MATEC Web of Conferences **409**, 00001 (2025) https://doi.org/10.1051/matecconf/202540900001
Concrete Solutions 2025

Durable and sustainable service life extension of existing concrete structures: a holistic approach within a life cycle perspective

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Abstract. Concrete structures are facing increasing aging, leading to a rising demand for maintenance, repair, strengthening and/or replacement. More than 70% of the damage to reinforced concrete structures is linked to reinforcement corrosion, which can affect the durability of the structure and the residual load-carrying capacity. Actions are required which consist of an adequate diagnosis, including the identification of damage mechanisms and causes, severity and the extent of defects, showing the cause and extent of the damage, (i) to determine the actual bearing capacity, (ii) to make an estimation of the technical durability and the residual service life and (iii) to select a durable repair technique and/or maintenance strategy (pro-active vs. re-active). Furthermore, in selecting a rehabilitation strategy for preventive maintenance and curative repair, it is important to not only focus on technical requirements and initial cost, but to consider the environmental impact and financial costs over the entire life cycle and the intended service life extension, by considering LCA/LCCA. This paper highlights the entire service life extension process, from diagnosis to the assessment of both technical durability and structural load-bearing capacity, with the aim of developing durable, sustainable and economically viable repair solutions. Several case studies with different functionalities, age and damage causes are discussed: (i) a blast furnace slag cement based concrete water reservoir with insufficient post-execution curing, which led to increased carbonation, and therefore required surface protection after 15 years of service, (ii) a 60-year-old high-rise building where reinforcement scans revealed a lack of reinforcement in the balconies and incorrect placement of reinforcement, leading to reduced load-bearing capacity, and therefore required additional strengthening, (iii) a jetty with high chloride contamination and where potential mapping indicates the need of proper electrochemical treatment. By analyzing these case studies, the need for a holistic approach to concrete repair is emphasized.

1 Introduction

In the late 60s and the mid-70s a significant amount of reinforced concrete (RC) structures were built (Figure 1). As these structures usually have a service life of approximately 50 to 60 years, in the upcoming decade we are expecting challenging times with regards to condition assessment, durable repair and rehabilitation of these structures (Figure 1). In the past decades numerous incidents of collapsed RC constructions have been reported worldwide [1-3] and it can be expected that the frequency of cases with collapsing concrete elements might increase in time if no actions are undertaken. Timely and forehanded interventions are essential to avoid the sudden failure of (civil) concrete structures such as Morandi bridge in Italy (2018) and Carola bridge in Germany (2024). According to numerous guidelines (e.g. the EN1504 standard) the need for an investigation and damage diagnosis prior to a durable repair method is an essential step in the entire service life extension and rehabilitation process in which the cause of the problem is adequately identified [4, 5]. It must be determined whether the structural integrity of the reinforced concrete elements is endangered or not, in order to avoid collapse.

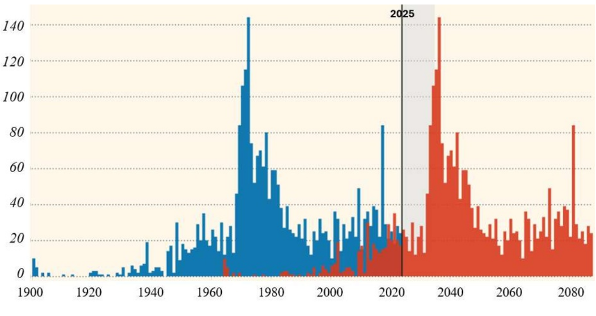

Fig. 1. Histogram of the constructed concrete bridges in Flanders (Belgium) (blue bar chart) and prognosis of the upcoming rehabilitation and service life extension program (orange bar chart).

The main cause of deterioration is corrosion of the reinforcement [6] as more than 70% of the damaged reinforced concrete structures are in a way related to corrosion, induced by carbonation of the surrounding cementitious matrix and/or chloride ingress (due to use of

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de-icing salts or marine environment). Corrosion affects the durability of a concrete structure [7], resulting in cracks and delamination of concrete parts due to the expansive nature of corroding steel. Furthermore, the structural safety and actual load bearing capacity of the structure can be jeopardized as corrosion reduces the cross section of the reinforcement bars in a uniform (due to carbonation) or local way (pitting corrosion initiated by chlorides). Besides a reduction of the steel area caused by corrosion, incorrect or insufficient positioning of the reinforcement, poor concrete quality (e.g. lack of curing, lower cement content, higher water content), leakage and malfunctioning joints and higher applied loadings are reported as the main contributing factors to the failure of RC structures [7].

Proper actions are required which consist of an adequate diagnosis and condition assessment, including the identification of the damage mechanisms and causes, severity and the extent of defects, showing the cause and extent of the damage [8]. For each case of an existing RC structure, it is essential (i) to determine the actual bearing capacity, (ii) to make an estimation of the residual service life and (iii) to select a durable and sustainable repair technique and/or define maintenance requirements (pro-active vs. re-active approach). Furthermore, in selecting a rehabilitation strategy for preventive maintenance and curative repair, it is important to not only focus on technical requirements and initial cost, but to consider the environmental impact and financial costs over the entire life cycle and the intended service life extension. With the European transition towards a circular economy and sustainable development goals in mind, implementing LCA/LCCA (life cycle assessment and life cycle cost analysis) in the repair decision-making process can create durable and sustainable added value [9]. Although we are facing challenging times with an emerging financial and environmental burden, doing nothing (neglective approach) is not an option.

To demonstrate the condition assessment and intervention strategy protocol, a diverse range of case studies encompassing varying functionalities, ages, and damage origins are being discussed in the paper. These include: (i) a blast furnace slag cement based concrete water containing complex suffering from accelerated carbonation due to inadequate post-construction curing, necessitating surface protection after 15 years of service; (ii) a 60-year-old high-rise building with balconies exhibiting deficient reinforcement placement and pitting corrosion due to mixed-in chlorides, compromising their load-bearing capacity and necessitating structural strengthening; and (iii) a jetty grappling with severe chloride contamination, where preliminary assessments suggest the necessity of electrochemical treatment. Through an in-depth analysis of these case studies, this paper underscores the critical need for a holistic approach to concrete repair.

