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An objective assessment strategy for industrial reinforced concrete structures – Case study.

Liesbet De Winter[1*], Mathias Maes[1], Tim Soetens[1], Bjorn Van Belleghem[1]

[1]SANACON bv, Ghent University spin-off, Nijverheidsweg 1/A, 9820 Merelbeke, Belgium

Abstract. Due to the booming construction wave between the 60´s and 80´s, a lot of reinforced concrete structures are now approaching their intended service life design. Concerning the industrial sector, an unforeseen shutdown of production processes due to urgent maintenance or repair actions of reinforced concrete structures can have major consequences for owners and managers. Therefore, it is important that they get a clear view of the current state of their structures in order to shift towards a more proactive maintenance. In order to recognize and assess both classic and more complex damage phenomena, a scientific approach to translate collected field data towards a service life and inherent repair strategies for the concrete structure is adopted by means of an in-house developed data analysis tool DIMCoSt (Durability Information Model for Concrete Structures). The objective assessment strategy is described in this paper by means of a case study. The subject of the case study is concrete storage silos. Although the applied methodology is mainly universal, each case has its own emphasis that must be considered regarding the decision-making process based on the owner's or manager's requirements. By means of this case, a scientifically based approach is presented which leads to objective decision-making regarding asset management and planning of maintenance or renovation activities.

1 Introduction

An objective assessment and decision-making strategy regarding concrete structures on industrial sites is explained by means of a case study. Four crucial steps in this strategy are: |1| the diagnosis: how to determine the nature, cause and extent of the damage; |2| assessment: how to assess the severity of the damage and |3| sustainable repair options: which repair actions need to be done to slow down or repair the damage and what is the impact on the operation of the company? |4| Review: is everything available to formulate an answer to the previous questions or does additional research need to be done? All these steps are discussed throughout this paper in more detail.

2 Subject of the case study

The concrete storage silos are located on an industrial site (port area), which has grown organically over time. The main reason for having a survey of their concrete silos is because of the safety towards their employees. Falling pieces of concrete were already observed in the past, which is usually a (rather late) reason for investigation. In this case two different construction phases are discussed. A part of the industrial site is shown in the floor plan Fig. 1. The structure from phase 1 was built in 1968 and consists of two concrete structures, namely a silo tower (1A) and 23 vertical silos in a silo block (1B). Construction phase 2 consists of 7 vertical silos and was added at the south side of construction phase 1 in 1972.

The silo tower (1A) from construction phase 1 has a height of 56 m. The silo block (1B) and silos of construction phase 2 are 44 m high. An overview of the site with indication of the silos from both phases is given in the google maps view in Fig. 2. The silo walls have a thickness of 20 cm.

Fig. 1. Google maps plan view of the storage silos from phase 1 and phase 2.

In order to reduce this paper complexity, this case only contains 2 construction phases out of a total of 6 different construction phases on the industrial site, which were all investigated at the same time.

* Corresponding author: liesbet.dewinter@sanacon.be

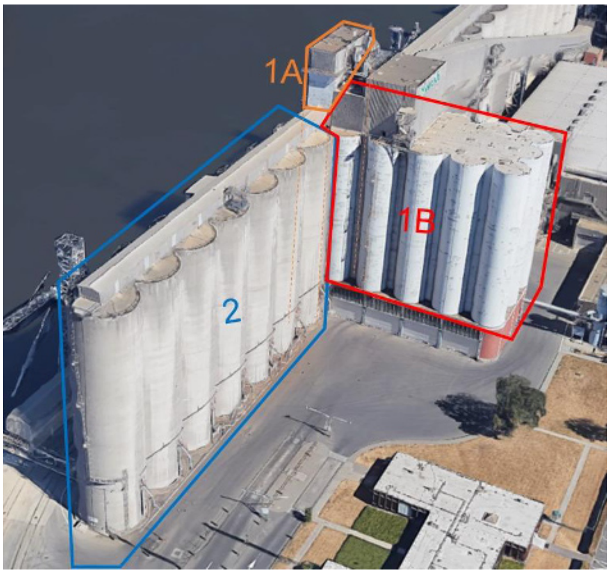


Fig. 2. Google maps view of the storage silos d.d. 16/03/2022.

