Material Performance Lessons

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Abstract

The use of advanced technologies applying chemical and mineral admixtures, combined with conventional raw materials, can provide cost effective solutions to meet many of the challenges facing the concrete industry, especially with respect to production of cost effective durable structures, without the necessity of application of exotic new cementitious systems. The mechanisms controlling the performance of such concretes and the concepts of mix design are generally understood. Yet, in spite of this know-how, the performance on site frequently fails to meet the potential of these concretes. Some of the causes for this gap are associated with the influence of field practices which are often ignored, as well as the dependence on specifying and monitoring of concrete quality only on strength. The present paper discusses the various parameters and strategies which need and can be considered to change this state of affairs, including amongst other, quantifying the limitations of strength based specifications for durability, and the use of models and laboratory tests to optimize mix design for durability by considering expected curing effects, sensitivity of mineral additives to provide durability enhancement in different types of exposure conditions, as well as prediction of cracking which is based on quantifying the development of residual stresses in structures at early ages.

1. Introduction

Remarkable advancement in the development of a variety of high performance cementitious materials has taken place in recent years. It includes low w/c ratio-high strength concretes, high performance fibre reinforced cements exhibiting improved ductility and even strain hardening, as well as compositions which are more environmentally friendly, containing increasing contents of by-products and mineral admixtures. The drive for these developments stems from the innovative nature of researchers in these fields as well as the appreciation, sometimes intuitive, that in order to obtain more cost/effective, durable and environmentally friendly structures and buildings, there is a need for innovative materials technologies. The advent of materials science, which is guiding the R&D in cementitious materials is enabling remarkable achievements.

The fruits of such development effort can be observed quite vividly, and its mark on the construction industry is obvious. Several examples can be cited: (i) The use of high strength concretes to achieve cost/effectiveness of the structural system and enhance its durability, (ii) Use of fibre reinforced concretes for crack control to replace steel mesh in structures such as tunnels linings and pavements, (iii) Application of self compacting concrete for cost/effectiveness of the construction process, (iv) Extensive use of fly ash and slag to comply with environmental requirements and take advantage of economical environmental incentives.

These technological achievements and applications create a sense of satisfaction in the scientific and engineering community, but at the same time there is a growing appreciation that these advanced technologies are not making their mark in practice to the extent expected based on their potential, and thus there is a sense of the existence of some gap between knowledge and practice.

A sense of a gap is also evident when assessing the experience with long term performance of reinforced concrete structures. This is perhaps the most significant performance characteristic which is of high priority in modern concrete construction. The lessons gained from field practices are that the long term performance actually obtained in practice is too frequently less than what one would expect based on our understanding of the mechanisms controlling durability. These mechanisms are a function of the chemistry and physics of the interactions of cementitious materials with the environment. Even when the codes are followed, and implied in them is at least 50 years of service life, we encounter structures where repair is needed quite early on.

A variety of reasons can account for these gaps, some of them being economic in nature and others associated with technical issues. Some of these issues and gaps will be reviewed in this paper, with specific emphasis on durability performance and the development of advanced cementitious materials for that purpose. The paper will address the gaps and the strategy to bridge them by considering the system at three levels:

(i) Materials properties and actual performance
(ii) Materials properties and their performance as affected by structural interactions, in particular cracking
(iii) Performance and modeling approach

2. Materials properties and actual performance

The guiding principle in concrete technology is the “water to cementing material ratio, w/cm” concept. This is to a large extent the major parameter in standards and specifications with regards to durability (specifications of maximum w/cm ratio for different exposures), and the parameter used for designing concrete mixes for strength, Mindess et al. [1], Neville [2].

However, there are numerous instances where this principle is not sufficient, or perhaps even misleading, especially when there is a “gap’ between the influence of w/cm ratio on strength and durability performance. Special attention will be given in this section to effects which are associated with the concrete cover and with the strategies of using mineral admixtures. Both of these are of considerable practical consequences and they represent two aspects of concrete technology which are at the focus of special attention:
(i) Field practices and the performance of concrete
(ii) Environmental regulations and incentives which are behind the drive to enhance the use of industrial by-products, in particular fly ash, ground granulated blast furnace slag (GGBS) and calcium carbonate filler.

2.1 Field Practices and Quality Control

The main influence of field practices is related to consolidating and curing of the concrete. The nature of the effects of these two is almost obvious to any concrete technologist, as they both affect the porosity of the concrete and thus, improper practices should lead to more porous concrete and this should be reflected in lower strength and lower durability performance. From the point of view of the “typical” concrete technologist, who has been trained to relate the concrete performances to strength, such influences could be monitored and assessed by quality control measures based on strength evaluation. Apparently this concept is correct in the sense that there should be a clear cut correlation between lower strength (due to poor site practices) and reduced durability performance. This can be readily demonstrated when one calculates the service life of a reinforced concrete structure, based on available models for chloride and carbonation induced corrosion, as a function of the strength of the concrete. The result of such a calculation is shown in Figure 1, Bentur [3]. It can be seen that for concrete strength levels above about 45MPa (which is usually the range of concrete quality specified when durability performance is required) a small change in strength is accompanied by a relatively large increase in service life. This implies that improper field practices which may lead to small reduction in strength (which is perhaps within acceptable limitations from the point of view of the specifications for structural performance) will result in a relatively large reduction in durability performance. This observation underlines the limitation of strength as a quality control measure and its failure to serve as a reasonable tool for assuring that indeed good field practices were applied, especially in concretes which are specified for durability performance.

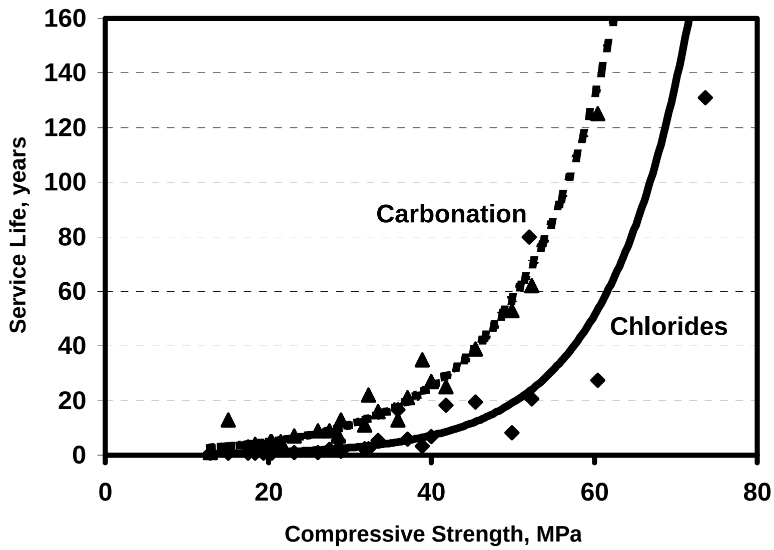

Figure 1: Calculated service life of a reinforced concrete structure for carbonation and chloride steel induced corrosion, after Bentur [3].

To provide more quantitative estimates and demonstration of this limitation, the effect of curing on strength and durability performance was studied and the results were presented in terms of mechanical performance (strength) and durability performance (service life predictions), Bentur [3], Bentur and Baum [4]. Durability performance was quantified by determining experimentally the effect of curing on chloride diffusivity and rates of carbonation, and based on these parameters and relevant models the service life was calculated. The influences of the curing procedures on strength and service life are presented in Figures 2 and 3, in terms of values which are relative to the ones derived for what is considered a “good curing practice”, namely curing continuously in water for one week.

It can be seen that improved curing procedures are associated with increase in strength and service life; yet the relative effect of curing on service life is much greater than on strength. Deficient curing will result in 10 to 20% reduction in strength and 30 to 60% reduction in service life. This reduction in service life can amount to as much as 20 to 40 years, and the consequences can be estimated to be much more critical than the reduction in strength.

Observations of this kind lead to some comments reflecting the practice and laboratory testing. The enhanced service life in “exceptional curing” (the 28 days ponding in standard laboratory curing) is indicative of the potential of a given concrete. This enhanced potential may be partially achieved only when the service conditions are wet, which is not always the case.

