VENCON 2.0: a Fast and Reliable Design Tool for Concrete Road Pavements (jointed and continuously reinforced applications)

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Abstract

In the Netherlands, concrete pavements varying from low volume country roads to dual carriage motorways, are designed with the 'VENCON 1.0' program. This program has been developed in the eighties and was originally meant for the design of jointed concrete pavements. An enquiry amongst the users revealed the need for an update towards the Windows operating system but also a technical update became necessary. The update also gave an opportunity to reconsider the design models, to incorporate CRCP design and 'standardise' input data such as the latest practical measurement data from axle loads and tyre spectra of vehicles as well as temperature gradients in the concrete pavement. The classical Westergaard formulae for thickness calculation of concrete layers were extended with Van Cauwelaert's closed form solutions to cater for bound base layers. The 'Delft University tensile member model' is used to design the reinforcement and to predict the cracking pattern of the continuously reinforced concrete pavement. This model is validated with in-situ measurements of two CRCP roads. The program distinguishes three user levels (junior, senior, expert). VENCON 2.0 was developed under CROW's responsibility and came onto the market in early 2005. It is used to design both jointed plain concrete pavement (JPCP) and continuously reinforced concrete pavements (CRCP). This paper provides an outline of the calculation process and the main improvements to the program.

Keywords: design, jointed (JPCP), continuously reinforced (CRCP), tensile member model

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Introduction

The ‘VNC design method’ has been used in the Netherlands for the design of concrete pavements ever since the early eighties. This program has been coded as the ‘VENCON’ design program for all kind of concrete roads in 1994. The program has 1,400 registered users and is renowned for its user-friendliness and the reliability of results. Social and technical developments have required to be updated and upgraded the program. Improvements include:

– raised axle loads and the increasing use of wide-base tyres
– the introduction and construction of continuously reinforced concrete pavements
– changes in the transverse sections of the road in accordance with Dutch Motorway Design Guidelines and the new Road Design Handbook (e.g. new safety measures)
– new insights into material and structural behaviour, resulting in improved design models

VENCON 2.0 – 'the next generation'

A significant improvement in upgrading to version 2.0 (CROW 2004) is the harmonisation of the design rules for elastically supported road pavements and industrial floors. The latter is regulated in Recommendation 36 ‘Designing elastically supported concrete floors and pavements (second revised edition)’ (CUR 2000) issued by the Dutch Centre for Civil Engineering Research (CUR). Article 1.2 of this Recommendation provides regulations, superseding the flaws in NEN 6720 (NNI 1995, 2004). However, in floor design the load duration is a factor in formulating the concrete strength. The long duration of temperature, shrinkage and creep in continuously reinforced concrete pavements require a long term concrete strength, whereas dynamic loads require a short-term strength of concrete. The short- and long-term strengths of both calculation methods are now harmonized including the effects of concrete fatigue caused by repeated traffic loads In the case of pavements made of jointed or continuously reinforced concrete, the strength criterion based on fatigue determines the thickness of the paving slabs (Bouquet et al. 2003).

Design of concrete pavements

Design principles. The design principle of a concrete pavement is similar to that of any structural design i.e. initiative phase, definition phase, (pre-)design, specifications, preparation tender and actual construction. The engineering part of a pavement design consists of determining the composition of the pavement, choosing the materials for the substructure, determining the layer thicknesses (the design) and further refinement.

The designer has to take account of the mechanical properties of concrete. The material is perfectly capable of absorbing compressive stress, but its capacity to resist tensile stress is limited. Measures to ensure that cracks in the concrete occur in a controlled fashion are:

- the application of joints in a jointed concrete pavement
- the incorporation of continuous longitudinal reinforcement into a jointless concrete pavement

Jointed concrete pavements require transverse joints at regular intervals. The joint acts as a programmed crack initiator when the concrete shrinks. The concrete cracks underneath the joint. In more heavily loaded concrete pavements, dowels are applied at the transverse joints to ensure adequate load transfer between the slabs. Tie-bars are always required in longitudinal joints to prevent the slabs from being driven apart.

Continuously reinforced concrete pavements have longitudinal reinforcement bars, positioned in the mid to upper portion of the slab. The permissible crack width and the strength class of the concrete, together with the loads imposed (shrinkage, temperature), determine the amount of longitudinal reinforcement. These steel bars ensure that the (transverse) crack pattern is evenly distributed. Transverse joints are not necessary in this case, but longitudinal joints are needed depending on the pavement width. A CRCP is more expensive, however, the lack of joints also means less maintenance.

Figure 1: Finished concrete pavement for a bus lane

Design factors. Design comprises the establishment of the thickness of a concrete pavement and the computational check. This is intended to generate both technically and economically acceptable solutions. Design should consider geotechnical and road engineering aspects. Geotechnical calculations are required when the subgrade is not able to bear loads or when uneven settlement is expected. In those cases, lightweight subbase material such as foam concrete or expanded polystyrene foam (EPS) is a solution (CROW 2002).

Road engineering calculations show that in the event of expected traffic load, the stresses and strains that occur inside and immediately underneath the concrete pavement are acceptable. The concrete thickness is determined on the basis of one of the following load situations:

- the traffic load on the transverse joint in jointed concrete and on a fine transverse crack in the case of continuously reinforced concrete
- the load on the longitudinal slab edge (in this case, a subbase and/or a central hard shoulder and/or emergency lane can be used to good effect to the thickness)

In the case of a motorway, the load on the longitudinal joint between the traffic lane and the deceleration or acceleration lane is also taken into account.

