Deflection of reinforced concrete beams simultaneously subjected to sustained load and reinforcement corrosion

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Summary

The bearing capacity and serviceability of reinforced concrete structures can be severe deteriorated by steel reinforcement corrosion. Recent research results show that deterioration of serviceability of structures due to increasing of deflections begins much earlier than structural collapse. The results presented in this paper are a part of a comprehensive experimental research. In this experiment, chloride induced corrosion of reinforcement embedded in reinforced concrete beam specimens is accelerated with alternation of dry and wet periods with salt water in an environmental chamber. Such alternation of wet and dry periods simulates natural process of reinforcement corrosion in chloride environment. Beam specimens were simultaneously subjected to sustained load and reinforcement corrosion. In this paper the method for computing the long-term deflections of beams exposed to reinforcement corrosion is proposed. The serviceability parameters which consider effect of localized reinforcement chloride corrosion on tension stiffening and reduced stiffness of corroded reinforcement were introduced. Mentioned serviceability parameters are expressed as a function of corrosion state of reinforcement and evaluated by results of experimental research. Corrosion state of reinforcement can be determined by measuring of reinforcement corrosion rates during the time of exposure of reinforced concrete members to corrosive environment. The relationship between corrosion state of reinforcement and deflections is established and evaluated by test results.

Keywords

reinforcement corrosion, beam deflections, serviceability parameters, experimental research.

Theme

structural design/tests - durability - reinforced concrete

1 Introduction

The consequence of reinforcement corrosion, except for reduction of the bearing capacity, is deterioration of serviceability of concrete structures. The two most important criteria for evaluating structure serviceability are the magnitude of deflections and width of cracks. The results of conducted research up to now [1-3] as well as the results of experiments conducted as part of the scientific research [4] show that reinforcement corrosion influences deformation and cracking of reinforcement elements subjected to bending. When the reinforcement is induced by chlorides, traces of reinforcement corrosion on the surface of structural elements, as are, for example, longitudinal cracks in concrete above the reinforcement and corrosion smears, appear in places or are not even visible, but still damaging effects of pitting corrosion of reinforcement can significantly influence the rest of the bearing capacity and serviceability of reinforced concrete structures. Transverse cracks upright to axis of reinforced concrete elements appear in elements strained by the moment of bending with or without longitudinal force if the stress in concrete oversteps the tensile strength of concrete [4].

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In elements subjected to sustained load and at the same time exposed to reinforcement corrosion, the increase of transversal cracks width and deflections due to reinforcement corrosion are observed. In this paper a part of comprehensive experimental research [4] is presented. The calculation method for determination of deflection of beams with corroded reinforcement is proposed.

2 Experimental research program

2.1 Test specimens and material properties

The beam specimens selected for the experiment had a cross-section of 80 × 120 mm and length of 2000 mm. They were reinforced with two bars having a diameter of 8 mm in the tensile zone, two bars of 6-mm diameter in the compressive zone of cross-section, and with stirrups of 6 mm in diameter spaced at intervals of 80 mm. The protective concrete cover to the depth of the reinforcement (stirrups) is 10 mm (figure 1). The bars of 6-mm nominal diameter were cold worked ribbed reinforcing steel bars, while those of 8-mm nominal diameter were hot rolled ribbed reinforcing steel bars.

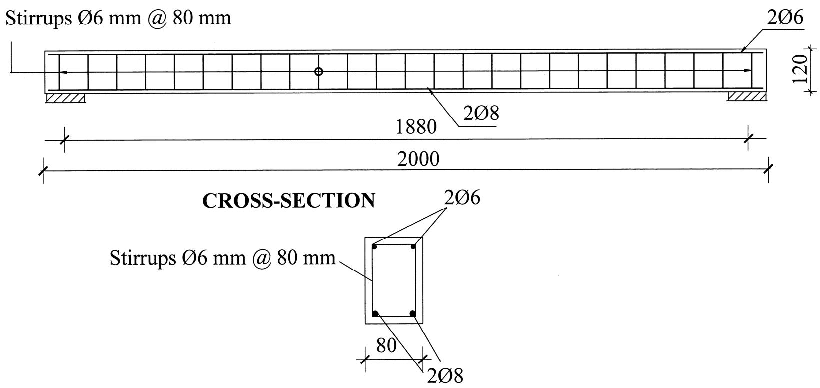


Figure 1: Beam specimen

Material properties (concrete and reinforcing steel) were determined from tests performed on at least three specimens. Mechanical properties of hardened concrete were tested according to methods [5-8] given in table 1. In table 2, mechanical properties of reinforcing steel were tested according to ISO 15630-1 [9].

