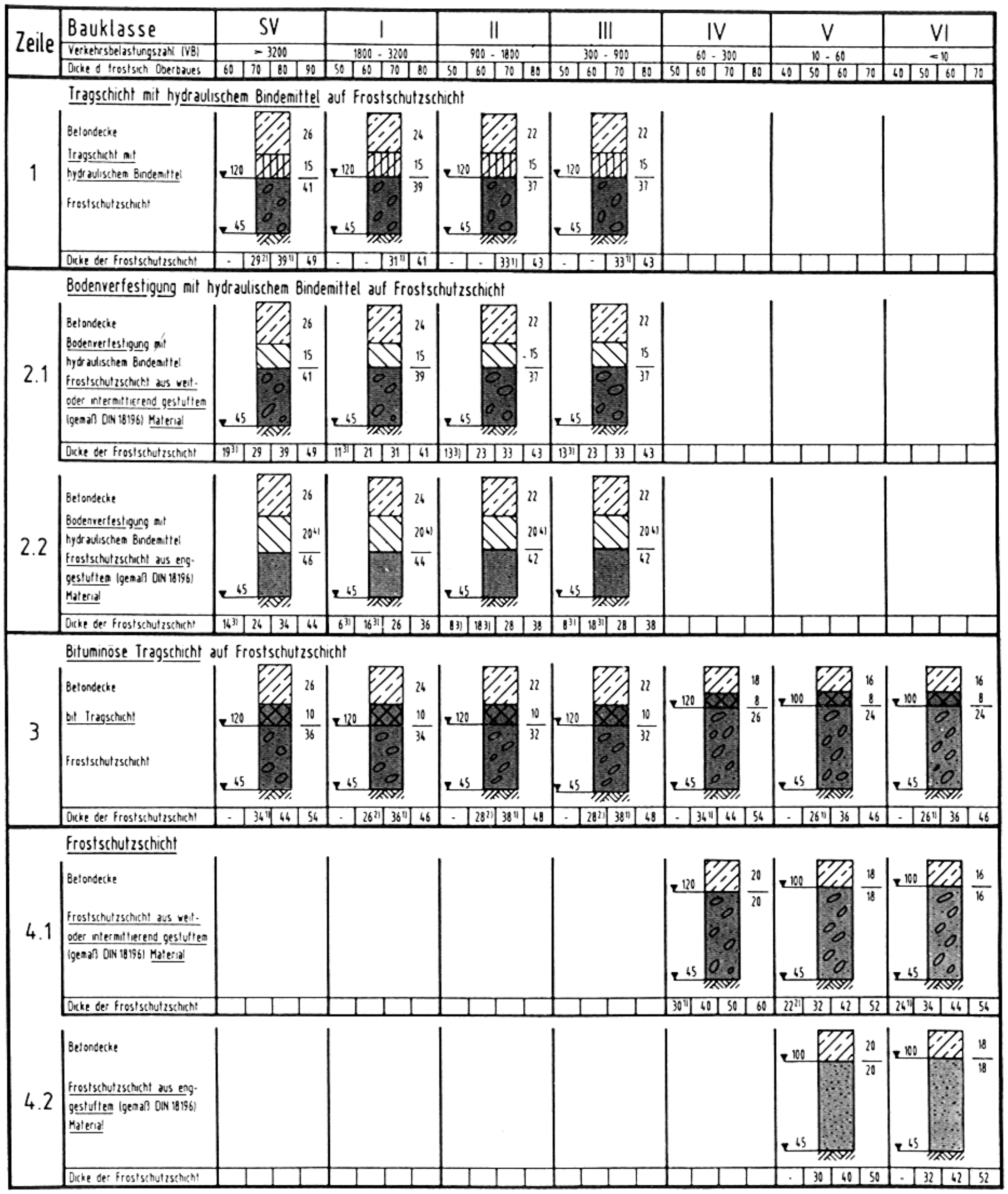
Figure 32. Additional plain concrete slab thickness where the slab does not extend 1 m or more beyond the edge of lanes carrying commercial vehicles (30).

4.2.4 The BRD-method

The German design method (31,32) for plain concrete pavement structures in fact is a catalogue with a limited number of standard structures, mainly determined on the basis of the very extensive experience with concrete pavement structures in Germany.
To be able to use figure 33 with standard plain concrete pavement structures one has to know the traffic loading (VB), expressed as the number of heavy vehicles (trucks and busses) per day at the design traffic lane in the first year of the design period (usually taken as 20 years). For every traffic loading VB some pavement structures, with various types of substructure, then can be chosen.
At top of every pavement layer a minimum static modulus of elasticity (E_{v2}) is required, for instance at top of the subgrade: $E_{v2} \geq 45$ N/mm^2; this E_{v2}-value has to be determined on the basis of a plate bearing test.

4.2.5 Applicability of empirical design methods

It has to be emphasized that one has to be very careful with the application of empirical design methods. They only can be applied with confidence in those areas where all relevant local circumstances (such as climate, type of traffic, axle loadings, material properties, construction details, drainage measures, construction techniques and equipment, etc.) are (nearly) the same as those in the areas/countries for which the method was developed. This means for instance that in developing countries the above described Northern-American and Western-European empirical design methods can only be used for a first global design, and certainly not for the final design of a concrete pavement structure.



[1] Mit rundkörnigen Mineralstoffen nur bei örtlicher Bewährung anwendbar
[2] Nur mit gebrochenen Mineralstoffen und bei örtlicher Bewährung anwendbar
[3] Nur bei Bodenverfestigung im Baumischverfahren ausführbar
[4] Mit zusätzlichen Maßnahmen zur gezielten Rißbildung (z. B. gemäß ZTVT-StB)

Figure 33. German catalogue with standard plain concrete pavement structures; thicknesses in cm (31,32).

4.3 Analytical design methods

The main design criterion in analytical design methods for (un)reinforced concrete pavement structures is cracking within the concrete top layer. Therefore in these design methods emphasis is laid on the calculation of the fatigue damage due to traffic and temperature loadings in the critical locations of the concrete layer. Figure 34 shows how such a fatigue analysis can be done for a plain concrete pavement structure (33).

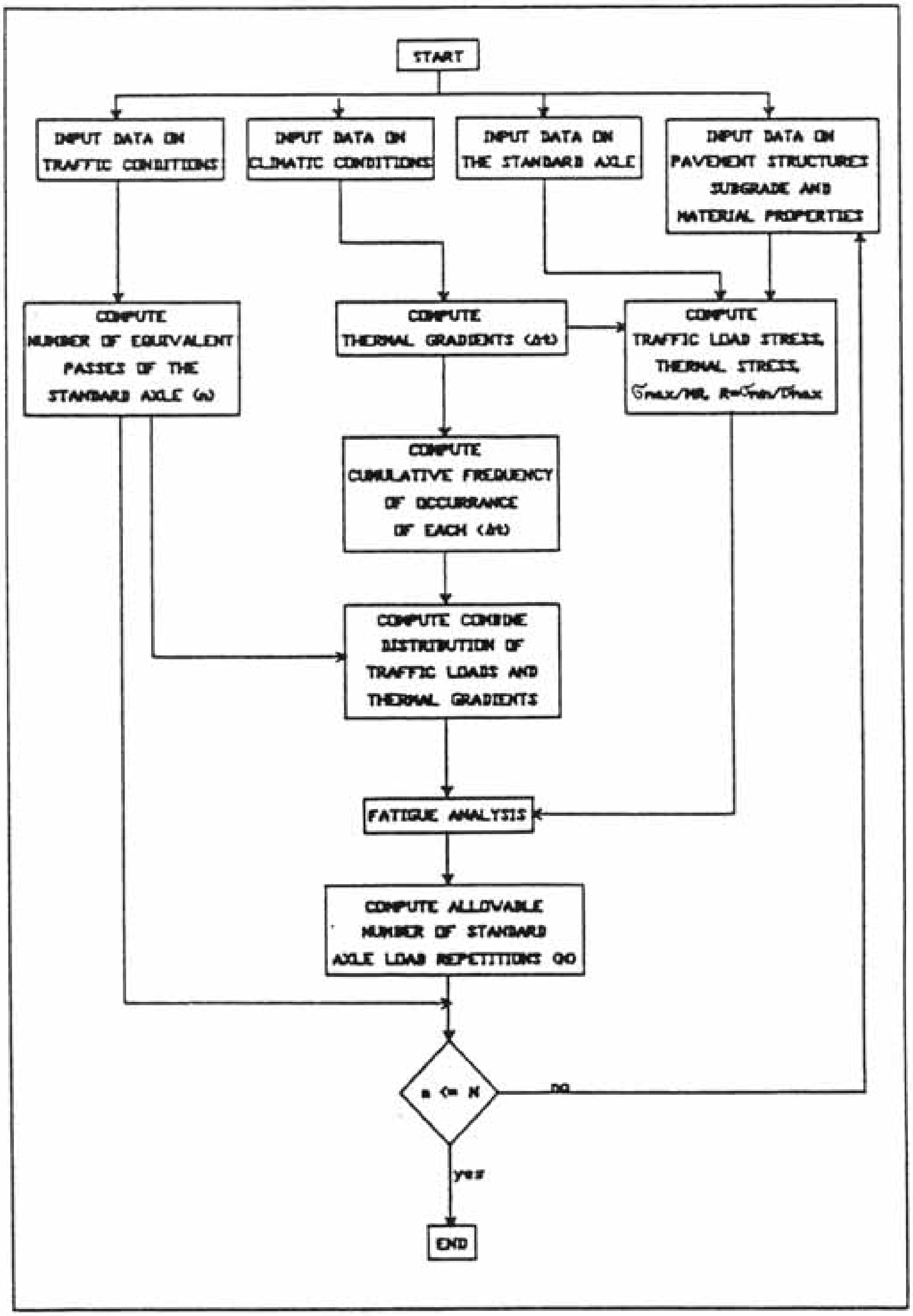


Figure 34. Flowchart for the fatigue analysis of plain concrete pavement structures (33).

In some analytical design methods (34) there is a second design criterion for plain concrete pavements, namely joint faulting (resulting in a bad serviceability due to unevenness at the joints).
In the design calculations this criterion is taken into account by limiting the deflection at the transverse joints due to the traffic loadings: the higher the number of (equivalent standard) axle loadings, the lower the acceptable deflection.

In some analytical design methods (33,34) the traffic loading is expressed as a predicted number (N_{eq}) of equivalent standard axle load repetitions (compare equation 45):

$$N_{eq} = \sum_i \left(L_i / L_{st}\right)^m n_i \tag{46}$$

where: L_i = axle load (kN)
L_{st} = standard axle load (kN)
n_i = number of repetitions of axle load L_i during the pavement life
m = load equivalency exponent; with respect to cracking in concrete: m = 10 to 20!

However, one is getting convinced that it is more correct to use the predicted number of repetitions of the various distinguished axle load groups (5,35).

Generally two locations of the concrete slab are assumed to be critical, namely the transverse edge in the wheel track (greatest number of traffic loading repetitions) and the centre of the longitudinal edge (greatest temperature gradient stresses).
The flexural tensile stresses due to the traffic loadings at these locations of the slab edge are calculated, usually by means of a Westergaard-equation for edge loading.
Independently, the flexural tensile stresses due to only a temperature gradient are calculated, usually by means of Eisenmann's theory.
Then the stresses due to the traffic loadings are linear added to those due to the temperature gradients, taking into account some temperature gradient frequency distribution, to find the maximal flexural tensile stresses.

In some more recent analytical design methods (33,35) these maximal flexural tensile stresses within (the edge of) the concrete slab are calculated by means of a finite element program. In these calculations the more realistic combined effect of a temperature gradient and a traffic loading is taken into account.

When having calculated the flexural tensile stresses due to the traffic loadings and the temperature gradients, the thickness design of the concrete pavement structure is done by means of Palmgren-Miner's fatigue law:

$$\sum_{ij} \frac{n_{ij}}{N_{ij}} = 1 \tag{47}$$

where: n_{ij} = predicted number of repetitions of flexural tensile stress σ_{ij} (due to an axle load L_i and a temperature gradient Δt_j).