2 Protocol for assessment of existing RC structures

2.1 Phase 0 – Tender stage: introduction and the need for assessment

The need for an evaluation and assessment of the existing structure prior to repair is crucial and therefore a clear and objective method for obtaining a diagnosis is necessary. Numerous guidelines for condition assessment and evaluation of existing (concrete) structures are available in the literature but are mainly a summary of general descriptions and lack objective and/or straightforward action plans. Based on a critical review of existing standards and guidelines, integrating existing knowledge and on-site experience, a stepwise protocol to evaluate the condition and actual bearing capacity is developed (Figure 2). This protocol, which can be seen as a sum of best practices, is particularly applicable on cantilevered reinforced concrete (RC) balconies, but the framework can be generalized for all RC structures. The evaluation of (i) the actual bearing capacity and (ii) the durability (with focus on corrosion damage) are included. The protocol consists of five consecutive phases: 0) tender stage, 1) inspection stage, 2) research stage, 3) assessment stage and 4) report stage.

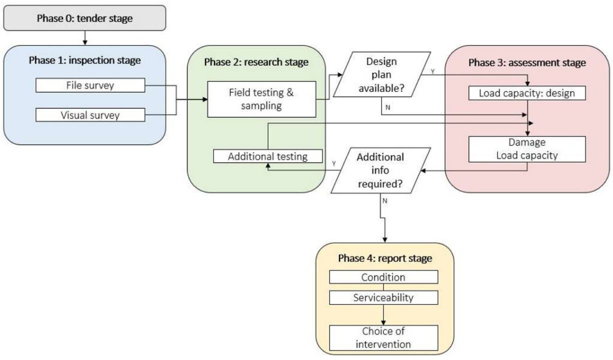


Fig. 2. Stepwise protocol for assessment of RC structures.

Prior to the investigation, it must be decided whether an assessment study is needed. Various scenarios exist for which it is urged to investigate an existing RC structure: damage can be expected due to the age of the building (end of service life is reached or approaching) or the environment (exterior, sea or saline), damage of the RC elements is already observed, assessment of the condition after a hazard or accidental loading, re-use of an old existing structure, extension or expansion of an existing RC structure, etc.

2.2 Phase 1 – Inspection stage: inventory, file investigation and visual inspection

The file investigation includes the collection of information on the original design (plans, reinforcement sketches, as-built reports, etc.). Information about circumstances during execution, maintenance and/or previous repair actions, previous inspection reports can be valuable and are also part of the file investigation. For existing structures, the collection of these documents can be a labour-intensive task. Furthermore, high chances exist on not finding data for aged buildings. In addition, the requirements of the owners also determine the preconditions of the investigation: execution time, available budget and desired lifespan of the RC structure.

A thorough visual inspection of the entire structure (top, bottom and front) with an inventory of observed damage (quantification and qualification) is an additional part of Phase 1. A schematic visualisation and summary of the considered and most expected defects and damage mechanisms, specifically intended for RC balconies, are depicted in Figure 4:

- Mechanical damage of concrete: M1 – M3;
- Physical damage of concrete: cracking due to plastic shrinkage (F1) or hindered (drying) shrinkage (F3), water ingress and crystallization (F5), freeze/thaw related damage (F6);
- Chemical damage of concrete: alkali aggregate reactions, thaumasite and/or ettringite formation (CH1-2)
- Corrosion-related damage due to the expansive nature of corroding steel reinforcement (Figure 3): carbonation induced (CO2) or chloride induced (CO3)

Corrosion, initiated by carbonation and/or chlorides, is the main deteriorating mechanism in most cases. Corrosion initiated by carbonatation is often visible at the underside or front of the RC elements, causing cracks and delamination of large concrete parts. Corrosion initiated by chlorides leads to local phenomena such as pitting corrosion, which reduces the reinforcement area and is often one of the main reasons for sudden RC structural failure and collapse. Therefore, the severity of corrosion-related damage is high. Moreover, in case of chloride induced corrosion, pitting corrosion may not be visible at the surface, at least in the early stages.

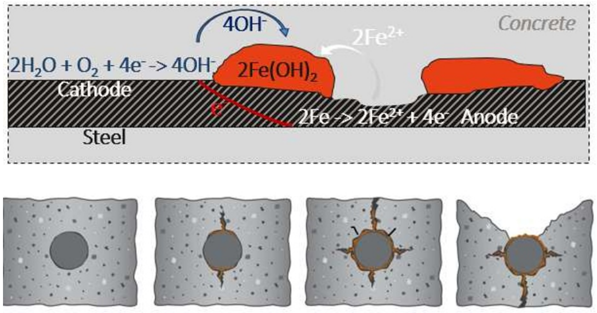


Fig. 3. Corrosion of steel in concrete: electrochemical processes and evolution of cracking and delamination.

Once the visual inspection of the balconies is finalized, the condition of the RC elements can be determined, and in the first stage entirely linked to visual observations. Each element is ranked with a score between 1 (excellent condition) and 6 (extremely poor condition). Once the deterioration mechanisms and damage causes of the RC balconies are determined by means of field testing, sampling and lab testing (Phase 2), the condition score of each balcony can be updated to the actual and proven damage history (Phase 3).

2.2 Phase 2 – Research stage: (non-) destructive testing and sampling

Several non-destructive (NDT) and (semi-)destructive tests (SDT/DT) can be used to identify the damage mechanisms and causes, to map the (potential) danger zones and to evaluate the remaining bearing capacity. Although proof loading may be a useful method to demonstrate the safety of existing structures this is not included in the protocol. The step-by-step method is intended to avoid overkill of destructive testing measurements.

The excessive phasing of the examinations can eventually lead to a higher cost and more disturbance. Additional investigations to support the design of repair techniques (e.g. concrete resistivity in case of cathodic protection) are also not included in the described protocol.

Based on the prior visual investigation of Phase 1, different areas are selected for Phase 2. The basic field testing and sampling procedure includes a combination of consecutive non-destructive (NDT) and destructive (DT) measurements.