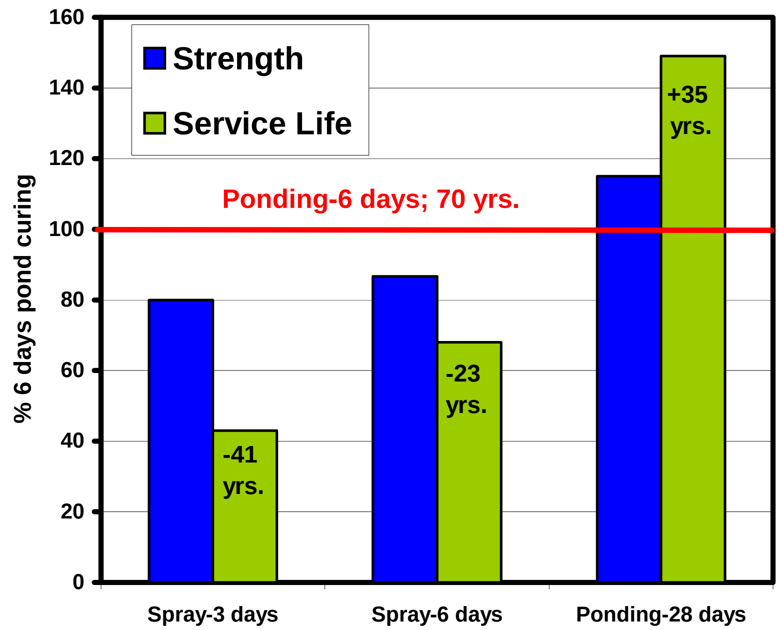

Figure 2: The effect of curing conditions on strength and service life in carbonating conditions, relative to curing by ponding for 6 days. The concrete strength is 50 MPa (100mm air dry cubes), after Bentur [3].

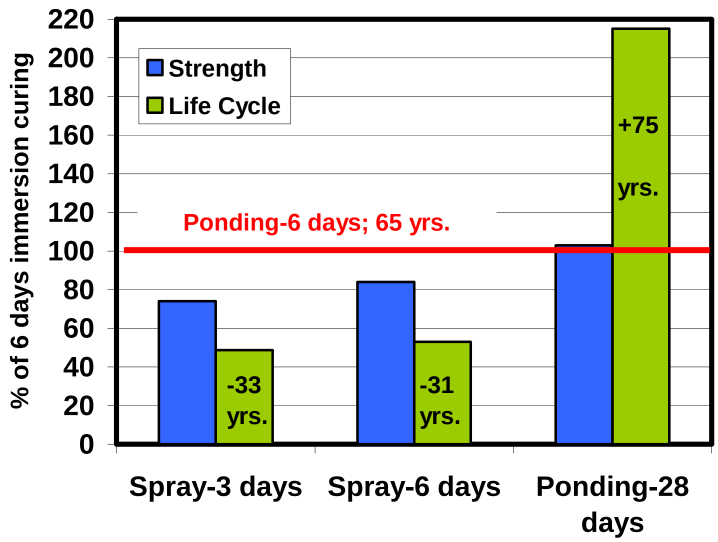

Figure 3: The effect of curing conditions on strength and service life in chloride conditions, relative to curing by ponding for 6 days. The concrete strength is 75 MPa (100mm air dry cubes), after Bentur [3].

The data shown here can demonstrate to practitioners and specifiers, that although there is a general trend which provides correlation between service life (i.e. durability performance) and strength, it is not sufficiently sensitive from a practical point of view: small differences in strength, which may be somehow tolerable, may be associated with much larger changes in service life, which are unacceptable, as they involve shortening of service life by more than 20 to 40 years. The data in Figures 2 and 3 can also bring across the message of the significance of curing on site, and the limitations of assessing deficient curing and its consequences by evaluating strength only.

The trends demonstrated above with respect to curing, reflect the response of the concrete cover to curing practices. It is expected that poor curing will have a much more limited influence on the concrete strength, since the core of the concrete does not dry out as fast as the cover, and the structural performance is to a large extent dependent on the properties of the bulk concrete, whereas durability is sensitive to the properties of the concrete cover.

Finishing of the concrete by trowels is a common practice for floor finishing, and the compaction of the surface by such means, after the bleed water has escaped, is known to be an efficient means for obtaining a floor surface which meets requirements such as being crack free and abrasion resistance. There is considerable field experience with such surface finishing techniques, but much less systematic studies demonstrating and quantifying such influences. Some indications of such influences can be scarcely found in the literature, and one of them is demonstrated in Figure 4, after a publication by Soroushian et al. [5]. It demonstrates the influence of surface finishing on elimination of plastic shrinkage cracking, and highlights the view of many practitioners that if good practices are applied in the construction process, there is no need for special means such as fibre reinforcement for plastic shrinkage crack control.

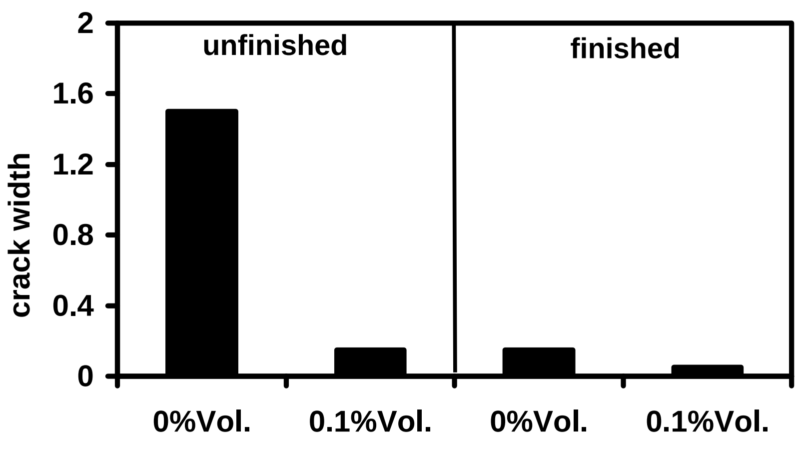

Figure 4: Effect of fibrillated polypropylene fibre and mode of finishing on plastic shrinkage cracking, after Soroushian et al. [5].

In order to deal with these issues there is an extensive R&D effort to develop and apply tests which could characterize the properties of the concrete cover, using techniques which could be sufficiently friendly to be applied on site. They are usually based on evaluation of penetration of gases or liquids, or characterization of electrical properties, in particular electrical resistance. A review and assessment of such tests is provided by publications of a RILEM Technical Committee dealing with the non-destructive evaluation of the properties of the concrete cover, RILEM TC 189-NEC, Torrent and Luco [6], RILEM TC 189-NEC [7]. An indication of the kind pf performance which can be achieved by such tests is presented in Figure 5 (after Kubens et al. [8]), showing how an air permeability tests (Torrent [9]) can provide indications for the increased permeability of the concrete cover due to deficient water curing. The sensitivity here is much greater than the one obtained by strength measurement, and the changes in the air permeability better correlate with changes in chloride penetration and carbonation. The limitations of many of these tests is that the permeability is very sensitive to the moisture conditions of the concrete cover, and for the test to be meaningful there is a need to develop methodologies to account for this effect. Several of those have been reported, Torrent and Luco [6], Romer [10].

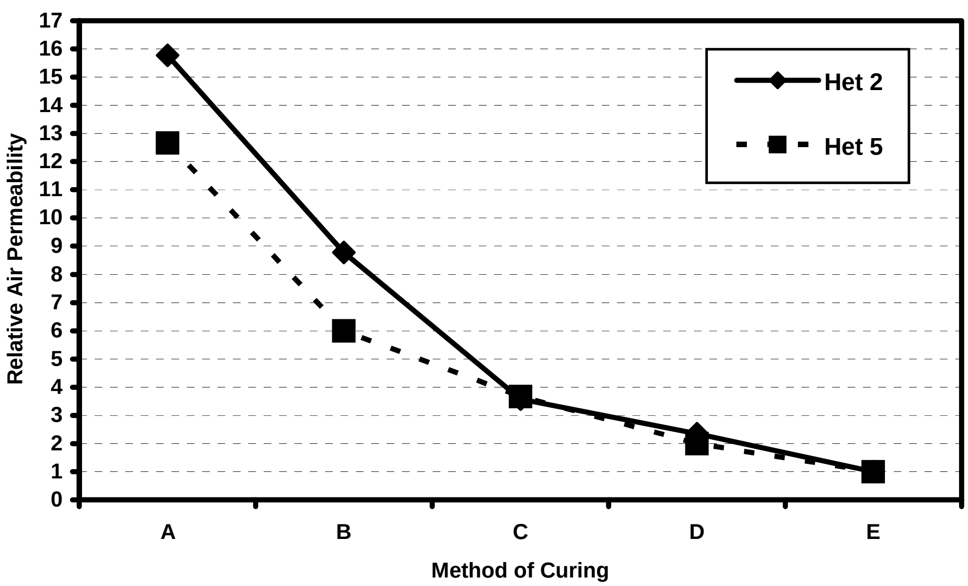

Figure 5: Effect of curing method on the relative air permeability coefficient of the concrete cover in a simulated laboratory test, for 20 MPa (Het 2) and 50 MPa (Het 5) concretes, after Kubens et al. [8].
E - 28 days water ponding curing; D - 7 days ponding water curing; C - 7 days of intermittent water curing; B - 3 days of intermittent water curing, A – continuous air curing after demolding at 1 day.

