Capacity of concrete structures with corroded reinforcement and prestressing tendons

Capacity of concrete structures with corroded reinforcement and prestressing tendons

M Sigvaldsen[1], G Ersdal[2*], G Markeset[3], S Samarakoon[2] and M Langeteig[4]

[1] Niras, Stavanger, Norway
[2] Department of Mechanical and Structural Engineering and Materials Science, University of Stavanger, Norway
[3] Oslo Metropolitan University - OsloMet, Oslo, Norway
[4] Norwegian Public Roads Administration, Stavanger, Norway

* Corresponding author: gerhard.ersdal@uis.no

Abstract. The main degradation mechanism for concrete structures is corrosion of the reinforcement and prestressing tendons. Management of structures with such degradation requires detailed understanding of their remaining strength and safety and if necessary, make a decision regarding repairs or replacement of the structure or components. Some simplified methods for estimating the residual capacity of concrete structures do exist, primarily based on a reduction of the flexural capacity equal to the percentage of the corroded area. In this paper, a more physical understanding and description of the influence of corrosion on the strength is investigated, based on a reduction of the area of the reinforcement and prestressing tendons both due to uniform corrosion and pitting corrosion. The results of these models are successfully compared to experimental results of concrete beams with corrosion. Particularly corrosion of post-tensioned tendons is a concern for concrete structures. Some disturbing examples of collapse of concrete bridges have been seen as a result of such corrosion. The paper highlights the importance of the significant strength loss of the reinforcement as a result of corrosion itself, but also the loss of ductility due to possible hydrogen embrittlement and hydrogen induced stress corrosion cracking. The paper also suggests sulphate reducing bacteria as a possible explanation to corrosion issues related to corrosion of post-tensioned tendon structures where no chloride is found. The aim of the paper is to propose a method to calculate a lower bound estimate of the remaining capacity of concrete beams with corrosion damage to reinforcement and to the prestressing tendons.

1. Introduction

A large number of existing structures are an essential part of infrastructure in the world, and these are of vital importance for the societies where they are being used. In many cases, these are degrading and in need of significant investments for strengthening, maintenance and repair to be safely used further. Replacement with new structures is also an option, but replacement might be both economically and environmentally unsound. Structures in operation are exposed to conditions of stress and environment that ultimately will degrade them from their initial state, and damage will accumulate until the structures may be judged to be no longer fit-for-service. If these degraded and damaged structures are not withdrawn from further service or being repaired, failure of some kind will eventually occur. In addition, the cost of the maintenance, inspection and repair needed to cope with this deterioration and damage will at some stage become unacceptable compared to the revenue that can be gained from the use of these structures.

Reinforced and prestressed concrete bridges are an important example of such infrastructures. Bridges in coastal areas or exposed to de-icing salt are deteriorating due to general and pitting corrosion, as they are prone to chloride-ingress. A large number of bridges were built in the 1960s and 1970s at a time where requirements for durability and robustness were less extensive than today and these are at present a challenge for engineers and owners.

In order to manage existing structures in a safe manner it becomes vital to know:

1. how structures change with age (including deterioration, damage and load changes),
2. how these changes, particularly to the condition, can be determined,
3. how their capacity can be determined as a result of degradation, damage and other changes,
4. how anomalies found in an existing structure can be repaired and mitigated, and
5. how the integrity of ageing structures should be managed.

This paper briefly describes how the performance of concrete structures change due to corrosion and how the ultimate flexural capacity of concrete structures can be described as a result of corrosion in reinforcement and prestressing tendons. The initiation of corrosion can be largely attributed to lack of concrete cover and poor-quality concrete. The effect of reinforcement and prestressing corrosion is a reduction in cross-sectional area leading to a reduced capacity.

The most common approach to estimate the residual capacity of a corroded concrete structures is to reduce the capacity according to the equivalent percentage reduction of the reinforcement cross-sectional area. This method will be compared to experimental data and a new method will be proposed.

In addition, other important issues related to safety of concrete structures will be discussed, including hydrogen charging, hydrogen embrittlement and hydrogen-induced stress corrosion cracking (HISCC). Loss of bond strength and volume expansion [1] is also a potential problem for concrete structures. This effect is not included in this paper and could potentially alter the choice of pitting corrosion model presented in this paper.

2. Corrosion of reinforcement and prestressing tendons in concrete structures

Chloride ingress is the most common cause of reinforcement corrosion. Such corrosion will result in a uniform thickness reduction but may also lead to local pitting corrosion. The effect of corrosion of reinforcement of medium strength steel include:

- Reduction of the area of the reinforcement bar due to both uniform and pitting corrosion (resulting in a reduction in the axial capacity of the reinforcement bar)
- Reduction of the bond strength between the reinforcement bar and the concrete due to the presence of corrosive residues.

Both these effects may reduce the flexural capacity of the concrete structure. In contrast, experience indicates that corrosion due to carbonatization primarily results in unform corrosion without pits.

Similarly, chloride ingress is the main cause of prestressing tendon corrosion. However, as prestressing tendons are manufactured from high strength steel, these are susceptible for hydrogen ingress, hydrogen embrittlement (HE) and stress corrosion cracking (SCC). Hydrogen embrittlement (HE) and stress corrosion cracking (SCC) may lead to significant loss in the ductility of the steel and may lead to unexpected and abrupt rupture at low deformations and loads.

Pre-tensioned prestressing tendons are normally cast in the concrete, similar to the reinforcement bars, and are assumed to primarily be affected by chloride ingress. In contrast, post tensioned prestressing tendons are normally placed in ducts in the concrete. These ducts are grouted after the post tensioning to protect the prestressing tendons against corrosion and to provide bond strength between the prestressing tendons and the concrete. Lack of grout and voids in the grout may result in aggregation of water which can result in corrosion [2].

More recently, some examples of corroded prestressing tendons without the presence of chlorides have been found [3]. Common in these cases is the presence of crumbled and pasty mortar, and this type of corrosion is to some extent called “Soft grout corrosion”. It is assumed that this type of corrosion occurs in partially filled cable ducts and that the crumbled mortar creates a humid environment instead of protecting the reinforcement against corrosion. No generally accepted explanation for this type of corrosion is available at present, but a possible explanation may be corrosion initiated by sulphate reducing bacteria (SRB). In one of the known cases of soft grout corrosion [4] leading to the failure of the prestressing tendons, elevated levels of sulphate ions were found in combination with high pH pore water, low content of chloride ions and water content in the grout.

3. Size, frequency and interaction of pitting corrosion

When estimating the flexural capacity of the corroded concrete structures, the area of the reinforcement bars and prestressing tendons need to be reduced according to the uniform corrosion experienced. In addition, the cross-sectional area needs to be reduced due to the pitting corrosion.

Fernandez et al [5] studied the pit depth versus corroded radius as a relation with the degree of corrosion and a large scatter was seen, with the mean pit depth versus corroded radius given by:

$$\frac{p_d}{r_1} = 2.318 \cdot c_d$$

where p_d is the pit depth, r_1 is the uniformly corroded radius and c_d is the degree of corrosion. The upper limit 95% percentile of pit depth is given by:

$$\frac{p_d}{r_1} = 3.755 \cdot c_d$$

As this paper is aiming to develop a reasonable flexural capacity of corroded concrete beams to the safe side, a reasonable choice is to use the upper limit 95% percentile pit depth.