A brief description of the visual observed deterioration and location of the collected samples is drawn up for each measuring location, illustrated by photographs on site. The dimensions of the concrete elements are measured and finishing layers are identified for which a combination of (non-)destructive techniques (ground-penetrating radar (GPR), core drilling) are necessary in some cases.

An accurate covermeter (electromagnetic rebar locator) is used to (i) determine the diameter, depth and spacing distance between the rebars and (ii) identification of the concrete cover. It is recommended to physically check and validate the cover locally in a destructive way, e.g. via viewing windows. For calculations of the bearing capacity, the characteristics of the reinforcement are determined. An incorrect positioning or deficiency of the main reinforcement (e.g. by errors during construction) is one of the main reasons contributing to an insufficient bearing capacity. It is therefore strongly advised to investigate the entire element in order to come to a reliable conclusion with regard to the bearing capacity. GPR can also be very useful to locate the reinforcement bars in case the thickness of finishing layers exceeds the capacity of the covermeter (approximately 60-80 mm) or different layers of reinforcement are observed. The location of the rebars at the bottom and front are determined for the residual life-time estimations (durability damage). The attributive properties of the reinforcement (diameter, characteristic steel yield strength) must be determined/validated in a destructive way, as they are an essential property needed for the determination of the actual bearing capacity.

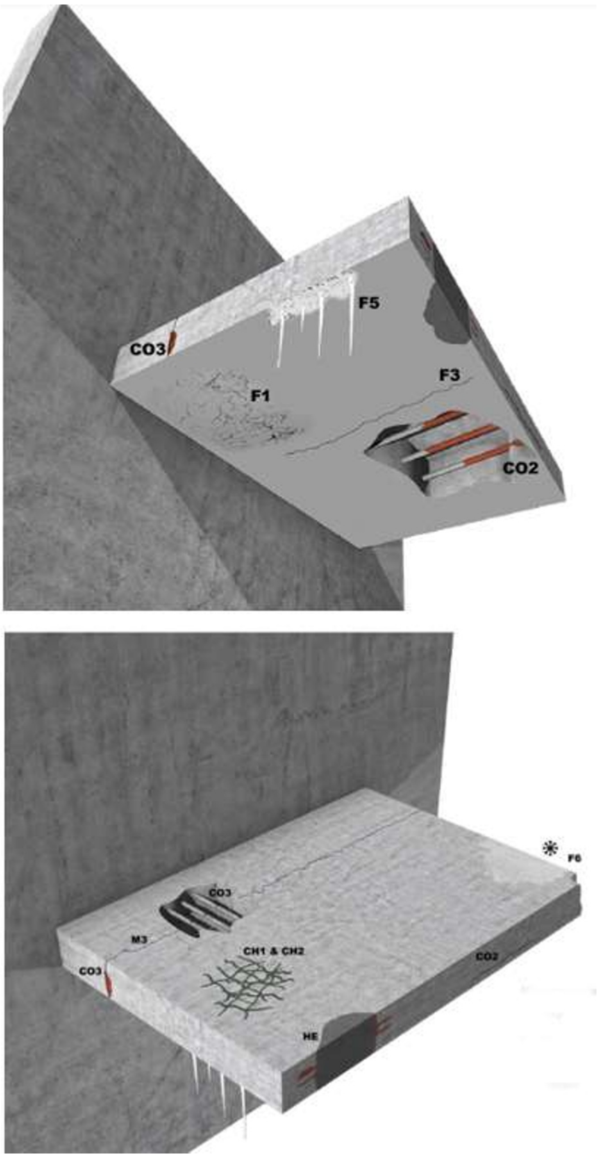


Fig. 4. Render of typical observed damage on cantilevered RC balconies.

In a next step, chloride identification and/or profiling are executed. Different approaches for sample extraction (core drilling, dust sampling) and chloride content determination (Volhard method (rapid chloride test RCT or colorimetric Hach Lange method) or potentiometric titration methods) are possible according to EN 14629. It is possible to extract dust at different levels of the concrete cover, but knowing the amount of chlorides at the level of the rebars is essential. The different levels are based on the previous depth determination by means of the cover meter. Mostly the chloride content as a percent of chloride ion mass by mass of concrete sample is determined. To convert this value into a percent of chloride ion mass by mass of cement content, a correction factor of approximately 8 (=2400/300) is applied. In case values of 1.0 m% (or higher) are found, there is a considerable risk of pitting corrosion of the steel. Values inferior to 0.4 m% appear to be rather harmless. Moreover, this critical chloride content C_{crit} depends on various parameters (exposure conditions, concrete related properties such as cement type, type of reinforcement, etc.). Furthermore, chlorides may have been initially admixed (by use of binding accelerator or polluted raw materials) or penetrated (use of de-icing salts, cleaning products, marine environment). However, the number of tests carried out may depend on the suspicion of whether or not chlorides will be present.

Corrosion damage initiated by carbonation is often visible at the front or bottom of the element which can result to dangerous situations of falling concrete parts and delamination. The bearing capacity is less affected, in comparison to corrosion initiated by chlorides (where pitting corrosion might occur), as corrosion by carbonation does not significantly reduce the diameter of the rebars. The most commonly used technique to determine the depth of the carbonation front is by applying an acid-base indicator (e.g. phenolphthalein) on a freshly created fracture surface as described in EN 14630. Note that phenolpthalein is considered to be carcinogenic and hence handling precautions are required

Additional testing can be performed: determination of the hardness and uniformity of the concrete cover by means of a rebound hammer. Weakened and delaminated areas can be identified by means of this non-destructive technique, especially if combined with ultrasonic pulse velocity testing. In case of an increased risk of corrosion (e.g. lack of concrete cover, high chloride concentration, etc.) potential mapping is a valuable technique to identify possible corroding hot spot areas (Figure 5). As local connection with the reinforcement is necessary, the technique is semi-destructive.

Furthermore, with regard to the assessment of the bearing capacity of the RC balconies (Phase 3), insight into the material characteristics can increase the reliability of the analysis. Therefore, a tensile strength test on an extracted piece of steel reinforcement (according to EN 15630-1) and/or compressive strength test on drilled cores to determine the strength of the existing concrete element (according to EN 13791) can be necessary. By combining non-destructive techniques (rebound hammer, UPV) and destructive strength determination of concrete, the number of cores to be drilled can be reduced [11].

