On-site monitoring for studying the effects of repair measures on corroding steel in chloride contaminated concrete

Fritz BINDER [1], Franz PRUCKNER [2]

[1] Asfinag Service GmbH, Vienna, Austria

[2] ZT-Büro Pruckner, Euratsfeld, Austria

Contact e-mail: fritz.binder@asfinag.at

ABSTRACT: Among the approximately 5,000 bridges, which are maintained by the road authority ASFiNAG, a total number of 1,500 objects are ramp bridges or overpass structures. Particularly their substructures are subject to increased chloride contamination. Sooner or later, this leads to a reduction in durability and affects the service life in the long term. Therefore, repair measures are necessary. In order to guarantee an optimal structural condition with the available budgets, the costs and effects of repair measures must be known.

This paper is dealing with data collected from an on-site monitoring and also with data from nearby weather stations. For several years, monitoring has been carried out on supports of a bridge exposed to a high chloride load. The sensor measuring principles used to monitor the corrosion behaviour and the sensor design will be explained. First measurement results of this method will be shown, and the seasonal course can be easily traced on the basis of the evaluated data. With this data it is possible to show many interrelations and dependencies to the corrosion rate.

The observations after the repair measures are of particular interest. Now a direct statement can be made about the effectiveness of measures and a performance index can be proposed.

Based on these in-situ measurement results, important statements can be made for future repairs measures.

1 GENERAL INTRODUCTION IN CORROSION INITIATING

Reinforced concrete structures can be found almost anywhere in many developed countries in the terms of roads, bridges, airports, shopping centres, sculptures and wharfs. Especially in the vicinity of bridges the danger of corrosion chlorides induced due to de-icing salt is very high. Reinforced concrete is a composite material, reinforced with steel bars cast en bloc with cement, aggregate and water. The result is a porous matrix of relatively inert aggregates, bound together by a cementitious network. In general, the steel bars in reinforced concrete structures are sufficiently protected from corrosion by a passive layer on the steel bar surface due to the highly alkaline solution in the concrete, Schueremans et al. (20017). Due to environmental influences this passive layer on the steel is destroyed over time. Foundations are typically affected by the chloride-induced corrosion where the chlorides are originating from the de-icing salt which comes within the spray water, Eichinger-Vill et al. (2010). When the front of chlorides exceeds a critical concentration at the reinforcement depth, the steel bars locally loose their passive layer, Breit et al. (2011). This area now offers points of attack for corrosion, Glass et al. (1997), Alonso et al. (2000). The result is a section loss of the reinforcing steel and, consequently, a reduction of load bearing capacity of the steel and the reinforced concrete as a whole. The metal dissolution by electrochemical corrosion is increased by a high electrolytic conductivity of the concrete covering the reinforcement. A characteristic feature of this process is the flow of galvanic currents. In order to identify when a structure is affected by corrosion an accurate and reliable monitoring regime needs to be available.

2 MONITORING REINFORCEMENT CORROSION IN CONCRETE

Monitoring allows the corrosion status of reinforced concrete to be determined and also to forecast its propagation. Nowadays, many techniques for assessment of steel corrosion in concrete are available. The simplest techniques which includes measurements of the electrochemical rebar potential, resistivity of concrete, content of chloride ions and depth of carbonation are only indirect methods to assess the corrosion. All these methods do not measure the rate of corrosion directly. However, there are also techniques available for measuring the rate of steel corrosion directly. Out of the many electrochemical techniques proposed for monitoring the corrosion rate of concrete reinforcement in practice, the most popular are the linear polarization resistance (LPR) and the use of galvanostatic pulse or transient analysis methods, Andrade et al. (2004). In this paper the LPR method has been chosen to gauge the reinforcement corrosion in practice.

LPR monitoring is an effective electrochemical method of measuring corrosion. Monitoring the relationship between electrochemical potential and current generated between electrically charged electrodes in a process stream allows the calculation of the corrosion rate. The LPR method is most effective in aqueous solutions and has proven to be a rapid response technique.