In addition, the pits will be distributed along the length of the reinforcement and the number of pits will be dependent on the degree of corrosion (c_d). At low corrosion levels the frequency of pits will be low, and the size will be limited and, at higher corrosion levels the frequency and size of pits will increase (hence, the distance between the pits will be smaller). For the simple beams (single span beams with only two reinforcement bars) evaluated in this paper, the assumption will be that the maximum pit depth occurs simultaneously for both reinforcement bars at or near the location of the maximum bending moment. This assumption may be somewhat conservative and will be validated in the comparison with experimental data.

The frequency of pits will influence the likelihood of coinciding pits in nearby rebars. Kioumarsi et al [6] illustrates this by defining the maximum bending capacity as a function the distance between pits on two nearby rebars (l_p) and the distance between these two rebars (l_r). Kioumarsi et al [6] indicate that for concrete structure where the ratio between l_p and l_r is above 1.25, this effect can be ignored. However, a relationship between l_p and the corrosion rate is not provided. Hence, in this paper this effect has been ignored for all corrosion levels and both reinforcement bars have been reduced to the maximum value.

4. Experimental testing of flexural capacity of corroded concrete structures

Experimental work was performed by Azad et al [7] and Al-Gohi [8] to test the flexural capacity of simple concrete beams where the reinforcement bars have been exposed to corrosion. Azad et al [7] tested 56 beams with dimension 150x150 mm reinforced by two 10 or 12 mm reinforcement bars, see Table 1. These 56 beams were grouped into 4 groups depending on concrete cover and reinforcement bar size. For each group, two uncorroded reference beams were tested and 2x6 beams of various corrosion exposure where tested. Al-Gohi [8] tested 48 beams of breadth 200 mm and height varying from 215 mm to 315 mm reinforced by two 16 and 18 mm reinforcement bars, see Table 2. After the flexural capacity tests the bars were cleaned to remove all rust products and then weighted to find the remaining net weight of steel

In these tests all beams had a cover of 40 mm. The 48 beams were grouped into 6 groups depending on the height and reinforcement bar size. As for the Azad tests, two uncorroded reference beams were tested for each group. Based on the findings from Azad et al [7] and Al-Gohi [8], the variation of residual flexural capacity (%) as a function of corrosion weight loss (%) is shown in Figure 1, indicating a clear decrease in the flexural capacity with corrosion weight loss, but with a significant scatter in the data.

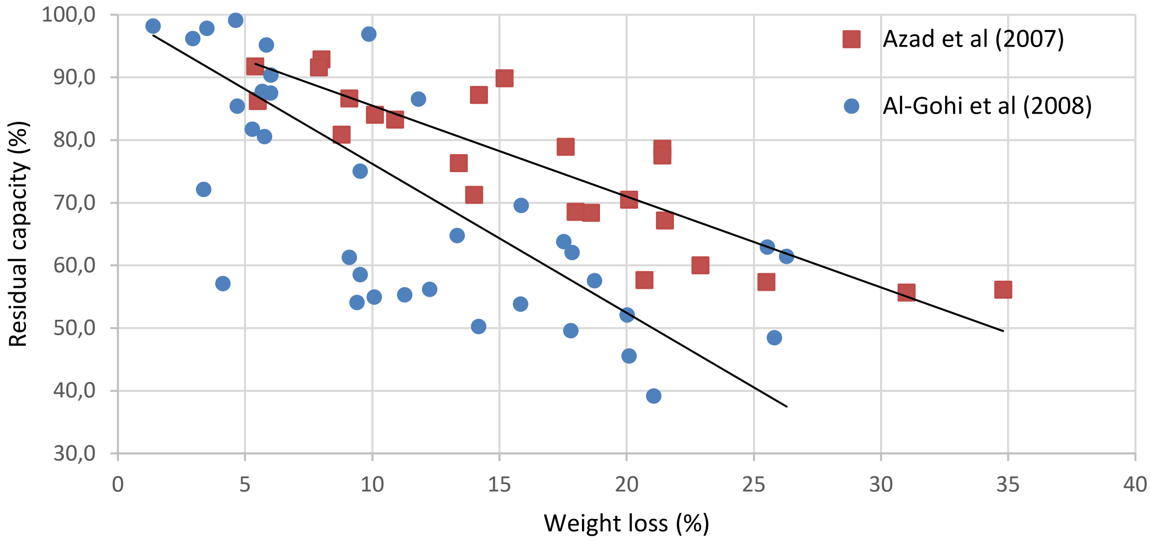

Figure 1: Remaining flexural capacity (%) as a function of corrosion weight loss (%) in steel reinforcement.

Table 1: Data for the experimental beams tested by Azad et al [7]

Beam Group	Beam Specimen	Width x height (mm)	Cover	Rebars	Weight loss %	Experimental capacity (kNm)
BT1	BT1-2-4	150x150	25 mm	2x10 mm	5.4	10.68
	BT1-3-4				14.2	10.15
	BT1-2-6				15.2	10.46
	BT1-3-6				21.4	9.15
	BT1-2-8				21.5	7.82
	BT1-3-8				31	6.48
BT2	BT2-2-4			2x12 mm	5.5	12.76
	BT2-3-4				8.8	11.97
	BT2-2-6				20.1	10.43
	BT2-3-6				14	10.55
	BT2-2-8				22.9	8.88
	BT2-3-8				25.5	8.49
BT3	BT3-2-4		40 mm	2x10 mm	8	10.92
	BT3-3-4				9.1	10.19
	BT3-2-6				10.1	9.88
	BT3-3-6				17.6	9.28
	BT3-2-8				21.4	9.12
	BT3-3-8				34.8	6.6
BT4	BT4-2-4			2x12 mm	7.9	12.03
	BT4-3-4				10.9	10.93
	BT4-2-6				13.4	10.02
	BT4-3-6				18.6	8.98
	BT4-2-8				18	9
	BT4-3-8				20.7	7.57

Table 2: Data for the experimental beams tested by Al-Gohi [8]

Beam		Width x height (mm)	Cover	Rebars	Weight loss %	Experimental capacity (kNm)
Group	Specimen					
1	B1-1	200x215	40 mm	2x16 mm	3.5	31.5
	B1-2				6	28.18
	B1-3				4.13	18.38
	B1-4				15.85	22.4
	B1-5				2.95	30.98
	B1-6				15.83	17.33
2	B2-1	200x655			11.82	36.58
	B2-2				9.86	40.95
	B2-3				18.74	24.33
	B2-4				17.53	26.95
	B2-5				25.53	26.6
	B2-6				25.81	20.48
3	B3-1	200x315			13.34	37.63
	B3-2				17.85	36.05
	B3-3				6.02	52.5
	B3-4				5.84	55.3
	B3-5				26.29	35.7
	B3-6				4.63	57.58
4	B4-1	200x215		2x18 mm	5.28	33.6
	B4-2				9.4	22.23
	B4-3				11.27	22.75
	B4-4				12.26	23.1
	B4-5				20.09	18.73
	B4-6				21.06	16.1
5	B5-1	200x265			9.1	31.15
	B5-2				9.53	38.15
	B5-3				9.53	29.75
	B5-4				5.76	40.95
	B5-5				14.18	25.55
	B5-6				17.8	25.2
6	B6-1	200x315			5.67	58.98
	B6-2				1.39	65.98
	B6-3				4.69	57.4
	B6-4				10.08	36.93
	B6-5				3.37	48.48
	B6-6				20.02	35