To summarise, a proper design approach should take into account:

- the road layout or geometry
- the strength properties of the concrete
- loads imposed, particularly by heavy traffic (vehicles with over 20 kN axle weight)
- stresses caused by temperature variations and pavement shrinkage
- the support of the concrete pavement due to the substructure consisting of subgrade, sand bed, subbase and any bituminous binder course

Design approach within VENCON 2.0

The program is a design or control tool for elastically supported road pavements. Input of the subgrade parameters, traffic and temperature loads and the properties of concrete and steel is required. To avoid excessive input work, the user can make a choice from a variety of user profiles with the design parameters of a number of common types of pavement in the Netherlands when starting up the program. The program has the following user profiles:

- motorway
- provincial highway
- municipal road
- low volume or country road
- bus lane

The design data and parameters are grouped under three tabs: 'road layout', 'substructure' and 'loads'. If necessary, it is easy to change and save the default settings. The program includes a large number of help texts and a number of technical drawings. To determine the layer thickness of the most common types of pavement in the Netherlands, the program has the user profiles referred to above: the user can quickly compute unusual situations by making a number of simple modifications. The results of the design of the most common types of pavement have now been incorporated into the revised version of the Dutch Cement Concrete Pavement Manual – Basic Structures (CROW 2005).

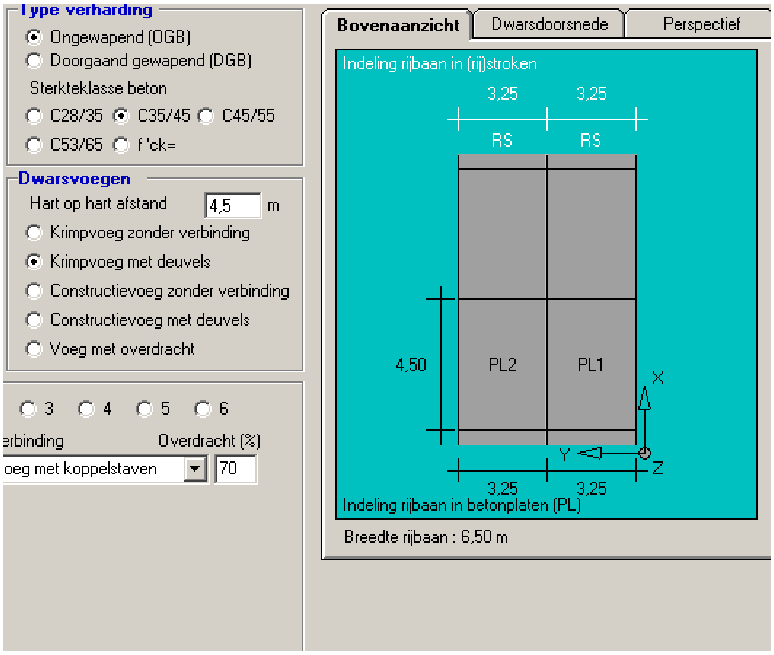


Figure 2: Part of the input screen under the 'Road lay-out' tab for a jointed concrete pavement

Lay-out of the pavement. The following factors play a role in determining the pavement's lay-out (Figure 3):

- the type of road and its use (this is linked to the traffic load)
- number and layout of the traffic lanes (position of the markings)
- type of pavement (jointed or continuously reinforced) and the strength class of the concrete
- number, position and type of longitudinal joints
- position and type of transverse joints

Design is particularly based on the cross section and the traffic flow over the traffic lanes. Wider traffic lanes are better in respect to the load distribution, since not all traffic passes in the same lane. A central hard shoulder and/or emergency lane will also be advantageous. The longitudinal joints must be positioned outside the wheel paths of the heavy traffic (see Figure 3). In this example for CRCP the emergency lane and right traffic lane have been constructed in one stage and the left traffic lane with the central hard shoulder in the next stage. That is why a longitudinal construction joint is needed. That would not be necessary when the total pavement width is constructed in one stage. Transverse joints in jointed concrete pavements are laid at intervals of max. 5 m.

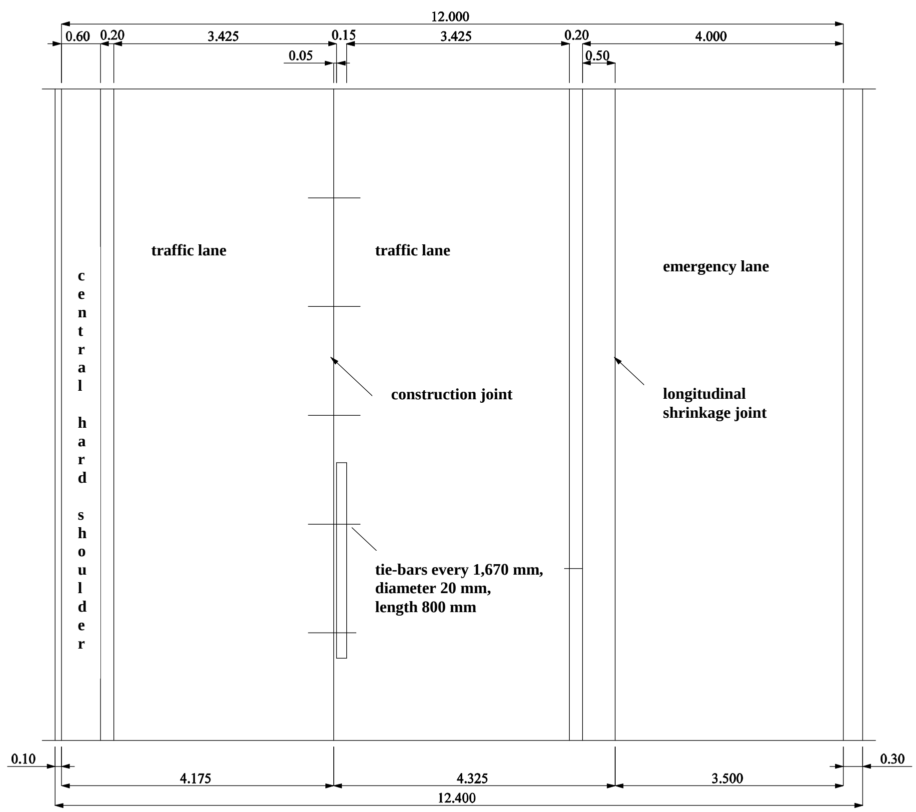
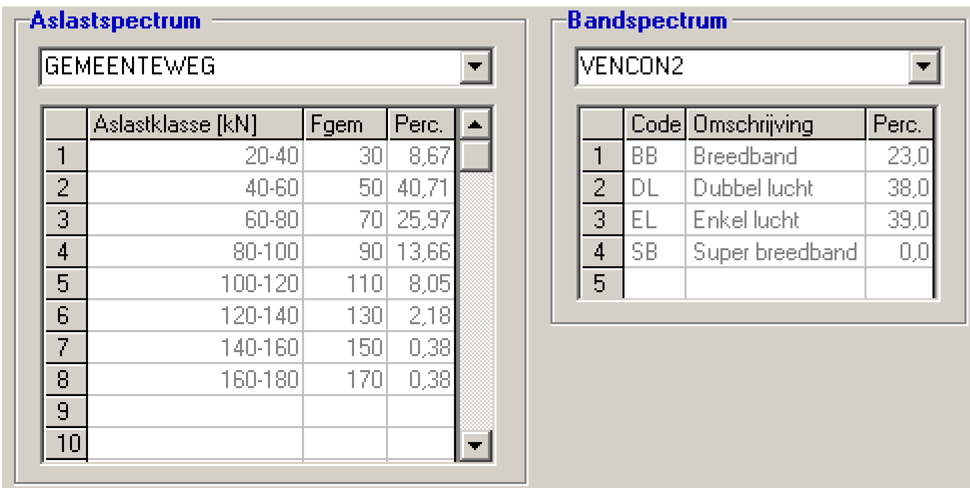