Table 1: Mechanical properties of hardened concrete

Property	Test methods	Shape and dimensions of test specimens	Mean value
Compressive strength (28 days), MPa	EN 12390-3 [5]	Cubes 150x150x150 mm	35.2
Modulus of elasticity (28 days), MPa	HRN U.M1.025 [6] (Croatian standard)	Prisms 120x120x360 mm	32833
Flexural tensile strength (28 days), MPa	EN 12390-5 [7]	Prisms 100x100x400 mm	4.56
The specific fracture energy by bending specimens, J/m^2	"RILEM" method [8]	Prisms 100x100x400 mm	79.54

Table 2: Mechanical properties of reinforcing steel (mean values)

Reinforcing steel	Yield strength*, MPa	Tensile strength* MPa	Total elongation at maximum force A_{gt}, %	Test method
Hot rolled ribbed bars	589	684	6.83	ISO 15630-1 [9]
Cold worked ribbed bars	573	607	2.13	

* Values determined according to real cross sections of bars

The program of the experiment included three levels of reinforcement corrosion. After each of corrosion levels of reinforcement had been approximately reached, the beam specimens were tested until they failed (table 3), and the actual corrosion state of reinforcement determined on the specimens with corroded reinforcement extracted from the beams. Value P_{corr} in table 3 is a depth of corrosion to the reinforcement relative to the original surface of uncorroded reinforcement. This value is determined from the results obtained by measuring the corrosion rate.

Table 3: Number of beam specimens and levels of corrosion with proposed depth of reinforcement corrosion

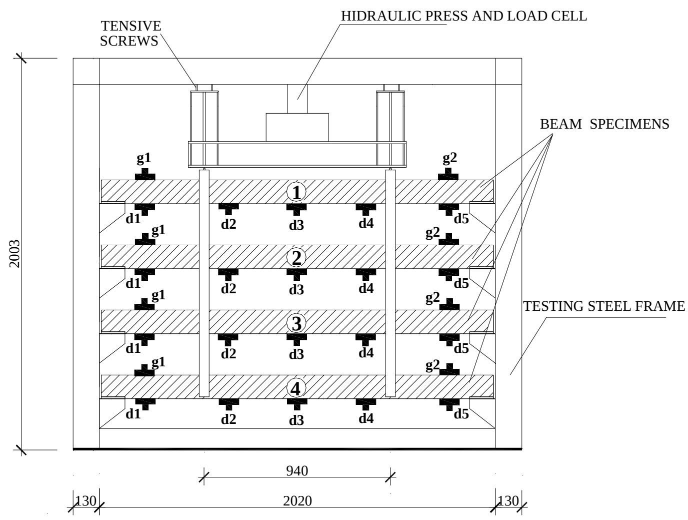
Level of corrosion	0	I	II	III
Depth of corrosion	Control specimens (not exposed to corrosion): P_{corr} = 0 mm	$P_{corr} \approx$ 0.05 mm	0.1 mm $\leq P_{corr} <$ 0.2 mm	$P_{corr} >$ 0.2 mm
Number of specimens	1 specimen at an age of 28 days tested to failure	1 each of beam specimens under sustained static load in environmental chamber for testing corrosion state of reinforcement and chloride penetration		
	3 specimens under sustained static load in laboratory conditions (20±2°C, RH 48%) tested to failure at the end of experiment	3 each of the specimens under sustained static load in environmental chamber tested to failure after the specimens had reached a particular corrosion level (I, II, III)		

2.2 Initiation and acceleration of the process of reinforcement corrosion

Reinforcement corrosion in the reinforced concrete beam specimens was initiated and accelerated, after it had started, by repeating the wetting and drying cycles in environmental chamber. The wetting cycles consisted of salt water spraying while the drying cycles included chamber heating to the temperature about 50°C and use of a fan to remove excess moisture from the chamber. One cycle took three days. First, specimens were sprayed with salt water (3.8% of sodium chloride to water mass).

