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Steel concrete composite bridges: Shrinkage effects in the slab cracking control

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I Abstract

The influence of shrinkage in the crack control of the concrete slab of a composite, steel and concrete, bridge deck is discussed.

The analysis is made comparing the Portuguese and the European standards. Besides this, the influence of the construction process on the stresses due to shrinkage and therefore, on the Cracking Limit State, is discussed.

Finally, some recommendations for the design and construction of composite bridge decks are made in order to reduce the effects of shrinkage.

II Introduction

The study of the influence of shrinkage at the serviceability limit state of a composite, concrete and steel, bridge deck implies the use of the existing codes to analyze and calculate the effects of shrinkage as an action. Hence, a brief reference to the Portuguese codes, Regulamento de Segurança e Acções e Regulamento de Estruturas de Betão Armado e Pré-Enforçado, and to the Eurocodes is done.

The concrete shrinkage is the volume variation of concrete, from the end of its compaction until a hygrometric equilibrium with the environment is reached. Besides the shortening of a concrete element, the shrinkage also comprises, when this element is kept in a humid environment, its expansion, which can be considered as part of the independent deformations of the strain state.

\[
\varepsilon_{cm}(t) = \varepsilon_{cs}(t) + \varepsilon_{ct}(t)
\]

where \( \varepsilon_{cs}(t) \) is the deformation due to shrinkage and \( \varepsilon_{ct}(t) \) is the deformation due to temperature variation.

Before the concrete hardening, when concrete did not yet reach its resistance, shrinkage occurs due to the settlement of the materials that are in the concrete mass and due to the water evaporation. This is called plastic shrinkage and will not be covered in this work. The most important shrinkage is the one happening after the concrete hardening as it is in the present study.

The most relevant shrinkage processes happening after the beginning of concrete hardening are:

The Thermal Shrinkage starting just after the concrete hardening which has a cooling time proportional to the square of its thickness. This phenomenon is not yet sufficiently known.
The Autogenous Shrinkage due to the temperature variation which develops much after the concrete hardening. This phenomenon is the main reason for the porosity of the ordinary concrete. The values of this shrinkage are usually less than $10^{-4}$, when the water to cement ratio is greater than 0.45. The Autogenous Shrinkage increases sharply for values of water to cement ratio less than 0.40, reaching the value of $3 \times 10^{-4}$. This shrinkage is only a small but not negligible cause for the premature cracking of concrete if it is not prevented since the beginning of the concrete hardening.

The Drying Shrinkage, starting since the beginning of the removal of the formwork, and varies between 2.0 and $6.0 \times 10^{-4}$. This shrinkage depends on several factors such as the thickness of the concrete piece, the porosity, the free water content in concrete, the mass volume and the binder type.

The stresses due to shrinkage are reduced with time because of the relaxation/creep effect. Therefore these two phenomena are considered together through the introduction of the creep coefficient in the calculation of the long term concrete characteristics.

In a composite, steel and concrete, structure the deformations of the concrete slab are restrained by the steel structure. Consequently, the well known primary and secondary effects of shrinkage appear. The primary effect is the isostatic part of the shrinkage; it takes place at the section level and corresponds to an imposed deformation to the section which gives rise to an self equilibrated strain distribution. If the structure is hiperstatic, the shrinkage induced curvature origins a hiperstatic moment diagram in equilibrium with the self equilibrated support reaction. These are the secondary effects of shrinkage.

The main reasons for the short term cracking of the concrete slab of a composite bridge deck are related to tension stresses due to external mechanical actions, in which the construction stage is included, and to tensile stresses in the concrete slab due to imposed deformation, such as shrinkage. Therefore, since this is one of the main causes for the concrete cracking, it should be reduced as much as possible, controlling the concrete composition, as well as the construction method and the conditions for a good cure.

At the intrinsic level, that at the level of the concrete composition, the most important factors to be controlled for shrinkage reduction are:

- The cement type;
- The aggregate nature;
- The water to cement ratio.

As far as the external factors are concerned, the following should be taken into consideration:

- The element geometry;
- The atmosfere humidity;
- The construction process.

According to the present codes, by establishing and controlling the minimum reinforcement and limiting the strains to a maximum value, the cracking is controlled. The values of the autogenous shrinkage are verified to be reduced by 15% due to the presence of the reinforcement.
III Design Codes

Until the European Standards to be available, specifically EN 1994 (Eurocode 4), the use in Portugal of any code concerning composite construction, namely for composite bridges, was very limited. Cracking control was made on the basis of “Regulamento de Estruturas de Betão Armado e Pré-Esforçado”, for reinforced concrete elements, in a similar way as now recommended by Eurocode 4.