N_{ij} = allowable number of repetitions of flexural tensile stress σ_{ij} (due to an axle load L_i and a temperature gradient Δt_j) to failure

The allowable number of load repetitions N can be found by means of the appropriate concrete fatigue relationship.
Table 12 and figure 35 show that there exist numerous of such fatigue relationships. The calculated fatigue damage, so the structural design of the concrete pavement, is very strongly influenced by the choice of the concrete fatigue relationship!

When the fatigue damage of an initial concrete pavement structure appears to be smaller or greater than 1, the pavement is over-designed and under-designed respectively.
By changing the layer thicknesses and/or material qualities, by means of an iteration process one can obtain a technical and economical optimal concrete pavement structure.

No.	Author	Fatigue law	Notes
1.	PCA (USA)	log N = 11.73 - 12.08 (σp/MR)	if (σp/MR) > 0.55
		log N = (4.2577/((σp/MR)-0.4325))^3.268	if (σp/MR) = 0.45÷0.55
		log N = unlimited	if (σp/MR) < 0.45
2.	Darter (USA)	log N = 16.61 - 17.61 * SR	failure prob.: 24%
3.	Veverka (Belgium)	log N = 20 - 20 (σ/MR)	σ=σp if slab lenght<6m σ=σmax if slab lenght>6m
4.	Iwama (Japan)	log N = 16.72 - 16.13 * SR	failure prob.: 15%
5.	VNC (Netherland)	log N = 12.6 [1 - 0.8 A/B]	A = (σmax-σmin)/MR B = 0.8 - σmin/MR
6.	Domenich. et al.(Italy)	log N = 10.48 (1 - SR)/(1-R)	
7.	Faraggi et al.(Spain)	log N = 11 (1 - SR)/(1-R)	
8.	Yao (China)	log N = 13.02 (0.944 - SR)/(1-R)	

Notation: SR = σmax/MR ; R = σmin/σmax ; σmax = σp + σΔt
σp = stress due to traffic load
σΔt= stress due to thermal gradients

Table 12. Examples of concrete fatigue relationships, used in concrete pavement design methods (33).

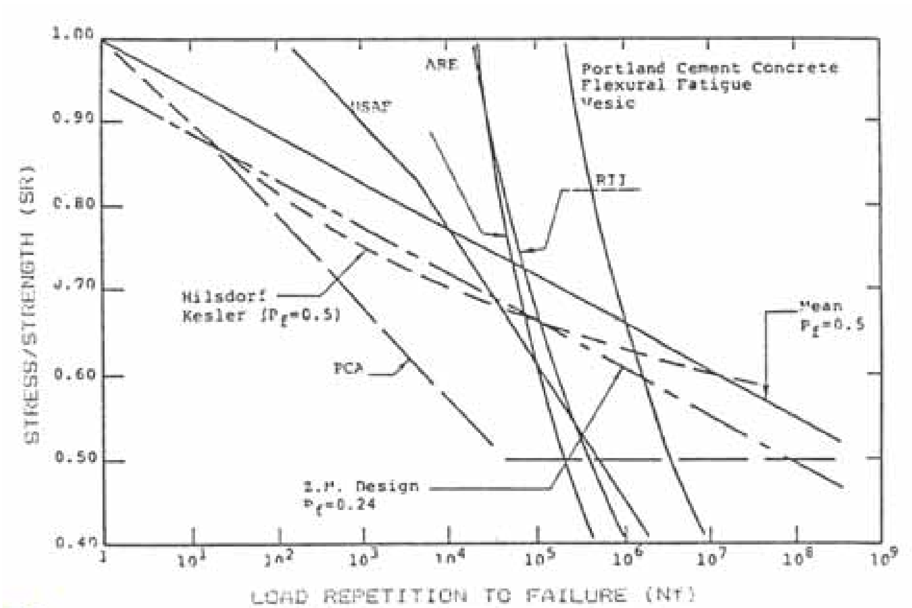


Figure 35. Examples of concrete fatigue relationships, used in concrete pavement design methods (33).

Generally spoken an analytical concrete pavement design procedure is preferable above an empirical design method, especially when there is a lack of experience with this type of pavement structure.

However, the quality of such an analytical design procedure is totally dependent on:

- the reliability of the values to be used for the various input parameters, such as the temperature gradient frequency distribution, the number of repetitions of every axle loading group, the concrete top layer characteristics (especially the flexural tensile strength and the fatigue relationship), the joint/crack (load transfer) characteristics and the substructure characteristics (especially the resistance to erosion and the modulus of substructure reaction)
- the quality of the theory or method used to calculate the flexural tensile stresses (and deflections) due to external loadings, such as traffic loadings, temperature gradients and unequal subgrade settlements; the finite element method is assumed to give the most accurate calculation results.

From the statements mentioned above it will be clear that it requires extensive material and analytical research to develop a sound analytical concrete pavement design method for local circumstances.

5. DUTCH DESIGN METHOD FOR CONCRETE PAVEMENTS

5.1 Introduction

In this chapter the development of the Dutch design method for concrete pavements will be described. The original design method, only for plain concrete pavements, was developed in the eighties by VNC (‘Vereniging Nederlandse Cementindustrie’, Cement Industry Association) and revised in the early nineties. In 2004 another revision was done but the design method was also extended for continuously reinforced concrete pavements; this current design method was published by CROW Technology Centre.

5.2 Original design method

The design model used in the original method for plain concrete pavements (11,36,37) is shown in figure 36. In the model two possibly critical locations of the most heavily loaded traffic lane are indicated:

- ZR = centre of the longitudinal edge of the concrete slab (free edge or longitudinal joint)
- VR = centre of the wheel track at the transverse joint with load transfer

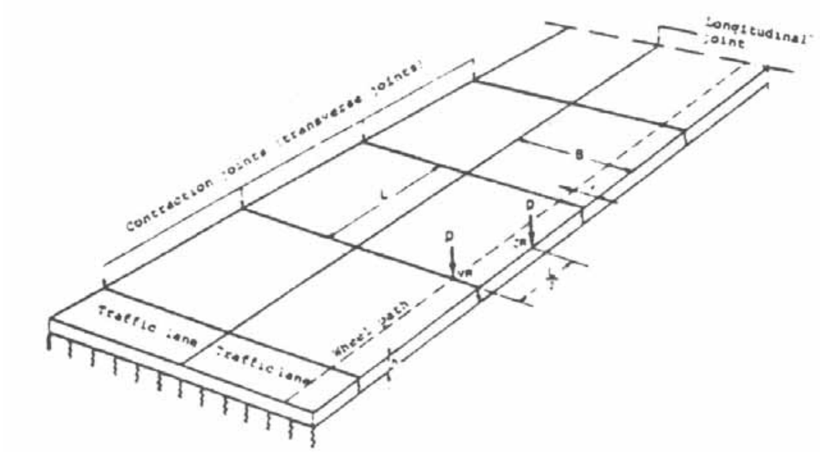


Figure 36. Design model for plain concrete pavements in the original method.

Two design criteria were used in the structural design of a plain concrete pavement:

1. a strength criterion, i.e. preventing the concrete pavement for cracking; the required thickness of the concrete pavement is found from a strength criterion which on one hand is determined by the occurring flexural tensile stresses under traffic and temperature gradient loadings and on the other hand by the fatigue strength of the concrete.
2. a stiffness criterion, i.e. preventing the development of longitudinal unevenness at the transverse joints (so-called joint-faulting): the required thickness of the concrete pavement is found from a stiffness criterion which on one hand is determined by the occurring deflection at the

transverse joints under traffic loading and on the other hand by the allowable deflection.

First of all a plain concrete pavement structure has to be assumed, which means that the length, width and thickness of the concrete slabs, the concrete quality, the type of joints, the thickness and modulus of elasticity of the base and sand sub-base, the modulus of subgrade reaction etc. have to be chosen. If it appears after the calculation that the assumed pavement structure does not fulfil the technical and/or economical requirements, then the analysis has to be repeated for a modified pavement structure. The flow diagram of the original design procedure is shown in figure 37.

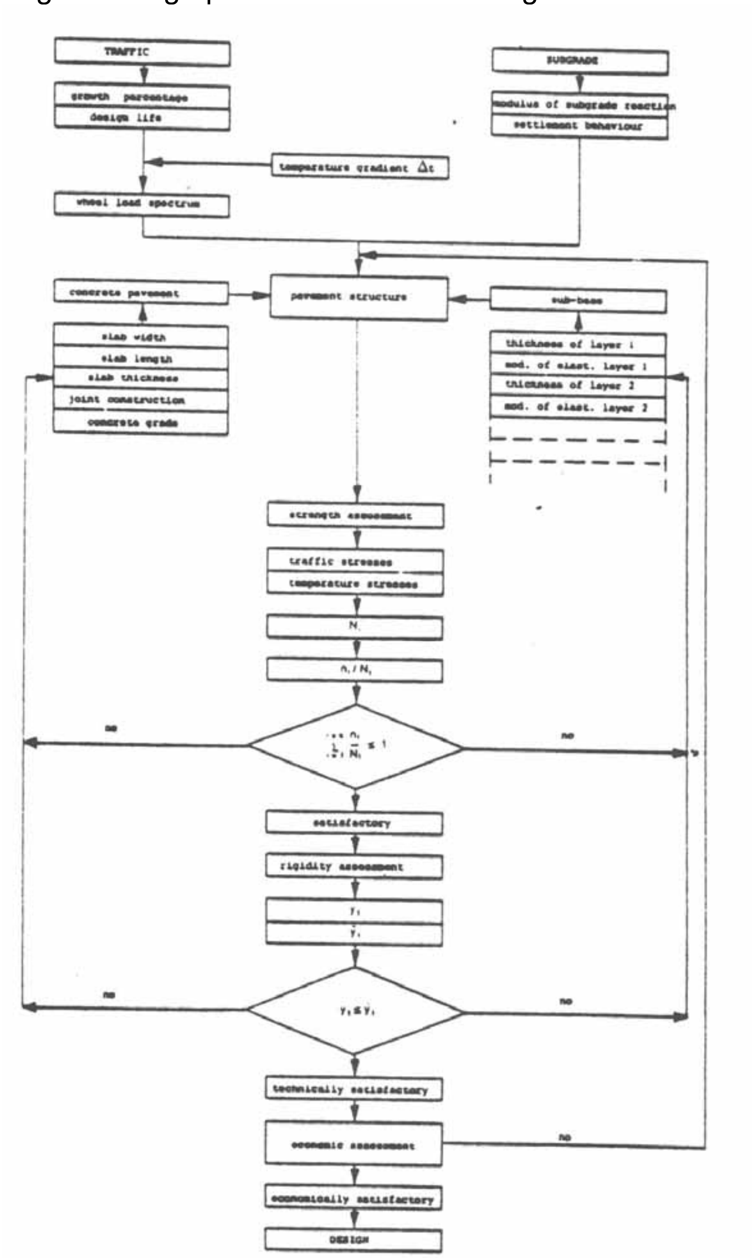


Figure 37. Flow diagram of the original design procedure for plain concrete pavements.

Only the technical aspects of the original design procedure will be briefly discussed here, the economical aspects will not be discussed.