When considering the effectiveness of the concrete cover one needs to take into account the cover thickness, in addition to the concrete quality which was discussed previously. Since measurement of the cover depth and setting the steel in place in the formwork to achieve the required depth are simple and straightforward, there is a tendency to take for granted that this characteristic of the cover depth is always achieved and is well controlled. However, the practice is quite different, and anyone with experience in monitoring of deteriorated structures is aware of the fact that quite frequently the deterioration is the result of lack of cover depth. This issue has been highlighted and reviewed by Neville [11], demonstrating that quite frequently the cover varies considerably from the specified values. This is the result of improper site practices and quality control, as well detailing of the steel reinforcement which make it unlikely to meet cover requirements. It can be shown that service life is proportional to the square root of the cover thickness, and thus deviations from specifications can be quite substantial in terms of the durability performance. For example, a deviation of 20% in cover thickness (e.g. from 50mm specified to actual 40mm) is expected to result in almost 36% reduction in the service life (the 40 mm cover will provide service life which is $(40/50)^2$ of the expected service life if the 50mm requirement was kept).

2.2 Utilization of Mineral Additives

The use of mineral additives in concrete has a long record, and a successful one, in particular with respect to additives such as fly ash, GGBFS and silica fume. In recent years metakaolin was added to this list, with the view of applications where high durability is required. The renewed drive for the use of these materials is obviously related to environmental regulations which are accompanied by economic incentives and sanctions. From the technological point of view the drive is accompanied also by the recognition of the advantages in performance which can be gained by such materials and their potential to provide solutions for durability requirements which can not be readily met with normal Portland cement. On top of these technical advantages there is an additional incentive for greater use of these materials, which is the advent of the ready mixed concrete industry with automated and computerized facilities which enable the production of concrete under high level quality control measures. These modern production facilities paved the way for more efficient use of mineral additives as one of the components of the concrete mix, which can be readily adjusted to meet a variety of requirements.

These developments are reflected in modern specifications and standards, such as the new EN 206, which provides guidelines for the use of such additives in the mix design of concrete, based on performance concepts. A common approach, which is also defined in the EN 206 standard, is quantification of the quality of the additive in terms of an efficiency *k*-factor. It is calculated on the basis of the effective water/binder ratio of the mix with the additive, $(w/b)_{eff}$, which provides the same performance as a reference concrete having water/cement ratio of $(w_0/c_0)_{ref}$:

$$(w_0/c_0)_{ref} = (w/b)_{eff} = w/(c+kA) \qquad (1)$$

where k is the efficiency factor, A and c are the additive and cement content, respectively, and w the water content.

If the reference concrete mix and the mix with the additives have the same water content (e.g. $w_0 = w$, implying that the additive does not lead to change in the water content required to maintain the same workability), the value of k can be interpreted simply as the amount of addition which is needed to replace 1kg of cement.

The "common wisdom" in the technical assessment of the contribution of these materials (and thus, their k value) is based on the densening of the pore structure when they interact with the Portland cement hydration products. On this basis, there is a tendency to use the rational which was discussed in the previous section, of relating the overall performance of these mineral additives to the strength enhancement which they can provide. Thus, the main clause defining the *k*-factor in EN 206 is based on strength performance. Although the EN 206 implies that the durability performance may require a different mix design, many practitioners use the *k*-value which is based on strength evaluation. The *k*-factor values recommended in EN206 are in the range of 0.2 to 0.4 for fly ash and 1 to 2 for silica fume.

Compilation of efficiency factors for strength, carbonation and chloride diffusivity of fly ashes in Figure 6, from a variety of references in the literature ([Babu and Rao [13], Babu and Kumar [14], Babu and Prakash [15], Papadakis and Tsimas [16,17], Papadakis [18,19], Cyr et al. [20], Oner et al. [21], Hobbs [22] and Bentur & Baum [4]), demonstrates the marked differences between their effectiveness for the various performances. The strength efficiency is in the range of 1 to 1.5 (much higher than the recommended values in EN 206), the carbonation efficiency is less than 0.6 and the chloride penetration resistance efficiency is about 2. Obviously, relying on strength as a measure for effectiveness for durability performance can be misleading. Even for the durability performance, significant differences are observed for different types of exposure conditions, with the fly ash being much more effective for chloride environment and much less effective for carbonating exposure conditions.

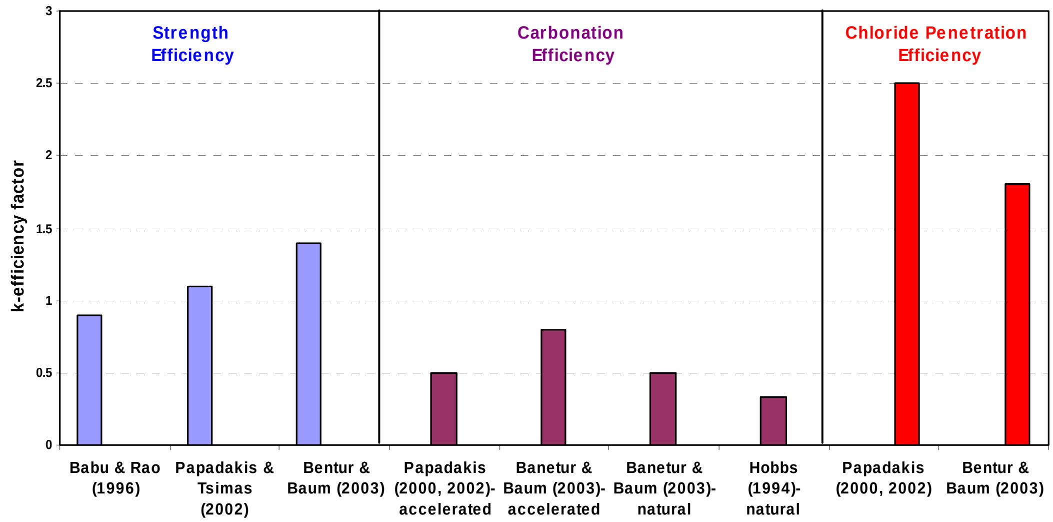

Figure 6: k-efficiency factors for strength, carbonation and chloride penetration for 90 days of fly ash concretes, based on data compiled from the literature, after Bentur [12].

When referring to the durability performance of fly ash substitution, there is a need to provide greater attention to the effect of deficient curing practices, which mainly influence the concrete cover, which is the zone where the effectiveness of the fly ash should be materialized. Simulation in the laboratory of the effect of various curing procedures on the effectiveness of the fly ash in providing strength and durability performances are shown in Figure 7. The data clearly demonstrates the small sensitivity to curing with regards to strength performance, and the extreme sensitivity when considering chloride environment with respect to chloride penetration. These extreme differences demonstrate again the limitations of using strength as a criterion for quality control in structures where the quality specified for the concrete is based on durability considerations.

The complexities highlighted above indicate that there is a need for more sophisticated approaches to effectively materialize the potential of mineral admixtures as a means for durability control The shortcomings of standards to provide the tools required have been highlighted by Thomas [23]. Clearly, the way to go here is by development of performance specifications, which include accepted test methods as well as criteria.