In each one of the referred codes, it is possible to proceed not only by an indirect cracking control (with a minimum reinforcement area and the use of different bar spacing and diameters) according to the maximum stress required and the environment in which the structure is in, but also to a direct control by the limit crack width calculation.

The meaning and calculation of the parameters in the following section may be obtained from each one of the codes.

III.1 “Regulamento de Estruturas de Betão Armado e Pré- Esforçado”

This code does not distinguishes shrinkage parts, it only considers a shrinkage reference value, that depends on the environment hygrometric conditions, the consistency of fresh concrete, the element notional size and one expression time dependent expressing the shrinkage evolution with the age of concrete that also depends on the element notional size,

\[
\varepsilon_{cso} (t, t_0) = \varepsilon_{cso} \left[ \beta_s (t_1) - \beta_s (t_0) \right]
\]

\( \varepsilon_{cso} \) – reference shrinkage value, equal to \( \varepsilon_{cso} = \varepsilon_{cso} \eta \);  
\( \beta_s (t_1), \beta_s (t_0) \) – coefficients expressing shrinkage value with the concrete age \( t \); assuming that shrinkage began at the age \( t_0 \)

The element notional size is equal to \( \lambda \frac{2A_c}{u} \)  

where,

- \( A_c \) is the element cross section area;
- \( u \) is the part of the element perimeter in contact with the environment;
- \( \lambda \) is a parameter depending on the environment hygrometric conditions.

The lower is the ratio between the element area and the perimeter, the higher are the shrinkage values at a certain concrete age since the shrinkage value varies inversely to the notional size value.

Besides this calculation, “REBAP” allows to ordinary cases the assimilation of the final shrinkage effects to a slowly and uniform temperature decrease of 15ºC.

To verify the cracking of concrete, “REBAP” provides for reinforced and partial prestressed concrete members:
A verification of cracking limit state (article 70º), in which the crack width calculated by equation II-9 should not exceed the set value on article 68º

\[ \omega_k = 1.7 \omega_m \]  \hspace{1cm} (III-2)

\( \omega_m \) being the medium crack width value equal to \( s_{rm} \varepsilon_{sm} \);

\( \omega_k \) being the the maximum crack width value specified on article 68º;

\( s_{rm} \) being the the medium distance between cracks;

\( \varepsilon_{sm} \) being the the maximum reinforcement strain.

According to article 68º, this maximum crack width value depends on the aggressiveness of the environment where the structure is located. The more aggressive the environment is, the lower is the crack width allowed, in order to provide a lower exposure to corrosion of reinforcement.

A verification of the limit state of decompression (article 69º), which is satisfied if, on the extreme fiber of the element section (the one that would be more tensioned), there is no tension stresses due to acting forces, with an elastic analysis of materials at the un-cracked stage;

A verification of the maximum compressive stress (article 71º). To this verification it is considered a limit, usually \( 0.8 f_{cd} \), that shall not be exceeded by stresses calculated for the infrequent load combinations.

To satisfy the cracking limit state is still necessary to verify the maximum bar spacing, as provided in article 91º of REBAP. These verifications are made, according to “RSA”, using infrequent (rare), frequent or quasi-permanent combinations depending on the aggressiveness of the environment.

**III.2 European Standard**

In the calculation of shrinkage effects by Eurocode 2: Part 1-1”, 3.1.4 (6), there is a clear difference between drying shrinkage and autogenous shrinkage, being the total strain provided by these two other components.

\[ \varepsilon_{cs} = \varepsilon_{cd} + \varepsilon_{ca} \]  \hspace{1cm} (III-3)

where \( \varepsilon_{cs} \) – is the total shrinkage strain;

\( \varepsilon_{cd} \) – is the drying shrinkage strain;

\( \varepsilon_{ca} \) – is the autogenous shrinkage strain.

These parameters depend on the notional size, as in “REBAP”. However this parameter, notional size, is calculated here in a different way, simply by \( \frac{2A_k}{u} \), not considering the dependent part of the environment hygrometric conditions. However, it yields, as in “REBAP”, the strain due to shrinkage varies inversely to the values of element notional size.
According to EC4 (article 5.4.1.1 (2)), a global elastic analysis at serviceability limit states should be performed, with appropriate corrections for non-linear analysis (due to concrete cracking, etc.). For this, a modular ratio shall be calculated, taking into account with the material creep coefficient. This coefficient depends on the structure load and is given by the expression 5.6, from article 5.4.2.2 (2)

\[ n_L = n_0 (1 + \psi \varphi) \]  

where: \( n_0 \) is the modular ratio \( E \frac{E_{cm}}{E_{cm}} \) for short-term loading;

\( E_{cm} \) is the secant modulus of elasticity of the concrete for short-term loading according to EN 1992-1-1:2004.