2.3 Measurement of reinforced concrete specimens' deflections and reinforcement corrosion

During the exposure to reinforcement corrosion, sustained load was applied to beam specimens in steel frames, which caused the occurrence of cracks in the concrete. The level of load was equivalent to 60% of the design flexural capacity according to Eurocode 2 [10]. The second criterion for the sustained load magnitude was occurrence of cracks with width of about 0.1 mm in tensile zone of beams. The cracks accelerated the time until the initiation of corrosion and the corrosion process itself. The second criterion is usual relations of quasi-permanent load for verification serviceability limit states (dead load and part of live load [11]) and design bearing capacity for structures. On the beam specimens, deflections and reinforcement corrosion parameters were periodically measured. The deflections were observed at the mid point of span. In the course of corrosion exposure in the environmental chamber, periodical measurements taken on reinforced concrete beam specimens included reinforcement corrosion parameters (corrosion rate, half-cell potential, and electric resistance) and deflections caused by loading and corrosion. The measurements were made using a integrated portable equipment whose operation is based on a galvanostatic impulse method [12, 13]. The beam specimens in the testing frames, and the arrangement and number of corrosion measurement points on the beam specimens are illustrated in figure 2.



①to④ ... marks of beam specimens
g1 to **2** ... corrosion measuring points on the top side of the beams
d1 to **d5** ... corrosion measuring points on the bottom side of the beams

Figure 2: Beam specimens in the testing frame and corrosion measurement points

To each degree of reinforcement corrosion, on whose samples of corroded reinforcement taken from the beams tensile testing was conducted, the corresponding value of diameter reduction of corroded reinforcement $\Delta\phi(P_{corr})$ was added, defined from the result of measuring the corrosion rate in the elapsed period [4, 13, 14] according to these expressions:

$$P_{corr} = \int_0^\tau 11.6 \cdot i_{corr}(t)dt = 11.6 \cdot \int_0^\tau i_{corr}(t)dt \quad (1)$$

$$\phi = 12.74 \cdot \sqrt{m_0 - 0.00785(\phi_{nom} P_{corr} - P_{corr}^2)\pi} \ , \quad (2)$$

$$\Delta\phi(P_{corr}) = \frac{(\phi - \phi_0)}{\phi_0} \cdot 100 \ , \quad (3)$$

where

P_{corr} - the mean corrosion depth in measuring area of sensors, i.e. corrosion depth calculated on the basis of the results of measuring the corrosion rate of the reinforcement in the period in which the progression of the reinforcement corrosion is monitored (according to expresssions (1) in microns, and in the expression (3) introduced in mm),

$i_{corr}(t)$ - corrosion rate in μA/cm^2 obtained by periodic measuring on beam samples, t is the lasting time of reinforcement corrosion in years,

τ - time (years) in which the mean depth of corrosion is calculated P_{corr},

m_0 - length mass of non-corroded reinforcement (g/mm), 0.00785 is the steal density (g/mm^3),

ϕ_{nom} and ϕ_0 - the nominal and original diameters of the non-corroded bars (mm) and $\Delta\phi(P_{corr})$ is reduction of the diameter of corroded bars in relation to original diameter of the non- corroded bars (%).

During the exposure of specimens to reinforcement corrosion which lasted 383 days, a total of 18 phases of measurements of corrosion parameters were conducted. Reduction diameter of reinforcement $\Delta\phi(P_{corr})$ is also calculated, for each phase of measuring the corrosion rates.

3 Numerical analyses

Numerical analyses are conducted in order to determine parameters which influence the behaviour of reinforcing concrete beams damaged by reinforcement corrosion under sustained load.

3.1 Constitutive models

Constitutive models used in these numerical analyses describe the relationships between stress and strain of materials. On the basis of failure ratios between uniaxial and spatial (3D) stress states, the program ABAQUS© determines the yield surface of concrete according to Lubliner [15], Lee and Fenves [16]. The testing of concrete for planar (2D) and spatial stress states of concrete were not conducted. Suggested values of the failure ratios which correspond to planar and spatial stress states of concrete according to Kupfer and Bangash [17] were used. In beams subjected to bending, the biggest contribution to their behaviour under load has relationships between stress and strain of concrete and steel in uniaxial stress state. Stress-strain diagrams for concrete in compression and bilinear diagrams of reinforcing steel determined on the basis of testing of concrete and reinforcing steel are used. Further on, the constitutive model of uniaxial stressed reinforced concrete is being discussed in more detail.