Examples of beams with corroded prestressing tendons are given in Kioumarsi et al. [9] which reports the experiments of among others Belletti et al. [10], Rinaldi et al. [11], Menoufy and Soudki [12] and ElBatanouny et al. [13]. The following experimental data is found relevant for this study:

- Group 1 by Belletti et al. [10] included tests of seven beams, including on uncorroded reference beam (PBN4P2) and three beams with properties that makes them unsuitable for this paper. The remaining four beams (PB4P7, PB4P8, PB4P13 and PB4P14) are corroded to different degrees and are included in this study.
- Group 2 by Vecchi et al [14], as reported by Kioumarsi et al [9], are based on naturally corroded beams, extracted from a cooling tower in a thermal power plant, where they have been in use

for 10 years and have been subjected to repeated wetting with seawater. The three beams were investigated and one of the beams was defined as the reference beam and the other two were evaluated to be corroded to a level of 5.7% and 9.3% corrosion respectively. The information provided is limited and for the experimental capacity is only reported for the first of these.

- Group 3 by Rinaldi et al [11], Group 4 by Menoufy and Soudki [12], Group 5 by ElBatanouny et al [13], Group 6 by Benenato et al [15] and Group 7 by Liu et al [16], as reported by Kioumarsi et al [9].

A summary of these experiments is presented in Table 3 and illustrated in Figure 2, similarly clearly indicating a loss of flexural capacity with increased weight loss.

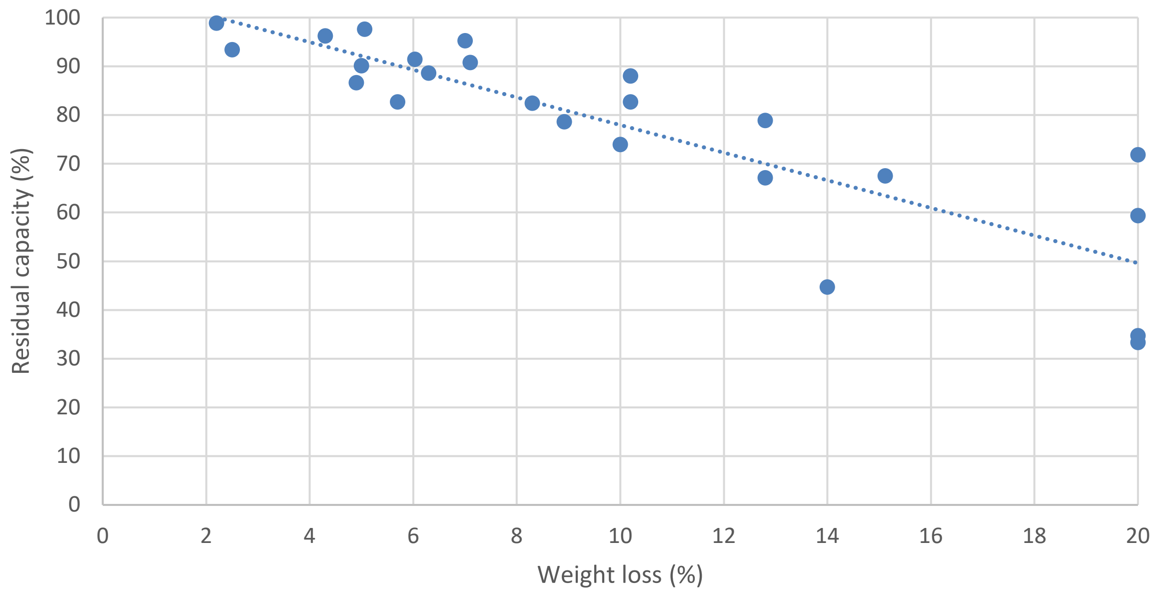

Figure 2: Remaining flexural capacity (%) as a function of corrosion weight loss (%) on prestressing tendons

An additional important aspect seen from these tests is the clear reduction in the deformation at failure for the corroded beams compared to the reference beams, indicating a clear loss of ductility in corroded prestressed tendons, commonly associated with hydrogen embrittlement and stress corrosion cracking in high strength steel.

Table 3: Experimental data for tested concrete beams with prestressing tendon reinforcement

Beam		Dimension (mm)	Length	Weight loss %	Experimental capacity (kNm)	Capacity reduction (%)
Group	Specimen					
1	PBN4P2	150x300	5400	0	82.71	Reference
	PB4P7			15.12	55.86	67.54
	PB4P8			8.92	65.04	78.64
	PB4P13			8.3	68.19	82.44
	PB4P14			6.03	75.62	91.43
2	1	150x300	5400	0	N.A.	Reference
	2			5.7	N.A.	82.7
	3			9.3	N.A.	N.A.
3	B7	200x300	3000	0	72	Reference
	B9			20	51.75	71.88
	B8			20	42.75	59.38
	B2			0	85.5	Reference
	B3			14	38.25	44.74
	B1			20	29.7	34.74
	B4			0	94.5	Reference
	B6			7	90	95.24
	B5			20	31.5	33.33
4	1	T-beam 100x(400)x 300	3600	0	45.7	Reference
	2			2.5	42.7	93.44
	3			5	41.2	90.15
	4			10	33.8	73.96
5	1 (U1)	T-beam 152x(610)x 381	4980	0	97.5	Reference
	2 (C5-0.8)			4.9	84.5	84.50
	3 (U2)			0	107.1	Reference
	4 (U3)			0	107.1	Reference
	5 (C1-0.4)			6.3	94.9	94.90
	6 (C2-0.4)			10.2	94.3	94.30
	7 (C3-0.4)			12.8	71.9	71.90
	8 (C4-0.4)			12.8	84.5	84.50
6	1	200x300	3000	0	84.5	Reference
	2			5.06	82.5	82.50
7	1 (B9)	150x250	2200	0	69.4	Reference
	2 (B7)			2.2	68.6	68.60
	3 (B3)			4.3	66.8	66.80
	4 (B2)			7.1	63.0	63.00
	5 (B5)			10.2	57.4	57.40

5. Model for area reduction of reinforcement and prestressing tendons due to corrosion

It is in this paper assumed that the area of reinforcement and prestressing tendons are reduced as a result of corrosion, firstly by the area reduction by a uniform corrosion and, secondly by pitting corrosion. The reduction of the area due to uniform corrosion is assumed to be described by:

$$A_{s\_corr\_uni} = \frac{A_s * (100 - wl)}{100}$$

where $A_{s\_corr\_uni}$ is the uniformly corroded area of a reinforcement bar and wl is the weight loss in percentage. The diameter of the corroded reinforcement bar ($\emptyset_{L\_corr\_uni}$) can then be calculated as:

$$\emptyset_{L\_corr\_uni} = \sqrt{\frac{4}{\pi} \cdot A_{s\_corr\_uni}}$$

The area loss due to pitting corrosion is less straight forward. Pits come in many shapes [17] and may be narrow and deep, shallow and wide, elliptical, subsurface and undercutting. Four different geometric models for the area reduction due to pitting corrosion are shown in Figure 3, including a) removing a wedge, b) removing the intersection between two circles, c) removing a segment and as a lower bound case d) reducing the diameter according to the pit size. In all these figures it is assumed that r_1 is the radius of the uniformly corroded reinforcement bar and r_2 is the pit depth.