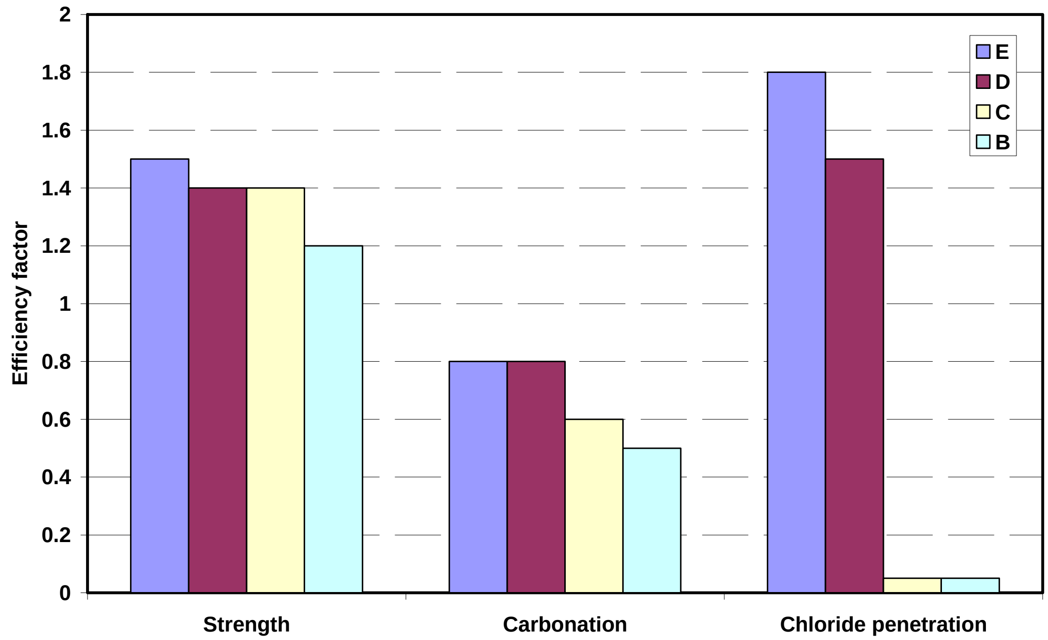

Figure 7: Effect of curing on the 90 days efficiency factor of type F fly ash determined for strength, carbonation and chloride penetration; E - 28 days water ponding curing; D - 7 days ponding water curing; C - 7 days of intermittent water curing; B - 3 days of intermittent water curing, based on the data in Bentur and Baum [4].

One such approach which is gaining momentum is based on durability indicators, which reflect in-situ properties of concrete with respect to different types of exposure conditions. This topic has been recently the subject of a dedicated workshop (Baroghel-Bouny et al. [24]) and two excellent review papers, provide an in-depth analysis of the potential of this approach (Baroghel Bouny [25], Alexander et al. [26]). Several indices have been developed based on properties characterized in tests such as chloride penetration, air permeability, oxygen permeability, water absorption, electrical resistance. For specified exposure conditions a suite of requirements which are combinations of several of these indicators have been set, as guidance for the performance required from the concrete.

3. Materials Performance in the Structure

The actual performance of the material in the structure can be quite different than that predicted by laboratory testing. Numerous factors can account for this difference, but perhaps the three most influential from a practical point of view are compaction, curing and cracking. Standard materials testing for quality control do not take these effects into account, and thus the actual performance in practice may deviate from the one expected based on the materials properties. In-situ testing is still limited in capturing such effects. However, some of these effects can be simulated in the laboratory and in doing so it is possible to demonstrate their consequences, and thus provide incentives for keeping good practices on site by setting the mechanisms to ensure that they are being followed. Examples of such simulations and their consequences was given in section 2 for curing effects. In the future, if test methods for in-situ monitoring of the quality of the concrete cover are developed, such as those discussed in section 2, it would be possible to rely on them for quality control. These could serve as alternatives to the current practice of specifying how curing should be done, and who is to oversee and inspect that the curing practices are actually being implemented on site.

The issue is much more complex when it comes to control and assessment of the influence of cracking in the structure. The onset of cracking and their extent is a complex function of materials properties, structural behavior and loading, as well as environmental conditions. Thus, to deal with this issue there is a need for a system approach to account for the properties of the materials, and consider them simultaneously with the whole structure. This can be done by modeling approaches, and some are highlighted in section 4. Yet, there is room to discuss in this section what might be the consequences of cracking on performance, in particular durability performance.

In a recent international workshop held in Evanston (Bentur et al. [27]) these issues were highlighted, and the discussion here follows many of the topics dealt with in that meeting. Some of the questions that need to be addressed are highlighted below:

- Are we indeed having more cracking problems with the use of modern concretes, and if so, what are the implications with regards to performance in conjunction with life cycle costs (based mainly on field observations and experience)?
- What is the significance of crack control in relation to durability?
- Are the current codes adequately reflecting the impact of cracking (requirements for limiting crack width in design codes, requirements for materials properties in tests such as shrinkage and restrained shrinkage)?

The greater sensitivity to cracking in high strength-low w/b ratio concretes has been of some concerns, although field observations are not always conclusive (e.g. French et al. [28], Schmitt and Darwin [29], Krauss and Rogalla [30], Saadeghvaziri and Hadidi [31]). Yet, extensive research has highlighted the increase in early age shrinkage (autogenous shrinkage) in such concretes which can be the cause for greater sensitivity to cracking, as clearly seen in several studies in which such concretes were investigated in restraining rigs, Bentur [32]. The mixed views on the cracking sensitivities observed on site might be perhaps be reconciled by considering the “residual stress” build-up, which was low relative to the strength in the past (“safe” curve in Figure 8), but increased to a levels close to the strength (“unstable” curve in Figure 8). This leads to a situation of sensitivity to cracking, with the transition from the “unstable” to the “cracked” condition becoming a “statistical” phenomenon, depending on many secondary effects (environment, settlement, etc.) which may easily tip the balance from unstable to cracked. This may explain the somewhat erratic nature of cracking observations on site.

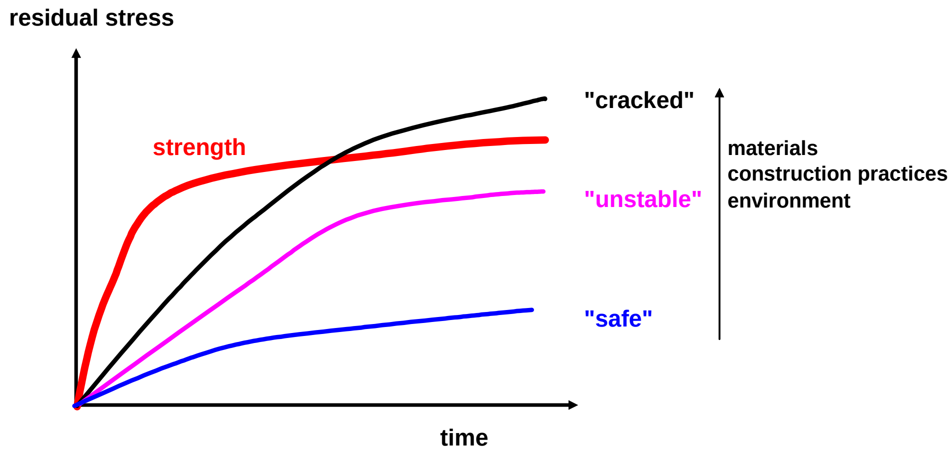

Figure 8: Schematic description of the residual stress curves, and their shift to the unstable zone due to demanding construction practices and changes in the concrete mix composition, after Bentur et al. [27].

The issue of crack control and durability has also attracted considerable attention, especially with respect to the requirements in standards and specifications, which usually limit crack width in the range of up to 0.3mm, depending on the exposure conditions. Several reports (Beeby [33], Schiessl [34], Ohta [35]) suggested that crack width is not an important parameter in controlling the overall durability of steel in concrete since it affects only corrosion initiation but not corrosion propagation. Apparently, such an approach is in contradiction with the requirements in standards for crack control (e.g. ACI [36,37]). This apparent conflict was addressed in the recent workshop (Bentur et al. [27]), and some made the comment that the rational behind the crack limitations in standards is geared to life safety as a minimum legal requirement and therefore the specifications for crack width are intended to assure safety but not serviceability.

Serviceability is a growing focus in terms of requirements for performance and costs during the life time of the structure, and it covers issues such as durability, water tightness and appearance. Thus, crack control to assure serviceability may be a more stringent requirement than currently specified in the codes. This can be clearly seen in several recent studies in which permeability through cracked concretes was evaluated (Lawler et al. [38], Aldea et al. [39,40], Edvardsen [41], Burlion et al. [42], Reinhardt and Jooss [43], Schiessl and Raupach [44], Francois and Arliguie [45]) showing that there is a threshold crack width below which permeability is extremely low, not much bigger than that of the uncracked matrix (Figure 9). This critical crack width is in the range of 50 to 100 μm. The concept of a critical crack width for durability, even if applicable, is unlikely to be a general one, and may depend on the type of performance; e.g. it would be different for water tightness, chloride penetration and aesthetics. Within this context one should consider the "microstructure" of the crack (e.g. tortuosity, continuity, etc.) which may be a more relevant measure than crack width, which is just an imprint on the surface of the concrete.