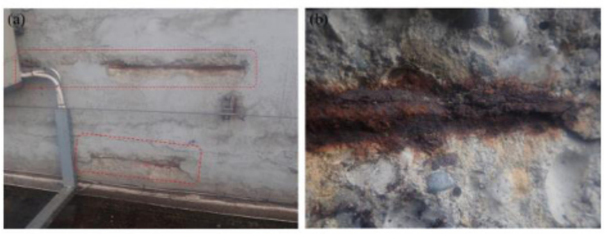
3 Diagnosis

An initial file study was carried out in order to collect historical information about the construction of the different phases, e.g. the method of construction and execution and any information known about some earlier (local) repairs.

Based on this file study, a customized inspection program was defined. Initially, a fixed grid was used for the visual inspection, the extraction of concrete core samples for laboratory testing and non-destructive measurements, but in the end, the accessibility of the locations was decisive. The visual inspection and the measurement grid have the sole purpose of informing the owner about the current state of the different constructions phases and compiling a sufficiently large dataset to be able to make a well-founded assessment (taking into account influence of height, orientation etc.).

3.1. Visual inspection

The damage visible at the walls of the storage silos was different for the different construction phases. For the silo tower from construction phase 1A, the damage was mainly characterized by rebars without any concrete cover, spalling of the concrete due to corroding steel reinforcement and damage of previously executed patch repairs as shown in Fig. 3 (a) and (b). For the silo block and the silos from construction phase 1B and 2 the main damage patterns were horizontal and vertical cracks, damage of previously executed patch repairs, spalling of the concrete and surface weathering as shown respectively in Fig. 3 (c), (d) and (e), (f).



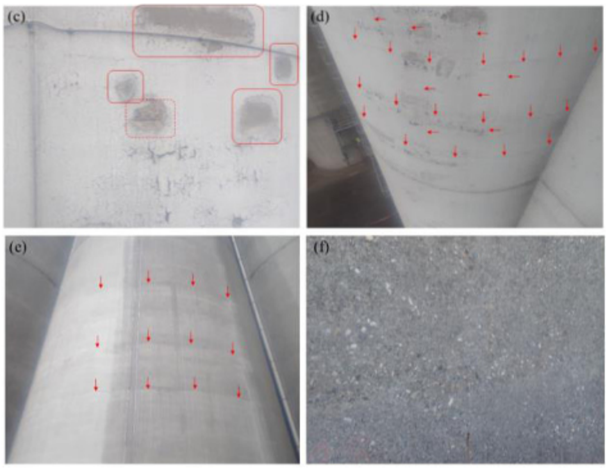


Fig. 3. Examples of damage patterns for the storage silos

3.2 Extraction of concrete cores and non-destructive measurements

An overview of the number of samples taken from the silos of each construction phase is given in Table 1. The samples were taken as much as possible on the different sides of the construction, taking into account the wind directions and accessibility.

Table 1. Concrete storage silos – Number of samples.

Construction phase	Cores	Dust drilling
1A	5	4
1B	4	3
2	5	4

The concrete cores with a diameter of 50 mm were used for the determination of the carbonation depth and chloride content in the concrete (see below). The carbonation depth based on the dust of drilling was determined in-situ by spraying phenolphthalein on a sample of freshly drilled concrete powder. A part of the extracted powder was also examined in the lab to determine the chloride content. At each location where the samples were taken, the concrete cover was also measured locally by an electromagnetic scanner. The results of the mean cover depth of the vertical and horizontal reinforcement in the walls are shown in Table 2.

Table 2. Concrete storage silos - Concrete cover depth of reinforcement.