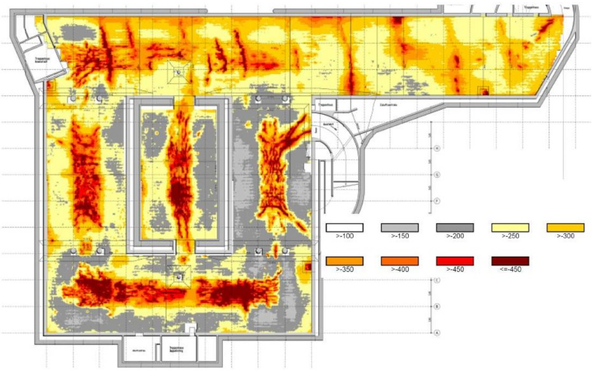


Fig. 5. Identification of corrosion hot spots utilizing semi-destructive potential mapping.

2.3 Phase 3 – Assessment stage: condition, damage and load capacity

2.3.1 Condition assessment

Once the inspection, the field and lab testing are finalized and the damage causes are identified, the condition of the RC elements is determined. Each element is ranked with a score between 1 (excellent condition) and 6 (extremely poor condition), based on the severity, extent and intensity of the damage.

The severity is directly linked to the identification of the damage mechanism. All types of concrete damage that can be expected for RC structures are classified: 1 = minor severity (e.g. coating issues), 2 = average severity (e.g. shrinkage-induced cracking) and 3 = critical severity (e.g. corrosion-related issues).

The extent of a defect is considered in relation to its size: incidental (<2%); local (2% to 10%); regularly (10% to 30%); substantially (30% to 70%); and general (>70%).

The intensity defines the stage of the defect, divided in three classes (Figure 6): initial ('In': damage process is initiated), advanced ('Ad': damage process is propagating and advanced) and final ('Fin': damage process is in its end stadium). By combining the severity, intensity and extent of the defect, a condition score can be linked to the investigated balcony (Figure 6): 1 = excellent condition, 2 = good condition, 3 = average condition, 4 = poor condition, 5 = bad condition, 6 = extremely poor condition. The defect that causes the highest score determines the total condition score of the element.

	Incidental <2%	Local 2-10%	Regular 10-30%	Substantial 30-70%	General >70%
In	1	1	2	3	4
Ad	1	2	3	4	5
Fin	2	3	4	5	6

Fig. 6. Condition score table for critical severity (3) defects.

After Phase 1 of the protocol, the condition score is solely based on visual characteristics. However, visual characteristics are no fully reliable indication of the bearing capacity and/or damage cause of the structure. Furthermore certain defects may be hidden by the finishing layers, which supports the fact that additional in-depth investigations by means of (non-)destructive techniques are critical in order to increase the reliability of the initial conditions score. Therefore, it is important to re-calculate an updated condition score based on an extended investigation survey or once the assessment stage of durability is finished. The bearing capacity is assessed separately, and is not included in the condition score. Besides the visual characteristics (the condition score), several factors (and combinations) can increase the risk of failure: the absence of an effective waterproofing, structure located in a marine environment [17], the age of the construction, a high span in combination with high loading and the use of de-icing salts are examples of indicators for a higher risk.

2.3.2 Assessment of damage and actual bearing capacity

Once the tender, inspection and research stage are performed and the condition score is determined, the acquired information can be used in order to come to a funded assessment of the damage (with regards to the durability of the structure) and the actual bearing capacity of the RC structure. This approach is visualized in Figure 7. Note that the protocol mainly focusses on corrosion damaged RC elements.

2.3.2.1 Durability-related damage assessment

First the durability-related damage is evaluated, in the scope of determining the probability of corrosion of the steel reinforcement to occur within a certain evaluation period t_{eval}. As corrosion might influence the bearing capacity of the RC elements, it is important to assess the durability first, prior to the assessment of the actual bearing capacity. Therefore, it is necessary to determine i) if corrosion can be expected within the evaluation period, and (ii) what mechanism might cause possible depassivation (chloride ingress and/or carbonation).

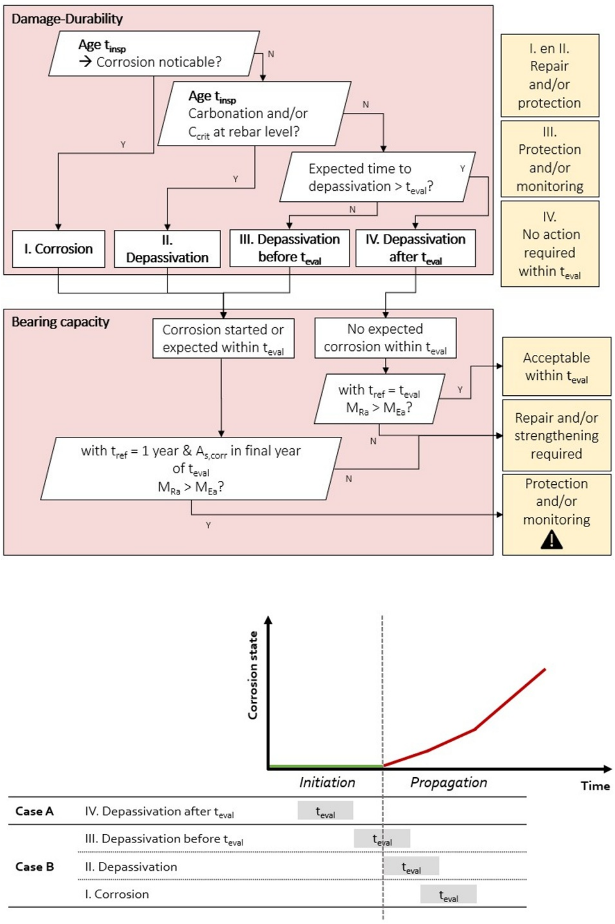


Fig. 7. Assessment protocol for damage/durability and bearing capacity of RC balconies, focus on corrosion.