First the cumulative number of heavy (truck) axle load repetitions on the most heavily loaded traffic lane (the design lane) during the desired pavement life (20 tot 40 years) has to be determined. This design traffic load is obtained in the following way:

a. average daily traffic in 2 directions during the design life: GEI vehicles
b. estimation of the maximum wheel load P_{max} (kN)
c. determination of the percentage of heavy traffic (Z_5); heavy traffic is defined as the total number of vehicles from the 5 heaviest wheel load groups, i.e. P_{max}, P_{max} – 10 kN, P_{max} – 20 kN, P_{max} – 30 kN and P_{max} – 40 kN; in general Z_5 is 5% to 8% of GEI, so the average daily number of heavy vehicles during the design life is $(Z_5/100)$.GEI
d. determination of average number of axles per heavy vehicle is GAA, where the following values serve as guidelines:
   heavy traffic with P_{max} = 90 kN: GAA = 3.2
   heavy traffic with P_{max} = 80 kN: GAA = 3.0
   heavy traffic with P_{max} = 70 kN: GAA = 2.8
   heavy traffic with P_{max} = 60 kN: GAA = 2.6
   heavy traffic with P_{max} = 50 kN: GAA = 2.4
e. the average daily number of heavy wheel loads in 2 directions (AWZ_5) is equal to: AWZ5 = $(Z_5/100)$.GEI.GAA
f. the total number of heavy wheel loads in 2 directions during the design life ($TAWZ_5$) is equal to: $TAWZ_5$ = AWZ_5.(design life in years).(number of working days per year)
g. by means of the directional factor vc the design traffic load (the total number of heavy wheel loads during the design life) for the design traffic lane (n) is obtained: n = vc.$TAWZ_5$
h. because of lateral wander the design traffic load in the wheel track ($n_{wheeltrack}$), point VR in figure 36, is taken as: $n_{wheeltrack}$ = 0.4 n, and the design traffic load in the centre of the longitudinal edge (n_{edge}), point ZR in figure 36, is taken as: n_{edge} = 0.1 n
i. a choice has to be made between two standard heavy wheel load frequency distributions (table 13); all wheels are assumed to have single tyres

Wheel load frequency distribution	P_{max}-40 kN (%)	P_{max}-30 kN (%)	P_{max}-20 kN (%)	P_{max}-10 kN (%)	P_{max} kN (%)	Total (%)
I	69	24	4	2.5	0.5	100
II	43	34	14	6.5	2.5	100

Table 13. Standard wheel load frequency distributions distinguished in the original design method.

The design temperature gradient Δt is taken as 0.05 ºC/mm. It is assumed that this temperature gradient is present in the plain concrete slab together with 15% of the design traffic load as mentioned under h ($n_{wheeltrack}$ and n_{edge} respectively).

Two design criteria are taken into account:

1. a strength (concrete fatigue) criterion for the centre of the longitudinal edge (point ZR in figure 36); the resulting loadings in this point are: in total 0.015.n heavy wheel loads (with wheel load frequency distribution according to table 13) together with a temperature gradient Δt = 0.05 ºC/mm
2. a stiffness (deflection) criterion for the centre of the wheel track at the transverse joint (point VR in figure 36); the relevant loading for this point is: in total 0.06.n heavy wheel loads (with wheel load frequency distribution according to table 13)

<u>Re. 1. Strength criterion for centre of longitudinal edge</u>

The tensile flexural stress at the bottom of the concrete slab in the centre of the longitudinal edge due to a wheel load is calculated with the original Westergaard equation for edge loading (equation 25):

$$\sigma = \frac{0.529\, P}{h^2} \left(1 + 0.54\, \upsilon\right) \left\{\log\left(\frac{E\, h^3}{k\, a_2^4}\right) - 0.71\right\} \qquad (48)$$

It is recalled that this original Westergaard equation for edge loading is not correct!

Both for a free edge and for a longitudinal contraction joint the load transfer W (paragraph 3.4.3, equations 39 and 40) is taken as 0%. This means that in equation 48 for P the values P_{max}, P_{max} – 10 kN, P_{max} – 20 kN, P_{max} – 30 kN and P_{max} – 40 kN have to be used for the 5 heaviest wheel load groups.

The tensile flexural stress at the bottom of the concrete slab in the centre of the longitudinal edge due to the temperature gradient Δt = 0.05 ºC/mm is calculated according to Eisenmann's theory (paragraph 3.3.3). The relevant equations from this theory are repeated here:

square slab: (0.8 ≤ L/W ≤ 1.2): l_{crit} = 228 h $\sqrt{E\, \alpha\, \Delta t}$ (49)

$$L' = L - \frac{2}{3} C \qquad (50)$$

$$C = 4.5 \sqrt{\frac{h}{k\, \Delta t}} \quad \text{if } C << L \qquad (51)$$

When the slab span L' is far smaller than the critical slab length l_{crit} (which is normally the case), then the flexural tensile stress is equal to:

$$\sigma = 0.85\ \sigma"_t = 0.85 \left(\frac{L'}{0.9\ l_{crit}}\right)^2 \sigma_t\ = 0.85 \left(\frac{L'}{0.9\ l_{crit}}\right)^2 \frac{1}{1-\upsilon}\frac{h\,\Delta t}{2}\alpha\ E \qquad (52)$$

In the original design method the following fatigue relationship is used for the plain concrete slab:

$$\log N_i = 12.6 * [1 - \frac{0.8\ (\sigma_{\max_i} - \sigma_{\min_i}) / f_{bk}}{0.8 - (\sigma_{\min_i} / f_{bk})}] \qquad (53)$$

where: N_i = allowable number of repetitions of wheel load P_i i.e. the traffic load stress σ_{vi} until failure when a temperature gradient stress σ_{ti} due to the temperature gradient Δt = 0.05 ºC/mm is present

σ_{mini} = minimum occurring flexural tensile stress (= σ_{ti})

σ_{maxi} = maximum occurring flexural tensile stress (= $\sigma_{vi} + \sigma_{ti}$)

f_{bk} = characteristic flexural tensile strength (N/mm^2) after 28 days of the concrete

The design criterion (i.e. cracking occurs) is the cumulative damage law of Palmgren-Miner:

$$\sum_i \frac{n_i}{N_i} = 1.0 \qquad (54)$$

where: n_i = occurring number of repetitions of wheel load P_i i.e. the traffic load stress σ_{vi} during the pavement life when a temperature gradient stress σ_{ti} due to the temperature gradient Δt = 0.05 ºC/mm is present

N_i = allowable number of repetitions of wheel load P_i i.e. the traffic load stress σ_{vi} until failure when a temperature gradient stress σ_{ti} due to the temperature gradient Δt = 0.05 ºC/mm is present

Re. 2. Stiffness criterion for centre of wheel track at transverse joint

To prevent longitudinal unevenness (joint faulting) at the transverse joints in the plain concrete pavement the deflection of the transverse edge in the wheel track (location VR in figure 36) due to the traffic loading should be limited. Taking into account the load transfer (joint efficiency W) at the transverse joint, according to Westergaard the deflection of the plain concrete slab at the transverse joint due to a wheel load P is (see equations 24 and 40):

$$w_l = \lambda \left(1 - \frac{W}{200}\right)\left(\frac{P}{kl^2}\right) \qquad (55)$$

where: w_l = deflection (mm) of the transverse edge of the loaded concrete slab

λ = parameter (-); in the case of one single wheel load on the concrete slab and a Poisson's ratio υ of concrete = 0.15 the λ-

value is 0.431
W = joint efficiency (%) with respect to deflections
P = wheel load (N); in the original method P = 50 kN = 50,000 N has to be used
k = modulus of substructure reaction (N/mm^3)
l = radius of relative stiffness (mm) of the concrete slab

The allowable deflection of the plain concrete slab at the transverse joint ($\overline{w_l}$, in mm) is dependent on the traffic loading:

$$\overline{w_l} = -0.3 \log N_{eq} + 2.7 \qquad \text{with} \quad \overline{w_l} \geq 0.3 \text{ mm} \tag{56}$$

where: N_{eq} = the total number of equivalent 50 kN wheel loads (= 100 kN axle loads) during the pavement life, calculated with the equation:

$$N_{eq} = \sum_i \left(P_i / 50\right)^4 n_i \tag{57}$$

where: P_i = wheel load (kN)
n_i = number of repetitions of wheel load P_i during the pavement life

The design criterion with respect to the stiffness of the plain concrete pavement is:

$$w_l \leq \overline{w_l} \tag{58}$$

For various combinations of concrete quality, plain concrete slab length, substructure condition, type of road and traffic loading, fatigue analyses were performed for the centre of the longitudinal edge of the concrete slab (similar to the calculation example in table 10) but using the equations 48 to 54. For each combination the result of the analysis was the required thickness of the concrete slab to obtain the desired pavement life.
Each pavement structure resulting from the fatigue analysis was checked for the stiffness criterion for the centre of the wheel track at the transverse joint, using the equations 55 to 58.
The results of all the calculations was a set of tables, giving the required thickness of the plain concrete pavement as a function of:
- the type of road (rural road, roads with 1 or 2 lanes, industrial roads, public transport bus lanes)
- the wheel load frequency distribution (spectrum I or II, see table 13)
- the maximum wheel load P_{max} (varying from 50 to 90 kN)
- the design traffic loading on the design traffic lane n (varying from 10^3 to 10^8)
- the concrete quality distinguished at that time (B30, B37.5 and B45)
- the length of the concrete slabs (varying from 3.5 to 5.0 m)
- the modulus of substructure reaction k (varying from 0.01 to 0.16 N/mm^3)

5.3 Revised design method

In the early nineties VNC ('Vereniging Nederlandse Cementindustrie', Cement Industry Association) decided to upgrade the original design method for plain concrete pavements. The methodology basically remained the same but the following major changes were included:

- new insights with respect to the behaviour of concrete
- new insights with respect to the calculation of traffic load and temperature gradient stresses in concrete slabs
- the strength criterion was applied to more potentially critical locations of the plain concrete slab

Due to developments with respect to computer hardware and software (MS-DOS), the revised design method for plain concrete pavements was released as the software package VENCON (38,39). This also enabled to make many input parameters a variable.
Following the changes in the revised design method are briefly explained.

The traffic loading is now expressed in terms of axle loadings (L) instead of wheel loadings (P), with L = 2P. The maximum axle load group is variable but should be related to the type of road. The highest axle load group that can be inputted is 180 – 200 kN (average 190 kN). Still only the 5 highest axle load groups are taken into account. The frequency distribution of these axle load groups is variable, although a default distribution is given (table 14).

Axle load group (kN)	Average axle load (kN)	Percentage heavy vehicles	
		range	average
80-100	90	100	100
100-120	110	15-25	20
120-140	130	8-15	12.5
140-160	150	5-10	7.5
160-180	170	2-5	3.5
180-200	190	1-2	1.5

Table 14. Indicative axle load frequency distribution.

Indicative values are also given for the average number of axles per heavy vehicle:
heavy traffic with maximum axle load L_{max} = 190 kN: GAA = 3.9
heavy traffic with maximum axle load L_{max} = 170 kN: GAA = 3.6
heavy traffic with maximum axle load L_{max} = 150 kN: GAA = 3.3
heavy traffic with maximum axle load L_{max} = 130 kN: GAA = 3.0
heavy traffic with maximum axle load L_{max} = 110 kN: GAA = 2.7
heavy traffic with maximum axle load L_{max} = 90 kN: GAA = 2.4

All the axles are provided wide single tyres. The radius of the circular contact area of a single tyre is calculated by means of the equation:

$$a = 10 \sqrt{(0.0014 * L + 51)} \quad (59)$$

where: a = radius of contact area (mm)

L = average axle load (N) of the axle load group

The total number of heavy axle loads in 2 directions during the design life is calculated in a similar way as in the original method, but now starting with the number of axle loads in the first year and next applying a (variable) growth factor. To find the design traffic loading on the design traffic lane not only the directional factor (vc) has to be applied but also a lane distribution factor (table 15).

Number of traffic lanes per direction	Percentage of trucks on design traffic lane
1	100
2	93
3	86
4	80

Table 15. Lane distribution factor.

With respect to the temperature gradient within the concrete slab the frequency distribution, given in table 16, was recommended.

Temperature gradient group (ºC/mm)	Average temperature gradient Δt (ºC/mm)	Percentage
0.000-0.005	0.00	71
0.005-0.015	0.01	17
0.015-0.025	0.02	6
0.025-0.035	0.03	3
0.035-0.045	0.04	2
0.045-0.055	0.05	1

Table 16. Recommended temperature gradient frequency distribution.