Construction phase	Reinforcement	Cover depth (avg ± st.dev.)
1A	Vertical Horizontal	35 ± 13 mm 28 ± 15 mm
1B	Vertical Horizontal	50 ± 15 mm 23 ± 8 mm
2	Vertical Horizontal	61 ± 17 mm 45 ± 9 mm

3.3 Laboratory testing

3.3.1 Carbonation depth

The determination of the carbonation depth on the concrete cores was determined according to the standard EN 14630:2007 [1]. The mean carbonation depth for the different construction phases, silo tower (1A), silo block (1B) of phase 1 and the silos of phase 2 is shown in Table 3.

Table 3. Carbonation depth based on cores and dust drilling

Construction phase	Carbonation depth (mm)			
	Min.	Max.	Avg.	St.dev.
1A	0	28	10	11
1B	0	80	21	22
2	0	90	23	28

When comparing these results of the mean carbonation depths to the mean cover depth, the risk for carbonation induced reinforcement corrosion in the current state was small but possible due to the large distribution in the results for the silo tower from phase 1 and the silos from phase 2. For the silo block of phase 1 the risk was higher. The probability (risk) of corrosion initiation of the reinforcement is further explained in part 3.4 and part 4.

3.3.2 Chloride content

At locations where a core was extracted from the structure, the chloride content of the concrete was determined by measuring the (total) chloride concentration at 3 different depths: at the surface, at the depth of the reinforcement and in the mass of the concrete (i.e. beyond the first layer of reinforcement). The determination based on the concrete drilling dust was only performed at the depth of the reinforcement, on the one hand because there was no logical external source of chlorides in the vicinity of the structure and on the other hand to save time and money. For the 3 structures (phase 1A, 1B and 2) a summary of the measured mean chloride contents at the corresponding depths is given in Fig. 4.

The results for the constructions of phase 1 show that the measured mean chloride contents were higher than 0.44 % by mass of cement at every depth. The range from 0.44 % to 1.13 % by mass of cement indicates a high risk for chloride induced reinforcement corrosion. With the exception of the peak at the depth of 30 mm, the chloride curves were fairly flat, which means that the presence of mixed-in chlorides during preparation of the concrete was very likely.

The chloride content measured in construction phase 2 was significantly lower than in phase 1. A maximum content of 0.23 % by mass of cement and a minimum of 0.12 % by mass of cement was measured. The risk for chloride induced reinforcement corrosion at the time of survey was negligibly small for this phase based on a critical chloride concentration of 0.4 % by mass of cement [2, 3].

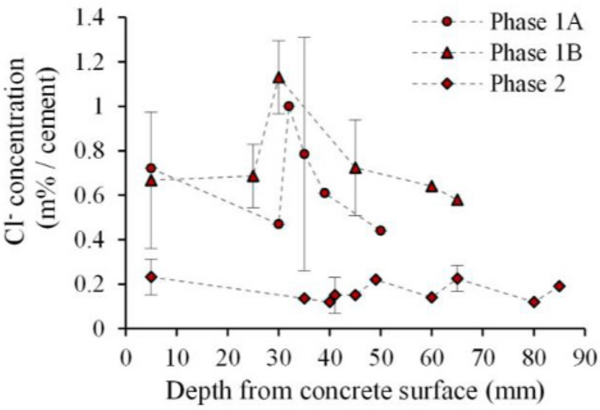


Fig. 4. Overview of the measured chloride contents

3.4 Cause of damage

The investigation has shown that the main cause of the damage for both construction phases was corrosion of the reinforcement. The phenomenon of reinforcement corrosion can occur with the simultaneous presence of the following three elements:

- Depassivation of the reinforcement,
- Moisture,
- Oxygen.

Depassivation of the reinforcement can take place by the influence of carbonation or by the presence of a sufficiently high content of chlorides or by a combination of both.

Depassivation of the reinforcement steel corresponds to the end of the initiation phase and is the moment when corrosion propagation can start. This is shown in Fig. 5. The probability of depassivation is the probability that phase 1 will be reached. The damage as a result of corrosion is visually perceptible from phase 2, i.e. the formation of cracks followed by phase 3, spalling of the concrete. Although the principle regarding the probability of depassivation and the general course is similar, the time span between the phases for corrosion initiated by carbonation and by chlorides differs. Visual damage will occur relatively more quickly with carbonation, while the loss of reinforcement section due to corrosion by chlorides may already be advanced before the damage is visually observed. Phases 3 and 4 of the chart will follow each other relatively more closely in case of corrosion initiated by chlorides in comparison with corrosion initiated be carbonation.