Figure 3: Longitudinal joint situation for CRCP in case of two traffic lanes with an emergency lane (right) and a central hard shoulder (left), dimensions in metre (CROW 2001)

Loads. When determining the thickness of jointed concrete roads, the number of repetitive loads imposed by heavy traffic during the life time of the pavement is highly significant. The traffic load is expressed as the number of heavy vehicles that will use the carriageway. The design load is determined by:

- the required design lifetime, the annual number of traffic days, the average daily traffic intensity over a road cross-section, the percentage of traffic on a single carriageway, the heavy traffic (axle loads over 20 kN) indicated in an axle weight distribution or spectrum, the number of axles on heavy vehicles and the annual growth in traffic
- the temperature variations in the paving slab, axle loads and tyre types that will be used on the road. The axle loads of heavy vehicles (20 kN and above) are categorized in axle load groups with classes of 20 kN. The tyre types single-wheel, dual-wheel, wide-base single-wheel and, since recently, the super wide-base single-wheel tyre are grouped in a spectrum, as are the temperature gradients
- the wandering of the traffic across the traffic lanes and over the longitudinal joints

The kinds of vehicles that actually drive on the road are closely connected to the kind of road to be designed, which can deviate from the standard axle load spectra included in the program. The actual expected axle loads must be used to compute the concrete thickness. The axle load spectrum may be estimated based on traffic counts or historical data, etc.


Aslastspectrum

GEMEENTEWEG

	Aslastklasse [kN]	Fgem	Perc.
1	20-40	30	8,67
2	40-60	50	40,71
3	60-80	70	25,97
4	80-100	90	13,66
5	100-120	110	8,05
6	120-140	130	2,18
7	140-160	150	0,38
8	160-180	170	0,38
9			
10			

Bandspectrum

VENCON2

	Code	Omschrijving	Perc.
1	BB	Breedband	23,0
2	DL	Dubbel lucht	38,0
3	EL	Enkel lucht	39,0
4	SB	Super breedband	0,0
5			

Figure 4: Example of an axle load (left) and tyre spectrum (right) for a municipal road

Substructure and subbase. The load bearing capacity of the subgrade together with the series of layers immediately underneath the concrete are expressed in a combined k-value (N/mm^3). The layer thicknesses and stiffness of all layers underneath the pavement determine the combined k-value (k_1) that depends on:

- the type of subgrade
- any embankment (of lightweight materials as well) and/or sand bed
- the type of subbase