Preventive life-extending and curative repair techniques are directly linked to the durability related condition of the element:

- If corrosion is clearly noticeable (I) proper and durable maintenance and repair actions are required. The same actions (II) are needed in case corrosion is not yet noticeable but initiation of depassivation already occurred (carbonation at the level of the rebar or a critical chloride content C_{crit} is reached).
- If initiation of depassivation is not yet reached but is expected within t_{eval} preventive actions and/or monitoring (III) of the structure is strongly advised.
- No actions (IV) are required if initiation of depassivation is not yet reached and not expected within $t_{eval.}$

Note that by means of full probabilistic approaches the probability of failure with regards to depassivation of the reinforcement can be determined and evaluated [12-13]. As input for this analysis, the age of the structure, the concrete cover, the depth of the carbonation front and chloride content profiling is needed and can be determined (non-)destructively in the preceding research stage (Phase 2).

2.3.2.2 Bearing capacity assessment

Once the durability-related damage is evaluated, the actual bearing capacity of the cantilevered RC balconies can be evaluated. To be able to evaluate the remaining bearing capacity several input parameters are necessary and need to be determined by means of the preliminary file survey (Phase 1) and/or on-site testing (Phase 2): the dimensions of the slab, the position of the reinforcement (i.e. based on the concrete cover and rebar diameter), the steel section, the material properties (steel and concrete strength) and the durability related condition of the element (concrete and/or steel damage). Calculations are performed in Ultimate Limit State (ULS). Insufficient load-bearing capacity due to reduction of steel section or by execution errors such as incorrect positioning of the reinforcement can cause sudden failure of the RC structure.

The comparison of the applied and the resistant bending moment (M_{Ea} vs. M_{Ra} respectively) is the considered and most dominant failure mechanism in this protocol.

At first, the remaining bearing capacity of the design is checked. This calculation can only be made possible if the file survey was successful and the necessary input parameters are known. If not, this check-up is not executable. Consecutively, depending on the outcome of the damage assessment, different scenarios are possible (Figure 7). For scenario A, if initiation of depassivation is not yet reached (I) and not expected within t_{eval}, no reduction of the steel section due to corrosion is expected and concrete quality is intact. A safe estimation is made for the concrete strength and steel quality without performing any on-site destructive sampling and lab testing. If there is no presumption of corrosion damage (no visible corrosion damage, carbonation front is lower the concrete cover, chloride content is lower than the critical value) and the remaining bearing capacity is sufficient (i.e. $M_{Ea} < M_{Ra}$), no additional inspection is needed and the bearing capacity is acceptable. However, if M_{Ea} exceeds M_{Ra} it might be useful to determine steel quality and concrete compressive strength on site rather than using estimated conservative values. If the bearing capacity is not sufficient, due to the incorrect positioning of the reinforcement, repair and/or strengthening measures should be prescribed.

In case depassivation is expected or already occurred within t_{eval} (scenario B, I-II-III), the bearing capacity is checked for a reference period t_{ref} of 1 year, taking into account a reduced steel cross section $A_{s,corr}$ of which it might be expected that corrosion reduced the amount of steel, hence reducing the resisting bending moment. Therefore, additional investigation is needed to evaluate the actual condition of the steel reinforcement (e.g. potential mapping, destructive visualization of the steel) in the final year of t_{eval}. If the bearing capacity is not sufficient in that case, due to the incorrect positioning of the reinforcement (i.e. reduced effective depth), repair and/or strengthening measures should be prescribed. Protection and monitoring are advised in the other case where the bearing capacity is sufficient, as the propagation of the corrosion process might worsen the condition rapidly.

The geometrical and material properties obtained during Phase 2 are used in this assessment stage. The dimensions (width, length, thickness) can be easily obtained on-site. If the material (steel/concrete) properties f_{yk} and f_{ck} are unknown a rather conservative estimation is made, e.g. based on the building age and corresponding building experience and regulations at that time. However, if additional testing is performed, the in situ characteristic value in situ $f_{ck,is}$ and/or $f_{yk,is}$ can be integrated into the calculations.

One of the key parameters is the effective depth of the reinforcement which has a decisive effect on the bearing capacity of the cantilevered slab as mentioned in [8, 11]. Generally, an average value of the effective depth is obtained from the reinforcement detection measurements. However, for existing structures, a wide range of individual results of the concrete cover can be found at the investigated location used for calculating a mean value of the effective depth. Therefore, an updated value is considered by taking into account the variance of the measurements in situ.

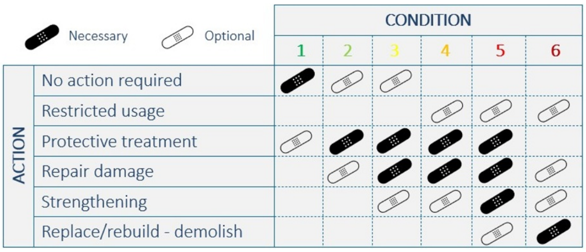
The steel section A_{s1} can determined by means of the file survey in combination with on-site testing for verification. If corrosion of the steel reinforcement is likely to occur or to be present (scenario B) the condition of the steel needs to be taken into account, e.g. by means of expressing the steel section as $A_{s,corr}$, of which its value is determined by means of additional testing and calculations. The corrosion process reduces the diameter and also might lead to a reduction in yield strength according to [14-16].

2.4 Phase 4 – Report stage: intervention decision

Once Phase 3 is finalized the conclusions of the investigation are summarized in an inspection report. This report includes the final condition score and complete durability-related damage diagnosis, evaluation of the actual bearing capacity and residual life estimation. Based on the results, recommendations to continuous monitoring, preventive maintenance and/or curative repair (possible techniques, materials, urgent intervention, etc.) based on the repair actions and principles described in EN 1504-9 should be created. Several concrete repair principles are available (Figure 8). It is important to link the acquired condition score to actions that have to be undertaken to preserve, maintain or restore the health and soundness of the investigated RC structure (Figure 9).