Two design criteria are again taken into account:

1. a strength (concrete fatigue) criterion, not only for the centre of the longitudinal free edge (point ZR in figure 36) but also for the centre of the longitudinal joint and for the centre of the wheel track at the transverse joint; for the centre of the longitudinal edge and longitudinal joint 5% - 15% of the heavy axle loads on the traffic lane are taken into account and for the centre of the wheel track at the transverse joint 40% - 50%; these traffic loadings, with their axle load frequency distribution, are combined with the temperature gradient frequency distribution
2. a stiffness (deflection) criterion for the centre of the wheel track at the transverse joint (point VR in figure 36); the number of heavy axle loads (with their axle load frequency distribution) at this point is 40% - 50% of the heavy axle loads on the traffic lane

<u>Re. 1. Strength criteria</u>

Around 1990 it had become clear that the original Westergaard equation, that was applied in the original design method, was incorrect (12,17,23,24,25,26).

Therefore in the revised method the new Westergaard equation for a circular tyre contact area (equation 27) is used to calculate the tensile flexural stress due to a wheel load P at the bottom of the concrete slab in the centre of the free edge:

$$\sigma = \frac{3(1+\upsilon)P}{\pi(3+\upsilon)h^2} \left\{ ln\left(\frac{E\,h^3}{100\,k\,a^4}\right) + 1.84 - \frac{4}{3}\upsilon + \frac{1-\upsilon}{2} + 1.18(1+2\upsilon)\frac{a}{l} \right\} \qquad (60)$$

For a <u>longitudinal free edge</u> the load transfer W (paragraph 3.4.3, equations 39 and 40) is taken as 0%.

For <u>longitudinal joints</u> the following values of the joint efficiency W are used:
- non-profiled construction joint with ty bars: W = 20%
- contraction joints with ty bars: W is calculated by means of the equation:

$$W = \{5.\log(k.l^2) – 0.0025.Wi – 25\}.\log N_{eq} – 20\log(k.l^2) + 0.01.Wi + 180 \qquad (61a)$$

For <u>transverse joints</u> the following values of the joint efficiency W are used:
- profiled construction joints or contraction joints, both without dowel bars: the joint efficiency W at long term is calculated by means of the equation:

$$W = \{5.\log(k.l^2) – 0.0025.L – 25\}.\log N_{eq} – 20\log(k.l^2) + 0.01.L + 180 \qquad (61b)$$

- profiled construction joints or contraction joints, both with dowel bars: the joint efficiency W at long term is calculated by means of the equation:

$$W = \{2.5.\log(k.l^2) - 17.5\}.\log N_{eq} - 10\log(k.l^2) + 160 \qquad (61c)$$

In the equations 61a, 61b and 61c is:
W = joint efficiency (%) at the end of the pavement life
L = slab length (mm)
Wi = slab width (mm)
k = modulus of substructure reaction (N/mm^3)
l = radius (mm) of relative stiffness of concrete layer (see paragraph 3.4.2)
N_{eq} = total number of equivalent 100 kN standard axle loads in the centre of the wheel track during the pavement life, calculated with a 4th power, i.e. the load equivalency factor $l_{eq} = (L/100)^4$ with axle load L in kN

The traffic load stress (σ_{joint}) at the bottom of the concrete slab in the centre of the longitudinal joint and in the wheel track at the transverse joint is calculated by means of the equation:

$$\sigma_{joint} = (1.01 – 0.01\, e^{0.03932.W})\,\sigma \qquad (62)$$

where: W = joint efficiency (%)
σ = flexural tensile stress at free edge (equation 60)

This approach means that in equation 60 the actual wheel loads (equal to halve of the actual axle loads) have to be inputted.

In the early nineties it also had become clear that Eisenmann's theory for calculation of the temperature gradient stresses did not yield the correct stresses which was especially the case for the (most important) greater temperature gradients (13,14). Therefore a new theory for the calculation of the temperature gradient stresses was developed (12). The most relevant equations of this theory (see paragraph 3.3.4) are repeated here.
The slab span in the longitudinal direction (L') and in the transverse direction (W') are equal to:

$$L' = L - \frac{2}{3} C \tag{63a}$$

$$W' = W - \frac{2}{3} C \tag{63b}$$

where: L' = slab span (mm) in longitudinal direction
W' = slab span (mm) in transverse direction
C = support length (mm), that is equal to:

$$C = 4.5 \sqrt{\frac{h}{k \ \Delta t}} \quad \text{if } C << L \tag{64}$$

In the case of a small positive temperature gradient Δt the flexural tensile stress σ_t at the bottom of the concrete slab in the center of a slab edge is equal to:

$$\sigma_t = \frac{h \cdot \Delta t}{2} \alpha \ E \tag{65}$$

where: h = thickness (mm) of the concrete slab
Δt = small positive temperature gradient (°C/mm)
α = coefficient of linear thermal expansion ($°C^{-1}$)
E = Young's modulus of elasticity (N/mm^2) of concrete

In the case of a great positive temperature gradient Δt the flexural tensile stress σ_t at the bottom of the concrete slab in the center of a slab edge is equal to:

longitudinal edge: $$\sigma_t = 1.8 * 10^{-5} \quad L'^2 / h \tag{66a}$$

transverse edge: $$\sigma_t = 1.8 * 10^{-5} \quad W'^2 / h \tag{66b}$$

The tensile flexural stress at the bottom of the concrete slab in the centre of a longitudinal edge or joint is the smallest value resulting from the equations 65 and 66a.

The tensile flexural stress at the bottom of the concrete slab in the centre of a transverse joint is the smallest value resulting from the equations 65 and 66b. To obtain the reduced temperature gradient stress in the centre of the wheel track at the transverse joint σ_t (calculated for the centre of the transverse joint) is multiplied by means of the reduction factor R (equation 20b):

transverse edge: $$R = \frac{4\,y\,(W' - y)}{W'^2} \qquad (67)$$

where: W' = slab span (mm) in the transverse direction
y = distance (mm) of the centre of the wheel track at the transverse joint to the nearest longitudinal edge minus 1/3 of the support length C

In the revised design method the following fatigue relationship is used for the plain concrete slab:

$$\log N_i = \frac{16.8\,(0.9 - (\sigma_{max_i} / f_{btg}))}{1.0667 - (\sigma_{min_i} / f_{btg})}\,] \qquad (68)$$

where: N_i = allowable number of repetitions of axle load L_i (= wheel load P_i) i.e. the traffic load stress σ_{vi} until failure when a temperature gradient stress σ_{ti} due to the temperature gradient Δt is present
σ_{mini} = minimum occurring flexural tensile stress (= σ_{ti})
σ_{maxi} = maximum occurring flexural tensile stress (= $\sigma_{vi} + \sigma_{ti}$)
f_{btg} = average flexural tensile strength (N/mm^2) after 90 days of the concrete:

$$f_{btg} = 1.4\,(1.6 - h/1000)\,(1.05 + 0.05\,f_{cc,k,o}) \qquad (69)$$

where: h = thickness (mm) of the concrete slab
$f_{cc,k,o}$ = characteristic cube compressive strength (N/mm^2) after 28 days (see table 3)

The design criterion (i.e. cracking occurs) again is the cumulative damage law of Palmgren-Miner:

$$\sum_i \frac{n_i}{N_i} = 1.0 \qquad (70)$$

where: n_i = occurring number of repetitions of axle load L_i (= wheel load P_i) i.e. the traffic load stress σ_{vi} during the pavement life when a temperature gradient stress σ_{ti} due to the temperature gradient Δt is present
N_i = allowable number of repetitions of axle load L_i (= wheel load P_i) i.e. the traffic load stress σ_{vi} until failure when a temperature gradient stress σ_{ti} due to the temperature gradient Δt is present

Re. 2. Stiffness criterion

Also the revised design method contains a stiffness criterion to prevent longitudinal unevenness (joint faulting) at the transverse joints in the plain concrete pavement. The deflection of the transverse edge in the centre of the wheel track (location VR in figure 36) due to the traffic loading should be limited. The occurring deflection is calculated by means of equation 71, which differs in two respects from the equation 55 included in the original design method:

- equation 71 takes into account the deflection caused by the other single tyre on the axis by means of the parameter Φ
- equation 71 takes into account that the occurring deflection not only depends on the magnitude of the axle (wheel) load but also on the number of heavy wheel loads through the parameter N_{eq}

$$w_l = 0.431.(1+\phi)\left(1-\frac{W}{200}\right)\left(\frac{L}{2kl^2}\right)(\frac{1}{8}.\log(N_{eq})+0.5) \quad (71)$$

where: w_l = deflection (mm) of the transverse edge of the loaded concrete slab

W = joint efficiency (%) with respect to deflections

L = axle load (N); in the revised method L = 100 kN = 100,000 N has to be used

k = modulus of substructure reaction (N/mm^3)

l = radius of relative stiffness (mm) of the concrete slab

N_{eq} = the total number of equivalent 100 kN axle loads (= 50 kN wheel loads) in the centre of the wheel track during the pavement life, calculated with the equation:

$$N_{eq} = \sum_i \left(L_i/100\right)^4 n_i \quad (72)$$

where: L_i = wheel load (kN)

n_i = number of repetitions of axle load L_i in the centre of the wheel track during the pavement life

Φ = parameter to include the effect of the other tyre on the axis:

$$\ln\phi = -0.02723(z/l)^3 + 0.04240(z/l)^2 - 0.6754(z/l) + 0.2570 \quad (73)$$

where: z = 1900 mm (= distance between the two single tyres on the axis)

$l = \sqrt[4]{\frac{E h^3}{12(1-\upsilon^2)k}}$ = radius (mm) of relative stiffness of the concrete layer

The allowable deflection of the plain concrete slab at the transverse joint ($\overline{w_l}$, in mm) is dependent on the traffic loading:

$$\overline{w_l} = 4.8\ e^{-0.35.\log Neq} \qquad (74)$$

The design criterion with respect to the stiffness of the plain concrete pavement is:

$$w_l \leq \overline{w_l} \qquad (75)$$

5.4 Current design method

5.4.1 Introduction

Around the year 2000 a further upgrade of the revised design method for plain concrete pavements was felt necessary. The reasons for this further upgrade were:

- new insights with respect to the behaviour of concrete
- new insights with respect to the calculation of traffic load stresses in concrete slabs
- availability of new measuring data with respect to types of tyres used on trucks, axle load frequency distributions and temperature gradient frequency distribution
- the change of pc software (Windows instead of MS-DOS) and availability of more powerful pc hardware

These new insights and data were included in the current design method that was released as the Windows software package VENCON2.0 (5). Because of the increased application of reinforced concrete pavements (with a porous asphalt wearing course) on motorways, VENCON2.0 is not limited to plain concrete pavements but it also includes continuously reinforced concrete pavements.
The VENCON2.0 has three user levels, i.e. junior, senior and expert. For the junior user only a few parameters are variable, on the other hand the expert user has the possibility to input the value of many parameters.

For plain concrete pavements the structural design methodology did not essentially change compared to the original and revised design method. The concrete strength criterion is however applied to more potentially critical locations of the plain concrete slab.
For the determination of the thickness of continuously reinforced concrete pavements the pavement is first considered as a plain concrete pavement, however with modified horizontal ‘slab’ dimensions (because of the presence of transverse cracks instead of transverse joints) and a great load transfer at the transverse cracks (that are very narrow). Having found the concrete thickness, based on the concrete strength criterion, the required longitudinal reinforcement (to control the crack pattern) is determined.
The stiffness criterion for the transverse joint or crack is not included anymore as experience has learned that in virtually all cases the strength criterion is dominant over the stiffness criterion.

Figure 38 gives an overview of the input and calculation procedure of the current Dutch design method. The items 1 to 11 mentioned in figure 38 are subsequently discussed in the next paragraphs.