3.1.1 Tensile stressed concrete in reinforced concrete element

Contribution of concrete in transmitting tensile forces between adjacent cracks (tension stiffening) is modeled with help of diagrams which define the relation of mean stress and mean strain in reinforced concrete element (including the width of the cracks). To obtain such diagrams, the CEB-FIP tension stiffening models are used [14,18] which are the basis of methods for the calculation of deflections and cracks according to Eurocode 2 [10]. In elastic range of reinforcement behaviour, the tension stiffening model according to MC 78 [18] is accepted, which is retained in the calculation for deflection according to Eurocode 2 and for the calculation of curvature (using the curvature and deflections) according to MC 90 [14]. To determine mean reinforcement strain after reaching the yield strength of reinforcement, the tension stiffening model according to MC 90 is taken.

To determine diagrams of mean stress and mean tensile strains of concrete, the equilibrium condition ($N = N_{cm} + N_{sm}$, in figure 3) is used which for the elastic range of reinforcement behaviour can be expressed as:

$$\sigma_{ctm} = \frac{(\sigma_s - \varepsilon_{sm} \cdot E_s) \cdot A_{s1}}{A_{c,eff}}, \tag{4}$$

where:

σ_{ctm} - mean tensile stress of concrete (between two cracks),

σ_s - tensile stress in reinforcement in cross-section with the crack,

ε_{sm} - mean strain of reinforcement,

E_s - modulus elasticity of reinforcing steel (or stiffness parameter of corroded reinforcement $E_{s,corr}$),

A_{s1} - cross sectional area of tension reinforcement,

$A_{c,eff}$ - the effective area of concrete in tension surrounding the reinforcement.

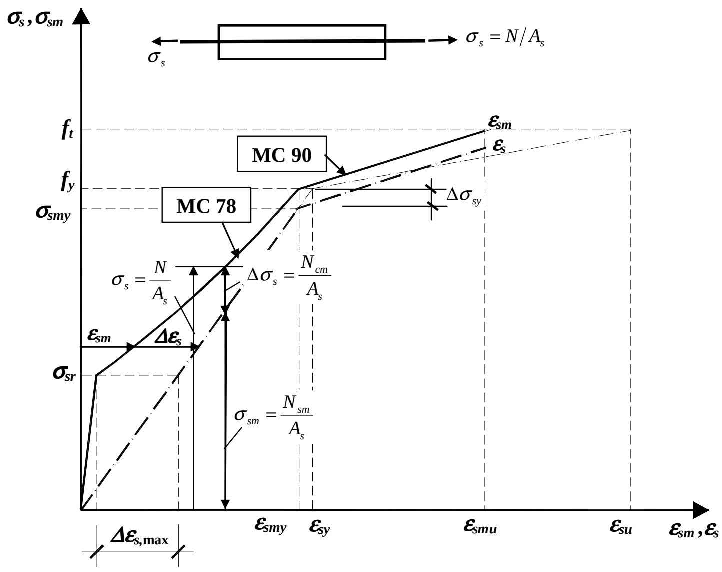


Figure 3: Tension stiffening model

In figure 3 N_{sm} and N_{cm} are the mean tensile forces in reinforcement and concrete, respectively.
Mean strain of reinforcement is determined with the help of the expression (15) according to [18]:

$$\varepsilon_{sm} = (1 - \zeta) \cdot \varepsilon_{s1} + \zeta \cdot \varepsilon_s, \tag{5}$$

where ζ is distribution coefficient with which the contribution of concrete between the cracks (tensile stiffening) is taken in consideration, ε_s is strain of reinforcement in the crack, and ε_{s1} is strain of reinforcement in uncracked condition. The distribution coefficient ζ is determined according to this expression [10, 18]:

$$\zeta = 1 - \beta \cdot \left(\frac{\sigma_{sr}}{\sigma_s} \right)^2, \tag{6}$$

where:

σ_s – tensile stress in reinforcement calculated on the basis of cracked cross-section,

σ_{sr} – tensile stress in reinforcement calculated on the basis of cracked cross-section under the loading conditions causing first cracking (the tensile strength of concrete is reached),

β – is a coefficient taking account of the influence of the duration of the loading or of repeated loading on the mean strain:

β = 1.0 for a single short-term loading,

β = 0.5 for sustained loads or many cycles of repeated loading.