When considering crack microstructure and the critical crack width (being less than 100 μm), it should be noted that in several studies of flow through cracks, it was concluded that in this range of width there is a process of crack healing which can take place when water is moving through it (Edvardsen [41], Reinhardt and Jooss [43], Schiessl and Raupach [44]). This mechanism may have to do with the complex microstructure of a crack, which when sufficiently fine may allow for self healing to take place effectively. Thus, the threshold crack width concept may have something to do with such self healing mechanisms.

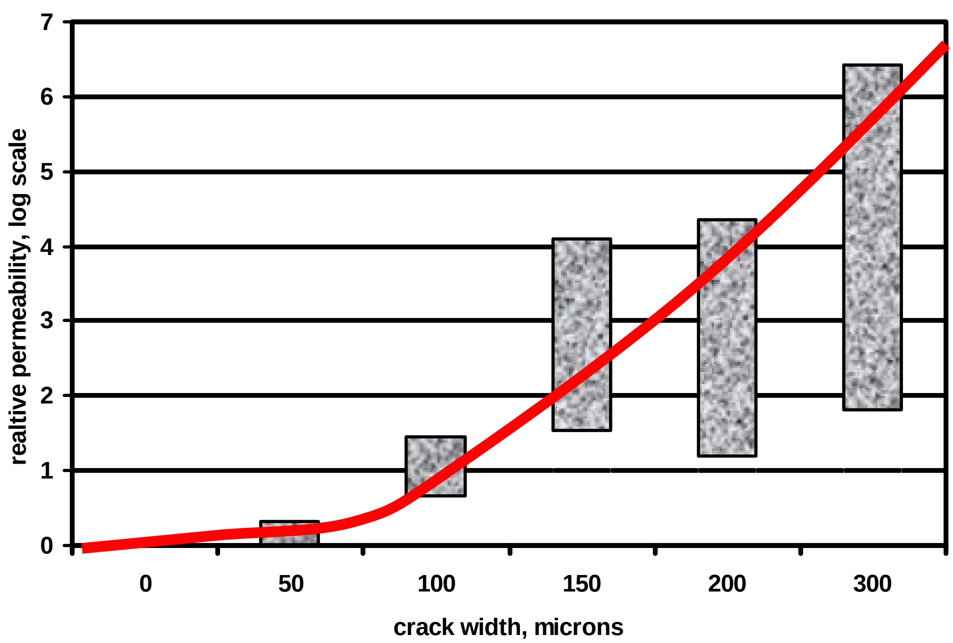

Figure 9: Effect of crack width on the relative permeability of water flow, based on compilation from the literature (Aldea et al. [39,40], Evardsen [41], Reinhardt and Jooss [43], Schiessl and Raupach [44], Burlion et al. [42]).

4. Performance and Modeling Approaches

It was already noted in the previous section that the overall performance of a structure with respect to durability may not be always effective unless a comprehensive approach is taken. For this to happen there is a need for modeling of a whole system in which the materials properties are important inputs, but not the only ones. In particular, two types of modeling are of interest with regards to performance:

(i) Modeling of the structure from the time of casting until maturity, to control cracking; at this stage, the properties of the materials, the construction practices and environmental conditions exert considerable influence, and they all need to be taken into account in the deign and the construction stages to assure crack control.

(ii) Modeling of durability performance taking into account that the overall performance should take into consideration several processes, physical-chemical-mechanical, which may take place at the same time.

4.1 Modeling for crack control

Cracking performance is usually in the domain of the structural engineer who applies design tools for the mature structure, namely after the concrete has achieved its matured properties. However, in practice we know that in many instances cracks can occur at earlier stages, and they are induced by environmental loading, such as shrinkage (autogenous in HSC as well as drying) and thermal (which may be high in HSC due to the high cementitious materials content). Even if these loadings are not leading to cracking they may induce residual stresses in the structure which may increase its susceptibility to cracking later on (Figure 8).

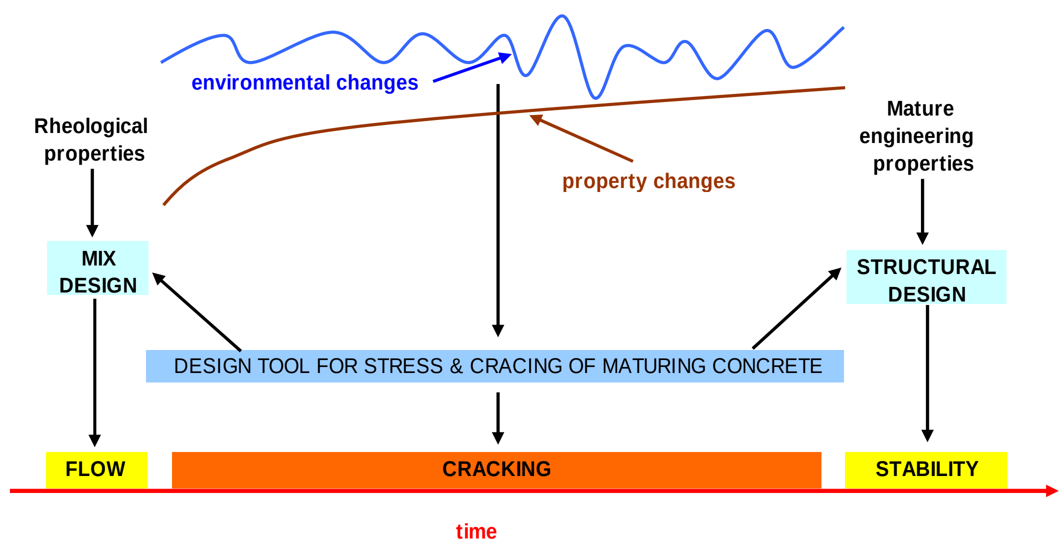

Figure 10: Schematic description of design tools for concrete and concrete structures and their range of applicability.

Cracking at early stages is dependent not only on the materials properties and environmental conditions, but also on construction practices (e.g. curing, formwork removal). Thus, modeling which takes into account all of these effects should provide inputs to the engineers in charge of the materials selection and the construction practice, and not only to the structural engineer. Examples of models of this kind are the HIPERPAVE II model developed in the US and the FEMASSE model developed in the Netherlands (Beek et al. [46], Schlangen et al. [47)]). The range of their applicability is shown schematically in Figure 10.

The applicability of these approaches was demonstrated in recent publications in which the cracking risks in bridge decks was calculated, Li et al. [48,49]. In this study the nature of the concrete in the deck was taken into account (0.3 to 0.5 w/c ratio, with and without shrinkage reducing admixture) as well as the girder on which it is placed (the restraining tensile stresses developed in the concrete in the deck depends on the restraint provide by the girder - higher restraint and stresses in the more rigid girder). Some results representing these effects are presented in Figure 11, showing that the changes in the mix design (SRA admixture) and the girder stiffness (HSC vs. NSC prestressed girders) can eliminate cracking. It should be noted however, that even when cracking is eliminated, there is a build up of a residual stress, but it is lower than the strength. Obviously this residual stress poses an inherent risk, and this can be quantified and serve as an estimate to cracking risk. Some values obtained by this calculation are shown in Figure 11.

These are examples demonstrating the power and need for such modeling. It can provide the platform for coordinating between the structural engineer, materials engineer and construction engineer who is in charge of the construction practices, to produce a crack free structure.