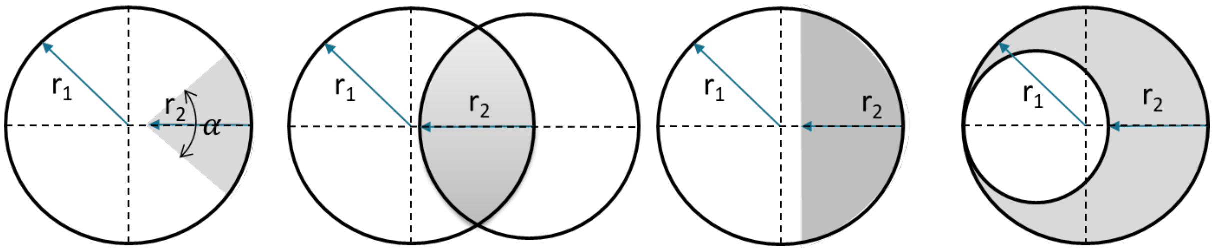

Figure 3: Possible models for the reduction of area of a circular element due to pitting corrosion, where r1 is the radius of the uniformly corroded bar and r2 is the pit depth: a) removing a wedge, b) removing an intersection between two circles, c) removing a segment and d) the lower bond method - reducing the diameter according to the pit size.

The geometric reductions as suggested in Figure 3 a) can be adjusted to represent narrow pits and wide pits, b) and c) are relevant for wide pits in addition to subsurface and undercutting pits and d) is a lower bound. The reduction of the reinforcement cross section for model c and d is given by:

- Removing a segment: $A_{seg} = {r_1}^2 \cdot \mathrm{acos}\left(1 - \frac{r_1}{r_2}\right) - \sqrt{2 \cdot r_1 \cdot r_2 - {r_2}^2} \cdot (r_1 - r_2)$
- The lower bound reduction: $A_{lower\_bond} = \pi \cdot \left(\frac{2 \cdot r_1 - r_2}{2}\right)^2$

The area reduction of the different models is illustrated in Figure 4.

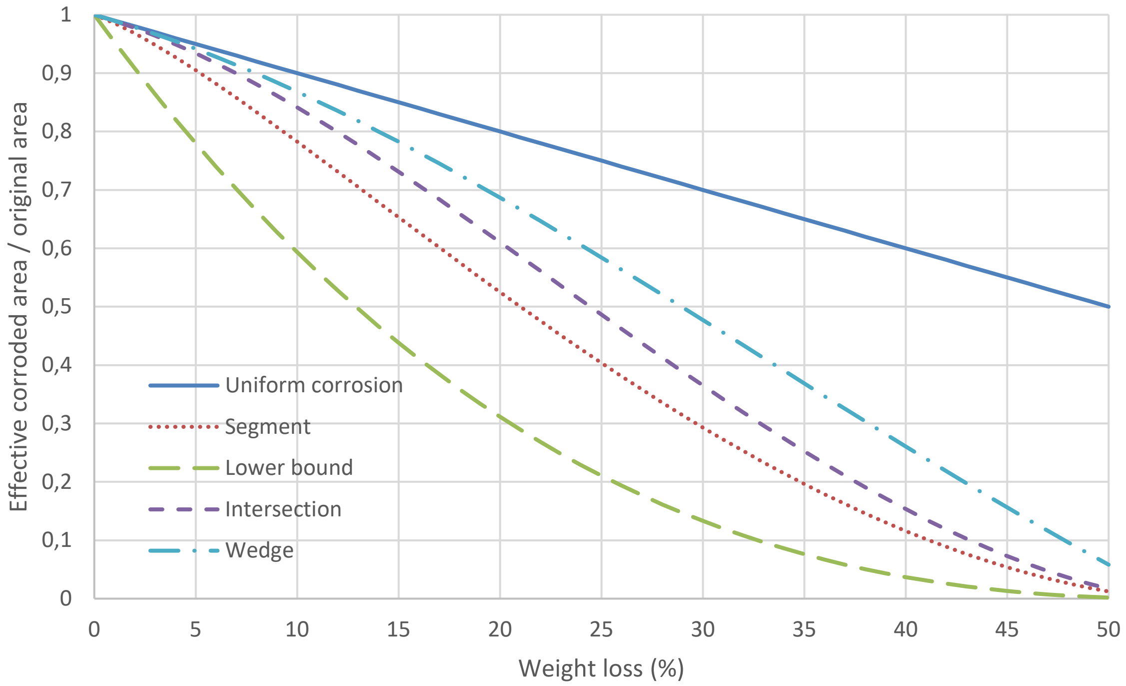

Figure 4: Area reduction according to different models; uniform corrosion and including the effect of pitting corrosion based on the segment method, lower case method, intersection between two circles and method of removing a wedge.

Prestressing tendons are made up of spirally wound wires and will corrode in a different way than a reinforcement bar. One possible approach is to assume a stepwise corrosion to the individual wire threads in the wire rope. The reduction of the cross section of each wire thread could be similar to the reinforcement bar described earlier. The stepwise approach assumes that:

- in step 1 the wires closest to the surface of the concrete will be the first to be exposed to general corrosion and pitting corrosion.
- In step 2, all the outer threads will be subject to corrosion.
- In step 3 there will be both corrosion on the inside and outside of the outer threads, in addition to the area between the threads.
- In the last step, step 4, all 7 threads are affected by corrosion.

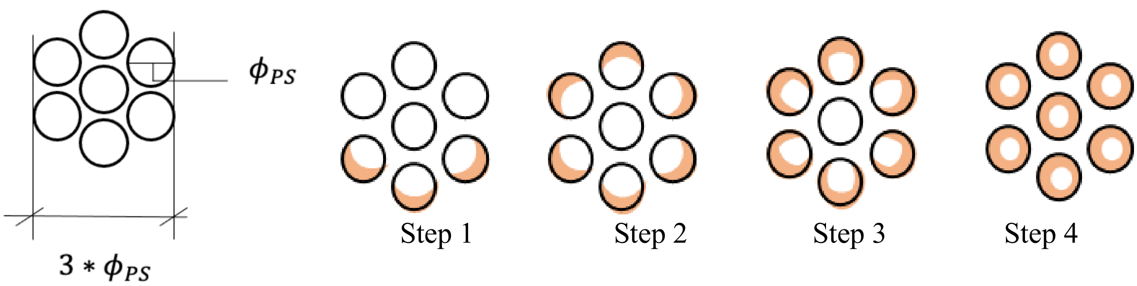

Figure 5: Illustration of the stepwise reduction of area of prestressing tendons

The area of the wire can then be described by $A_p = \pi * \left(\frac{3*\phi_{PS}}{2}\right)^2$, where ϕ_{PS} is the original diameter of the individual wire thread. Similarly, the reduced area of the wire is described by the corroded wire thread diameter $\phi_{PS_{corr}}$. A suggested method for calculating the area of the wire in the different steps may be as provided in Table 4.