For both construction phases, the cause of the damage was corrosion of the reinforcement. However, the damage mechanism that initiated the corrosion was not the same. In case of construction phase 1 the corrosion of the reinforcement was induced by a combination of chloride ingress and carbonation. Regarding construction phase 2, the chloride content in the concrete was low and the only possible cause of the observed reinforcement corrosion was depassivation by carbonation.

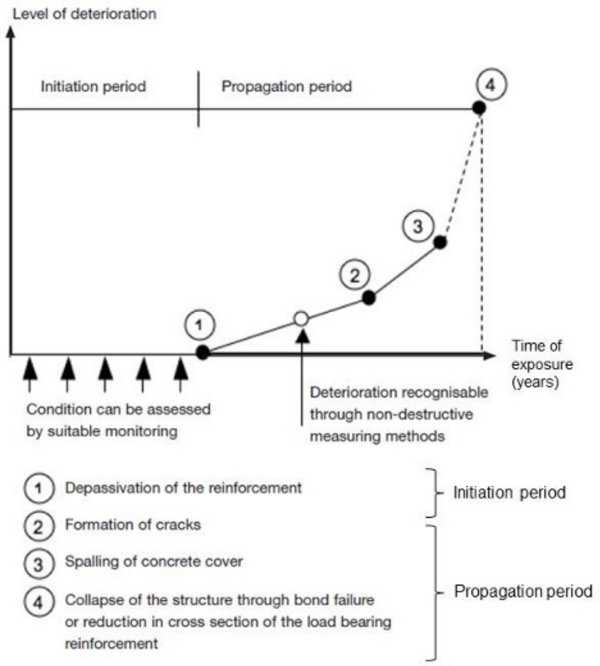


Fig. 5. Deterioration processs of reinforcement corrosion by Tuutti [4]

4 Assessment

Knowing the cause of the damage, the owner also needs to know which construction has a higher or lower priority for repair, based on the severity of the damage.

In order to decide on repair priority, the data analysis tool DIMCoSt (short for Durability Information Model for Concrete Structures) was developed and is used to determine both the probability (risk) of corrosion initiation of the reinforcement in the current state (P_f) and the remaining Service Life (SL_r) in terms of depassivation.

4.1 Risk of depassivation and remaining service life

The analysis provides an indication of the risk of development of both visible and not (yet) visible damage (latent damage) as explained in Fig. 5. In this way a more detailed and nuanced picture is obtained.

The distributions and the estimated evolution of the carbonation depth and chloride content in time are taken into account for the different construction phases. The models used in the DIMCoSt application, both for carbonation and for chloride induced corrosion are essentially based on the fib bulletin 34 [5]. A P_f of 30 % is used as the acceptance criterion as the results are substantiated by sampling [6].

Based on the applied acceptance criterion, the expected service life of the structure is estimated. The difference between the expected service life and the age of the construction at the time of investigation, provides an indication of the remaining service life (SL_r). If the applied acceptance criterion has already been exceeded at the time of the investigation, the expected service life is less than the age of the construction, the remaining service life is not shown as a negative value, but as N/A.

The results regarding the probabilities of depassivation and the remaining service life are shown in Table 4.

Table 4. Data analysis – Average probabilities of depassivation of the reinforcement.

Construction phase	$P_{f,CO2}$ [%]	$P_{f,Cl}$ [%]	SL_r [years]
1A	16	91	N/A
1B	34	100	N/A
2	12	0	> 100

As can be concluded from Table 4, the risk for reinforcement corrosion (mainly chloride induced) was very high for the concrete structures from phase 1 (1A and 1B). Therefore, their remaining service life was equal to 0. For the construction of phase 2, the residual life was still more than 100 years.

It is therefore clear that an urgent repair for phase 1 was needed. To get a better idea of the costs associated with a possible repair, the various repair options should first be compared.