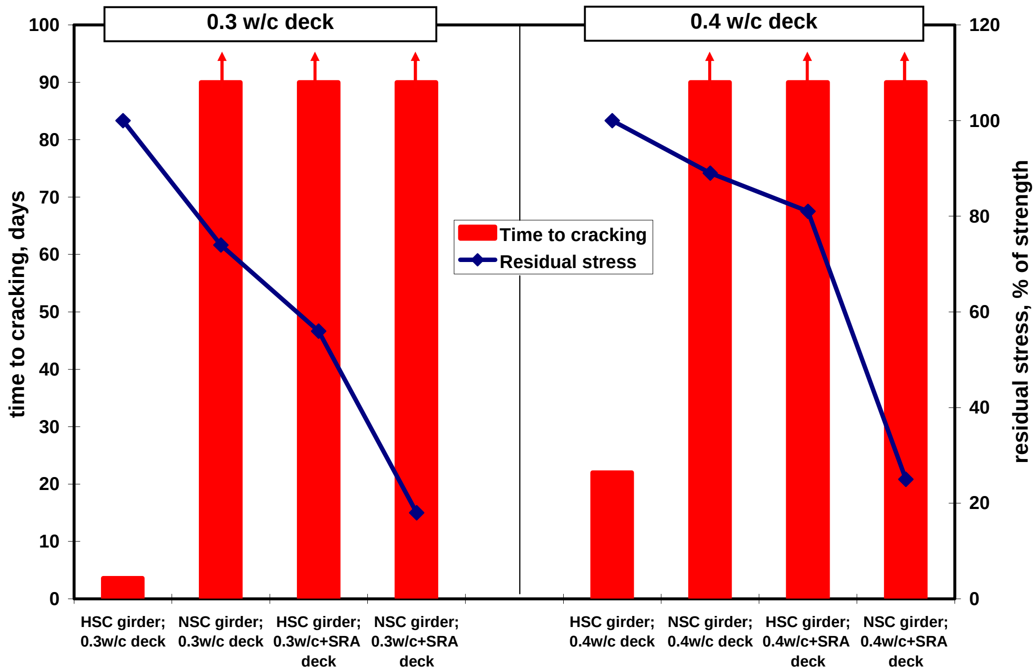

Figure 11: Time to cracking and residual stresses in bridge decks, compiled from the data in Li et al. [48,49] for 0.3 and 0.4 w/c ratio deck concrete, with and without shrinkage reducing admixture (SRA), cast on girders which are made from prestressed normal or high strength concretes (NSC and HSC, respectively). Modeling runs were carried out up to 90 days, and thus the 90 days bars indicate that no cracking occurred within that time period.

4.2 Example of cracking and residual stresses in a precast pretensioned girder

In order to achieve greater durability, owners are now expecting that finished products, such as precast pretensioned elements, be totally “crack free”. This requires more detailed analysis, considering the different stages in the construction of members. Figure 12 shows the cross section of a 2300 mm deep bridge girder that was produced in an outdoor precasting plant in eastern Canada, Khan et al. [50]. The predicted temperatures in the steam-cured girder, just before the removal of the steel forms are shown in Fig. 12(a).

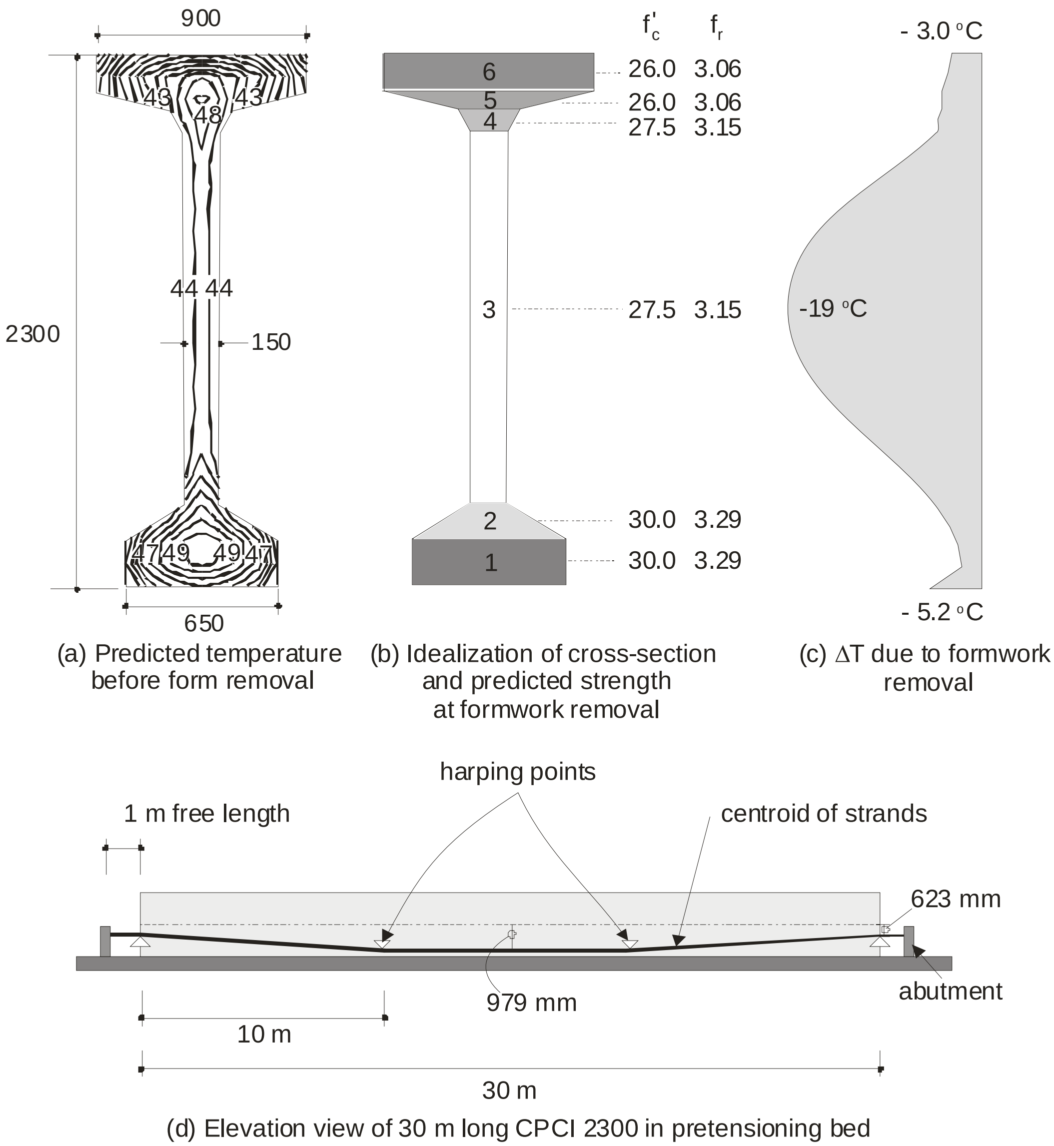

Figure 12: Precast pretensioned bridge girder during construction, after Khan et al. [50].

The predicted concrete strengths in different parts of the cross section are given in Fig. 12(b), Khan et al. [51]. After formwork removal (at an age of 20 hours), it was assumed that the ambient temperature was 5ºC to simulate a low temperature during production of some girders. The predicted changes in temperatures after the removal of the forms are shown in Fig. 12(c). The maximum temperature drop in the web occurs about one hour after stripping the forms, with the thin web experiencing a drop of 19ºC, while the thicker flanges have temperature drops of about 3ºC and 5ºC. At this stage, the girder will have tensile stresses in the web due to the large thermal gradient in the web, relative to the flanges. In addition to these induced internal thermal stresses, the beam will also experience a significant tensile restraining force due to the fact that the 36-13 mm diameter pretensioning strands are fixed at the stressing abutments during the sudden cooling of the girder (see Fig. 12(d)).

The longitudinal restraining force in this case can be estimated on the basis of compatibility of deformations, and its value was found to be 479 kN. Figure 13(a) illustrates the strain conditions and stresses in the concrete determined from a plane sections analysis using the computer program Response 2000 (Bentz and Collins [52]) and assuming that the curvature of the girder at the time of form removal was zero.