"Smeared cracking" model is used, at which the mean strain of concrete ε_m (including the width of cracks) equals to mean strain of reinforcement ε_{sm}. The effective area of concrete in tension after stabilizing the cracking $A^{st}_{c,eff}$ is deterimined according to the recommendation of Eurocode 2 [10] and MC 90 [14]:

$A^{st}_{c,eff} = (h - x)/3 \cdot b = (12 - 3.17)/3 \cdot 8 = 23.55$ cm^2, x is the height of compressive zone of cross-section.

To conduct the numerical analyses, somewhat bigger effective area of concrete in tension is taken, which takes the behaviour of concrete in consideration, from occurrence of the first crack, over the phase of occurrence of cracks to the stabilization of cracking. On the basis of conducted numerical analyses of deflections for short-term load (without influence of reinforcement corrosion), and in comparison to real measured values of deflections, the effective area of concrete in tension $A_{c,eff}$ = 28.66 cm^2 is taken.

3.2 Numerical model for analyses

The symmetry of system at the modelling was used (figure 3). The zone between longitudinal and transversal plane of symmetry was comprised in the numerical model, hence the quarter of a beam with introduction of adequate boundary conditions.

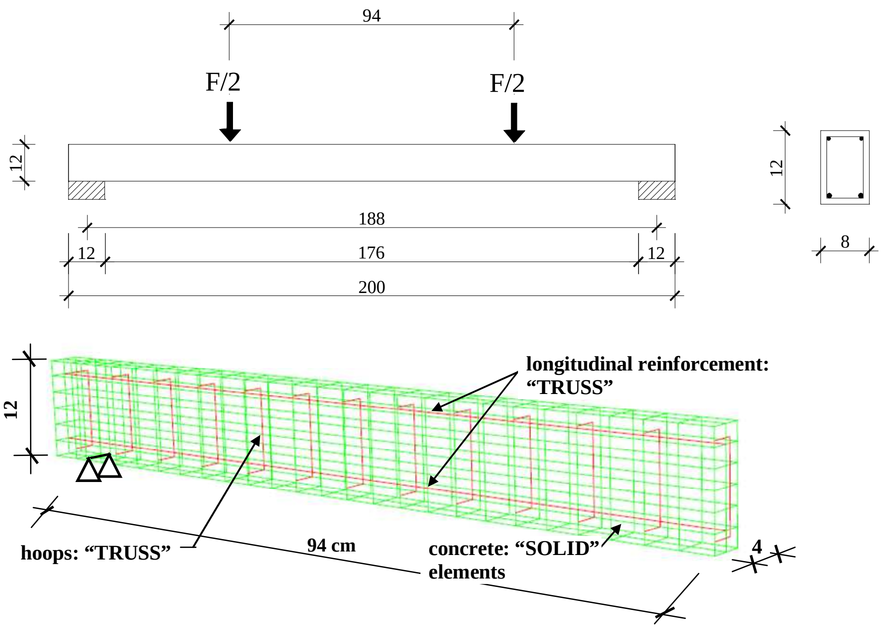


Figure 4: Scheme of load and numerical model for numerical analyses of beams

3.3 Influence of the localized corrosion of reinforcement

Within the scientific research [4], corrosion of reinforcement, which was induced by chlorides, was localized (pitting) corrosion. On the basis of conducted observations, the following can be established:

1. Corrosion damage is not evenly distributed along the reinforcing bars. The consequence is that there is no general reduction of strength and stiffness of bonding between the concrete and reinforcement, but it occurs only locally, in zones of greater damage.

2. The zones of pitting corrosion generally coincide with the zones of cracks in concrete which occurred because of bending of samples of beams. Strain of the tensile reinforcement is the greatest in cross-sections with the crack.

3. The shape and intensity of the reinforcement damage occurred because of pitting corrosion were uneven and irregular. On real structures it is not possible to do a thorough and detailed topography of corrosion damage along the reinforcing bars damaged by pitting corrosion.

All above listed points (from 1 to 3) objectively represent the difficulties in case when one would want to comprise local characteristics of corrosion damage with the numerical model, according to intensity and to their allocation. However, the problem can be observed and as the influence of pitting corrosion of reinforcement to contribution of concrete in tension between cracks (tension stiffening). The effect of pitting corrosion of reinforcement to tension stiffening can be explained with following two mechanisms:

1. Local reduction of cross sectional area of tension reinforcement leads to reduction of its stiffness, and therefore the change contribution of concrete in tension between cracks.

2. Local reduction of bond strength and stiffness also reduces contribution change contribution of concrete in tension between cracks.