5 Sustainable repair

The concept for the different construction phases of the storage silos will consist of a combination of different repair options. The most suitable repair option should be chosen depending on the cause of the damage, the intended remaining service life, the intended maintenance-free period and the cost of the repair. These are some decision-making criteria that depend on the long term vision and strategy of the building owner/manager. In order to be able to make a substantiated decision, an estimation of the cost of the required concrete repair can be useful. Next, the different repair options are compared in function of the estimated quantities.

5.1 Repair options

Depending on the intended remaining life extensions (i.e. the estimated maintenance-free period after intervention) and the different technical options, mainly 4 general repair strategies are proposed:

- Strategy 1: Local traditional repair + local sacrificial anodes,
- Strategy 2: Local traditional repair + local sacrificial anodes + Coating,
- Strategy 3: Local traditional repair + full surface galvanic cathodic protection (GACP) system.
- Strategy 4: Local repair + full surface impressed current cathodic protection (ICCP) system.

5.1.1 Strategy 1

A local, more traditional, concrete repair consists of repairing only the currently visible damaged parts. Additional sacrificial anodes are used to avoid ring anode effects. It should be noted that the passivity of the reinforcement in the areas without visible damage is not restored in this strategy. The development of reinforcement corrosion is possible in these areas and can result in new areas with visible damage in 5 to 10 years. Another important remark is the investment cost which increases after each intervention. The maintenance-free period of this strategy is estimated at approximately 5 to 10 years.

5.1.2 Strategy 2

In addition to Strategy 1, a watertight vapor permeable coating is applied (cf. EN 1504-2) [7] on the entire concrete surface to prevent further penetration of CO_2 and reduce the moisture content in the concrete. An additional note to this strategy: coatings need a periodic maintenance after about 15 years. Monitoring can indicate whether the coating is still sound. The maintenance-free period of this second strategy is assumed as approximately 15 years.

5.1.3 Strategy 3

Strategy 3 consists of repairing only the currently visible damaged parts of the concrete structure by a local concrete repair. Supplementary a galvanic cathodic protection (GCP) system is provided on the entire concrete surface. The expected service life of a galvanic system depends on the corrosion state of the reinforcement. The system performance decreases with increasing reinforcement corrosion. The system can be monitored but cannot be adjusted. The maintenance-free period of this third strategy is approximately 20 years.

5.1.4 Strategy 4

Strategy 4 also consists of a local repair, just like strategy 3, but additionally an active CP system with impressed current (ICCP) is applied on the entire surface of the structure to stop the corrosion process. By installing an ICCP system, adjustments and monitoring are possible. However, this is offset by a higher initial cost. The maintenance-free period of this fourth strategy is approximately 30 to 50 years, depending on the applied system.

5.2 A well-founded choice of repair strategy

5.2.1 Construction phases with a probability of depassivation < 30 %

In theory, the structures from construction phase 2 still have a long time before the predetermined acceptance criterion of $P_f = 30$ % is exceeded. This can be derived from the remaining service life analysis outcome as shown in Table 4. However, limited damage to this construction phase is already visible. The damaged areas are possibly locations where the concrete cover is locally lower than the carbonation depth due to "impurities" in the concrete such as honeycombs or cracks. For this phase the current situation can be summarized that damage phenomena have occurred, but by applying preventive measures the damage can be kept under control. The most appropriate repair strategy for this construction phase is therefore strategy 1 or 2.

5.2.2 Construction phases with a P_f > 30 %

For the constructions of phase 1 (1A and 1B) the primary cause for corrosion of the reinforcement is initiated by chlorides. Chloride contents higher than 1.00 % by mass of cement are measured. The various repair strategies as formulated in 5.1 are still the most appropriated repair methods. However, each strategy has an associated maintenance-free period and a certain chance of possible additional damage.