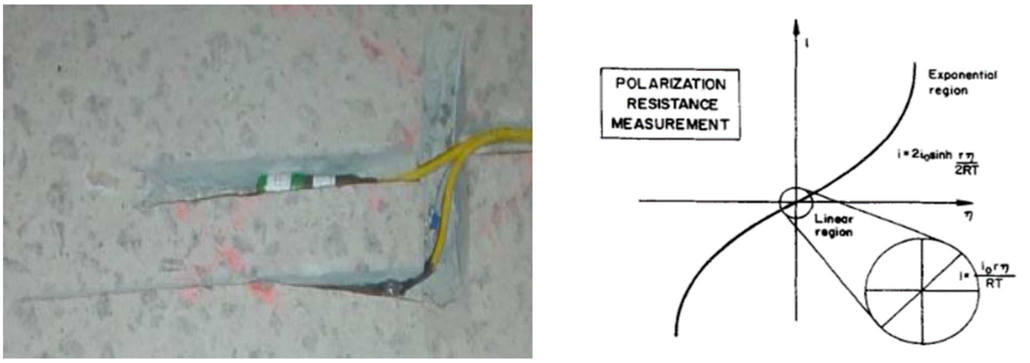

Figure 1. Embedding reference and counter electrode in concrete as well as establishing a reinforcement contact used fort the LPR-method (left) and linear plot of the polarization curve (right) Andrade et al.(1996).

A three-electrode probe is established in the concrete cover, with the electrodes being electrically isolated from each other and the reinforcement. A small section of the steel rebar (working electrode, WE) is polarized ±20 mV (ΔE) from the open circuit potential (OCP) using the counter electrode (CE) and the reference electrode (RE) and the resulting polarization current (ΔI) is measured. A polarization curve is obtained where a segment of this curve is linear. The slope of this linear curve segment is called the polarization resistance R_P, which is the ratio of the applied potential (ΔE) and the resulting current level (ΔI):

$$R_P = \frac{\Delta E}{\Delta I} \tag{1}$$

If the reinforcement section (WE) is corroding at a high rate with the metal ions passing easily into solution, the defined polarization applied to the reinforcement section will produce a high current, hence a low polarization resistance. This corresponds to a high corrosion rate. R_P is reversely proportional to the corrosion current, I_{corr} as first described by Stern et al (1957) by means of a constant, which is denoted as B. For corroding steel in concrete, a value of B=26 mV is used, whereas B=52 mV for passive steel (Andrade et al. 1978).

The corrosion current density is then calculated as:

$$I_{corr} = \frac{B}{R_P} \tag{2}$$

where I_{corr} is the corrosion current I_{corr} needs to be converted into a corrosion current density i_{corr} which is directly related to the corrosion CR rate by Faradays law:

$$CR \left[\frac{\mu m}{a}\right] = 1.16 \cdot i_{corr} \left[\frac{mA}{m^2}\right] \tag{3}$$

Integrating the measured corrosion rate over time should allow to estimate the corrosion rate over the entire measurement period for the particular location on the reinforcement.
1 mA/m² corresponds to a mass loss of approximately 9 g/m²a and a penetration rate of about 1.6 µm/a. In other words, a corrosion current density of 1 mA/m² corresponds to 11.6 µm steel section loss per year. On structures exposed to the atmosphere, the corrosion rate can range from a negligible quantity below 2 µm/a, to very high values above 100 µm/a. The range of the corrosion rate strongly depends on the carbonation or chloride contamination of the concrete as well as on the moisture content of the concrete. Ranges of i_{corr} regarding the corrosion risk is discussed in Andrade et al. (1996). Based on the time to reach a certain cross-section loss, four corrosion levels are proposed:

1. Negligible ($i_{corr} < 1$ mA/m²),
2. Low ($1 < i_{corr} < 5$ mA/m²),
3. Moderate ($5 < i_{corr} < 10$ mA/m²) and
4. High ($i_{corr} > 10$ mA/m²).