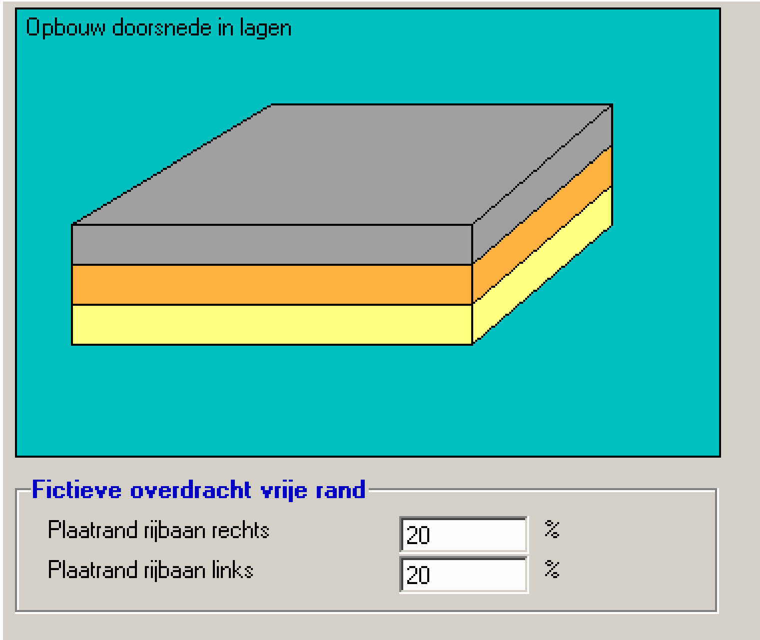


Figure 5: Diagram of layer structure with fictitious load transfer at the edges

Tensile member model

The reinforcement in concrete structures is generally designed to meet with all the rules presented in the design codes. These rules arise from requirements with respect to safety, esthetics, durability and serviceability. With regard to esthetics, durability and serviceability, cracking is one of the major issues. The occurrence of cracks is inherent to reinforced concrete structures. In continuously reinforced concrete slabs, the concrete cover is about 100-130 mm if the longitudinal reinforcement is situated at mid-slab depth.
In the last decades, theoretical models were developed for crack width calculation which are applicable for these types of concrete slabs. (e.g. Noakowski 1978, 1985; Fehling and König 1988). These models are based on a formulation describing the actual physical behaviour. Fundamental experimental research and finite element (FE) analyses provided the information needed to describe the basic components.

Several researchers have described the interaction between steel and concrete. The analytical models are based on a model presented by Noakowski (1985). Noakowski presented an analytical solution for both the transfer length (l_t) and the crack width (w_{cr}) for a not fully developed crack pattern. Use was made of a "power function" to present the bond stress-slip relation:

$$\tau_b = a\delta^b$$

At known steel stresses just before (σ_{s0}) and after cracking ($\sigma_{s,cr}$) the crack width and the transfer length were found to be:

$$w_{cr} = 2[\frac{1+b}{2}\frac{d_s}{4}\frac{1}{aE_s}\sigma_{s,cr}(\sigma_{s,cr} - \sigma_{s0})]^{\frac{1}{1+b}}$$

$$l_t = \frac{w_{cr}E_s}{(1-b)\sigma_{s,cr}}$$

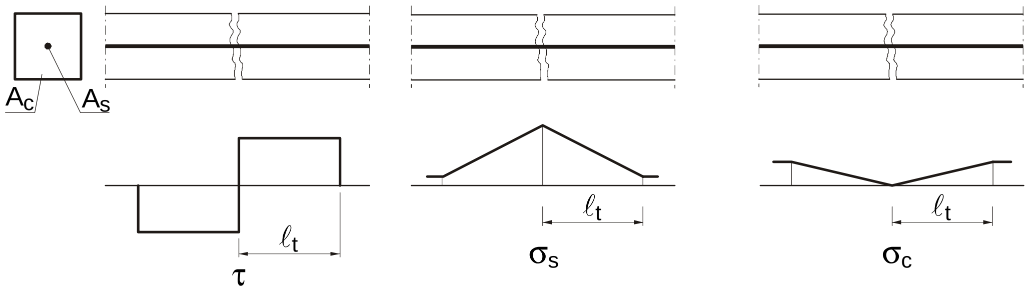


Figure 6: Cracked reinforced concrete tensile member: steel and concrete stresses over the transfer length in case a constant bond stress is assumed (CEB 1967).

Noakowski assumed that the crack pattern is fully developed when all the crack spacings vary between l_t and $2l_t$. The mean crack spacing is then:

$$l_m = 1.5l_t$$

It was assumed that no new cracks form if the load is increased beyond the cracking load, i.e. the mean crack spacing remains unchanged. According to Noakowski's model the tension stiffening is constant:

$$\Delta\varepsilon_s = 0.75\frac{1+b}{2}\varepsilon_{s,cr}$$

Based on the experimentally observed load-elongation behaviour of a reinforced tensile member (Hartl 1977), see also Figure 7, the mean crack width is:

$$w_m = l_m\varepsilon_{sm} = 1.5l_t(\varepsilon_s - \Delta\varepsilon_s)$$

Calculation process

Jointed concrete pavements. The concrete structure is modelled as a system of hinged, connecting slabs on an elastic support. The software distinguishes the following load cases for one slab:
- load on the wheel track position of the transverse joint
- load on the (centre of the) free (slab) edge (with limited load transfer)
- load on the (centre of the) longitudinal joint(s) between the traffic lanes

and in the case of motorways, also the load between the traffic lane and the (central) hard shoulder and deceleration and acceleration lane. Designing a concrete pavement with this program is based on strength and fatigue criteria for concrete. When calculating the strength, the stresses caused by traffic load and temperature variations across the thickness of the pavement are determined. The traffic stress at the bottom of a slab is calculated according to the Westergaard theory and for a bound base with the Van Cauwelaert multi layer slab model (Stet 2004). The thermal stress is calculated according to the Eisenmann theory. For different types of joint and subbase, the load transfer between the concrete slabs can be varied according to the Teller and Sutherland model (Stet 2004).

Westergaard's theory to determine traffic stresses. VENCON 2.0 enables to quantify the traffic stresses in an elastically supported concrete slab according to Westergaard's formulae ('New formula 1948'). For a wheel load P on a slab edge:

$$\sigma_e = \frac{3(1+\upsilon)P}{\pi(3+\upsilon)h^2}\left\{\ln\left(\frac{Eh^3}{100ka^4}\right)+1.84-\frac{4}{3}\upsilon\frac{1-\upsilon}{2}+1.18(1+2\upsilon)\frac{a}{l}\right\}$$

when ν = 0.15 this yields in:

$$\sigma_e = 0.3486\frac{P}{h^2}\left[\ln\left(\frac{Eh^3}{100ka^4}\right)+2.065+1.5340\left(\frac{a}{l}\right)\right]$$

where:

$$l = \sqrt[4]{\frac{E*h^3}{12(1-\nu^2)k}} = \sqrt[4]{\frac{E*h^3}{11.73*k}}, --> provided\,\nu = 0.15$$

and:

σ_e =	bending stress at the bottom of the concrete slab, caused by wheel load P at the 'free' slab edge (N/mm^2), there is no load transfer to an adjacent slab
P =	wheel load (N)
p =	contact stress (N/mm^2)
a =	radius (mm) of the circular contact area
F =	axle load (N) = 2*P
E =	elastic modulus of concrete (N/mm^2)
ν =	Poisson ratio of concrete (-), this value is 0.15
h =	thickness of concrete slab (mm)
k =	combined support of subbase (N/mm^3)
l =	relative radius of elasticity (mm) of the concrete slab

Eisenmann's formula to determine thermal stresses. Prof. Eisenmann has developed a method for calculating stresses due to a positive temperature gradient in a concrete slab. The critical thermal stress is determined as the minimum value, which results from:

$$\sigma_t = \frac{10^{-5}}{2} * h * E * \Delta t$$

In this case, the slab is in full contact with the support. However, when the slab is concave and tilts from the support, the thermal stress is calculated as follows:

$$\sigma_t = 1.8 * 10^{-5} * \frac{\left(L - 3 * \sqrt{\frac{h}{\Delta t * k}}\right)^2}{h}$$

With:

h = concrete slab thickness (mm)
E = elastic modulus of concrete (N/mm^2)
Δt = positive thermal gradient (°C/mm)
k = combined support of subbase (N/mm^3)
L = length of concrete slab or transverse joint spacing (mm)

Definition of Teller and Sutherland to account for load transfer in joints and due to base layer. The load transfer is defined as LTEδ = w_o/w_b, where w_o is the deflection or setting of the unloaded slab edge and w_b the loaded slab edge. In the Netherlands, the definition of Teller and Sutherland is the most widely used. In joints a certain load transfer takes place towards the adjacent slab. The effect of load transfer in a joint (W) is defined by Teller and Sutherland as:

$$W = 100 \frac{2w_o}{w_o + w_b}$$

where:

W = joint efficiency (%) with regard to deflections (elastic vertical movement)
w_b = deflection (mm) of the joint edge of the concrete slab directly loaded by the wheel load
w_o = deflection (mm) of the joint edge of the adjacent concrete slab

For the design of jointed concrete pavement the load transfer in the joints is incorporated in Westergaard's formula by using a reduced wheel load P_r in stead of the real actual load P. VENCON 2.0 uses the formula of Van Cauwelaert and Stet (Stet, 2004) for calculating the load transfer in joints. When load transfer and stress transfer act equivalently, the reduced wheel load P_r is calculated as follows:

$$P_{reduced} = \left(1 - \frac{1}{2} W / 100\right) P = \left(1 - \frac{W}{200}\right) P$$

Contrary to the original Westergaard formulae, the program does not consider a free edge situation on a free longitudinal joint. The slab-on-Pasternak model was used to determine the contribution of a granular subbase and a stabilised layer. Calculations have shown that the load transfer is approx. W=20%, in the case of a granular subbase, and W=35% in the case of a stabilised layer (see table 1).

Table 1: Load transfer values that are applied in the program

Position on the concrete slab	W in %	$P_{reduced}$ (*P)
Longitudinal shrinkage joint with tie-bars	70	0.650
Longitudinal joint when using granular subbase	20	0.900
Longitudinal joint when using bound subbase	35	0.825
Transverse shrinkage joint without dowels	35	0.825
Transverse shrinkage joint with dowels	80	0.600
Free slab edge on extended bound subbase	70	0.650
Terminal joint without dowels	20	0.900
Terminal joint with dowels	60	0.700
Fine-mesh transverse cracks in CRCP	90	0.550

A fictitious load transfer is applied to compensate for the overestimation of the response at the slab edge when applying the Winkler model to a subbase. The level of load transfer can be varied.

Strength of concrete. Concrete can withstand compressive stresses very well. However, tensile stresses will lead to premature cracking, if a slab has inadequate thickness. In the design of concrete roads, the dynamic loads must also be taken into account. Hence, the concrete tensile strength is based on the average (28-day) short term strength:

$$f_{ct,d,0} = f_{ct,d,\infty} / 0.7 = 1.3\ [1.05 + 0.05\ (f_{cc,k,0} + 8)] / \gamma_m$$

in which:
$f_{ct,d}$ = calculation value of the tensile strength
$f_{cc,k,0}$ = characteristic cube compressive strength
γ_m = material factor for concrete under tensile conditions = 1.2

As a result of the bending moment, flexural tensile stresses occur. In the Dutch and European codes the relation between flexural tensile strength ($f_{ct,fl}$) and direct tensile strength (f_{ct}) are defined as a function of the slab thickness as follows (h in mm):

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

Fatigue modelling approach. The design thickness of concrete roads and pavements depends on traffic and temperature induced stresses (due to the positive gradient) during pavement life will not lead to fatigue cracking. The number of load cycles till failure at an axle load class *i* and a gradient class *j* is computed according to the UEC-formula (CROW, 1999):

$$\log N(i,j)_f = \frac{12.903\left(0.995 - \frac{\sigma_{\max}}{f_{br}}\right)}{1.000 - 0.7525\frac{\sigma_{\min}}{f_{br}}} \quad \textit{provided that } 0.50 \le \frac{\sigma_{\max}}{f_{br}} \le 0.833$$

in this formula the parameters are:

$N(i,j)_f$ = the maximum number of load cycles until failure at an axle load class i and a gradient class j
σ_{max} = $\sigma_v + \sigma_t$: traffic stress and thermal stress (N/mm^2)
σ_{min} = σ_t: thermal stress (N/mm^2)
f_{br} = 28-day average short-term flexural tensile strength (N/mm^2)

Example: with h = 300 mm and concrete C 28/35, f_{br} = 1.3 $f_{ct,d,0}$ = 1.69 x 3.2 / 1.2 = 4.51 MPa.