Reduction of contribution of concrete in tension between cracks because of pitting corrosion of reinforcement is considered using the modified distribution coefficient ζ at the calculation of mean strain of reinforcement:

$$\zeta = 1 - k_{corr,p} \cdot \beta \cdot \left(\frac{\sigma_{sr}}{\sigma_s} \right)^2 , \tag{7}$$

where $k_{corr,p}$ is the coefficient which takes into account the effect of pitting corrosion of reinforcement to contribution of concrete in tension between cracks.

Coefficient $k_{corr,p}$ is determined with the help of numerical analyses at the model shown in Figure 4. Sustained load in the experiment and in the calculations were two concentrated forces F = 4.21 kN.

In constitutive model of tensile stressed concrete in reinforced concrete element which is described in chapter 4.1.1 modified mean strain of reinforcement according to expression (11). The change of coefficient $k_{corr,p}$ leads to the change of diagram of mean stress and mean strain of tensile stressed reinforced concrete (($\sigma_{ct,max}/\sigma_{ctm}$)-$\varepsilon_m$ diagrams in Figure 5.), which is the constitutive model of tensile strained concrete with corroded reinforcement for the numerical analyses.

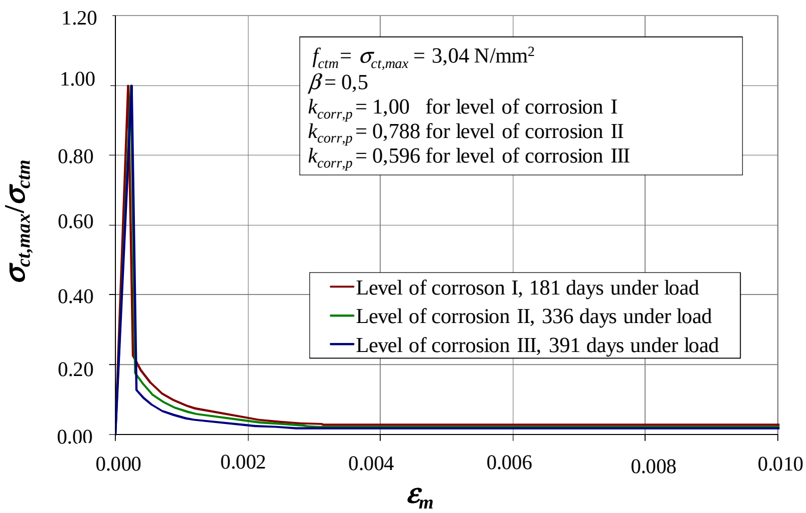


Figure 5: Diagrams **($\sigma_{ct,max}/\sigma_{ctm}$)-$\varepsilon_m$** of tensile stressed concrete at the sustained loaded samples of beams exposed to reinforcement corrosion in the climate chamber (f_{ctm} is the mean value of tensile strength of concrete)

In numerical analyses, the measured data about the properties of corroded reinforcement for each degree of corrosion was used. The mechanical properties of reinforcement were determined by tensile tests of samples of uncorroded and corroded reinforcement. The mechanical properties are obtained regarding to original cross sectional area of uncoroded bars. For the calculation of deflections (and cracks) the stiffness parameter E_{corr} is an important characteristic, i.e. the modulus of elasticity of corroded reinforcement.
The numerical model was firstly calibrated by comparison of calculated and measured values of deflections of uncorroded samples of beams. The excellent agreement of calculated and measured values of deflections with mean difference less than 1% was established.
Numerical analyses are repeated with different coefficients $k_{corr,p}$ till the deflection obtained with numerical analyses was equalized with the mean measured value of deflection in observed moment of time. In this way, the values of coefficient $k_{corr,p}$ is determined (Figure 5).
Shrinkage and creep strains of concrete were considered as functions of time according to the expressions given in Eurocode 2 [10].

4 Calculation of deflection of elements with corroded reinforcement

For calculation of deflection of elements damaged by reinforcement corrosion, the modification of the method according to Eurocode 2 [10] can be suggested. The modification consists of the following:

1. Geometric characteristics of cross-section for uncracked and cracked condition are calculated with original cross sectional area of uncoroded reinforcing bars (or at practical calculation with nominal cross sectional area), but with the usage of stiffness parameters of corroded reinforcement $E_{s,corr}$ instead of the modulus of elasticity of uncorroded steel for reinforcing $E_{s,corr}$.
2. Expression (5) for the calculation of the distribution coefficient is substituted with the expression (12), according to which the modified distribution coefficient is calculated.