3 THE STRUCTURE

For an in situ determination of the corrosion of the reinforcement, the environmental impact has also been monitored. Such a monitoring system was installed on columns in different heights on an overpass located in the Austrian motorway network of ASFiNAG. The overpass which was planned in 1980 has a length of about 100 m and shows a tunnel-like characteristic. The beams are supported by the abutment and the columns are situated between the direction lanes of the motorway. In 1994, concrete spalling was patched and the 21 columns were coated with an epoxy-based coating up to a height of 2.0 m. The substructure, especially the columns, showed severe damage to the concrete structure at the time of investigation in 2010, such as spalling and cracks over 0.3 mm wide as well as traces of rust in the contact and splash zones. In the course of the renovation in 2014, the substructure of the property was comprehensively renovated. Different repair systems were used for the columns. In the course of this project, some additional sensors were installed to better assess the effect of the individual repair methods.

4 RESULTS AND DISCUSSION

Figures 2 and 3 show the course of the corrosion potential in the spray and splash zone overtime. The seasonal variation can be clearly observed. Figure 2 shows the measured values of a column whose concrete surface was sealed with a coating after intervention wheras figure 3 shows the results from a hydrophobic treatment. In the figures, the time of intervention is clearly recognizable. The gray shadowed area in the figures marks the time of intervention additionally. Since this time a significant reduction of the corrosion potential occur. The marked limits are taken from ASTM C876-15 (2015) and indicate an improvement from below -438 mV, the level with high corrosion possibility, to approximately -350 mV. This can also be seen in the box plots of the two measured exposure zones (see right section of the figures). The variation of the values has remained approximately the same.

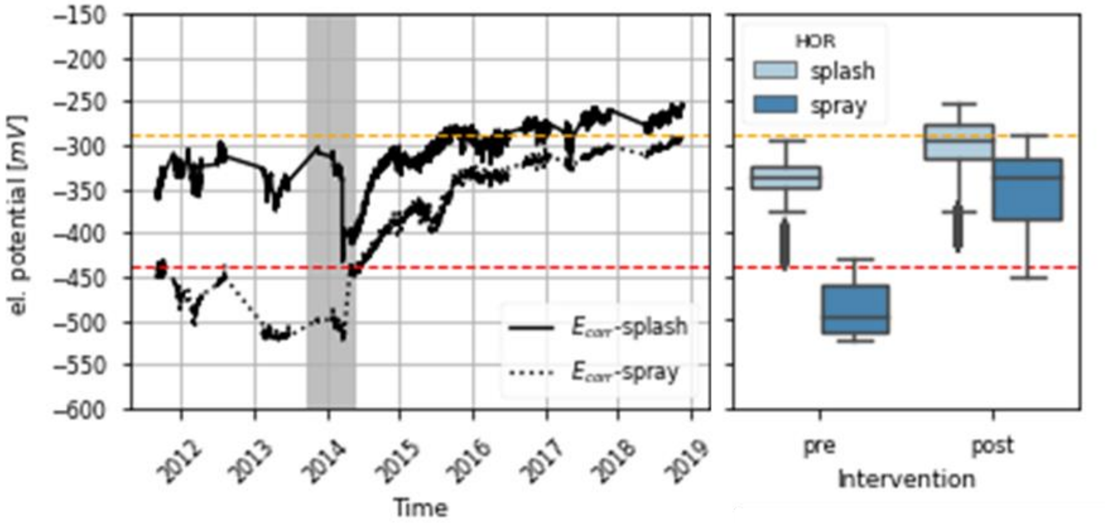

Figure 2: Development of the corrosion potential (left) at different heights and statistical description thereof (right) for column #03 repaired with a coating.