Table 4: Stepwise reduction of area of prestressing tendons

Step	Formula for area of wire rope
1	$A_{p\_corr\_1} = \frac{4}{7} \cdot A_p + \frac{3}{7} \cdot \frac{1}{2} \cdot A_p + \frac{3}{2}\left(A_{corr\_uni} - A_{corr\_seg}\right)$
2	$A_{p\_corr\_2} = \frac{1}{7} \cdot A_p + \frac{6}{7} \cdot \frac{1}{2} \cdot A_p + \frac{6}{2}\left(A_{corr\_uni} - A_{corr\_seg}\right)$
3	$A_{p\_corr\_3} = \frac{1}{7} \cdot A_p + 6 \cdot \left(A_{corr\_uni} - A_{corr\_seg}\right)$
4	$A_{p\_corr\_4} = 7 \cdot \left(A_{corr\_uni} - A_{corr\_seg}\right)$

6. Calculation of the flexural capacity of concrete beams with corroded reinforcement

Generally, the flexural capacity of an intact concrete beam is given by [18]:

$$M_{Rd} = F_{cc} * \left(d - \frac{s}{2}\right) + F_{sc} * (d - d')$$

where it is required that the resulting internal forces are in equilibrium, given by $F_{st} = F_{cc} + F_{sc}$, where $F_{st} = f_{yk} \cdot A_s$ is the force created by reinforcement in tension, $F_{cc} = f_{ck} \cdot b \cdot s$ is the force created by the concrete in compression and, $F_{sc} = f_{sc} \cdot A_s{}'$ is the force created by the reinforcement in compression. f_{yk} is the yield strength of the reinforcement bars, f_{ck} is the compressive strength of the concrete material, As is the area of the reinforcement in tension, As´ is the area of the reinforcement in compression, b is the breadth of the concrete beam and $s = 0.8 \cdot x$, the height of the concrete area in compression.

In order to establish F_{sc}, the compressive stress in the reinforcement is needed and is given by $f_{sc} = \varepsilon_{sc} \cdot E_s$. Based on Figure 6, ε_{sc} can be determined graphically to be $\frac{0.0035}{x} = \frac{\varepsilon_{sc}}{x-d'}$ and hence $\varepsilon_{sc} = 0.0035 \cdot \frac{x-d'}{x}$.

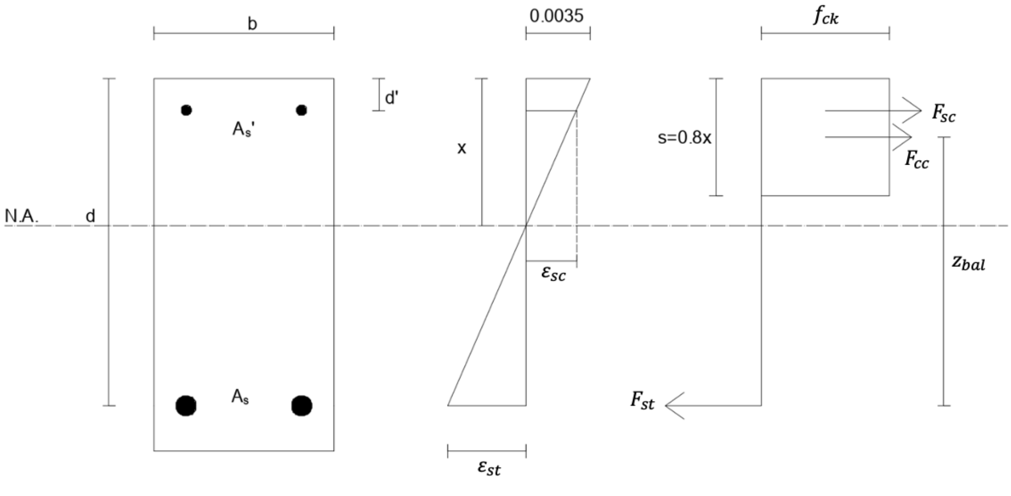

Figure 6: Tension and stresses in reinforced concrete beam, redrawn based on Mosley et al [18]

The elements in the formula for equilibrium is then given by:

$$f_{yk} \cdot A_s = f_{ck} \cdot b \cdot s + f_{sc} \cdot A_s'$$

$$f_{yk} \cdot A_s = f_{ck} \cdot b \cdot 0.8 \cdot x + \varepsilon_{sc} \cdot E_s \cdot A_s'$$

$$f_{yk} \cdot A_s = f_{ck} \cdot b \cdot 0.8 \cdot x + 0.0035 \cdot \frac{x - d'}{x} \cdot E_s \cdot A_s'$$

Solved for x and inserting into formula for the flexural moment capacity gives:

$$M_{Rd} = F_{cc} \cdot \left(d - \frac{0.8 \cdot x}{2}\right) + F_{sc} \cdot (d - d')$$

Estimates of the flexural capacity according to this model are compared to the experimental data in Figure 7 using the segment method for area reduction, and in Figure 8 using the lower-bound method for area reduction.

The segment method to reduce the area due to pitting corrosion, as shown in Figure 7, provides an unbiased estimate and should as such be an ideal method, but is overestimating the flexural capacity in a large number of cases.

The lower bound method to reduce of the area, as shown in Figure 8, provide results reasonably on the safe side, underestimating the mean value by a factor of 0.89 and a CoV of 12%.

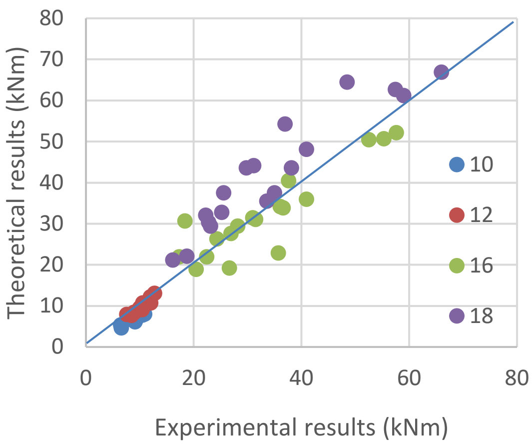

Figure 7: Flexural capacity of corroded beam using segment reduction of area for different diameters (10mm -18mm) of reinforcement bars (Bias 1.02 and CoV=0.22)

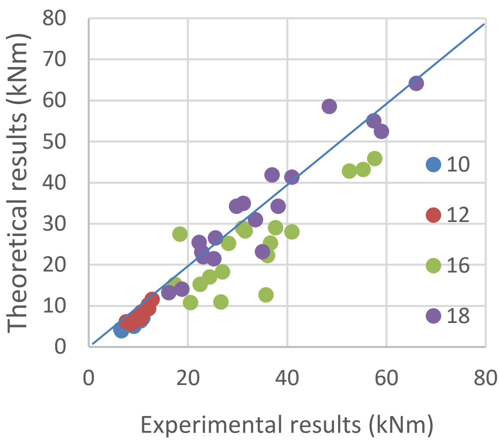

Figure 8: Flexural capacity of corroded beam using lower bound reduction of area (Bias 0.89 and CoV=0.12 for diameter 10 mm and 12 mm bars and Bias 0.86 and CoV=0.25 for diameter 16 mm and 18 mm bars)

7. Calculation of the flexural capacity of concrete beams with corroded prestressing tendons

Previous work on the flexural capacity of concrete beams with corroded prestressing tendons includes [9-13]. From the experimental data in these references, it is clear that there are two effects that are of vital importance to the capacity of corrosion-exposed beams. The first is the reduction in capacity with increased corrosion. The second issue is the loss in ductility, where the ductility of the beams with low corrosion (< ~4%) is similar to the ductility of the reference beams, while the more heavily corroded beams have significantly lower ductility. In the experiments, only average corrosion has been registered, and no information about the size, depth and occurrence of pits are provided. Hence, it is not possible give any exact estimate of the pitting corrosion and its effects on the flexural capacity.