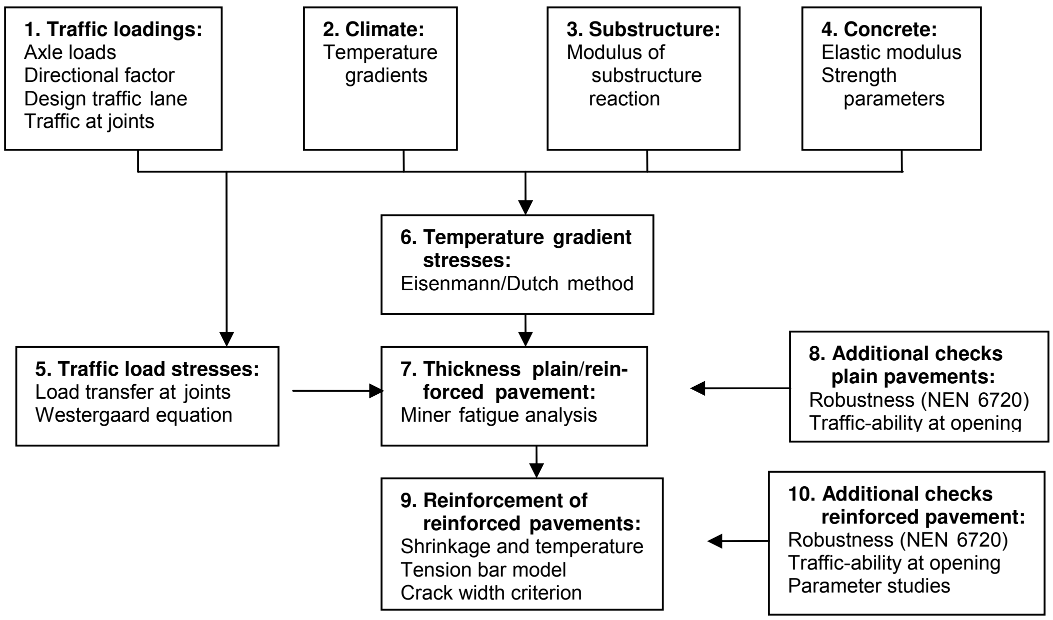


Figure 38. Flow chart of the structural design of plain/reinforced concrete pavements according to the current Dutch design method.

5.4.2 Traffic loadings

The traffic loading is calculated as the total number of axles per axle load group (> 20 kN, so only heavy vehicles such as trucks and busses are taken into account) on the design traffic lane during the desired life of the concrete pavement. These numbers are calculated from:

- the desired pavement life in years (usually taken as 20 to 40)
- the number of working days per year (usually taken as 260 to 300)
- the number of heavy vehicles per working day in the first year on the road (both directions together)
- the division of the heavy vehicles per direction; for roads having one carriageway the directional factor depends on the width of the carriageway, for roads having two carriageways the directional factor is taken as 0.5 (table 17)
- in the case that there is more than 1 traffic lane per direction: the percentage of the heavy vehicles on the most heavily loaded lane (the design traffic lane) (table 18)
- the average number of axles per heavy vehicle (table 19)
- the average percentage of growth of the heavy traffic over the desired pavement life (usually taken as 0% to 5%)

Number of carriageways	Width of carriageway (m)	Directional factor
1	3.0	1.00
	3.5	0.95
	4.0	0.85
	4.5	0.70
	≥ 5.0	0.50
2	-	0.50

Table 17. Directional factor as a function of the number and the width of carriageways of the road.

Number of traffic lanes per direction	Percentage of heavy vehicles on design traffic lane
1	100
2	93
3	86
4	80

Table 18. Percentage of heavy vehicles on design traffic lane as a function of the number of traffic lanes per direction.

Axle load group (kN)	Average axle load (kN)	Axle load frequency distribution (%) for different types of road						
		heavily loaded motorway	normally loaded motorway	heavily loaded provincial road	normally loaded provincial road	municipal main road	rural road	public transport bus lane
20-40	30	20.16	14.84	26.62	24.84	8.67	49.38	-
40-60	50	30.56	29.54	32.22	32.45	40.71	25.97	-
60-80	70	26.06	30.22	18.92	21.36	25.97	13.66	-
80-100	90	12.54	13.49	9.46	11.12	13.66	8.05	-
100-120	110	6.51	7.91	6.50	6.48	8.05	2.18	100
120-140	130	2.71	3.31	4.29	2.70	2.18	0.38	-
140-160	150	1.00	0.59	1.64	0.83	0.38	0.38	-
160-180	170	0.31	0.09	0.26	0.19	0.38	0.00	-
180-200	190	0.12	0.01	0.06	0.03	0.00	0.00	-
200-220	210	0.03	0.01	0.03	0.00	0.00	0.00	-
Average number of axles per heavy vehicle		3.5	3.5	3.5	3.5	3.5	3.1	2.5

Table 19. Default axle load frequency distributions for different types of roads.

In the case that no real axle load data are available, for a certain type of road the default axle load frequency distribution given in table 19 can be used. These frequency distributions are based on axle load measurements in the years 2000 and 2001 on a great number of provincial roads in the Netherlands. In comparison to the revised method (table 14) the axle load group 200-220 kN has been added to the axle load spectrum for main roads.
In contrast to the revised method (where only the 5 heaviest axle load groups were taken into account) in the current method all the truck axles are considered.

The legal single axle load limit in the Netherlands is 115 kN for driven axles (with dual tyres) and 100 kN for non-driven axles. The legal axle load limit for a dual and triple axle system (two or three axles rather close to each other) is dependent on the distance between the axles; if the distance between two adjacent axles of a tandem or triple axle system is 1.8 m or more, then the legal limit is 100 kN per individual axle, and if the distance between two adjacent axles is smaller than 1.8 m the legal axle load limit is smaller than 100 kN.
Table 19 makes clear that also in The Netherlands there are quite some overloaded axles and these really should be taken into account when designing a concrete pavement.

Both in the original and in the revised design method all the axles were provided wide single tyres. In the current design method, however, different types of tyre are included:

- single tyres, that are mounted at front axles of heavy vehicles
- dual tyres, that are mounted at driven axles, and sometimes at trailer axles
- wide base tyres, that are mostly mounted at trailer axles
- extra wide wide base tyres, that in future will be allowed for driven axles

Every tyre contact area is assumed to be rectangular. In the Westergaard equation for calculation of the traffic load stresses however a circular contact area is assumed. The equivalent radius of the circular contact area of a tyre is calculated by means of equation 76 (compare equation 59):

$$a = b \sqrt{(0.0014 * L + 51)} \quad (76)$$

where: a = equivalent radius of circular contact area (mm)
b = parameter dependent on type of tyre (table 20)
L = average axle load (N) of the axle load group

Type of tyre	Width of rectangular contact area(s) (mm)	Value of parameter b of equation 76
Single tyre	200	9.2
Dual tyre	200-100-200	12.4
Wide base tyre	300	8.7
Extra wide wide base tyre	400	9.1

Table 20. Value of parameter b (equation 76) for different types of tyre.

For the type of tyre also some default frequency distributions are included in the current design method (table 21).

Type of tyre	Frequency distribution (%)	
	roads	public transport bus lanes
Single tyre	39	50
Dual tyre	38	50
Wide base tyre	23	0
Extra wide wide base tyre	0	0

Table 21. Default tyre type frequency distributions for different types of roads.

5.4.3 Climate

In the years 2000 and 2001 the temperature gradient has continuously been measured on the newly build stretch Lunetten-Bunnik on the motorway A12 near Utrecht in the Netherlands. The continuously reinforced concrete pavement has a thickness of 250 mm and the measurements were done before the porous asphalt wearing course was constructed. Based on these measurements it was decided to include the default temperature gradient frequency distribution shown in table 22 (compare table 16) in the current design method.

Temperature gradient class (ºC/mm)	Average temperature gradient Δt (ºC/mm)	Frequency distribution (%)
0.000 – 0.005	0.0025	59
0.005 – 0.015	0.01	22
0.015 – 0.025	0.02	7.5
0.025 – 0.035	0.03	5.5
0.035 – 0.045	0.04	4.5
0.045 – 0.055	0.05	1.0
0.055 – 0.065	0.06	0.5

Table 22. Default temperature gradient frequency distribution.

5.4.4 Substructure

The modulus of substructure reaction k is calculated in the way as already explained in section 3.2:

$$k= 2.7145.10^{-4} (C_1 + C_2.e^{C3} + C4.e^{C5}) \quad (77)$$

with: $C_1 = 30 + 3360.k_o$

$C_2 = 0.3778 (h_f – 43.2)$

$C_3 = 0.5654 \ln(k_o) + 0.4139 \ln(E_f)$

$C_4 = -283$

$C_5 = 0.5654 \ln(k_o)$

k_o = modulus of subgrade/substructure reaction at top of underlying layer (N/mm^3)

h_f = thickness of layer under consideration (mm)

E_f = dynamic modulus of elasticity of layer under consideration (N/mm^2)

k = modulus of substructure reaction at top of layer under consideration (N/mm^3)

The boundary conditions for equation 77 are:

- $h_f \geq 150$ mm (bound material) and $h_f \geq 200$ mm (unbound material)
- every layer under consideration has an E_f-value that is greater than the E_f-value of the underlying layer
- $\log k \leq 0.73688 \log(E_f) – 2.82055$
- $k \leq 0.16$ N/mm^3

5.4.5 Concrete

The relevant concrete properties for the thickness design of the plain or reinforced concrete pavement are the stiffness (Young's modulus of elasticity) and the flexural tensile strength.

Young's modulus of elasticity E_c is calculated with the equation (eq. 8):

$$E_c = 22250 + 250 \cdot f_{cc,k,o} \quad (N/mm^2) \quad \text{with } 15 \leq f_{cc,k,o} \leq 65 \tag{78}$$

where: $f_{cc,k,o}$ = characteristic cube compressive strength (N/mm^2) after 28 days for loading of short duration (table 3)

The concrete strength parameter included in the current Dutch design method is the mean flexural tensile strength ($f_{ct,fl,o}$), that is a function of the thickness h (in mm) of the concrete slab and the characteristic cube compressive strength $f_{cc,k,o}$ (eq. 6):

$$f_{ct,fl,o} = 1.3 \, [(1600 - h)/1000)] \, [1.05 + 0.05 \, (f_{cc,k,o} + 8)]/1.2 \quad (N/mm^2) \tag{79}$$

5.4.6 Traffic load stresses

As already explained in section 3.4.3 the load transfer W in joints/cracks can be incorporated in the design of concrete pavement structures by means of a reduction of the actual wheel load P_{act} to the wheel load P (to be used in the Westergaard equation) according to:

$$P = \left(1 - 1/2 \, W/100\right) P_{act} = \left(1 - \frac{W}{200}\right) P_{act} \tag{80}$$

In the current design method the following values for the load transfer W are included:

- free edge of a plain or reinforced concrete pavement (at the outer side of the carriageway):
  - W = 20% in the case that a unbound base is applied below the concrete pavement
  - W = 35% in the case that a bound base is applied or W = 70% in the case that a widened bound base is applied
- longitudinal joints in plain or reinforced concrete pavements:
  - W = 20% at non-profiled construction joints without ty bars in plain concrete pavements on a unbound base
  - W = 35% at non-profiled construction joints without ty bars in plain concrete pavements on a bound base
  - W = 70% at contraction joints with ty bars in plain concrete pavements
  - W according to equation 81a at profiled construction joints in plain concrete pavements
  - W = 35% at joints in reinforced concrete pavement (where always an asphalt layer is applied below the concrete pavement)
- transverse joints/cracks in plain or reinforced concrete pavements:
  - W = 90% at cracks in reinforced concrete pavement

- W according to equation 81a at profiled construction joints or contraction joints, both without dowel bars, in plain concrete pavements:

  $W = \{5.\log(k.l^2)–0.0025.L–25\}.\log N_{eq}–20 \log(k.l^2)+0.01.L+180$ (81a)

- W according to equation 81b at profiled construction joints or contraction joints, both with dowel bars, in plain concrete pavements:

  $W = \{2.5.\log(k.l^2)-17.5\}.\log N_{eq}-10\log(k.l^2)+160$ (81b)

In the equations 81a and 81b is:
W = joint efficiency (%) at the end of the pavement life
L = slab length (mm)
k = modulus of substructure reaction (N/mm^3)
l = radius (mm) of relative stiffness of concrete layer (see paragraph 3.4.2)
N_{eq} = total number of equivalent 100 kN standard axle loads in the centre of the wheel track during the pavement life, calculated with a 4th power, i.e. the load equivalency factor $l_{eq} = (L/100)^4$ with axle load L in kN

Similar to the revised method also in the current design method the new Westergaard equation for a circular tyre contact area (equation 27) is used to calculate the tensile flexural stress due to a wheel load P at the bottom of the concrete slab along a free edge, along a longitudinal joint, along a transverse joint (plain concrete pavements) and along a transverse crack (continuously reinforced concrete pavement):

$$\sigma = \frac{3(1+\upsilon)P}{\pi(3+\upsilon)h^2} \left\{ ln\left(\frac{E\,h^3}{100\,k\,a^4}\right) + 1.84 - \frac{4}{3}\upsilon + \frac{1-\upsilon}{2} + 1.18(1+2\upsilon)\frac{a}{l} \right\} \quad (82)$$

where:
σ = flexural tensile stress (N/mm^2)
P = wheel load (N), taking into account the load transfer (equation 80)
a = equivalent radius (mm) of circular contact area (equation 76 and table 20)
E = Young's modulus of elasticity (N/mm^2) of concrete (equation 78)
υ = Poisson's ratio of concrete (usually taken as 0.15)
h = thickness (mm) of concrete layer
k = modulus of substructure reaction (N/mm^3) (equation 77)
l = $\sqrt[4]{\frac{E\,h^3}{12(1-\upsilon^2)k}}$ = radius (mm) of relative stiffness ofconcrete layer

5.4.7 Temperature gradient stresses

In the current design method the calculation of the stresses resulting from positive temperature gradients is done in the same way as in the revised method (section 5.3). The most relevant equations of this theory are repeated here.