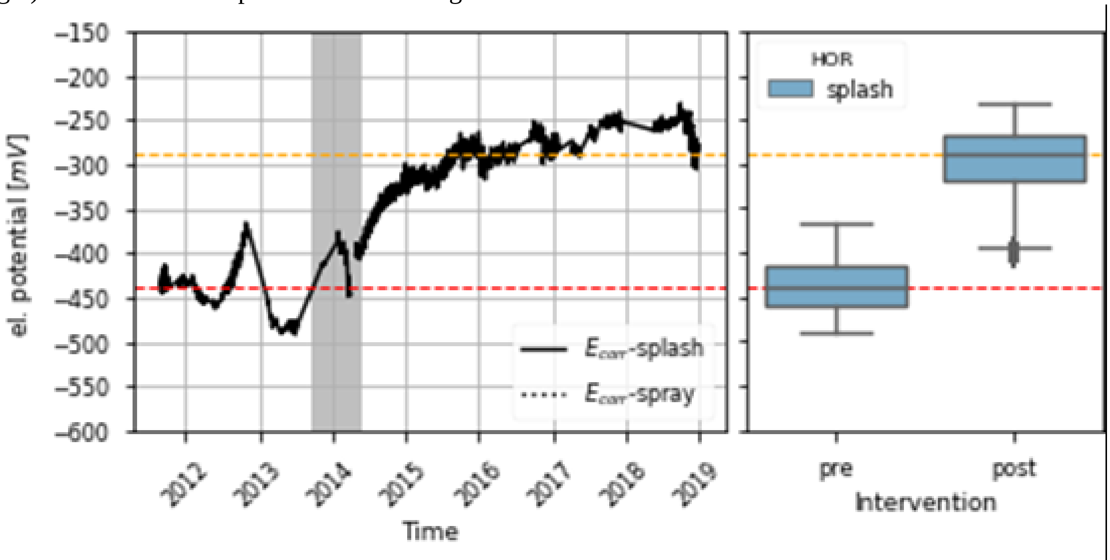

Figure 3: Development of the corrosion potential (left) at different heights and statistical description thereof (right) for column #17 repaired with hydrophobic treatment.

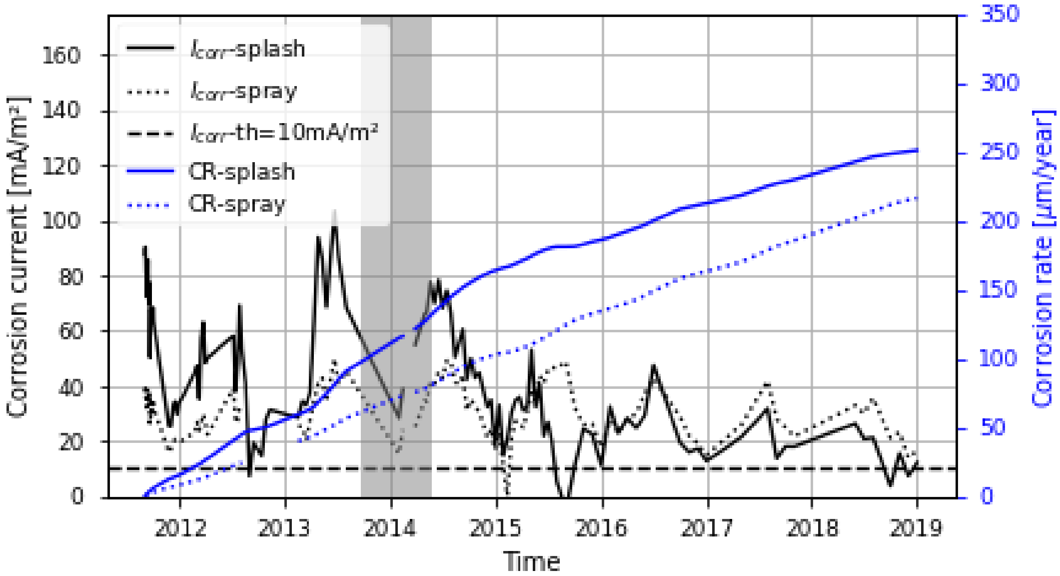

Figure 4: Development of the measured corrosion current density and the derived cumulative corrosion rate over the time of measurement for two exposure zones of column #03 repaired with coating