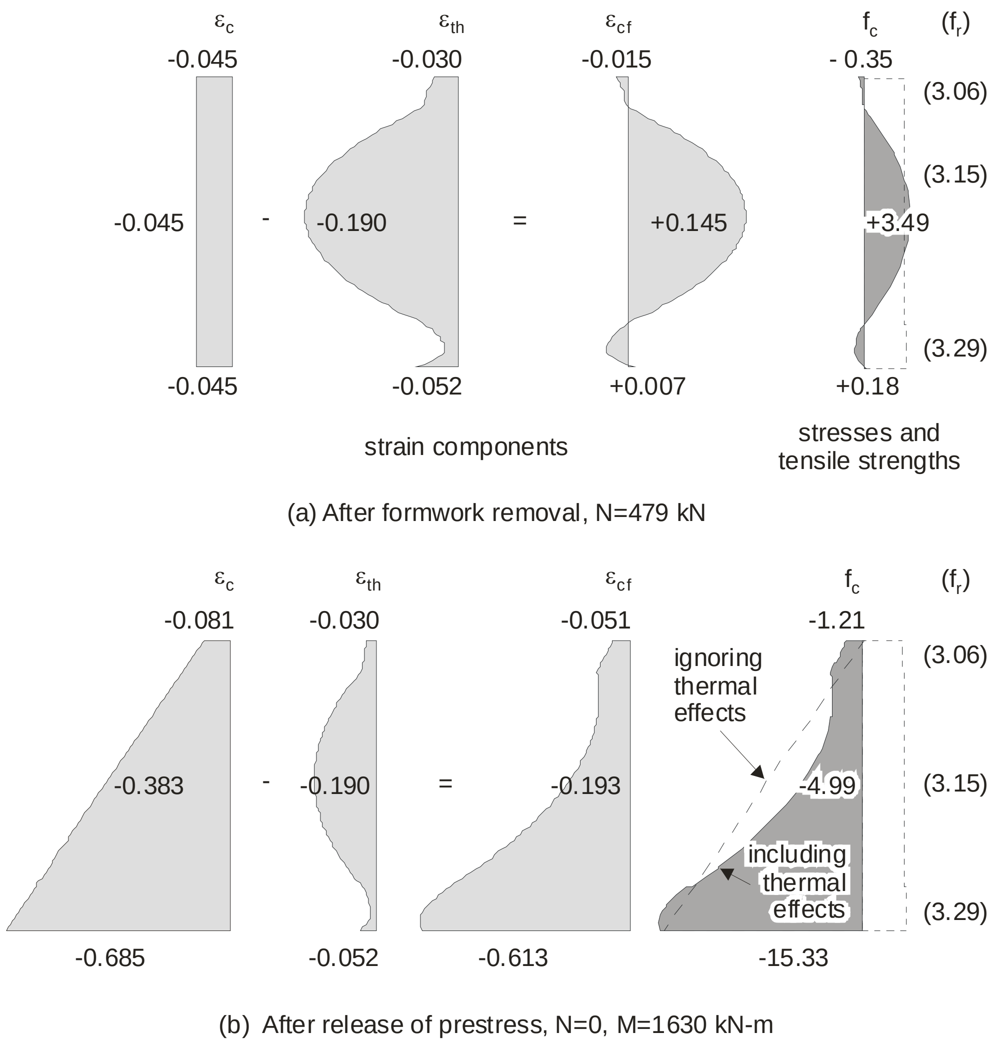

Figure 13: Predicted concrete strains and stresses in precast pretensioned girder

The strains causing stress in the concrete, ε_{cf}, are determined by subtracting the thermal strains, ε_{th}, from the total strains, ε_c. In the analysis it was necessary to use the early-age properties (Mitchell et al. [53]) for the concrete in the different parts of the cross section (see Fig. 13(b)). The predictions indicate that a maximum tensile stress of 3.49 MPa is reached in the central portion of the web. The modulus of rupture of the web is about 3.15 MPa and hence cracking is predicted. A separate analysis assuming minimum amounts of longitudinal reinforcement in the web, results in a predicted 600 mm long, vertical crack, with a maximum width of 0.13 mm as experienced in some girders at the precasting site. This example illustrates the need to provide minimum amounts of longitudinal reinforcement in the web regions and demonstrates that a very significant residual tensile stress is created in the web. Figure 13(b) shows the predicted concrete stresses immediately after the release of the prestressing and compares the resulting stresses, with and without thermal effects. This residual tensile stress can lead to early shear cracking of the girder in service. Creep effects would reduce these stresses with time. The fact that the prestressing causes a significant compressive stress in the web would result in closing of the cracks. It has been demonstrated that autogeneous healing of the crack interfaces can take place if the web is kept moist, Zia and Caner [54]. The sequence of detensioning of the inclined and the straight strands must also be considered, Kannel et al. [55]. One way of decreasing the restraint effects of the strands during formwork removal is to increase the free strand length at the ends of the girder. A minimum total free strand length of 10% of the bed length is recommended, Kannel et al. [55].

4.3 Durability Performance

Durability performance is often specified in terms of the resistance of the concrete to penetration of deleterious substances. A notable example is chloride penetration which is quantified by the ASTM C 1202 by determining the penetration of chlorides into a concrete specimen under an electric potential gradient; the quality of the concrete is quantified in terms of the charge which passes after 6 hours or the resistivity which is measured at the commencement of the test. These two parameters were shown to correlate with diffusivity.

However, it should be born in mind that the performance of the concrete to steel corrosion will be dependent on other factors too, such as chloride threshold value, the build up of chloride on the concrete surface and the change of the concrete diffusivity over time. Thus, relying just on the concrete diffusivity may be misleading, and there is a need to take the whole corrosion process into account for assessment of performance. Several models have been developed for this purpose. Some of them are quite sophisticated and take into account numerous chemical, physical and environmental factors (e.g. [Marchand [56]) while others involve some simplifications which make them particularly attractive for engineering (e.g. LIFE 365 Model, Thomas and Bentz [57]).

There is a need to consider such overall modeling, since it is not sufficient to compare between the properties of materials if decisions are to be made to chose the alternative which will provide the best cost/effective overall durability performance. An example demonstrating this system is the replacement of cement with fly ash to provide enhanced durability to the steel in chloride exposure conditions. The apparent and simplistic way for the design with fly ash would be based on comparing the penetration resistance as expressed in terms of the efficiency *k*-factor for penetration in Figure 6. However, when the overall process is considered, there are other influences that should be taken into account, such as the reduced level of cement content in the fly ash mix which may be associated with reduction in the threshold chloride level required for depassivation, as well as the effect of time, which is expected to lead to greater reduction in the chloride diffusion coefficient in the fly ash concrete, due to the continued pozzolanic reaction. The impact of the pozzolanic reaction may be slow, but its influence may be quite marked if time spans of 10 to 50 years are considered. This reduction will obviously depend also on the environmental conditions, which need to be wet for significant pozzolanic reaction to occur. The recommended relation in the LIFE 365 model to account for the diffusion coefficient is:

$$D(t) = D_{ref}\left(\frac{t_{ref}}{t}\right)^{m} \qquad (2)$$

Higher values of *m* in equation (2) imply greater reduction over time of the diffusion coefficient. In the life-365 model *m*=0.2 was suggested for Portland cement and *m*=0.5 for a system with 40% fly ash. Recently, it was suggested that the reduction in diffusion coefficient for fly ah concrete occurs mainly in the first 2-3 years, and values of *m* lower than 0.35 should be used, Aldykiewicz et al. [58].

These considerations suggest that relying on the efficiency *k*-factor determined from the chloride penetration test (i.e. a materials property) may not adequately represent its overall effect and efficiency with regards to the protection of the steel. In order to estimate the differences between the two, some sensitivity calculations were carried out based on the LIFE 365 model. This analysis was intended to determine the time to initiation using different assumptions, in order to calculate the efficiency *k*-factor on the basis of the effect of the chloride on the initiation time. The data in Bentur and Baum [4] was used for this purpose. The assumptions made are the following:

(a) Chloride threshold: proportional to the actual cement content of the mix (0.4% by weight of cement), implying lower values for the fly ash mixes in which cement was replaced.

(b) Reduced diffusion coefficient with time: several scenarios were modeled: (i) reduction similar to Portland cement concrete (i.e. *m*=0.2) which implies no continued excessive pozzolanic reaction beyond 90 days, (ii) *m*=0.52 recommended in the Life 365 program for 40% fly ash substitution, and (iii) intermediate value of *m*=0.35, as suggested in Aldykiewicz et al. [58].

The efficiency *k*-factor values calculated on this basis are outlined in Figure 14. If no pozzolanic reaction is expected (i.e. *m*=0.2, which may be the case in dry environmental conditions where the concrete cover is kept dry) the *k*-efficiency factor calculated for service life is much smaller than that calculated on the basis of the diffusion coefficient. However, if pozzolanic reaction is assumed to occur (i.e. *m*=0.35 which is more typical of humid environmental condition) than the *k*-efficiency factor is much higher for service life than for diffusion. Values of *m*=0.50 will result in even much greater efficiency coefficients for fly ash, bigger than 3, but in view of the data reported by Aldykiewicz et al. [58] this should be questioned. This demonstrates the significance of modeling when assessing the effect of changes in the concrete composition, and the limitation in relying on a single materials property, even if it seems to be a relevant one.