The slab span in the longitudinal direction (L’) and in the transverse direction (W’) are equal to:

$$L^{'} = L - \frac{2}{3} C \qquad (83a)$$

$$W^{'} = W - \frac{2}{3} C \qquad (83b)$$

where: L’ = slab span (mm) in longitudinal direction
W’ = slab span (mm) in transverse direction
C = support length (mm), that is equal to:

$$C = 4.5 \sqrt{\frac{h}{k \ \Delta t}} \quad \text{if } C << L \qquad (84)$$

In the case of a small positive temperature gradient Δt the flexural tensile stress σ_t at the bottom of the concrete slab along the edge, joint or crack is equal to:

$$\sigma_t = \frac{h \cdot \Delta t}{2} \alpha \ E \qquad (85)$$

where: h = thickness (mm) of the concrete slab
Δt = small positive temperature gradient (°C/mm) (table 22)
α = coefficient of linear thermal expansion ($°C^{-1}$) (usually taken as 1.10^{-5})
E = Young’s modulus of elasticity (N/mm^2) of concrete (equation 78)

In the case of a great positive temperature gradient Δt the flexural tensile stress σ_t at the bottom of the concrete slab along the edge, joint or crack is equal to:

longitudinal edge: $\sigma_t = 1.8*10^{-5} \quad L^{'2} / h$ (86a)

transverse edge: $\sigma_t = 1.8*10^{-5} \quad W^{'2} / h$ (86b)

The flexural tensile stress at the bottom of the concrete slab along the edge, joint or crack is the smallest value resulting from the equations 85 and 86a (longitudinal edges and joints) or the equations 85 and 86b (transverse joints and cracks).
A reduction of the temperature gradient stress in the wheel track at the transverse joint/crack by means of the reduction factor R (equation 20b) is not applied anymore. This means that the temperature gradient stresses, calculated by means of the equations 83 to 86 for the centre of the edge/joint/crack, are taken for every location at the edge/joint/crack.

5.4.8 Thickness plain/reinforced concrete pavement

In the case of <u>plain concrete pavements</u> on a 2-lane road the strength analysis is carried out for the following locations of the design concrete slab:

- the wheel load just along the free edge of the slab
- the wheel load just along the longitudinal joint between the traffic lanes
- the wheel load just before the transverse joint

In the case of a multi-lane road (e.g. a motorway) in addition the strength analysis is also done for:

- the wheel load just along every longitudinal joint between the traffic lanes
- the wheel load just along the longitudinal joint between the entry or exit lane and the adjacent lane

In the case of <u>continuously reinforced concrete pavements</u> the strength analysis is done for two locations of the design concrete 'slab':

- the wheel load just before a transverse crack
- the wheel load just along a longitudinal joint

Both for plain and reinforced concrete pavements the flexural tensile stress (σ_{vi}) at the bottom of the concrete slab due to the wheel load (P_i) in each of the mentioned locations is calculated by means of the Westergaard equation (eq. 82), taking into account the appropriate load transfer (joint efficiency W, equations 80 and 81) in the respective joints/cracks.

Both for plain and reinforced concrete pavements the flexural tensile stress (σ_{ti}) at the bottom of the concrete slab due to a positive temperature gradient (Δt_i) in each of the mentioned locations is calculated by means of the equations 83 to 86.
In the case of plain concrete pavements the horizontal slab dimensions (length L, width W) are determined beforehand.
In the case of reinforced concrete pavements the width W of the 'slab' is determined beforehand (distance between free edge and adjacent longitudinal joint or distance between two longitudinal joints), the length L of the 'slab' is arbitrarily taken as 1.35*W, with L ≤ 4.5 m.

Both for plain and reinforced concrete pavements the structural design is based on a fatigue analysis for all the mentioned locations of the pavement. The following fatigue relationship is used (eq. 7):

$$\log N_i = \frac{12.903\,(0.995 - \sigma_{\max_i} / f_{ct,fl,o})}{1.000 - 0.7525\,\sigma_{\min_i} / f_{ct,fl,o}} \quad with\ 0.5 \le \sigma_{\max} / f_{ct,fl,o} \le 0.833 \tag{87}$$

where: N_i = allowable number of repetitions of wheel load P_i i.e. the traffic load stress σ_{vi} until failure when a temperature gradient stress σ_{ti} is present

σ_{mini} = minimum occurring flexural tensile stress (= σ_{ti})

σ_{maxi} = maximum occurring flexural tensile stress (= σ_{vi} + σ_{ti})

$f_{ct,fl,o}$ = mean flexural tensile strength (N/mm^2) after 28 days for loading of short duration

The design criterion (i.e. cracking occurs), applied on every of the above-mentioned locations of the plain or reinforced concrete pavement, again is the cumulative fatigue damage law of Palmgren-Miner:

$$\sum_i \frac{n_i}{N_i} = 1.0 \qquad (88)$$

where: n_i = occurring number of repetitions of axle load L_i (= wheel load P_i) i.e. the traffic load stress σ_{vi} during the pavement life when a temperature gradient stress σ_{ti} due to the temperature gradient Δt_i is present

N_i = allowable number of repetitions of axle load L_i (= wheel load P_i) i.e. the traffic load stress σ_{vi} until failure when a temperature gradient stress σ_{ti} due to the temperature gradient Δt_i is present

The determination of the occurring number of load repetitions on the design traffic lane has been explained earlier (see also tables 17 to 19). In a similar way the occurring number of traffic loads on the other traffic lanes can be determined or estimated.
Lateral wander within a traffic lane is not taken into account when analyzing a transverse joint or crack.
When analyzing a longitudinal free edge or longitudinal joint the number of traffic loads just along the edge or joint is limited to 1%-3% (free edge) or 5%-10% (every longitudinal joint) of the occurring total number of traffic loads on the carriageway (so not the design traffic lane).

5.4.9 Additional checks plain concrete pavements

After the determination of the thickness of the plain concrete pavement two additional checks can be performed:
1. check for robustness ('unity check')
2. check for the traffic-ability at opening of the pavement

Re. 1. Check for robustness
The check for robustness ('unity check') is done according to NEN 6720 (1) for the ultimate loading condition. For the loading case 'slab edge' it is controlled whether the flexural tensile stress due to the ultimate wheel load (P_{UWL}), taking into account the load transfer W at the longitudinal edge (equation 40), remains below the flexural tensile strength for loading of long duration.
P_{UWL} is calculated as:

$$P_{UWL} = \left(1 - \frac{W}{200}\right) P_{act} \,.\, y_l \,.\, DLF \qquad (89)$$

where: P_{UWL} = ultimate wheel load (kN), to be used for calculation of the flexural tensile stress σ_{UWL}

P_{act} = actual ultimate wheel load (kN)

W = load transfer at longitudinal edge (%)
γ_l = 1.2 is load facor on the wheel load (40)
DLF = Dynamic Load Factor (40)

The flexural tensile stress σ_{UWL}, calculated by means of Westergaard (equation 82), due to the ultimate wheel load P_{UWL} should be smaller than the mean flexural tensile strength $f_{ct,fl,\infty}$ for loadings of long duration (see equation 4):

$\sigma_{UWL} \leq f_{ct,fl,\infty} \quad \rightarrow$

$\sigma_{UWL} \leq [(1600 - h)/1000)] \,.\, f_{ct,d,\infty} \quad \rightarrow$
$\sigma_{UWL} \leq [(1600 - h)/1000)] \,.\, 0.9 \,.\, [1.05 + 0.05\,(f_{cc,k,o} + 8)]/1.2 \quad (N/mm^2)$ (90)

where: h = thickness of plain concrete pavement (mm)
$f_{ct,d,\infty}$ = mean tensile strength for loading of long duration (N/mm^2)
$f_{cc,k,o}$ = characteristic (95% probability of exceeding) cube compressive strength after 28 days for loading of short duration (N/mm^2), see table 2

Re. 2. Check for traffic-ability at opening of pavement
The check for the traffic-ability concerns the determination of that wheel load that does not cause any fatigue damage at the opening of the plain concrete pavement. The calculation is done for the loading case ‘transverse joint’, and takes into account the temperature gradient stress present at the transverse joint.
For the strength the mean flexural tensile strength for loadings of short duration ($f_{ct,fl,o}$) is used (equation 6), however taking into account the fatigue factor λ:

$f_{ct,fl,o} = [(1600 - h)/1000)]\; f_{ct,d,o} / \lambda =$

$= 1.3\; [(1600 - h)/1000)]\; [1.05 + 0.05\,(f_{cc,k,o} + 8)]/(1.2 \,.\, 1.4) \approx$

$\approx 0.9\; [(1600 - h)/1000)]\; [1.05 + 0.05\,(f_{cc,k,o} + 8)]/1.2 \quad (N/mm^2)$ (91)

where: h = thickness of plain concrete pavement (mm)
$f_{ct,d,o}$ = mean tensile strength for loading of short duration (N/mm^2)
$f_{cc,k,o}$ = characteristic (95% probability of exceeding) cube compressive strength after 28 days for loading of short duration (N/mm^2), see table 2
λ = 1.4 is fatigue factor for concrete under tension, subjected to more than 2.10^6 load cycles (40)

Taking into account the temperature gradient stress σ_t the allowable traffic load stress σ_v is equal to:

$$\sigma_v = (f_{ct,fl,o} - \sigma_t) / (1.2 \,.\, \text{safety factor} \,.\, \left(1 - \frac{W}{200}\right)) \quad (92)$$

where: $f_{ct,fl,o}$ = mean flexural tensile strength for loadings of short duration (N/mm^2), equation 91
σ_t = temperature gradient stress (N/mm^2), calculated with the equations 83 to 86
W = load transfer at transverse joint (%)

On basis of the traffic load stress σ_v the allowable wheel load P is calculated by means of Westergaard (equation 82).

5.4.10 Determination of the reinforcement of reinforced concrete pavements

Initially a reinforced concrete pavement is indefinitely long. Therefore the pavement is always subjected to imposed shrinkage of the hardening concrete, sometimes in combination with temperature changes of the (partially) hardened concrete. It is assumed that the strains and curvatures resulting from shrinkage (plus temperature changes) are totally obstructed because of the friction of the concrete pavement with the underlying base layer.

<u>Shrinkage</u>

The value of the specific shrinkage strain ε_r is calculated according to NEN 6720 (1).