Concrete damage	Reinforcement corrosion
1. Protection against ingress	7. Preserving or restoring passivity
2. Moisture control	8. Increasing resistivity
3. Concrete restoration	9. Cathodic control
4. Structural strengthening	10. Cathodic protection
5. Increasing physical resistance	11. Control of anodic areas
6. Resistance to chemicals	

Fig. 8. Repair principles P1-6 for concrete damage and P7-11 for corrosion-related damage of RC structures.



● Necessary ○ Optional

		CONDITION					
		1	2	3	4	5	6
ACTION	No action required	●	○	○			
	Restricted usage				○	○	○
	Protective treatment	○	●	●	●	●	
	Repair damage		○	●	●	●	○
	Strengthening			○	○	●	○
	Replace/rebuild - demolish					○	●

Fig. 9. Relation between the condition of an RC structure and its acquired service life-extending actions.

To increase the sustainability of service life-extending strategies of existing RC structures, selecting an appropriate repair technique should consider both the environmental impact and the cost over the life cycle of the preferred lifetime extension of the construction [9, 17]. This state-of-the-art holistic approach contrasts with the current practice where repair techniques are usually chosen as a function of the available budget without considering a life cycle perspective. Given its prominence in academic research and policy documents, it is important that the concept of the circular economy is subjected to critical analysis and is assessed quantitatively by including a life cycle perspective. Methods like life cycle assessment (LCA) have proven their value for making well-informed decisions concerning the optimization of the environmental performance of products, processes and services. Combining LCA and life cycle cost analysis (LCCA) endorses a holistic approach to decision-making. Unfortunately, the availability of studies supporting decision-making for the rehabilitation of existing concrete structures through LCA and LCCA is almost non-existent. In general, there is a lack of LCA results of service life-extending concrete maintenance and repair techniques. Furthermore, most of the literature on LCA for concrete buildings focuses on (i) new 'green' construction or (ii) energy refurbishment. Only a few studies consider the environmental impact of building system repair. However, it can be stated that the rehabilitation of existing structures will be one of the key factors in Europe to reduce the environmental impact in the future. Overall, the available studies (i) include mostly only one of the two assessment methods (LCA or LCCA), (ii) consider a limited number of maintenance or repair interventions and/or (iii) are mainly case dependent. Moreover, there are also uncertainties linked to the long-term effect of some interventions regarding the End-of-Life (EoL), such as the deterioration mechanisms of concrete repair techniques.

3 Case studies

3.1 Context

To demonstrate the added value of the developed protocol for assessment of existing RC structure, three case studies with different functionalities, ages and damage causes are being discussed: (i) a 60-year-old high-rise building where reinforcement scans revealed a lack of reinforcement in the balconies in chloride-contaminated concrete and incorrect placement of reinforcement, leading to reduced load-bearing capacity, and therefore required additional strengthening; (ii) a blast furnace slag cement based concrete water reservoir with insufficient post-execution curing, which led to reduced quality of the concrete cover, increased carbonation, and therefore required surface protection after only 15 years of service to reach the desired service life of 50 to even 100 years; (iii) the pillars of a jetty with high chloride contamination and potential mapping indicate the need for proper electrochemical treatment. By analyzing these case studies, the added value of a holistic approach within a life cycle perspective to concrete repair is emphasized

3.2 Cantilevered RC balconies in high-rise building

A high-rise building situated in Brussels with cantilevered RC balconies (NW oriented), constructed in 1961, has 17 floors and each floor has a length of 33 meters and with cantilevered RC balconies (span 1.5 meters, thickness of 100 mm covered with a cementitious screed of 30 mm). Additional info can be found in [8,10]. According to the original design documents the elements are reinforced with longitudinal tension rebars with a diameter of 10 mm and spacing distance 100 mm, (totalling a steel cross section of 864 mm²/m), with a cover of 20 mm: the design value of the effective depth equals 75 mm. These rebars are anchored in the RC beams/plates of the building. No details were found on the concrete strength class or the steel quality.

An in-depth field survey and investigation was performed on 9/17 floors of which F1, F8 and F16 are selected for the assessment demonstration of this paper. First of all, high chloride contents (exceeding the critical chloride content) were found at the level of the rebars for floors F8 and F16, most likely caused by the use of $CaCl_2$ as a binding accelerator (evidenced by a lack of concentration gradient). According to the protocol, it therefore can be assumed that depassivation and corrosion is most likely to be present in t_{eval}: also corrosion damage visuals are clearly noticeable Assessment with regard to durability-related damage leads to the conclusion that repair and protection will be needed (action I and II). As corrosion is initiated by mixed-in chlorides all floors are ranked with condition score 5, as corrosion damage has a critical severity, the process is believed to occur generally (>70%) due to the presence of mixed-in chlorides and the corrosion process is already in an advanced stage.

Fig. 10. Failing coating and joint + local rust stains + crack parallel to building (floor F16).

As original plans were available, the design of the RC balconies can be evaluated. A recalculation was executed based on the partial factors e.g. according to Eurocode regulations and FIB Bulletin 80 and amounts up to 3.97-5.41 kN/m², which is sufficient.

For the selected floors, the upper reinforcement was detected by a covermeter, the non-destructively obtained values were checked by core drilling and the diameter of the rebars was confirmed destructively. For F1 lower values for the mean effective depth were found (57,6 mm), F16 revealed higher values (87,2 mm), where 77,1 mm was found for F8. The first assessment was done with the assumption that no chlorides were present and corrosion was not expected. The insufficient bearing capacity of F1 is a result of the low position of the rebars. For floors F8 and F16 the maximum reduction of the reinforcement area due to corrosion Q_{corr} for which applied M_{Ea} = resistant M_{Ra} with a variable load $q_{k,rem}$ of 2.5 kN/m² is determined and amounts to 16% for F8 and 25% for F16. Additional testing was executed: chloride profiling, carbonation depth determination, potential mapping in combination with destructive core drilling for visualization of the steel rebars and concrete strength verification. Although a parametric study revealed that the effect of concrete strength on the actual bearing capacity is limited, core drilling and testing and rebound hammering were performed on several floors (Figure 11). A high variation in strength among floors was noticed in combination with a high standard deviation (up to 10 MPa). It was also revealed that using the suppliers based correlation curve of the Schmidt hammer for strength estimation is unacceptable as it clearly overestimated strength.