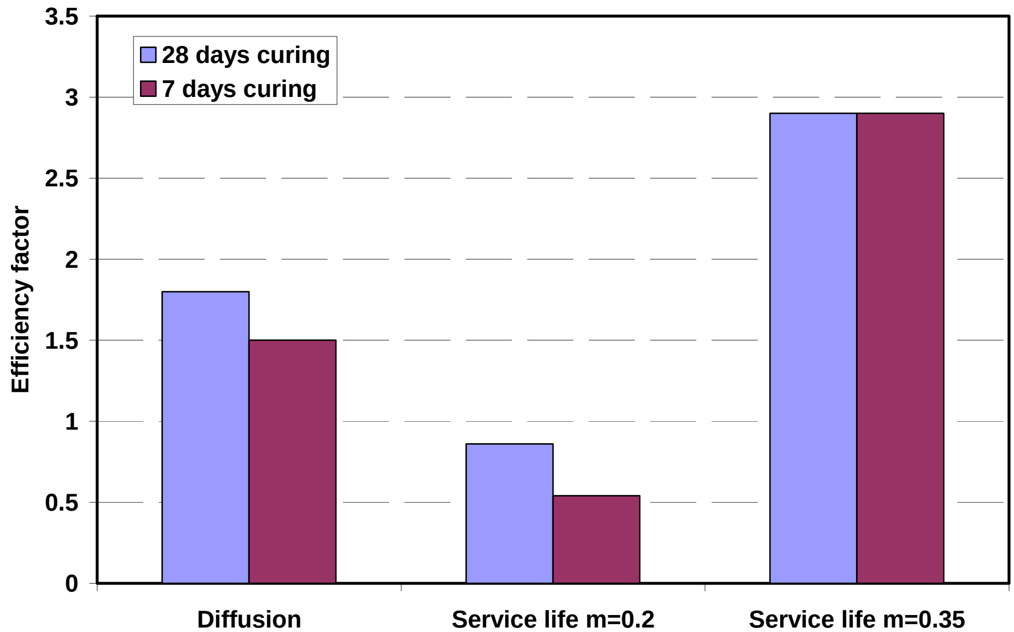

Figure 14: Fly ash efficiency *k*-factors for durability of steel corrosion in concrete in chloride environment, after Bentur [12].

Another demonstration of the significance of modeling, with regards to the material design of a concrete mix to achieve cost/effective durability, is in combining different types of admixtures in a single mix, with each having a different mechanism for promoting durability performance. In the case of chloride corrosion in concrete, the mix design can affect several factors which will control the service life, Bentur et al. [59,60]:

- Fly ash and silica fume to reduce the diffusion coefficient; reduction by an order of magnitude is feasible.

- Water repelling admixture which reduces the build up of chlorides on the surface of the concrete in wetting/drying exposure conditions; it is well documented that the surface chloride concentration build-up is proportional to the capillary absorption coefficient, and the latter can be reduced by a factor in the range of 3 to 5 when water repellent admixtures are incorporated in the concrete.
- Corrosion inhibitors which elevate the chloride threshold value at the steel surface; increases of up to about 10kg/m^3 have been reported.

Each of these three means had been advocated for use for durability enhancement. However in order to develop a mix deign where all three are used at the same time, to optimize for durability performance, there is a need to consider their combined influence, and this can only be done by modeling. Recently, the LIFE 365 model was used for this purpose by providing inputs on chloride surface build up, diffusion coefficient and chloride threshold which are influenced by the three technologies, Bentur et al. [59,60]. The output of this modeling was quantified in terms of service life (time to depassivation) and the protection life cycle cost (which is the life cycle cost based on the additional cost of mix over the 0.40 w/c mix required for marine structures by standards, Figure 15). It can be seen that the incorporation of such additives leads to increase in the initial cost of the concrete, but this is accompanied by a marked reduction in life cycle cost. This is a demonstration of the use of such modeling for optimizing mix design of concrete to assure long term performance and provide the rational for the optimum choice which is based on life cycle cost and not on initial cost. Unfortunately, initial cost carries weight with decision makers. However, the use of tools, which are not overly complex, as demonstrated here, can provide the incentive for promoting the application of considerations based on life cycle cost to justify the use of high quality concretes.

5. Conclusions

1. The development of extremely high performance cementitious materials, such as the ones with w/b ratio less than 0.2 and strain hardening properties paves the way for a “step function” for producing structures which are much more cost effective in terms of short and long term performance. However, for this potential to materialize there is a need for in-depth coordinated effort of all the disciplines involved in construction to develop new design methods, construction systems and codes.

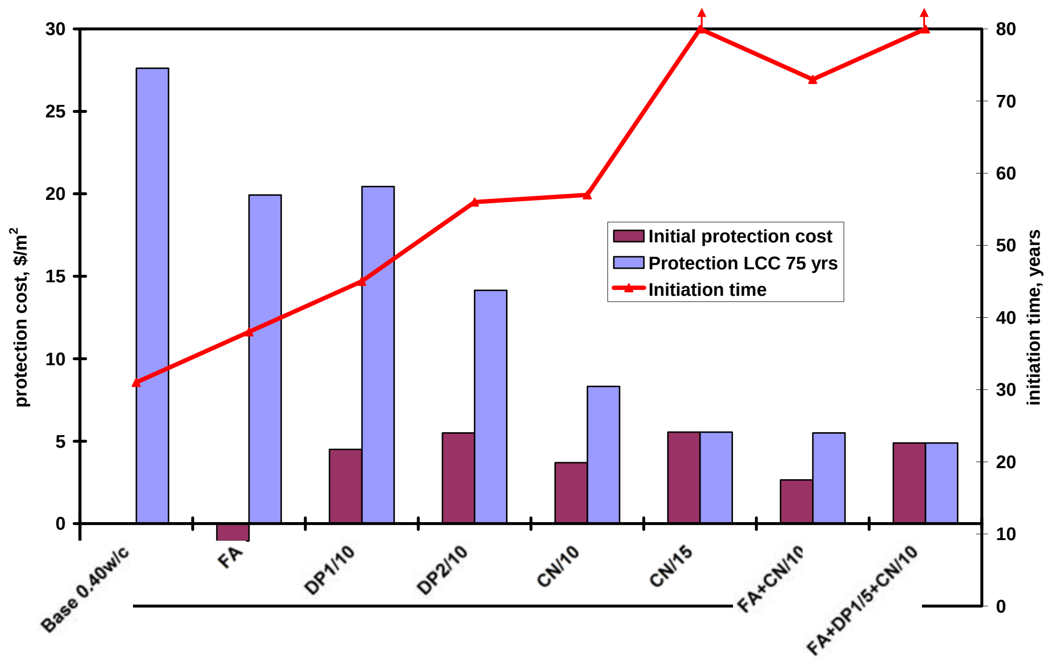

Figure 15: Prediction of costs (initial and life-cycle) and corrosion initiation times for a bridge deck with 75-year design life, after Bentur et al. [59].

2. The use of advanced technologies applying chemical and mineral admixtures, combined with conventional raw materials, e.g. portland cement and aggregates, enables the production of more environmentally friendly concretes and high performance ones. These available and advanced technologies can provide cost effective solutions to meet many of the challenges facing the concrete industry, especially with respect to production of cost effective durable structures.
3. The mechanisms controlling the performance of such concretes and the concepts of mix design are generally understood and can be modeled reasonably by tools which are of engineering nature. Yet, in spite of this know-how, the performance on site frequently fails to meet the potential of these concretes. Some of the causes for this gap are associated with the influence of field practices which are often ignored, and the dependence on specifying and monitoring of concrete quality only on strength.
4. The effect of relaxed field practices on the performance of concretes can be quantified, especially when durability is the issue. Current modeling tools can be used to estimate the effect of deficient curing and concrete cover thickness on service life, and if used properly can provide the rational for enforcing and monitoring "good practice". Quantification in terms of service life effects of poor field practices can provide a basis for the cost/effectiveness of such practices and calculation of compensation if they are lacking.
5. The combined use of chemical and mineral admixtures can provide flexible tools for adjusting the mix design for a variety of durability

performances. For this to happen and be implemented properly there is a need to apply modeling tools, such as the ones used to predict cracking and corrosion of reinforcement. Models which have been developed in recent years can provide outputs of performance (e.g. service life, cracking), with inputs of concrete mix design, construction practices, structural details and environmental conditions during the period when the concrete matures and later on. Thus, such modeling can serve as a platform for the mix design of concrete and the use of concretes of advanced properties, with the higher initial cost being justified in terms of life cycle cost.

6. In order to materialize the potential for the flexible design of concrete properties by chemical and mineral additives, there is a need to consider the actual performance of the concrete required, and for that purpose characterizing the concrete quality in terms of one parameter, which is usually strength, is insufficient, or can be misleading. Means for performance testing, and specifications of the concrete quality by more than one performance requirement (e.g. performance indicators) can now pave the way for more rational concrete design to effectively meet requirements. Such tools can now strengthen the performance approach for the design of concrete, and there is a need to start incorporating them in standards.
7. In order to minimize the risk of cracking during construction, measures should be taken to reduce the tensile stresses due to thermal and shrinkage effects. These measures will also reduce the residual tensile stresses and hence will reduce the risk of cracking in structural elements due to applied loads in service.

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