In the calculation of the occurring shrinkage strains a linear development over the thickness (h) of the concrete pavement is assumed. The occurring strains are:
- at the top: $\varepsilon_{rb} \leq \varepsilon_r$
- at the bottom: $\varepsilon_{ro} \leq \varepsilon_r$

The boundary condition is that the occurring shrinkage strain at the top of the concrete pavement (ε_{rb}) is greater than the occurring shrinkage strain at the bottom of the concrete pavement (ε_{ro}): $\varepsilon_{rb} \geq \varepsilon_{ro}$.
The imposed deformations due to shrinkage of the hardening concrete thus are:
- average shrinkage strain: $\varepsilon_{rm} = (\varepsilon_{rb} + \varepsilon_{ro}) / 2$ (93)
- negative curvature: $\kappa_r = (\varepsilon_{rb} - \varepsilon_{ro}) / h$ (94)

A shrinkage has to be input as a negative value, so in general will be valid $\varepsilon_r < 0$ and $\kappa_r < 0$.

Before cracking of the reinforced concrete pavement the average shrinkage strain ε_{rm} results in a normal tension force N_r and the negative curvature κ_r results in a bending moment M_r.

<u>Temperature changes</u>

In the reinforced concrete pavement stresses occur due to actual temperatures (T_b at the top and T_o at the bottom of the pavement) that are different from the temperature distribution for which the concrete slab is free of stress. The stress-free temperature distribution is schematized by means of a ‘reference temperature’ at the top (T_{rb}) and at the bottom (T_{ro}) of the concrete slab. The temperature differences in the concrete slab are thus:

- at the top: $\delta T_b = T_b - T_{rb}$
- at the bottom: $\delta T_o = T_o - T_{ro}$

The occurring strains are:

- at the top: $\varepsilon_{Tb} = \alpha \, . \, \delta T_b$
- at the bottom: $\varepsilon_{To} = \alpha \, . \, \delta T_o$

The imposed deformations through the temperature in a concrete pavement with a thickness h are:

- average temperature strain: $\varepsilon_T = \alpha\,(\delta T_b + \delta T_o) / 2 = \alpha\,\delta T$ (95)
- curvature: $\kappa_T = \alpha\,(\delta T_b - \delta T_o) / h = \alpha\,\Delta T$ (96)

where: α = coefficient of linear thermal expansion of concrete
δT = average change of temperature in the concrete slab
ΔT = average temperature gradient in the concrete slab

Before cracking of the reinforced concrete pavement the average temperature strain ε_T results in a normal (tension or compression) force N_T and the curvature κ_T results in a bending moment M_T.

Tension bar model

In the Dutch design method (5) a model for the design of the longitudinal reinforcement of a reinforced concrete pavement is included that is explained below. This model, the reinforced tension bar model (figure 39), was developed at the Section Concrete Structures of the Delft University of Technology and it is more extensively described in (41,42).

It is assumed that in the uncracked phase (**phase I** in figure 39) of the ‘indefinitely’ long reinforced concrete slab the imposed deformations through shrinkage and temperature changes are completely obstructed.
The most unfavorable situation is taken as the starting point. This means that not only shrinkage is taken into account but also a decrease of the temperature of the concrete pavement that may occur e.g. in summer during the night or in winter. In elastically supported concrete slabs the cracking, caused by the obstruction of the imposed deformations, will always be initiated at the top of the concrete slab.

The obstructed strains and curvatures due to shrinkage and change of temperature result in the bending moment M and the tensile force N in the concrete pavement (figure 40).

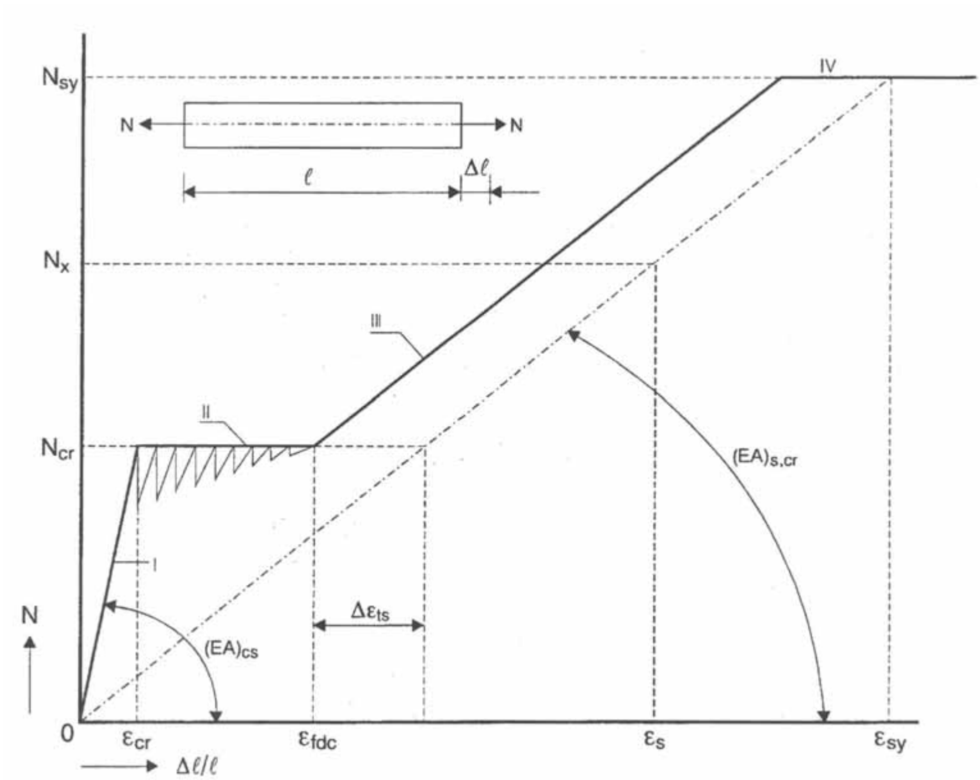


Figure 39. Force-deformation relationship for a reinforced tension bar (41).

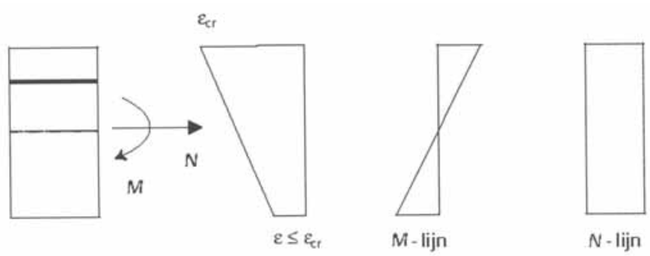


Figure 40. Bending moment M and normal tensile force N due to obstructed strains and curvatures in reinforced concrete pavement just before cracking.

Cracking will occur if the strain at the top of the reinforced concrete pavement ($\varepsilon_{rb}+\varepsilon_{Tb}$) reaches the critical value (ε_{cr}):

$$\varepsilon_{rb}+\varepsilon_{Tb} = \varepsilon_{cr} = \sigma_{cr} / E_c \qquad (97)$$

The tensile stress σ_{cr} in the concrete slab just before the moment of cracking (**end of phase I** in figure 39) is taken as 60% of the mean tensile strength after 28 days for loadings of short duration (see equation 2) to take into account that cracking starts well within 28 days after construction of the concrete layer:

$$\sigma_{cr} = 0.60\ f_{ct,m,o} = 0.54\ [1.05 + 0.05\ (f_{cc,k,o} + 8)] \quad (N/mm^2) \qquad (98)$$

The central tensile force N in the concrete pavement just before the moment of cracking, N_{cr}, is equal to:

$$N_{cr} = \sigma_{cr} \cdot A_c \quad (99)$$

where: A_c = cross-sectional area of concrete

Just after the moment of cracking (**start of phase II** in figure 39) the bending moment M (figure 40) has disappeared at the location of the crack. The present horizontal forces are now the force N_s in the reinforcement steel, the friction force N_f (as the horizontal deformations due to the cracking are partly obstructed because of the friction between the concrete pavement and the underlying base layer) and the central tensile force N_{cr} due to an interaction of forces occurring at the moment of cracking.

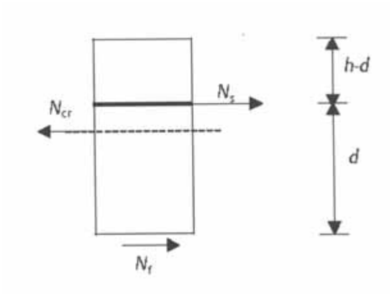


Figure 41. Concrete pavement (thickness h) with a single reinforcement at distance d above the bottom of the concrete layer.

The balance of the horizontal forces ($\Sigma N = 0$) results in (figure 41):

$$N_s + N_f - N_{cr} = 0 \quad (100)$$

Balance of the bending moments relative to the reinforcement steel leads to:

$$N_{cr} (d - h/2) - N_f \, d = 0 \quad (101)$$

It follows from the equations 100 and 101 that the tensile force N_s in the reinforcement steel is equal to:

$$N_s = N_{cr} \, h / 2d \quad (102)$$

In the case of a <u>central</u> reinforcement and a central tensile force the reinforcement steel stress $\sigma_{s,cr}$ in the crack just after the moment of cracking is equal to:

$$\sigma_{s,cr} = \sigma_{cr} (1 + n\omega) / \omega \quad (103)$$

where: σ_{cr} = concrete tensile stress just before cracking (equation 98)
ω = percentage of reinforcement:
$\omega = A_s / bh$
with: A_s = cross-sectional area of reinforcement steel

b = width of the concrete slab
h = thickness of the concrete slab
n = ratio of Young's modulus of steel (E_s) and that of concrete (E_c):
$n = E_s / E_c$

In the case of an eccentric reinforcement, considering equation 102, the reinforcement steel stress $\sigma_{s,cr}$ in the crack just after the moment of cracking is equal to:

$$\sigma_{s,cr} = \sigma_{cr} (1 + n\omega)\, h / 2\, d\, \omega \qquad (104)$$

During cracking the reinforcement steel stress in the crack increases to $\sigma_{s,cr}$. Due to a further increase of the shrinkage and due to a decrease of the temperature a further small increase ($\Delta\sigma_s$) of the reinforcement steel stress occurs, resulting in a total steel stress:

$$\sigma_s = \sigma_{s,cr} + \Delta\sigma_s \qquad (105)$$

The increase $\Delta\sigma_s$ of the reinforcement steel stress in the phase of the uncompleted crack pattern (phase II in figure 39) and in the phase of the completed crack pattern (phase III in figure 39) is described as a 2nd order parabola:

$$\Delta\sigma_s = E_s\, (\varepsilon_{max} - \varepsilon_{cr})^2 / 2\, (\varepsilon_{sy} - \varepsilon_{cr} - \Delta\varepsilon_{ts}) \qquad (106)$$

where: ε_{max} = maximum strain due to shrinkage and decrease of temperature
ε_{cr} = concrete strain just before cracking (equation 97)
ε_{sy} = f_s / E_s = theoretical strain at flow of reinforcement steel (see figure 39)
$\Delta\varepsilon_{ts}$ = $\varepsilon_{s,cr} - \varepsilon_{fdc}$ = decrease of steel strain due to 'tension stiffening' (see figure 39)
$\varepsilon_{s,cr} = \sigma_{s,cr} / E_s$
$\varepsilon_{fdc} = (60 + 2.4\, \sigma_{s,cr})\, 10^{-6}$ = steel strain when reaching the completed crack pattern (phase III in figure 39)

The friction force N_f near the crack is equal to (equation 101):

$$N_f = N_{cr}\, (d - h/2) / d \qquad (107)$$

If this friction force cannot be mobilised, the initially centric normal force will gradually move upward at decreasing N_f.