\( \varphi \) is the creep coefficient according to EN 1992-1-1:2004.

\( \psi \) is the creep multiplier depending on the type of loading.

According to the same article, clause (8), in regions where the concrete slab is assumed to be cracked, the primary effects due to shrinkage may be neglected in the calculation of secondary effects, as well as in the calculation of stresses, (article 7.2.1 (4)), since the isostatic moments are zero in the cracked areas.

In terms of cracking verification, the EC4 provides, not just the limit for cracked widths, see article 7.4.1 (1), leaving to EC2, but also, conservatively, allows in the same article, item (3), a minimum reinforcement area and a bar spacing or diameters not exceeding the crack width limits, defined indirectly. The crack width limit is given according to the element class exposure being this value equal to 0.3mm, for bridges. However, instead of a conservative analysis, it is possible to calculate the maximum crack width according to article 7.3.4 of Eurocode 2.

According to this standard, the crack width may be calculated from the following expression:

\[ w_k = s_{r,\text{max}} \left( \varepsilon_{\text{sm}} - \varepsilon_{\text{cm}} \right) \]  

where: \( s_{r,\text{max}} \) is the maximum crack spacing;

\( \varepsilon_{\text{sm}} \) is the mean strain in the reinforcement under the relevant combination of loads, including the effect of imposed deformations and taking into account the effects of tension stiffening.

\( \varepsilon_{\text{cm}} \) is the mean strain in the concrete between cracks.

The minimum reinforcement for non-prestressed sections subject to significant tensile stresses due to restraint imposed deformations, just like shrinkage in combination or not with direct loads, is given in the article 7.4.2 (1) of Eurocode4 by the following expression:

\[ A_s = k_k_k f_{\text{ct,eff}} \frac{A_s}{\sigma_s} \]  

where: \( f_{\text{ct,eff}} \) is the mean value of the tensile strength of the concrete effective at the time when cracks may first be expected to occur.

\( k \) is a coefficient which allows for the effect of non-uniform self-equilibrating stresses.
k_s is a coefficient which allows for the effect of the reduction of the normal force of the concrete slab due to initial cracking and local slip of the shear connection.

k_c is a coefficient which takes account of the stress distribution within the section immediately prior to cracking.

\( \sigma_s \) is the maximum stress permitted in the reinforcement immediately after cracking. This stress value may be obtained by the tables present in Eurocode 4 relating this value with the maximum bar diameter and crack widths allowed. These stresses \( \sigma_s \) are calculated taking into account the effects of tension stiffening of concrete between cracks (art. 7.2.1 (6) – Eurocode 4), if there are any direct loads on the structure.

\[
\sigma_s = \sigma_{s,0} + \Delta \sigma_s
\]  

\( \Delta \sigma_s \) is the area of tensile zone caused by direct loading and primary effects of shrinkage immediately prior to cracking of the cross section.

The diameters taken from the table shall be modified, to a maximum used diameter value, as follows

\[
\phi = \phi^* \frac{f_{ct,eff}}{f_{ct,0}}
\]  

in which \( \phi^* \) is the diameter from the table and

\( f_{ct,0} \) is a reference stress equal to 2.9N/mm²

**IV Models for shrinkage structural analysis**

As previously referred, in an hiperstatic structure, shrinkage induces an imposed deformation and a self-equilibrated stress distribution to the section, namely by primary effects of shrinkage, and a diagram of hiperstatic moments in balance with a group of self-equilibrated support reactions, called secondary effects of shrinkage. The structure may be solved and the hiperstatic unknowns may be found by the force or by the displacement method.