Kioumarsi et al [9] proposes a formula for the reduction in flexural capacity with increasing corrosion level, based on a curve fitting to the mean value of the available experimental data. The fitted curve is split for weight loss due to corrosion of 0-7% ($y = 1 - 0.0172 \cdot x$) and for weight loss due to corrosion of 7-30% ($y = 1.18 \cdot e^{-0.042 \cdot x}$).

In this paper, the model of area reduction presented earlier, including the stepwise reduction of the strands in the prestressing tendons, are used and compared with the experimental tests [9-13] as shown in Figure 9 for area reduction according to the segment method and Figure 10 for the lower-case method. The formula by Kioumarsi et al [9] is also included in these figures.

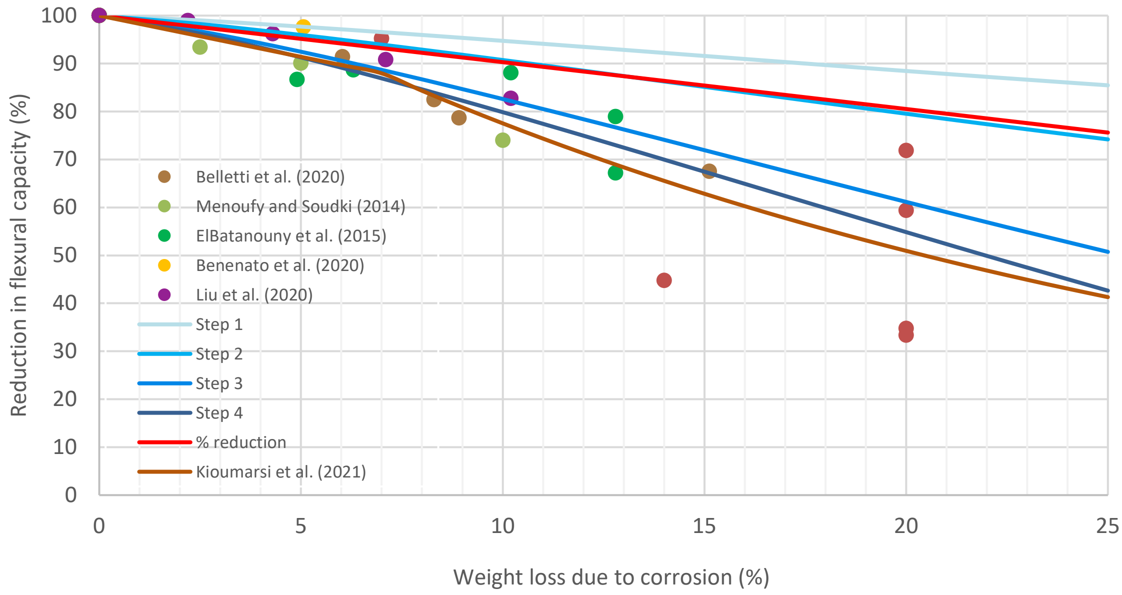

Figure 9: Reduction in flexural capacity at increasing corrosion rate, area reduction according to the segment method.

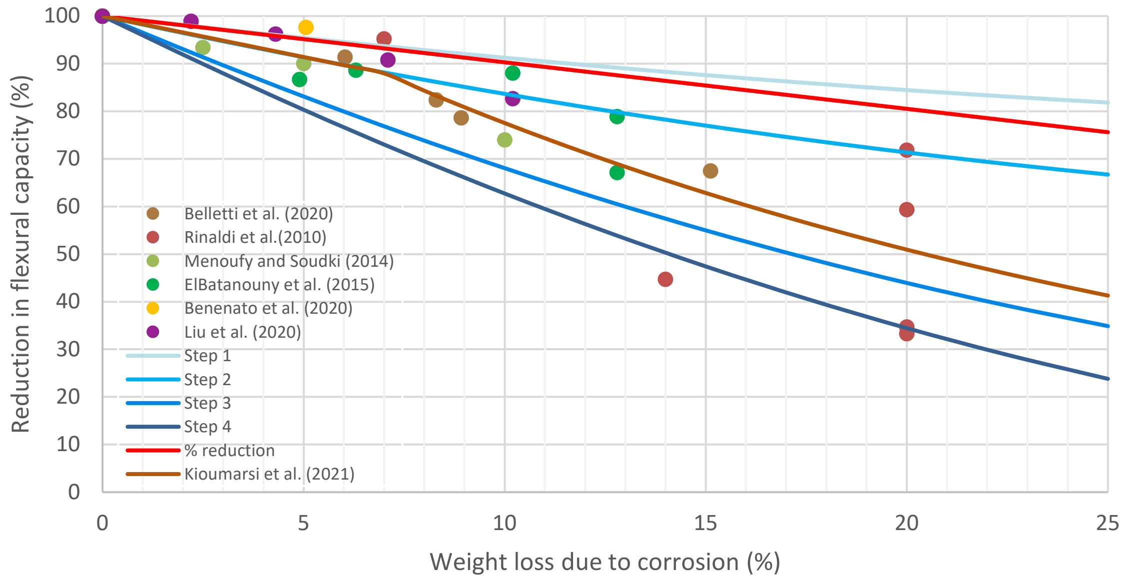

Figure 10: Reduction in flexural capacity at increasing corrosion rate, area reduction according to the lower bound model.

IOP Conf. Series: Materials Science and Engineering 1201 (2021) 012052 doi:10.1088/1757-899X/1201/1/012052

Based on Figure 9 and Figure 10 it can be concluded that:

- The segment method is overestimating the capacity of the concrete beams, at least for corrosion weight loss over 7%. For lower corrosion weight losses, the step 4 can be used, but it is counterintuitive and hence not recommended. Hence, we recommend using the lower-case method for area reduction, which is assumed further.
- Step 2 seems to give a mean value for the capacity of concrete beams with corroded prestressing tendons up to a corrosion level of 7%, and hence coincides with the Kioumarsi et al [9] formulae. However, as a design curve step 2 may be slightly unconservative.
- Step 3 seems to provide a safe estimate of the capacity up to a corrosion level of approximately 13%
- Step 4 seems to provide a reasonable safe estimate of the capacity over 13% weight loss due to corrosion.

The traditional method for estimating the flexural capacity, based on the percentage reduction of the reinforcement, results in a very unconservative estimate of the capacity compared to test results with increasing weight loss, particularly for weight losses above 8%. The curve fitted formula by Kioumarsi et al. [9] is corrected at 7% and thus provides a better estimate. However, also this formula overestimates the capacity compared to the test results, as this is intended as a mean curve.

For the assessment of actual existing structures with known corrosion weight loss, the proposed method provides a safe estimate of the flexural capacity, and it is also the only method that is based on a reasonable physical understanding of the problem.