After cracking through the obstructed imposed deformations the reinforced concrete pavement arrives in the uncompleted crack pattern (**phase II** in figure 39). The mean crack width w_{om} is then equal to (41,42):

$$w_{om} = 2\, [(0.4\, \varnothing / (f_{cc,m,o}\, E_s))\, \sigma_{s,cr}\, (\sigma_{s,cr} - n\, \sigma_{cr})]^{0.85} \qquad (108)$$

where: $f_{cc,m,o}$ = mean cube compressive strength after 28 days for loadings of short duration (equation 1)
$\varnothing$ = diameter of the reinforcement steel

$\sigma_{s,cr}$ = tensile stress in reinforcement steel in crack just after cracking (equation 104)
σ_{cr} = tensile stress in concrete slab just before cracking (equation 98)
n = E_s / E_c

The maximum crack width $w_{o,max}$ in the phase of the uncompleted crack pattern is equal to:

$$w_{o,max} = \gamma_{so}\ \gamma_{\infty}\ w_{om} \leq w_{all} \quad (109)$$

where: γ_{so} = factor to take care of the variation of the crack width; in the uncompleted crack pattern is valid: γ_{so} = 1.3
γ_{∞} = factor to take care of loadings of long duration or cyclic loadings:
for $\sigma_s \leq 295$ N/mm^2: γ_{∞} = 1.3
for $\sigma_s > 295$ N/mm^2: $\gamma_{\infty} = 1 / (1 - 9\ \sigma_s^3\ 10^{-9})$
w_{all} = maximum allowable crack width

According to (2) the following environmental classes are valid for continuously reinforced concrete pavements: XC4, XD3 and XF4. Taking into account (1) the allowable crack width w_{all} for reinforced concrete pavements is equal to:

$$w_{all} = 0.2\ k_c \quad \text{(mm)} \quad (110)$$

where: $k_c = c / c_{min}$ $(1 \leq k_c \leq 2)$
with c = actual concrete cover (mm) on the reinforcement steel
c_{min} = minimum concrete cover (mm) on the reinforcement steel: c_{min} = 35 mm

As in practice the actual concrete cover c always is greater than 70 mm, the parameter k_c will be equal to 2 and hence the allowable crack width w_{all} will be equal to 0.4 mm.
The longitudinal reinforcement has to be designed such that the allowable crack width w_{all} is not exceeded.

In practice a continuously reinforced concrete pavement remains in the uncompleted crack pattern (**phase II** in figure 39). Increasing obstructed deformations (due to further shrinkage of the concrete and low temperatures) result in an increasing number of cracks, so a decreasing mutual distance between the cracks, while the crack widths remain constant.

Would the reinforced concrete pavement ever reach the completed crack pattern (**phase III** in figure 39) then further increasing obstructed deformations result in increasing crack widths while the number of cracks does not change anymore.
It will be clear that the reinforced concrete pavement never should arrive in **phase IV** (figure 39) where increasing deformations result in flow of the reinforcement steel. To prevent this situation the percentage of longitudinal reinforcement should always be greater than a certain minimum percentage (see below).

The above explained reinforced tension bar model has been validated on the continuously reinforced concrete pavements constructed on the motorways A5 and A50 in the Netherlands (43).
The pavement on the motorway A5 consists of 250 mm concrete of quality C28/35 = B35 and has a centric reinforcement Ø 16-135 (percentage of reinforcement = 0.60%).
The pavement on the motorway A50 also has a thickness of 250 mm but of concrete quality C35/45 = B45. This concrete pavement has a centric reinforcement Ø 16-120 (percentage of reinforcement = 0.67%).
The model results showed a good agreement with the in-situ measurement results, both with respect to the average crack width as with respect to the variation in crack width. Therefore the reinforced tension bar model was included in the current Dutch design method for concrete pavements (5).

Results

The figures 42 and 43 show some calculation results obtained with the reinforced tension bar model (42). These results are valid for a 250 mm thick continuously reinforced concrete pavement with a single longitudinal reinforcement, located at a height e (= eccentricity) above the middle of the concrete layer. The concrete quality is C35/45 (B45). The starting points for the calculations are the concrete shrinkage according to (1) and a linear decrease of the temperature of 25ºC.

Figure 42 shows, for reinforcement steel bars with a diameter Ø = 16 mm, the maximum crack width $w_{o,max}$ (vertical axis) as a function of the percentage of longitudinal reinforcement (horizontal axis) for various values of the eccentricity e. The allowable crack width w_{all} (0.4 mm, except in the case of a very great e-value) is indicated in the figure. For centric reinforcement (e = 0 mm) the required percentage of longitudinal reinforcement appears to be 0.61%. The greater the eccentricity e, the lower the required percentage of reinforcement.
There is however a minimum percentage of longitudinal reinforcement to prevent flow of the steel, and for concrete quality C35/45 this minimum percentage is 0.47% (see table 23) that is also indicated in figure 42.

Concrete quality	C28/35 (B35)	C35/45 (B45)	C45/55 (B55)
$\omega_{o,min}$ (%)	0.41	0.47	0.54

Table 23. Minimum percentage of longitudinal reinforcement to prevent flow of steel bars (44).

Figure 43 shows the required percentage of longitudinal reinforcement (horizontal axis) as a function of the eccentricity e of the reinforcement (vertical axis), for reinforcement steel bars with a diameter Ø = 16 mm and Ø = 20 mm. For centric reinforcement (e = 0 mm) the required percentage of reinforcement is 0.61% and 0.65% for steel bar diameters Ø = 16 mm and Ø = 20 mm respectively.

In the Dutch design method for concrete pavements (5) the eccentricity is limited to 25 mm.

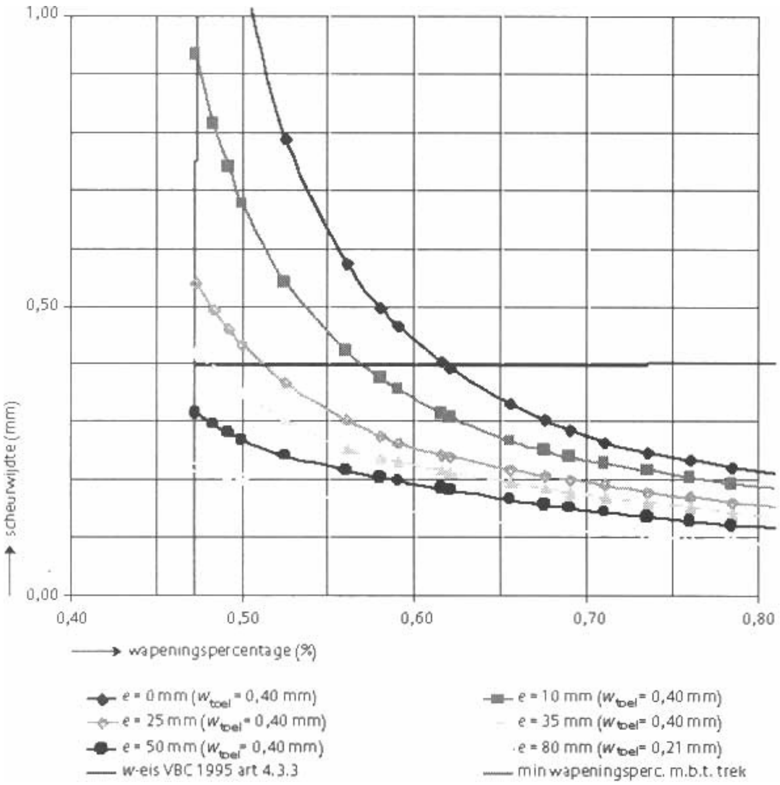


Figure 42. Relationship between maximum crack width and percentage of longitudinal reinforcement for various eccentricities (250 mm concrete C35/45, steel bars Ø = 16 mm) (42).

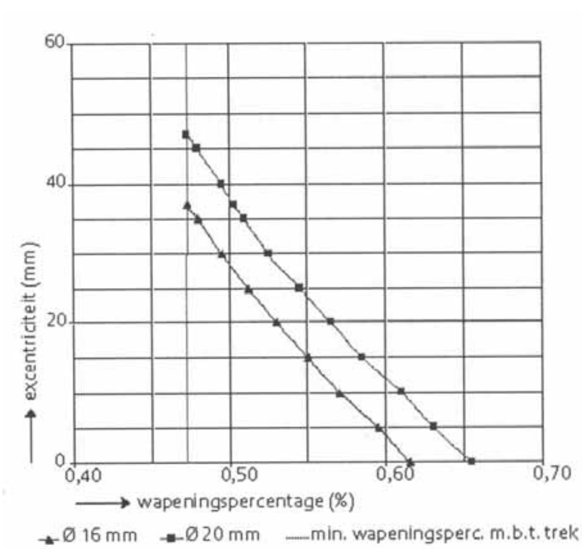


Figure 43. Required percentage of longitudinal reinforcement as a function of the eccentricity for steel bars Ø = 16 mm and Ø = 20 mm (250 mm concrete C35/45) (44).

5.4.11 Additional checks reinforced concrete pavements

After the determination of the thickness (see section 5.4.8) and the reinforcement (see section 5.4.10) of the reinforced concrete pavement two additional checks can be performed: the check for robustness ('unity check') and the check for the traffic-ability at opening of the pavement. These checks are done in the same way as for plain concrete pavements (see section 5.4.9).

Furthermore, the expert user of the program VENCON2.0 can investigate the effect of a number of variables on the crack pattern (more specifically: the crack widths) that develops in the reinforced concrete pavement. These variables are:

- the concrete quality
- the thickness of the concrete pavement
- the diameter of the steel reinforcement bars and the location of the reinforcement
- the shrinkage and creep characteristics of the concrete
- the temperature of the concrete during construction and during cold periods

5.5 Influencing factors

The relative effect of various input parameters on the thickness design of plain and reinforced concrete pavements cannot be analysed directly with the VENCON2.0 software package. It can however be done using the basic equations (especially the equations 80 to 88). In this paragraph an example is given of such an analysis is given with respect to the strength analysis for the centre of the longitudinal free edge of a 2-lane road.

The same basic data is used as in the example of the fatigue analysis in paragraph 3.6. For this 'reference case' a total fatigue damage of 0.62 was calculated (table 10).

Here the effect of small deviations of some individual (so not combined) design parameters on the total fatigue damage analysed:

- modulus of substructure reaction k 10% greater/smaller
- wheel loads P 10% greater/smaller
- temperature gradients Δt 10% greater/smaller
- thickness concrete pavement h 5%/10% greater/smaller
- mean flexural tensile strength of concrete $f_{ct,fl,o}$ 5%/10% greater/smaller

The calculated total fatigue damage for each of these cases is given in table 24.

On the basis of table 24 the relative concrete pavement life as a function of the value of the various parameters can easily be calculated. The results are given in table 25.

	10% smaller	5% smaller	reference case	5% greater	10% greater
Modulus of substructure reaction k	0.724	-	0.620	-	0.518
Wheel loads P	0.163	-	0.620	-	2.187
Temperature gradients Δt	0.422	-	0.620	-	0.748
Thickness of concrete pavement h	∞ (20.353)	∞ (2.974)	0.620	0.123	0.033
Mean flexural tensile strength of concrete $f_{ct,fl,o}$	∞ (11.385)*	∞ (2.385)*	0.620	0.189	0.065

* if omitting upper boundary condition of fatigue relationship (equation 87)

Table 24. Total fatigue damage of concrete pavement as a function of the value of individual design parameters in calculation example.

	10% smaller	5% smaller	reference case	5% greater	10% greater
Modulus of substructure reaction k	0.86	-	1.0	-	1.20
Wheel loads P	3.80	-	1.0	-	0.28
Temperature gradients Δt	1.47	-	1.0	-	0.83
Thickness of concrete pavement h	0 (0.03)*	0 (0.21)*	1.0	5.04	18.79
Mean flexural tensile strength of concrete $f_{ct,fl,o}$	0 (0.05)*	0 (0.26)*	1.0	3.28	9.54

* if omitting upper boundary condition of fatigue relationship (equation 87)

Table 25. Relative concrete pavement life as a function of the value of various individual parameters in calculation example.

The tables 24 and 25 clearly show that the thickness of the concrete pavement, the flexural tensile strength of the concrete and the magnitude of the wheel loads are the most important influencing factors with respect to the concrete pavement life. This necessitates for instance to input in the structural design of concrete pavements the traffic loadings as realistic as possible. The tables also show that a somewhat thicker (e.g. 10 mm) concrete pavement yields a much longer pavement life.
The values of the modulus of substructure reaction and the temperature gradients have a much smaller effect on the thickness design of concrete pavements.

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