Corrosion parametars of serviceability $E_{s,corr}$ and $k_{corr,p}$ are function of states of corroded reinforcement (Figures 6 and 7), i.e. the corrosion reduction of reinforcement diameter $\Delta\phi(P_{corr})$ determined from results of measuring of corrosion rates according to the expressions (1) to (3).

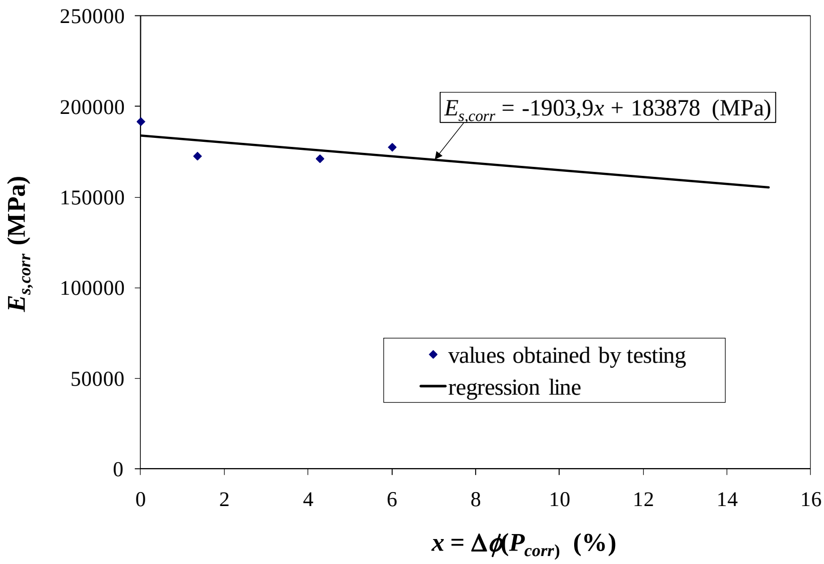


Figure 6: Stiffness parameter of hot rolled reinforcement samples of beams as the function of reinforcement corrosion

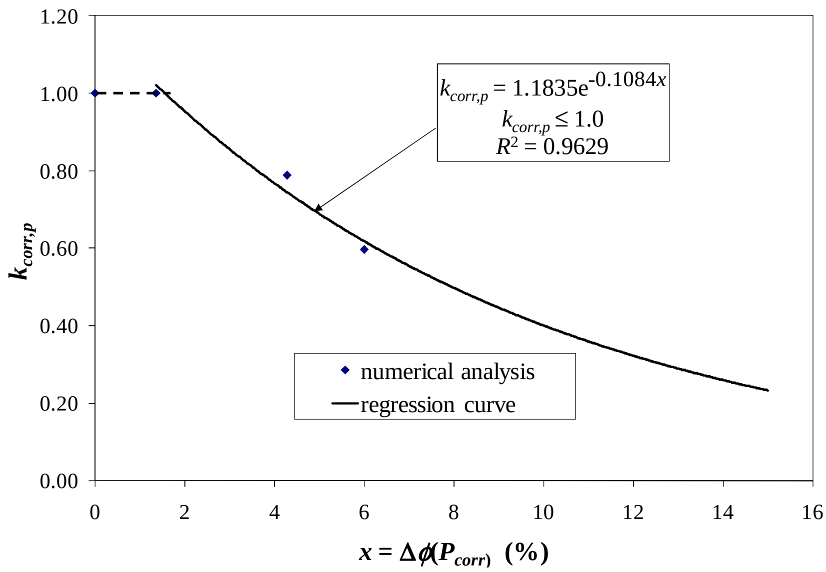


Figure 7: Coefficient $k_{corr,p}$ of beams as the function of reinforcement corrosion

At the sustained load, it is necessary to consider the long-term strain of concrete because of shrinkage of concrete and creep of long-term stressed concrete at the calculation of deformation of reinforced concrete elements. The value of strain of concrete under constant stress is obtained as the sum of elastic strain and strain because of creep of concrete [14] according to the expression:

$$\varepsilon_c(t,t_0) = \sigma_c(t_0) \cdot \left(\frac{1}{E_c(t_0)} + \frac{\varphi(t,t_0)}{E_c(28)} \right), \tag{8}$$

where $\sigma_c(t_0)$ is constant stress in concrete achieved in the moment t_0, $E_c(28)$ is tangent modulus of elasticity of 28 days old concrete, $E_c(t_0)$ is tangent modulus of elasticity of concrete in time t_0, $\varphi(t,t_0)$ is creep coefficient of concrete aged t which was loaded in the moment t_0, t_0 is time of application of load to concrete and t is observed time.