8. Ductility of corroded prestressing tendons

Ductility is a desired property of structures and prevents brittle fractures of structures. For prestressed structures, there are several damage mechanisms that can lead to reduced ductility. As earlier mentioned, the experimental data clearly indicates a reduction in ductility with increasing corrosion in addition to the reduction in capacity. The cause of the reduction in ductility for prestressed reinforced concrete is believed to be hydrogen embrittlement, hydrogen induced stress corrosion cracking (HISCC) and pitting corrosion, and these often occur simultaneously [19].

American Concrete Institute [20] distinguishes between various causes and regards pitting and HISCC to be easily distinguished, and that pitting corrosion is the major cause of collapses. However, The International Federation for Structural Concrete [21] attributes HISCC as the main cause of fractures in prestressing reinforcement in structures. There may be several explanations for this difference in conclusions, but both ACI and *fib* consider pitting and HISCC to be major issues for prestressed structures.

Hydrogen embrittled steel has been tested in various experiments, Han et al [22], Jia et al [23], to examine the ductility and strength. In general, steel charged with hydrogen achieves approximately 70-80% of the capacity of the reference steel, while the ductility is significantly reduced, especially for hot-rolled steel.

Golkar [24] summarizes these effects and clearly shows that ductility decreases with increasing hydrogen content, as illustrated in Figure 11. From 0.6-0.7 ppm hydrogen to 1 ppm hydrogen there is a sharp reduction in ductility. At 1 ppm hydrogen, the material has become brittle. The strength is also reduced abruptly, but not as sharply as the ductility, and there is still a 40-50% capacity left at 3 ppm hydrogen charging.

Pitting may also result in a more brittle behaviour of the prestressing tendons, as significant pitting and consequent rupture of one wire will lead to a significant increase in the tension of the remaining wire-rope. Hence, the rupture will occur at lower forces and will appear as a reduction in ductility.

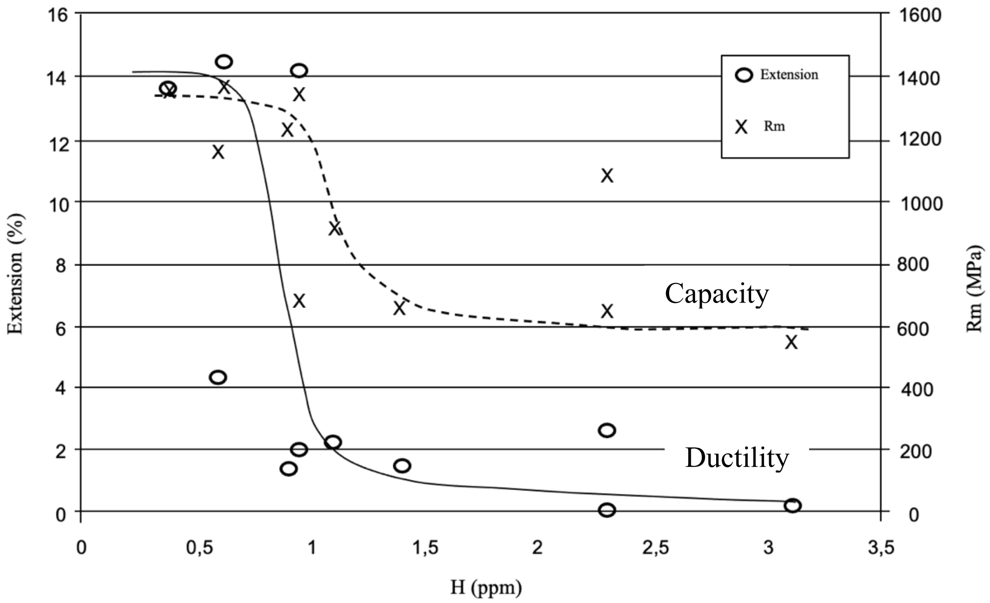

Figure 11: Capacity and ductility at different levels of hydrogen charging, based on Golkar [24]

9. Conclusions and recommendation for further work

This paper has addressed the process of how corrosion affects the flexural capacity of reinforced and prestressed concrete beams. There is at present no standardized method to calculate the flexural capacity of an ageing and corroded concrete structure. The method being used to some extent at present assumes that the percentage loss of the reinforcement due to corrosion, based of inspections, can be used directly as a percentage reduction in the remaining capacity. Experimental data, presented in this paper, indicates that for both types of reinforcement, the capacity is overestimated by this model of reducing the capacity in accordance with the percentage reduction of reinforcement cross section.

A simple method for estimating the flexural capacity has been proposed, partly based on Sigvaldsen [25]. The method proposed is based on a reduction in the reinforcement cross section, both due to uniform and pitting corrosion. Based on measurements of pit sizes as a function of average reinforcement corrosion performed by Fernandez et al [5], a calculation model is proposed that takes into account pitting corrosion. Due to the significant scatter found in the measurements by Fernandez et al [5], a 95% fractile value is used to provide a lower bound estimate of the capacity. The same type of reduction of the cross section has been used both for reinforcement bars and prestressing wire strands. The total reduction in the prestressing tendons has been proposed by including corrosion on the different strands in a stepwise process.

The resulting flexural capacity formula proposed in this paper is compared to a large number of experimental data and provides a reasonably good lower bound estimate of the capacity of simple beams (two reinforcement bars and one prestressing tendon).

For larger long spanned beams with larger cross-sectional area and increased number of reinforcement bars and prestressing tendons, it is unlikely that the largest pit will occur on the same location for several rebars. Hence, a correction for the reduced likelihood of simultaneous pits may be required. However, the available experiments do not provide data for such beams. Hence, insufficient data exists to confirm or deny that such effects will occur. Consequently, the total reinforcement area is relevant in the calculation model as it cannot be assumed that all reinforcement bars are equally corroding. Hence, this need to be seen as a simplification in the proposed model.

Loss of cross-sectional area for prestressing tendons is normally assumed to be more critical than loss of cross-sectional area of normal reinforcement. This, as prestressing tendons are normally made from high strength steel, with yield strength of two to three times the yield strength of normal reinforcement and a loss of area of prestressing tendons will as a result give an effect two to three times that of normal reinforcement. In addition, due to the high strength of the prestressing reinforcement, these are also exposed to other degradation mechanisms such as hydrogen embrittlement (HE) and hydrogen-induced stress corrosion cracking (HISCC). In an acidic environment where pH drops below 7, atomic hydrogen may form as part of the corrosion process and penetrate into the steel. Such an acidic environment can occur in pits and may as a result lead to HISCC. HISCC can lead to an abrupt reduction in the ductility of the prestressing tendons and, hence, brittle fractures at low strain values. HISCC may as a result lead to sudden failures of concrete structures, without any of the common warning signals in the form of large deflections.

Future work is required to include the effect of the loss of bond strength due to corrosion and its effect on the capacity of concrete bridges. If the loss of bond it properly included in the capacity model it could influence the choice of pitting corrosion loss model presented in this paper. In addition, position and frequency of pits in the longitudinal direction on different rebars and its effect on the capacity need to be included in this model for larger concrete beams with several reinforcement bars. The effect of loss of capacity and ductility as a result of hydrogen charging on prestressing reinforcement is of a major concern, and more knowledge is needed to fully assess the safety of concrete bridges, including hydrogen embrittlement and HISCC. As mentioned earlier in this paper, capacity and ductility loss due to pitting corrosion on prestressing reinforcement has also been seen in connection with the so-called soft grout corrosion in cable ducts. No accepted explanation is so far given for this type of corrosion, but it is in this paper suggested that sulphate reducing bacteria may cause such corrosion. One paper reporting this type of corrosion has indicated the presence of sulphate reducing bacteria, but further work is required to confirm of reject this hypothesis.