5.2.3 Estimation of the required concrete repair

For the investigated concrete structures, the general probability of depassivation of the reinforcement due to carbonation ($P_{f,CO2}$) and chlorides ($P_{f,Cl}$) has been determined. The average probability of depassivation ($P_{f,A}$) based on both damage mechanisms is taken for further calculations. Based on the probability of depassivation, the total area to be repaired (A_{rep}) is estimated by multiplying the total exposed concrete surface of construction phase (A_{tot}) with the average probability of depassivation ($P_{f,A}$). The repair volume (V_{rep}) is subsequently determined by multiplying the surface to be repaired (A_{rep}) by the average concrete cover (c_A), which is increased by 25 mm to take into account that the removal of the concrete must take place past the reinforcement. The results of these calculations are shown in Table 5.

Table 5. Estimation of the area of concrete repair

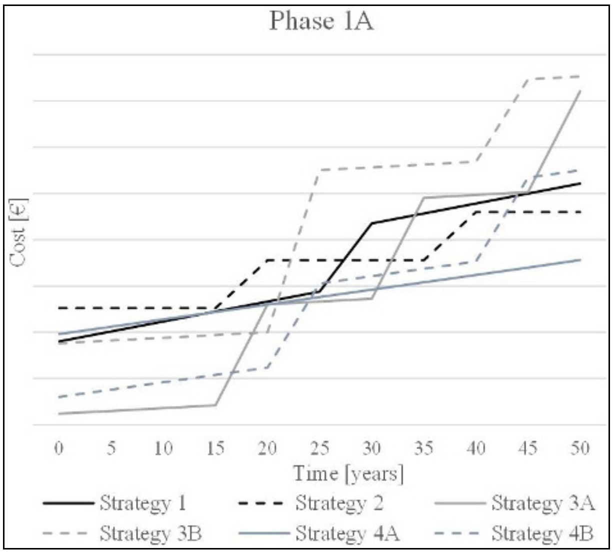
Construction phase	A_{tot} [m²]	$P_{f,A}$ [%]	$A_{rep.}$ [m²]
1A	1180	54	635
1B	5510	67	3690
2	12100	6	725

Table 6. Estimation of the volume of concrete repair

Construction phase	c_A + 0.025 [m]	V_{rep} [m³]	V_{rep} [dm³]
1A	0.055	35	35000
1B	0.050	185	185000
2	0.070	50	50000

5.2.4 Cost estimation

In order to present a clear financial view to site owners, the estimated cost of the various repair strategies is plotted as a function of time, considering the maintenance-free period. Strategy 1 in which a traditional concrete repair is performed and strategy 2 additionally with a coating need little explanation as to the method of execution. In the cases of strategy 3 and strategy 4 different execution options are possible. Using a zinc foil (3A) or zinc mesh (3B) are two options in case of strategy 3 and are further elaborated. In case of strategy 4 also two execution methods have been selected, titanium mesh in mortar overlay (4A) and application of a conductive coating (4B). An estimation of the costs is made in
Fig. **6** for these 4 strategies with variations in execution and taking into account the maintenance-free period. All charts cover a time period of 50 years.