The concrete slab will fail when the cumulative 'damage contributions' caused by the various combinations of wheel load and temperature gradient, equal 1.0. This is according to the hypothesis of linear damage by Palmgren-Miner:

$$\sum_{i-1}^{10}\left[\sum_{j=1}^{10}\frac{ni,j}{Ni,j}\right]=1.0$$

In this formula is:
n(i,j) = no. of passages of a wheel load class i during design life, combined with a certain temperature gradient in the concrete slab;
N(i,j)= allowable no. of passages of a wheel load class i during design life, combined with a certain temperature gradient in the concrete slab.

Continuously reinforced concrete pavement. A new feature of the program is to incorporate the design of a continuously reinforced concrete pavement. Once the slab thickness has been determined on the basis of the strength criterion for jointed concrete (in fact cracked concrete), the required amount of longitudinal reinforcement is calculated with a specially developed model (Van Breugel et al. 1996) in such a way as to meet the crack width criterion for CRCP:

permissible crack width: $w_{permissible} = 0.2\ k_c$ (mm)

with $k_c = c/c_{min}$ ($1 \leq h_c \leq 2$), where c the present concrete surface cover and the required minimum cover c_{min} = 35 mm.
In the case of reinforcement located almost centrically in a concrete pavement, the surface cover (in relation to the upper side) is c>>35 mm, so k_c = 2 and hence $w_{permissible}$ = 0.40 mm (measured at the concrete surface). This permissible crack width depends on the environmental conditions like moist, frost, de-icings salts and the surface cover of the concrete above the reinforcement steel.

A sufficient amount of reinforcement is required to withstand the imposed longitudinal deformation in the surface of the concrete slab and to prevent yielding or even breaking of the steel at the cracks.
An inherent feature of the design of a reinforced concrete pavement is the control of crack formation. Theoretical and experimental research has produced models that describe the cracking pattern (crack width and crack spacing). To use Noakowski's theory in practice, coefficients, cracking stresses etcetera must be defined. Definitions and relevant material properties are described in detail by Van Breugel (1998). Therefore, the results will be presented briefly.

The 'Delft University tensile member model' (Van Breugel et al. 1996) is used to check the cracking pattern of the continuously reinforced concrete pavement. This model is validated recently using crack observations in two motorways constructed in CRCP (Braam, Horeweg 2003 and Braam 2004). In this model, deformation is used to cause cracks. The shear stress/slip relationship between the reinforcing steel and the concrete is the basis for the cracking behaviour. The steel stress after crack formation is calculated on the basis of that relationship and the cracking power in a cross-section. The attachment stresses ensure that the steel stress is reduced from a peak value (the steel stress in a crack) to the steel stress in the zones that are not cracked. The very first crack will occur at the end of section I (see Figure 7). Initially, there is an 'incomplete' crack pattern, which means that the initial crack width is limited.

As the imposed deformation increases, more transverse cracks will develop while the crack width remains constant. As a result, the average crack spacing decreases during the period of use. A saw tooth pattern (section II in Figure 7) is caused because the formation of a new crack leads to a reduction in the elastic rigidity of the tensile member. This crack pattern occurs in *continuously reinforced concrete* pavements and is referred to as an incomplete crack pattern.

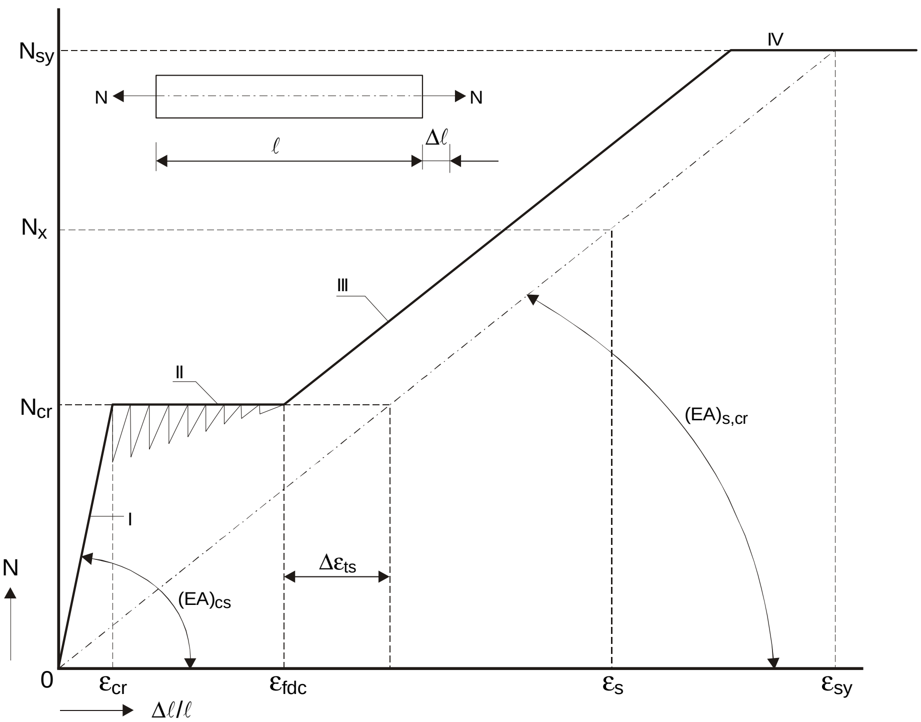


Figure 7: Simplified tensile force-strain diagram of a reinforced concrete tensile member (Noakowski 1985), based on experimental results by Hartl (1977).

As stated before, the tensile force-strain relation of a reinforced concrete tensile member is presented in Figure 7. Stage I is the phase in which the member behaves linear-elastically, defined by the stiffness in the uncracked stage, $(EA)_{cs}$. If the cracking force N_{cr} (imposed load) or the crack strain ε_{cr} (imposed deformation) is reached, the first crack is formed. Now, the member is in stage II; the not fully developed crack pattern. In case of an imposed deformation, the deformation can be increased, which results in the gradual formation of new cracks. It is assumed that all cracks are formed at the same cracking force.