This paper has exclusively investigated the flexural capacity of corroded concrete beams, and further work to investigate the shear capacity of such beams exposed to corrosion is similarly required for the safe assessment of existing concrete structures.

Acknowledgement: This work is primarily based on the master thesis by Marie Sigvaldsen at the University of Stavanger. However, in this paper some adjustments and extensions have been made regarding the modelling of uniform and pitting corrosion. The opinions expressed herein are those of the authors, and they should not be construed as reflecting the views of any institutions the authors may be associated with.

References

[1] Adasooriya, N. D., Samarakoon, S. and Gudmestad, O. T. (2018), “Corrosion propagation phase and bond strength degradation of reinforced concrete structures: State of the art”, in Int. J. Comp. Meth. and Exp. Meas, vol. 6, no. 3, pp. 499-514, 2018.

[2] Rogers, R. A., Wotherspoon, L., Scott, A. and Ingham, J. M., (2012) “Residual strength assessment and destructive testing of decommissioned concrete bridge beams with corroded pretensioned reinforcement”, in PCI Journal, vol. 57, no. 3, pp. 100-118, 2012.

[3] Bathen, L. (2021) «Spennkabler og korrosjon», Norsk betongforening: Betoncast nr 14. 21/4 2021 (In Norwegian)

[4] Permeh, S., Vigneshwaran, K. K. K., Echeverría, M., Lau, K. and Lasa, I. (2017), “Corrosion of Post-Tensioned Tendons with Deficient Grout, Part 2: Segregated Grout with Elevated Sulfate Content”, in CORROSION, vol. 74, no. 4, pp. 457-467, 2017.

[5] Fernandez, J. M. Bairán and A. R. Marí, (2016), “Mechanical model to evaluate steel reinforcement corrosion effects on $\sigma - \varepsilon$ and fatigue curves. Experimental calibration and validation”, in Engineering Structures, vol. 118, pp. 320-333, 2016.

[6] Kioumarsi, M. M., Hendriks, M. A. N., Köhler, J. and Geiker, M. R. (2016) “The effect of interference of corrosion pits on the failure probability of a reinforcced concrete beam”, in Engineering Structures, vol. 114, pp. 113-121, 2016.

[7] Azad, A. K., Ahmad, S. and Azher, S. A. (2007), “Residual strength of corrosion-damaged reinforced concrete beams”, in ACI Materials Journal, vol. 104, no. 1, pp. 40-47, 2007.

[8] Al-Gohi, B. H. A. (2008), “Time-dependent modeling of loss of flexural strength of corroding RC beams”, MSc Thesis, King Fahd University of Petroleum & Minerals, Saudi Arabi, 2008.

[9] Kioumarsi, M.M., Benenato, A., Ferracuti, B. and Imperatore, S. (2021), “Residual Flexural Capacity of Corroded Prestressed Reinforced Concrete Beams”, in Metals, vol. 11, no. 3, 442, 2021.
[10] Belletti, B., Rodríguez, J., Andrade, C., Franceschini, L., Montero, J. S. and Vecchi, F. (2020) “Experimental tests on shear capacity of naturally corroded prestressed beams”, in Structural Concrete, vol. 21, no. 5, pp. 1777-1793, 2020
[11] Rinaldi, Z., Imperatore, S. and Valente, C. (2010), “Experimental evaluation of the flexural behavior of corroded P/C beams”, in Construction and Building Materials, vol. 24, no. 11, pp. 2267-2278, 2010.
[12] Menoufy, A. E. and Soudki, K. (2014), “Flexural behavior of corroded pretensioned girders repaired with CFRP sheets”, in PCI Journal, vol. 59, no. 2, pp. 129-143, 2014.
[13] ElBatanouny, M. K., Nanni, A., Ziehl, P. H. and Matta, F. (2015), “Condition Assessment of Prestressed Concrete Beams Using Cyclic and Monotonic Load Tests”, in ACI Structural Journal, vol. 112, no. 1, pp. 81-90, 2015.
[14] Vecchi, F., Belletti, B., Franceschini, L., Andrade, C., Rodríguez, J., Sánchez Montero, J. (2020), “Flexural tests on prestressed beams exposed to natural chloride action”, In Proceedings of the CACRCS DAYS 2020, Capacity Assessment of Corroded Reinforced Concrete Structures, Workshop On Line, 1–4 December 2020.
[15] Benenato, A.; Ferracuti, B.; Imperatore, S.; Kioumarsi, M.; Spagnuolo, S. Behaviour of prestressed concrete beams damaged by corrosion. In Proceedings of the CACRCS DAYS 2020, Capacity Assessment of Corroded Reinforced Concrete Structures, Workshop On Line, 1–4 December 2020.
[16] Liu, Y.; Fan, Y.; Yu, J.; Li, Q. (2020), “Flexural behavior test of corroded prestressed concrete beams under chloride environment”, Acta Mater. Compos. Sin. 2020, 37, 707–715. (In Chinese)
[17] Chen, E., Berrocal, C.G., Löfgren, I. and Lundgren, K. (2020), “Correlation between concrete cracks and corrosion characteristics of steel reinforcement in pre-cracked plain and fibre-reinforced concrete beams”. In Mater Struct 53, 33 (2020).
[18] Mosley, B., Bungey, J. and Hulse, R. (2012), Reinforced Concrete Design to Eurocode 2, 7th Ed. Basingstoke: Palgrave MacMillan, 2012.
[19] Nürnberger, U. (2002), “Corrosion inducting failure mechanisms of prestressing steel”, in Materials and Corrosion, vol. 53, no.8, pp. 591-601, 2002.
[20] ACI (2001), ACI Committee 222 Corrosion of Prestressing Steels, ACI, USA, ACI 222.2R-01, 2001, pp 7-8
[21] fib (2003), “fib Bulletin No. 26 - Influence of material and processing on stress corrosion cracking of prestressing steel – case studies”, International Federation for Structural Concrete (fib), Lausanne, Switzerland. p 3.
[22] Han, J., Nam, J. H. and Lee, Y. K. (2016), “The mechanism of hydrogen embrittlement in intercritically annealed medium Mn TRIP steel”, in Acta Materialia, vol. 113, pp. 1-10, 2016.
[23] Jia, H., Zhang, X., Xu, J., Sun Y. and Li, J. (2019), “Effect of Hydrogen Content and Strain Rate on Hydrogen-induced Delay Cracking for Hot-Stamped Steel”, in Metals, vol. 9, no. 7, p 798, 2019.
[24] Golkar, F. (2008), “Study on the effect of hydrogen content on the mechanical properties in press-hardened boron steel”, MSc Thesis, Luleå Tekniska Universitet, Sweden, 2008.
[25] Sigvaldsen, M. (2021), “Flexural capacity of corrosion damaged reinforced and prestressed concrete structures”, MSc Thesis at University of Stavanger, Norway (in Norwegian).