![Isostatic stresses diagram](image)

Fig. IV-1 – Isostatic stresses diagram [13]

The isostatic stresses in concrete are tensile stresses, increasing with the approach to the centre of gravity of the homogenous section, due to the equivalent isostatic positive moment induced by tension deformation. These tensile stresses in concrete and compressions in structural steel are maximum at the fiber at contact between steel and concrete. In this stress analysis, the induced curvatures in the structure are not considered. The elastic isostatic stresses are calculated based on the deformation compatibility in the section, with base on the principle of conservation of plane sections.
Being shrinkage totally restrained, it creates in the concrete slab a blocking force of the shrinkage deformation equal to

\[ N^0_c = \varepsilon_{cs} \frac{E_n}{n} A_c \]  

\( \frac{E_n}{n} = E_{c,ef} \) is the effective elasticity modulus of the homogenized section; \( \varepsilon_{cs} \) is the deformation due to shrinkage and \( A_c \) the concrete cross section. This force makes at the level of the centre of gravity a group of forces composed by a force \( N_0 = N^0_c \) and a moment \( M_0 = N^0_c a_{ch} \) with \( a_{ch} \) the distance between the centres of gravity of the concrete slab of the homogenized section and the centre of gravity of the homogenized section.

The stress state at the concrete slab equivalent to the blockage force is uniform and equal to

\[ \sigma^0_c = \varepsilon_{cs} E_{c,ef} ; \sigma^0_a = 0 \]  

At the end section, the blockage force, resulting from \( \sigma^0_c \), induces an axial force \( N_0 \) and a bending moment \( M_0 \), which shall be cancelled. This requires the application at the centre of gravity of the homogenized section and a pair of forces equal but with opposite direction to the restriction force, \( N_0 \), and the moment, \( M_0 \). It is then obtained, in the homogenized section, a stress equivalent to these internal forces composed by a part due to axial force and another due to the moment:

\[ \sigma^I = \frac{N_1}{A_h} + \frac{M_1 z}{I_h} = N^0_c \left[ -\frac{1}{A_h} + \frac{a_{ch} z}{I_h} \right] \]  

The final stress distribution is obtained by adding these two isostatic parts

\[ \sigma^I = \sigma^0 + \sigma^I \]  

Taking into account the real concrete section, stresses shall be affected by their modular ratio coefficient \( \sigma^I = \sigma^I / n \).

Substituting equations (IV-2) and (IV-3) in (IV-4) the isostatic final stresses are

\[ \sigma^I_c = \varepsilon_{cs} \frac{E_n}{n} \left[ 1 + \frac{A_c}{n} \left( -\frac{1}{A_h} + \frac{A_{ch} z}{I_h} \right) \right] \]  

\[ \sigma^I_a = \varepsilon_{cs} \frac{E_n A_c}{n} \left( -\frac{1}{A_h} + \frac{a_{ch} z}{I_h} \right) \]  

These stress effects are relevant at serviceability limit states, but may be neglected for the ultimate limit states verifications.
The hiperstatic moments that appear on the continuous beam due to the curvature induced by shrinkage are represented in the following figure

![Image of a continuous beam with labeled moments and forces]

**Fig. IV-2 - Hiperstatic moments in a continuous beam [13]**

The concrete slab is subjected to tensile stresses, due to restrained shrinkage effects. Emphasis is made on the fact that total stresses are reduced from the upper fibre of concrete until the interface steel-concrete. This is due to the hiperstatic effects of shrinkage, as these negative moments induce stresses in the upper face of concrete much superiors to the ones of the restrained face, not compensating the isostatic moment. For the same reason, the hiperstatic negative moment induces compressions in the lower fibre of the steel beam and tensions at the upper fiber which added to the isostatic stresses, results to an increase of compression stresses on the steel beam from the upper to the lower fibres.

According to Eurocode 4 – Part 2, the effects of concrete cracking are only considered if the stress at the extreme tensioned fibre is more than 2xf_{ctm} or, in a simplified way, if the ratio between adjacent spans, minor/major, in a continuous beam is at least equal to 0.6, the cracking of concrete in 15% of the span for each side of the internal supports may be considered. In these sections, in a cracked analysis, the primary effects of shrinkage are not taken into account, meaning that, the total effects are equals to the hiperstatic ones in 15% of the spans over supports, and equal to the sum of isostatic with hiperstatic effects at mid-span.

When calculated from a cracked analysis, due to concrete cracking, the shrinkage hiperstatic moments are approximately half the moments from an un-cracked analysis. In this way, at mid-span, the stresses in concrete have a lower tension value than when obtained by an un-cracked analysis, as the isostatic part is always constant. The final stresses vary with the hiperstatic moment of the section.
V Influence of shrinkage at the serviceability limit states

V.1 Differences in codes

In EC4 – Part 2, 7.4.2 (5), it is referred the minimum reinforcement area in sections subjected to imposed deformations, in combination or not with direct loads, calculated with equation III-6, is made in the sections where the stresses are tensile stresses in concrete, according to the characteristic combination of actions.