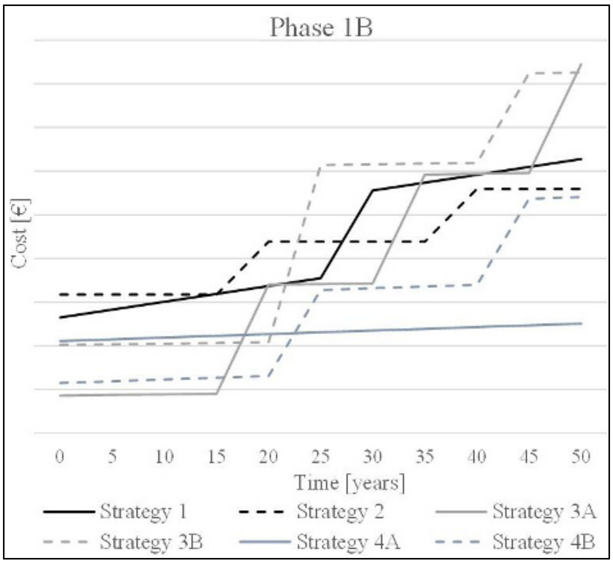


Fig. 6. Cost estimation per strategy in time

Certain assumptions were made to obtain the graphs in
Fig. **6**. In the case of strategy 1, it is assumed that additional damage occurs every 5 years and that an additional 5 % of the initially unrepaired concrete surface needs to be repaired. After 25 years, 25 % of the repaired concrete must be repaired again. Strategy 1 also implies sacrificial anodes in the edge of the repair. In strategy 2, the coating must be restored after 15 years. This involves removing and reapplying the coating. In the case of the zinc foil (strategy 3A) it is assumed that after 15 years the system must be reapplied. The zinc mesh (strategy 3B) will need to be replaced after 20 years, as will the activation mortar overlay. After 40 years the cost will only be half as much as the zinc will react slower. For the ICCP systems (strategy 4), in addition to the annual maintenance, the conductive coating and the top coat (strategy 4B) will have to be replaced after 20 years. Strategy 4A, titanium mesh in mortar overlay (4A), requires only an annual maintenance for the first 50 years.

5.2.5. Choice of repair strategy

Based on the graphs in
Fig. **6** the site owners preferred strategy 4A (titanium mesh in mortar) or strategy 4B (conductive coating). The major advantage of strategy 4A is the minimum of maintenance this system requires. In case of strategy 4B the economic advantage is a lower initial cost and the costs are more spread over time. Indeed, both systems have their pros and cons. Eventually, strategy 4 (full surface ICCP) was chosen.

6. Additional research

Although the most suitable repair technique is defined (based on damage cause and aimed service life), the next step is to determine whether the application of CP is technically possible and feasible for phase 1.

In the context of the additional concrete research, the following investigations were carried out:

- half-cell potential measurements,
- reinforcement type and configuration
- electrical continuity of the reinforcement and
- the condition of the reinforcement.

In case of the structures of phase 1A and 1B, 3 and 12 additional zones were investigated, respectively.

6.1. Potential mapping

Half-cell potential measurements were performed by means of a $Cu/CuSO_4$ reference electrode [8] in a grid with dimensions of 0.5 m x 0.5 m. The results are visualized in a potential mapping colour plot in Fig. 7 for phase 1A and in Fig. 8 for phase 1B.