Stage II ends when the crack pattern has become fully developed. The mean steel strain is ε_{fdc} (fdc = fully developed crack pattern). The tensile member is now in stage III. The tension stiffening ($\Delta\varepsilon_{ts}$) is assumed constant throughout this stage. Therefore, the force-strain relation is parallel to the force-strain relation of the reinforcement only (axial stiffness $(EA)_{s,cr}$). In case of an imposed load, the transition from stage II to stage III theoretically takes place immediately because it is assumed that all cracks are formed at the same cracking load. In stage III, an increase of load or imposed deformation results in the widening of already existing cracks; no new crack are formed.

Maximum steel stresses occur in the cracks ($\sigma_s = N/A_s$). Between cracks, steel stresses are lower because bond stresses between the concrete and the reinforcement result in force transfer from the steel to the concrete. If the steel stress in a crack reaches the yield stress of the steel, an uncontrolled widening of existing cracks occurs (stage IV). Crack width models are now no longer valid.

The mean crack width in stage II is described by the following formula:

$$w_{cr} = 2[\frac{1+b}{2}\frac{d_s}{4}\frac{1}{aE_s}\sigma_{s,cr}(\sigma_{s,cr} - \sigma_{s0})]^{\frac{1}{1+b}}$$

which - combined with the material properties presented by Van Breugel (1998) - results in the following expression:

$$w_{mo} = 2[\frac{0.4d_s}{f_{ccm}E_s}\sigma_{s,cr}(\sigma_{s,cr} - n\sigma_{cr})]^{0.85}$$

In the previous formula, $n\sigma_{cr}$ is the steel stress just before cracking; $\sigma_{s,cr}$ is the steel stress directly after cracking. The latter is calculated on the basis of the tensile force equilibrium in a cracked cross-section. The mean cube compressive strength is f_{ccm} = C-value + 8 MPa. The C-value is defined by the strength class of the concrete (based on 150 mm cubes). Cracking is assumed to take place at a concrete tensile stress of $0.75f_{ctm,0}$ if cracking occurs relatively fast, i.e. within several hours after the load is applied. If, however, cracks occur due to long-term loading or to a gradually increasing imposed deformation, $\sigma_{cr} = 0.60f_{ctm,0}$. The mean short-term concrete tensile strength to be used in these expressions is:

$$f_{ctm,0} = 1.05 + 0.05\,f_{ccm}$$

The transition from stage II to III takes place at a mean strain:

$$\varepsilon_{fdc} = (60 + 2.4\sigma_{s,cr}).10^{-6}$$

In stage III, the mean crack width is:

$$w_{mv} = \frac{\Delta l_m}{E_s}(\sigma_s - 0.5\sigma_{s,cr})$$

In the previous formula, σ_s is the steel stress in a crack under the load considered for crack width control. The mean crack spacing is:

$$\Delta l_m = 1.8w_{mo}\frac{E_s}{\sigma_{s,cr}}$$

In case of a not fully developed crack pattern (stage II), there is strictly speaking no ‘mean crack spacing’. However, the area over which a crack influences steel and concrete stresses is known. This area extends over l_t, the transfer length, from both crack faces, see Figure 6. The total length of this area is thus $2l_t$. As stated before, the mean crack spacing in a fully developed crack pattern is related to the transfer length: $\Delta l_m = 1.5l_t$.

The theory presented before is based on the calculation of the mean crack width. In codes, crack width control, an item dealt with in the serviceability limit state, is based on the characteristic crack width, also taking into account the influence of long-term and/or repeated loading:

$$w_k = \gamma_s \gamma_\infty w_m$$

In this formula, w_k is the 95% upper bound value of crack width; w_m is the mean crack width. In a not fully developed crack pattern (stage II) the ratio between the characteristic and the mean value is $\gamma_s = 1.3$. In a fully developed crack pattern (stage III), $\gamma_s = 1.5$. In both stages, long-term and/or repeated loading is dealt with by $\gamma_\infty = 1.3$.

The calculation of CRCP in the program starts by calculating the required slab thickness. Next, the amount of reinforcement is calculated for ‘pure’ tension forces, while the crack width is checked. Should the crack width exceed the permissible one, the amount of reinforcement is increased in a step-by-step manner until the crack width satisfies. The longitudinal joints divide the slab into relatively small longitudinal strips. The theoretical slab thickness calculated for the cross section, increased by an allowance for construction tolerances, forms the input for this model. In the longitudinal direction, the slab needs to be reinforced due to the considerable friction forces (‘imposed deformation’) with the substructure. The program then calculates the required amount of reinforcing steel, which absorbs the tensile stresses caused by imposed longitudinal deformation in the travelling direction of the paving slab. When the concrete cracks, the reinforcing steel has to withstand the force without yielding. Together with the centrally located reinforcing steel and the required reinforcement percentage, the crack width can be controlled in the utility limit minimum value condition. The crack width check conforms with NEN 6720 (NNI 1995, 2004) for durable concrete structures.

The load caused by imposed deformation (epsilon ε) consists of thermal effects on the concrete due to cement hydration and due to shrinkage of the concrete. Article 6.4.1 of CUR Recommendation 36 (CUR 2000) recommends that the shrinkage be incorporated either as 100% shrinkage across the slab thickness or as a shrinkage gradient because the top of the surface dries out more than the bottom. The shrinkage on the top is then 90% and 60% on the bottom (percentage of the max. shrinkage according to NEN 6720). The Delft tensile member model has been validated in practice with measurements (Braam, Horeweg 2003 and Braam 2004).

Table 2 shows the required amount of longitudinal reinforcement for various thicknesses, as indicated after calculation with the program (Bouquet et al. 2004). The rods are attached to ø 12-700 mm supports placed at an angle of 60° on the road axis.