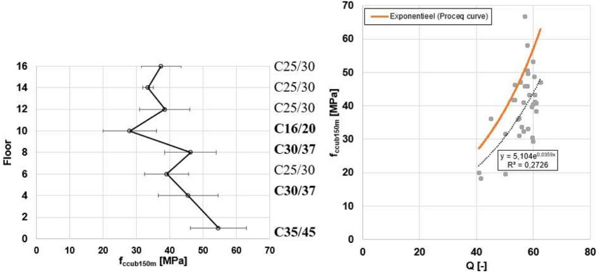


Fig. 11. Compressive strength determination by means of core drilling (DT) and rebound hammer (NDT).

As high chloride contents (>2m%$_{cement}$) were found at F8 and F16 it can be expected that corrosion already started. A clear difference in potential mapping between the two floors is noticeable (Figure 12): the observed crack on F16 accelerates the corrosion process, which was also observed visually: Q_{corr} approximately 20% for F16, negligible for F8.

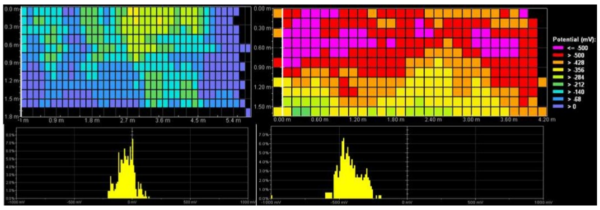


Fig. 12. Potential mapping of floors F8 (left) and F16 (right).

The protocol revealed that floor F1 and floor F16 (insufficient bearing capacity as a result of the corroding rebars in chloride-contaminated concrete) need adequate repair and strengthening due to lower positioning of the rebars (F1) and severe corrosion due to crack parallel to the building on the top level of the cantilevered balcony (perpendicular to the steel reinforcement) (F16) respectively.

To finalize, the economic and environmental impact of five commonly used repair techniques, i.e. patch repair, conventional repair, galvanic cathodic protection, impressed current cathodic protection, and total replacement was determined (detailed calculations in [18]). For a brief extension of the buildings' service life (up to 5 years), patch repair emerges as the most favourable option, maintaining the existing condition with minimal intervention. However, to reach a longer service life extension (up to 40 years), conventional repair or cathodic protection emerge as competitive options where the entire renewal of concrete elements exhibits the highest overall environmental and financial impact.

3.3 Water retaining structure

A potable water retaining concrete structure, constructed in 2010, consists of RC walls (4 elements: length 25 m, height 5 m, thickness 0.30 m), RC columns (41 elements, diameter 0.40 m, height 5 m) and a RC slab on top (25x25 m², thickness 0.30 m). The concrete was designed according to EN206 regulations (exposure classes XC3-XF3-XA3, 320 kg of blast furnace CEM III/B 42.5, W/C-ratio 0.50, porphyry aggregates), steel quality BE500 with a minimal concrete cover of 30 mm in correspondence to EN1992-1-1. The RC elements were constructed in winter/spring, without any proper demoulding strategy and curing actions. Only 14 years after construction it was found that the surface of the walls and the columns that come in direct contact with the water had a brittle skin and experienced leaching led deterioration, debonding and dislodging of aggregates in the inside of the reservoir, up to a depth of 5 mm. This phenomenon of softening of concrete in contact with potable water has been reported in other studies [20].

No corrosion of the steel reinforcement in the walls and columns inside the reservoir is expected as negligible chloride content and carbonation depth were found. According to the protocol, it therefore can be assumed that depassivation and corrosion are most likely not to be present in t_{eval}. Assessment with regard to corrosion-related damage leads to the conclusion that no further action is required (action IV). However, this only applies to the elements in the reservoir, with no distinction in conclusion between the area in direct contact with the water and the area above the water line (4 m in height) as the relative humidity is higher than 90% which significantly slows down CO_2 diffusion, lowers the air permeability and the carbonation rate.

However, the parts of the outer wall elements experience severe carbonation, with carbonation rates up to 5-7 mm/y$^{0.5}$ (carbonation depth of 20-28 mm), which is very high even for blast furnace slag cement based concrete. It was found that the execution of the elements in wintertime (snow, temperatures below 5°C) was performed without the application of proper insulation and heating. Poor workmanship after demoulding, with no proper curing actions, led to an inferior quality of the skin-crete. According to EN 13670 a curing period of at least 7 days should have been applied. Additional testing by means of a portable air permeability tester revealed that the quality of the concrete in the outer layers was rather poor (high permeability rating, Figure 13).

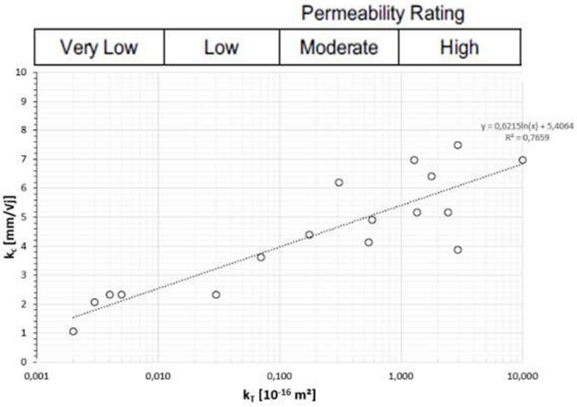


Fig. 13. Correlation between carbonation coefficient and air permeability.

Rebar scanning revealed high variations in concrete cover (Figure 14). According to the protocol, it can be assumed that depassivation and corrosion is most likely to occur in t_{eval} in certain areas with insufficient cover, as full probabilistic Monte Carlo simulation revealed that carbonation induced corrosion can occur within 15 years (totalling a service life lower than 30 years!) if no protection is applied. Also, it was advised to further monitor the evolution of the condition of the RC wall elements (action III).