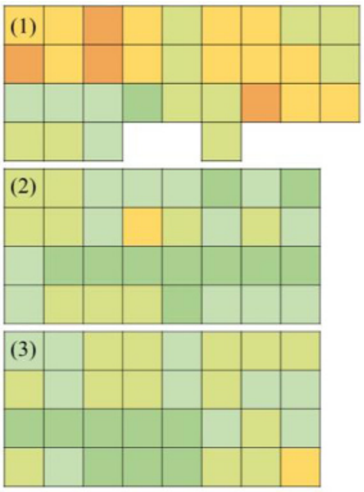


Fig. 7. Half-cell potential mapping phase 1A

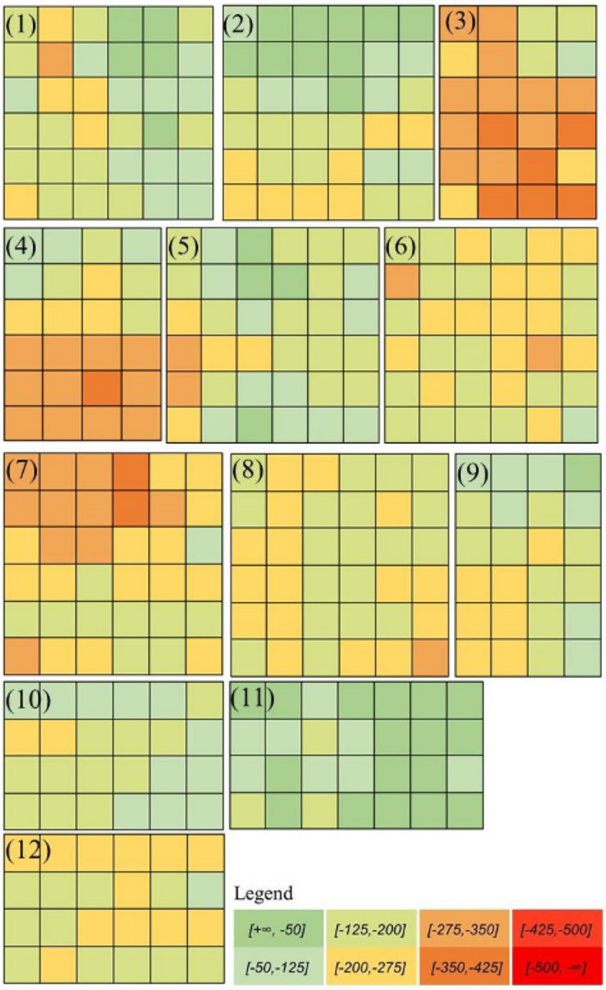


Fig. 8. Half-cell potential mapping phase 1B

The half-cell potential measurement results indicate an increased possibility of active corrosion for most of the zones.

6.2. Electrical continuity

In order to verify the feasibility of the installation of a CP system, the electrical continuity of the steel reinforcement was checked for both structures. The electrical resistance for phase 1A ranged from 0.67 Ω to 0.80 Ω, for phase 1B from 0.56 Ω to 1.57 Ω. The limit for having electrical continuity according the norm EN ISO 12696 [9] is 1 Ohm. Only a slight exceedance is observed for phase 1B, but the reinforcement can still be regarded as continuous. Both execution phases have electrical continuity of the reinforcement.

6.3. Condition of the reinforcement

Based on the half-cell potential measurements, the condition of the reinforcement was checked by locally exposing steel reinforcement bars. This location is always chosen at the point where the most negative steel potential is measured and in a zone where no rust or cracks are present at the surface. Additionally, the section of the reinforcing bars has been measured. In case of phase 1A, smooth steel has been observed with a nominal diameter of 10 mm and a spacing of 250 mm between the horizontal bars and 250-300 mm between the vertical bars. Phase 1B has ribbed reinforcing bars with a diameter of 10 mm vertical and a diameter of 14 mm horizontal. The spacing is respectively 300 mm and 200 mm. An example of the state of the reinforcement for each phase is shown in Fig. 9.

Fig. 9. State of the reinforcement of phase 1A (left) and phase 1B (right)

6.4. Technical feasibility

The application of ICCP is the most appropriate technique for phase 1 (both silo tower and silo block) to prevent the degradation of the silos in long term. For almost all investigated locations the reinforcement has progressed into a moderate to severe degree of local or global corrosion.

The application of ICCP is considered to be the most efficient. In case of execution the following technical aspects are relevant.

- The existing reinforcement is electrically continuous for all examined reinforcement zones.
- Active corrosion of the reinforcing steel was found at all locations. This mainly concerns the horizontal bars, which is the main reinforcement, but with the lowest concrete cover.
- The reinforcement amounts are uniform for each phase (1A and 1B) and are relatively low. A density of approximately 0.25 m^2steel/m^2concrete and 0.30 m^2steel/m^2concrete must be taken into account in the design of the ICCP system.

7. Conclusions

Concrete research must be carried out in an adequate and objective manner so the necessary parameters in making a good choice of repair, can be determined. The high impact on budgets and possible decommissioning of the structures are the main reasons why this is an important step in the maintenance of the structures.

DIMCOST offers a framework to store all the data, to compare it stochastically and thus to get an objective idea of the actual situation based on failure probabilities and service life calculations. The information on failure probabilities makes it possible to estimate the amount of necessary concrete repair and the associated costs. The service life calculations give a numerical priority to the different construction phases.

In this way building managers and site owners now have all the information they need to make an informed decision and plan for maintenance. For this case it is known that phase 1 requires an urgent intervention and a thorough repair strategy such as ICCP is preferred. Phase 2 is still in good condition, so maintenance does not have to be planned immediately. A periodic visual inspection of structures where no immediate action is taken, can provide a safe working environment and a quick observation when damage is increasing.

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