Table 2: Minimum longitudinal reinforcement FeB 500 (centric position) required if the crack width requirement of 0.4 mm is met

Concrete strength	C 35/45	
Nominal diameter rods (mm)	16	20
Minimum reinforcement percentage (% of the cross-section) at $w_{permissible}$ = 0.40 mm	0.62	0.66
Specified thickness of the concrete layer		
230 mm		
- min. reinforcement amount (mm^2/m)	1438	1518
- max. centre-to-centre distance rods (mm)	140	207
250 mm		
- min. reinforcement amount (mm^2/m)	1547	1650
- max. centre-to-centre distance rods (mm)	130	190
270 mm		
- min. reinforcement amount (mm^2/m)	1674	1782
- max. centre-to-centre distance (mm)	120	176
Overlap length longitudinal rods (mm)	370	450

The reinforcing steel is usually laid centrically in the slab of a continuously reinforced concrete pavement. On certain conditions, the program enables the user to position it somewhat higher up in the cross-section. This can also contribute to further optimisation of CRCP roads.

Validation for CRCP

Two concrete highways under construction were selected for crack width measurements: the A5 (between the two cities of Haarlem and Hoofddorp) and A50 (Oss – Eindhoven), both in the Netherlands (Braam, Horeweg 2003 and Braam 2004). Both highways consist of 250 mm thick concrete slabs having a longitudinal reinforcement and hence no transverse joints. The longitudinal reinforcement is positioned halfway down the thickness of the slabs. The slabs are placed on a asphalt concrete interlayer which is supported by a 250 mm thick granular subbase. Design and construction details of the A5 are presented by Hermans et al. (2003).

Parts of the A5 concrete slabs selected were cast in August 2002. Longitudinal reinforcement consisted of Ø16 mm bars, spaced at 135 mm. The concrete strength class was C28/35 (characteristic 28-day 150 mm cube compressive strength). The mixture was no-slump concrete (slump < 40 mm). In November 2002, seven transverse cracks were selected. Crack widths were measured on the concrete surface directly over and in between the longitudinal reinforcement bars. Use was made of a plastic card with lines of different thickness to measure crack widths. Crack widths were rounded to the nearest 0.05 mm.
The location of the measurements had been inspected seven days after casting. Cracks existing at that moment had been marked. All seven transverse cracks inspected had been marked, so they had been formed within seven days after casting.

One part of the A50 selected was cast in September 2002. Longitudinal reinforcement was Ø16 mm, spaced at 120 mm; concrete strength class was C35/45 (characteristic cube compressive strength). In November 2002, six transverse cracks were selected and crack widths were measured following the same procedure as used for the highway A5 project. In June 2003, a second part of the A5 was selected. Crack widths were measured 1, 2, 4, 6 and 19 days after casting. This procedure provided insight into the concrete age at the instant of cracking, the crack width at that instant and the development of crack width in time.

Evaluation of measurements and calculations

Crack widths over and in between the reinforcing bars. For all 13 transverse cracks, the mean crack widths over and in between the bars were calculated. Analysis demonstrated that there was no statistically significant difference between both mean crack widths of an individual transverse crack. If was therefore concluded that the distance from the concrete surface to the nearest reinforcing bar has no influence on the crack width.

Crack widths measured and calculated. In the cracking model previously described, the concrete properties at the instant of cracking are of major importance. For instance, for cracking at an age of two days, tensile and compressive strength of the concrete are approximately 50% of the 28-day strength. For cracking at an age of seven days, this is 90% and 80% for the tensile and compressive strength, respectively (Reinhardt 1985). Especially the influence of the tensile strength is very important because of its impact on the steel stress after cracking. Calculations were therefore performed for cracking assumed to occur at both ages. Since measurements were done several months after cracking was supposed to occur, the influence of long-term loading was accounted for by multiplication of the short-term crack width by a factor 1.3.

Discussion and conclusions of the crack width validation

The results of the on-site validation demonstrated good agreement between theory and experiments: for both projects, the measured mean crack widths are within the interval of crack widths calculated for cracking at two concrete ages. As known, information on the material properties, thus concrete age, at the instant of cracking is of major importance to enable comparison between theory and experiment. The measuring results have demonstrated that the theoretical crack width is increased by about 50% if the concrete age at cracking is seven instead of two days.

The concrete age at the moment of cracking was investigated into detail in a second test series. It was demonstrated that the initial crack width increases as the concrete age increases. This is in line with the theory. However, crack width increases in time in such a way, that the influence of the concrete age at the instant of cracking diminishes. All crack widths therefore tend to reach the same final value. It was therefore suggested not to take into account the concrete age at cracking and to apply the theory presented using 28-day concrete strengths. Good agreement between the measurements and the theory was observed.

If the crack widths measured are compared with the allowable crack width, Dutch design codes present an allowable basic long-term characteristic value of 0.2 mm. Because of the large concrete cover applied in the slabs of A5 and A50, it is permitted to double this limit value to 0.4 mm (Dutch code). None of the characteristic values that can be derived from the measurements exceed this limit.

It can be concluded that the cracking model predicted crack widths rather well and sufficiently safe. Also the variation in crack width was in accordance with theoretical assumptions. With regard to the concrete cover, it is advised not to make any adjustments to the model; the theoretical model as presented before is also applicable in the case of a concrete cover of 125 mm.

Findings

As of 2005, VENCON 2.0 provides the concrete road-construction sector in the Netherlands with an up-to-date tool for comparing paving variants quickly and reliably. The structure and the clear presentation of input and output data make it a highly user-friendly design program. At expert level, the program also calculates the maximum axle load that can be permitted on the (early age) concrete pavement. The background report on the cd-rom included with the program contains all formulae and calculation models, and the program's calculation process is explained step-by-step (Stet 2004).

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