The first step consists in an un-cracked analysis so it is possible to compare the final stresses in tensioned sections, calculated to the characteristic combination of actions, with the resistant stress \( f_{\text{cm}} \). If these stresses are higher than \( 2 \times f_{\text{cm}} \) a cracked analysis of the section shall be done.

The shrinkage, induces tensile stresses at the concrete slab, independently of the section it refers to, so, it has always an unfavorable effect on the concrete sections at supports, since every direct loads cause negative moments on these sections, increasing the tensile stresses. On the contrary, at mid span its effect may be favorable or unfavorable, as it presents the same signal or opposite signal to stresses induced by the permanent loads applied.

After an analysis of the characteristic combination, taking account of the concrete shrinkage or not, it is verified that at support sections the shrinkage effects may be sufficient to induce cracking of concrete on the structure, that is, making the stresses values higher than the limit for an un-cracked analysis. This limit value, \( f_{\text{cm}} \), recommended by “REBAP” is more conservative (half of the value) than the one recommended by the Eurocode, \( 2 \times f_{\text{cm}} \). Having in mind these limit values, although the stress values obtained by an analysis based on the Eurocodes are higher than based on the “RSA”, this analysis may not be more conservative to proceed to a cracked analysis since the resistant comparison value is also higher than with an analysis made by the Portuguese regulation, still in force.

The shrinkage effects are favorable in compression stresses, reducing their values, therefore it shall not be considered for the calculation of stress combinations in positive moments at deck’s mid span (when only the internal span is loaded); on the other hand, when variable actions, i.e., traffic loads, induce negative moments at the concrete slab at mid span, that is, when only the end spans are loaded, shrinkage increases the tension stresses in concrete and shall be considered in this calculation.

For the control of cracking the design reinforcement stresses shall be known and, according to EC2 and EC4, shall be considered the quasi-permanent load combination; according to “RSA” and “REBAP”, it is considered the load combination corresponding to each exposure class: to a environment with a moderate risk of reinforcement attack, a frequent load combination shall be used.

When cracking control is made by the design of a minimum steel reinforcement by equation III-6, the value of \( \sigma_s \) shall be discussed because if \( \sigma_s = f_{sk} \) is adopted, the value of the minimum reinforcement is the less possible. This value does not correspond to the real value, once the stresses in the structures
never present this value but a lower one, leading to reinforcement values higher than this one. These stresses are taken from the tables referred above and are dependent of the adopted reinforcement bars and the limit crack width allowed.

Being the stresses in the steel reinforcement obtained from an un-cracked analysis higher to the ones obtained from a cracked analysis (once the first analysis is conservative), the minimum reinforcement value from the first analysis it will be inferior to the area that the second analysis leads to, according to equation III-6.

Another factor to keep in mind is, although forces and stresses, calculated for the traffic loads are higher when calculated by Eurocode with respect to “RSA”, cracking control, made by a direct crack width design, is more unfavourable by “REBAP” than by the Eurocode. This happens due to the following factors: the first one corresponds to the environment aggression considered, that being moderately aggressive, the crack width limit is 0.2mm, while by EC, for bridges, the maximum value allowed is 0.3mm. On the other hand, for quasi-permanent load combinations the reduction factor for traffic loads is 0 in the Eurocode, yielding to lower stress values than when the calculation is made for the frequent combination, by “RSA”.

V.2 Sequence of construction

The structural behaviour is largely influenced by the adopted construction sequence, and particularly by the casting plan, changing the static system during construction and co-existing different ages of concrete in the final structure. This phenomenon leads to a variation of resistant characteristics of concrete along the time.

When studying two different casting plans, one in an AB sequence and another in an ABC sequence, it is verified that stresses and deformations on the structure are much influenced by the construction method.