3.4 Harbour jetty

A jetty in Port of Antwerp, constructed in 1970, consists of 40 hollow prefabricated cylindrical RC pillars (outer diameter 60 cm, thickness 12 cm). The distance between the water line and the top of the pillar is approximately 1.80 m. A condition assessment of the pillars was performed by means of an extensive testing program: concrete cover rebar scanning, chloride profiling on three different levels (-1.0 m below water, +0.40 m and +1.0 m above water), potential mapping and concrete resistivity testing via Wenner probe.

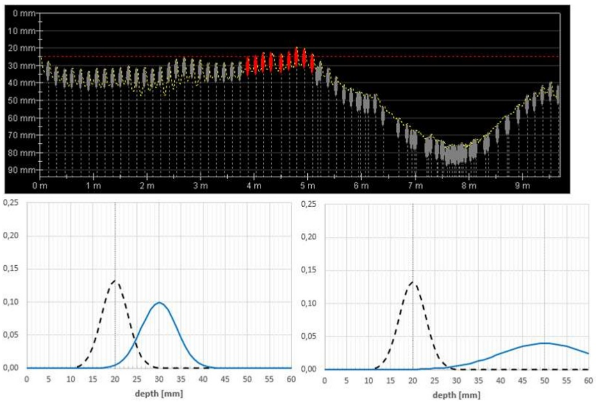


Fig. 14. High variation in concrete cover and full probabilistic Monte Carlo simulation results (dotted line: depth of carbonation front, full blue line: concrete cover).

The minimal concrete cover is 34 mm, mean value 39 mm (standard deviation of 3 mm), which is acceptable. Very high concentrations (>2 m% compared to cement mass) were found at the level of reinforcement for the levels located under water. For the levels located at or above the waterline, the chloride concentrations at the level of the reinforcement are medium to high (0.4-1.0 m% compared to the cement mass). Minimal concrete resistivity is between 41 and 88 kΩcm.

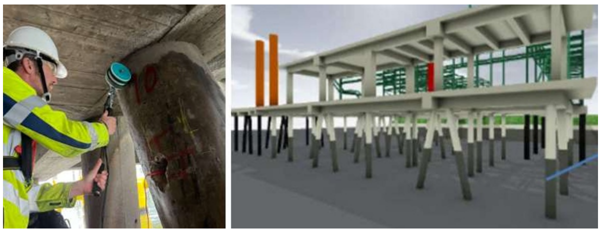
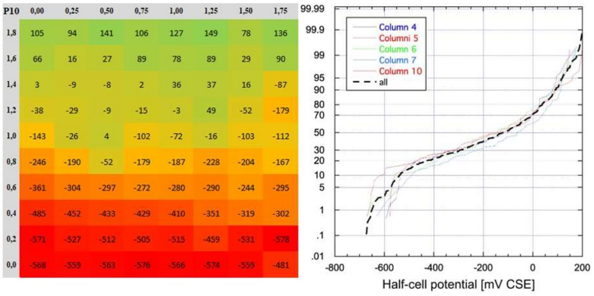


Fig. 15. Execution of potential mapping via one-wheel-electrode ($CuCuSO_4$) + schematic overview of the jetty.

The main focus of this brief discussion will be on the potential mapping that was performed on 5 selected pillars. Potentials below -400 mV were found that are influenced by water saturation up to a height of +0.4 m, with a broad region comprising 50% of the data between (-400 mV and 0 mV). A region with potentials more positive than 0 mV (about 30% of all data) was located higher than +1.3 m (Figure 16).

For each pillar on level +1.20 m (for connection with the multimeter) and +0.40 m (in the splash zone), the rebars were destructively liberated and examined: no corrosion was found. The rebars were non-ribbed, with a diameter of 7-8mm.

The resistivity was measured via Wenner probe on level 1.20 m (after the rebar detection, so the probe was positioned in between rebars).



P10	0,00	0,25	0,50	0,75	1,00	1,25	1,50	1,75
1,8	105	94	141	106	127	149	78	136
1,6	66	16	27	89	78	89	29	90
1,4	3	-9	-8	2	36	37	16	-87
1,2	-38	-29	-9	-15	-3	49	-52	-179
1,0	-143	-26	4	-102	-72	-16	-103	-112
0,8	-246	-190	-52	-179	-187	-228	-204	-167
0,6	-361	-304	-297	-272	-280	-290	-244	-295
0,4	-485	-452	-433	-429	-410	-351	-319	-302
0,2	-571	-527	-512	-505	-515	-459	-531	-578
0,0	-568	-559	-563	-576	-566	-574	-559	-481

Fig. 16. Potential map of pillar P10 and overview of the frequency diagram.

According to the protocol it can be assumed that depassivation due to chloride ingress has already set in, but has not led to reinforcement corrosion yet. It was therefore advised to protect (action II-III) and monitor the condition of the pillars by means of cathodic protection. An overall recommendation for civil structures that are not easy to reach or to inspect is applying integrated sensors that can monitor the condition from the start, e.g. by means of fiber optics [19], internal RH measurement, concrete resistivity or steel potential evolution mapping. However, this very interesting topic lies out of the scope of this paper.

4 Conclusion

The need for sustainable rehabilitation (maintenance and repair) of existing concrete structures is urgent and due to the aging of buildings and civil structures, there will be a great need for preventive and/ or curative interventions in the upcoming decades. More than 70 % of the damage to reinforced concrete (RC) structures is linked to reinforcement corrosion, which affects the durability and sustainability of the structure. Besides a reduction of the steel area caused by corrosion, incorrect or insufficient positioning of the reinforcement (i.e. lack of concrete cover), poor concrete quality execution (e.g. lack of curing, lower cement content, higher water content), leakage and malfunctioning joints, inadequate design, concept and choice of building material and building details and increased environmental and applied loadings are reported as the main contributing factors to the failure of RC structures.

Proper actions are required which consists of an adequate diagnosis and condition assessment, including the identification of the damage mechanisms and causes, severity and the extent of defects, showing the cause and extent of the damage. For each case of an existing RC structure, it is essential (i) to determine the actual bearing capacity, (ii) to make an estimation of the residual service life and (iii) to select a durable and sustainable repair technique and/or define maintenance requirements (pro-active vs. re-active approach).

To demonstrate the added value of the developed protocol for assessment of existing RC structure, three case studies with different functionalities, age and damage causes were discussed.

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