The creep coefficients as the function of time are used, which were determined according to the method in EN-a 1992-1-1 [10]. With so calculated creep coefficients, the tangent modulus of elasticity $E_c(28) \approx 1{,}05E_{cm}$ [10] for the calculation of creep strain of concrete is used. If the $E_c(t_0) \approx E_c(28)$ is taken, the following is obtained:

$$\varepsilon_c(t,t_0) = \sigma_c(t_0) \cdot \left(\frac{1+\varphi(t,t_0)}{E_c(28)} \right), \tag{9}$$

where the expression in brackets can be expresed as effective modulus of elasticity of concrete:

$$E_{c,eff} = \frac{E_c(28)}{1+\varphi(t,t_0)}. \tag{10}$$

The influence of shrinkage strain of concrete is considered with the help of total shrinkage strain for free shrinkage of concrete according to Eurocode 2 [10]. In Figure 8 the measured and calculated values of deflection as functions of time (t–t_0) in days passed after the application of load to samples of beams (using the parameters $E_{s,corr}$ and $k_{corr,p}$ in Figures 6 and 7) are shown.

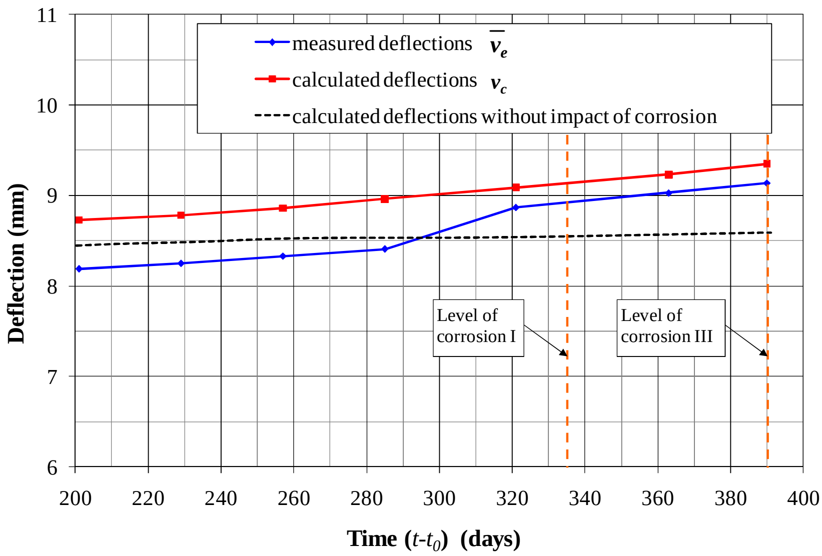


Figure 8: Measured and calculated magnitudes of deflections of samples of beams

Measured and calculated values of deflections are proportional in observed period. According to Figure 8 it can be expected that observed proportional trend will be preserved in the following period of time (i.e. for greater reinforcement corrosion). Calculated deflections are 2 to 4% greater than the measured ones, which is acceptable deviation.

5 Conclusion

Calculated values of deflection determined by the presented method are very well coinciding with the measured values of deflection. As the reduction of diameters of reinforcing bars caused by corrosion increases in time, so do the corrosion parameters of serviceability, which are used in the presented method for the calculation of deflections, are functions of time. The corrosion reduction of diameters of bars (state of reinforcement corrosion) can be determined on the basis of measured corrosion rates in observed period of time. Therefore, the suggested method can be used for predicting the deflections at the evaluation of service life in view of the criteria of structural serviceability. The application of suggested method on real structures should be combined with detailed investigations and measurements on the structure. If the monitoring of the structure exposed to reinforcement corrosion is conducted, within which the periodic measurement of deflections (and cracks) is conducted, and the periodic or continuing measuring of parameters of reinforcement corrosion, then the corrosion parameters of serviceability can be valued and calibrated according to measured values of deflections. This way, the prediction of the magnitude of deflections in the following period of serviceability of a structure can be improved.

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