In an AB sequence, where the spans are first casted and only after the support sections, the stresses induced to the structure are lower than the ones induced when the casting starts in an end span until 1/5 of the adjacent span and until the hole structure is casted, in an ABC sequence. Despite stresses, deformations are dependent on the structure spans ratios; to small ratios between adjacent spans, it is shown that ABC sequence is less favourable than the AB sequence, in contrary to what was expected. Besides, when a modular ratio for short term loading is used, the stresses induced at the concrete slab increase due to an increase of the element stiffness; it may well decide on the kind of analysis to be made: in sections where it wouldn’t be necessary to proceed to a cracked analysis with a long term modular ratio, \( n_L \), may be necessary based on an analysis made with the referred short term modular ratio, \( n_0 \). The stresses in the steel beam reduce when adopting \( n_0 \), once the concrete slab takes more forces than with the modular ratio \( n_L \) because of its higher stiffness; the deformations, considering the modular ratio for short term loading, reduce due to the increase of slab resistance.
VI Conclusions / Design and construction recommendations

By the comparison made between “REBAP”/“RSA” and the European Standards, it is verified the quantification of the minimum reinforcement is very different in each code. By the first one, a minimum longitudinal reinforcement is calculated by the equation $A_t = \frac{\rho b d}{100}$, not making any difference for minimum reinforcement for control of cracking. However, the EC4 considers, not only with the geometric characteristics of the concrete element, but also the stress state the structure is subjected to and the materials mechanics resistance as exposed in equation III-6.

Another factor leading to more conservative results in cracking control by the EC4 than by the Portuguese codes is the consideration of autogenous and drying shrinkage separately which leads to shrinkage values more approached to the real ones. Nevertheless, there is a shrinkage part that is not considered, the thermal shrinkage, which occurs at the young age of concrete, and which depends on the thickness of the slab, the kind of cement adopted and the casting conditions.

As it was discussed, the shrinkage affects largely the stresses observed in a concrete slab of a composite deck and can even induce cracking in the slab. Thus, measures to reduce shrinkage effects shall be adopted. The cracking control in a composite structure may be done by one of two ways: by the control of the concrete tensile stresses or by the limitation of crack widths.

In the first case, the control is done either during the construction process, by the limitation of external loads, by the use of difference of level of supports or by the slab casting by segments (this casting allows to control cracking at the support sections, making it similar to the cracking in the middle spans); either shrinkage control, specially in a short term.

In the second case, for the limitation of crack widths the following measures shall be adopted

- Provide minimum shrinkage reinforcement by the existent regulation, namely the Eurocodes, as it has been referred along this report;
- Limit the tensile stresses in the reinforcement in cracked sections; the EC4 allows an increase of the bar diameters when the cracking results from imposed deformations, as it is shrinkage;
- Limit the bar diameter and the bar spacing; the steel percentage required increases with the adopted bar diameter.

Besides the materials characteristics, shrinkage is also largely influenced by the geometry of the concrete element. Thus, when limiting the thickness element to 30cm, if possible, one may control the thermal shrinkage and its effects, avoiding a large difference of temperature between steel and concrete during hardening. This measure significantly reduces the thermal effects in concrete.

On the other hand, to control the autogenous shrinkage, firstly the concrete characteristics that it depends on, like the nature and the grain size of cement shall be controlled; and secondly, the water/cement ratio. To reduce the effects of this part of shrinkage a concrete with a ratio a/c>0.45 shall be
obtained, once the stresses in the liquid phase vary in opposite way to the pores sizes in the interface with the gas phase. Once this kind of shrinkage is a material phenomenon, uniform in the element volume, one phase of casting only is recommended or, in the case of bridges with composite deck, the use of precast slab elements; with this, there is no generation of mechanical effects, avoiding the restraint of imposed deformations caused by the linking connectors of the composite element and, consequently, the precocious concrete cracking.

Once the drying process of a concrete element starts immediately after removing the forms and depending on the curing of concrete conditions and the element dimensions, it is recommended a good curing, knowing that the drying process of an element with a sealed face corresponds to the drying of an element with double thickness. The concrete resistance when the forms are removed shall be higher than 16MPa and this process shall only take place after 24h.

To proceed to a greater cracking control it is proposed in [16]
- The use of a cement with a weak aluminate content;
- Complement the concrete grain size with a limestone;
- To optimize the density of the granular skeleton; and
- To reduce the initial water content using a superplastifiant.

In a constructive level, in [16] one advises
- To remove the forms of the concrete in a way that can provide the drying of the slab faces;
- The use of longitudinal pre-stress, that allows to completely avoid the slab cracking in service;
- The precast, which annuls the shrinkage effects at the young age of concrete; or casting in situ, leaving windows near the connectors, assuring this connection in 2nd phase.

To a short term verification of concrete cracking, the shrinkage effects, calculated in construction phase, shall be added to the temperature variation effects between the upper fiber of the steel beams and the lower fiber of the concrete slab. The shrinkage calculation in construction phases shall be done considering $n=6$, what may lead to higher tensile stresses on the concrete slab and therefore to the cracking of the slab.

VII References

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