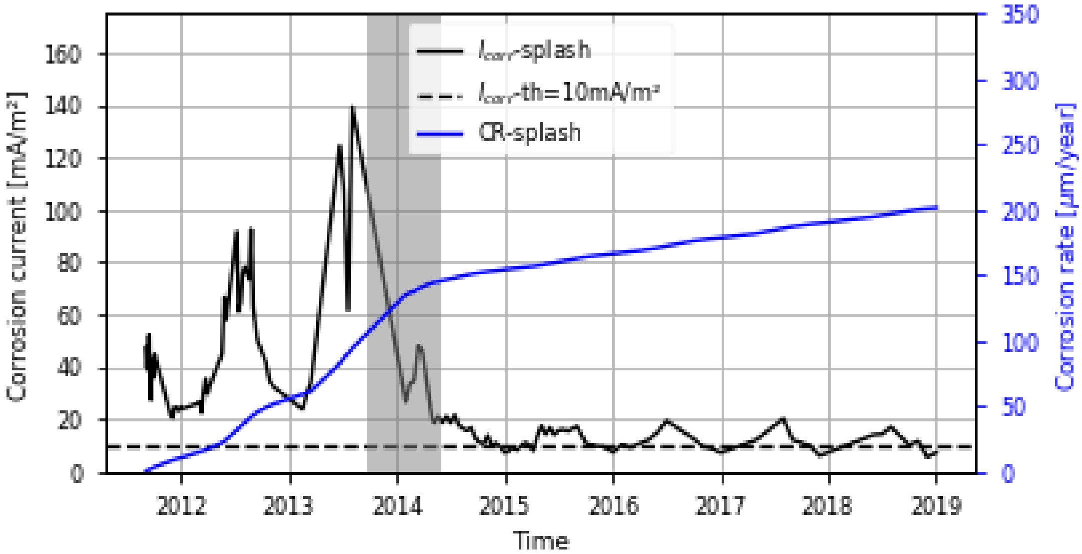

Figure 5: Development of the measured corrosion current density and the derived cumulative corrosion rate over the time of measurement for column #17 repaired with hydrophobic treatment.

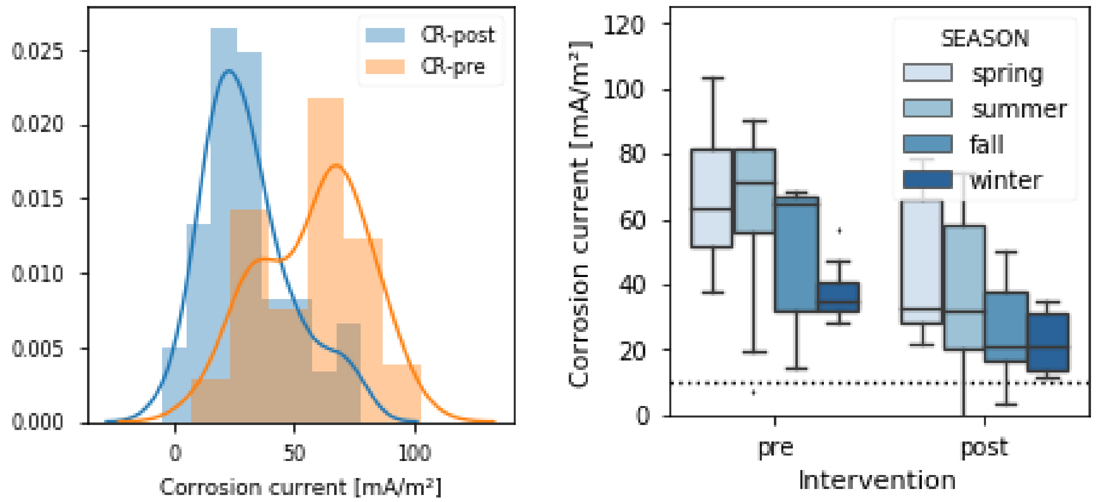

Figure 6: Density plot (left) and boxplot categorized into seasons (right) before (pre) and after (post) repair treatment with coating of column #03.

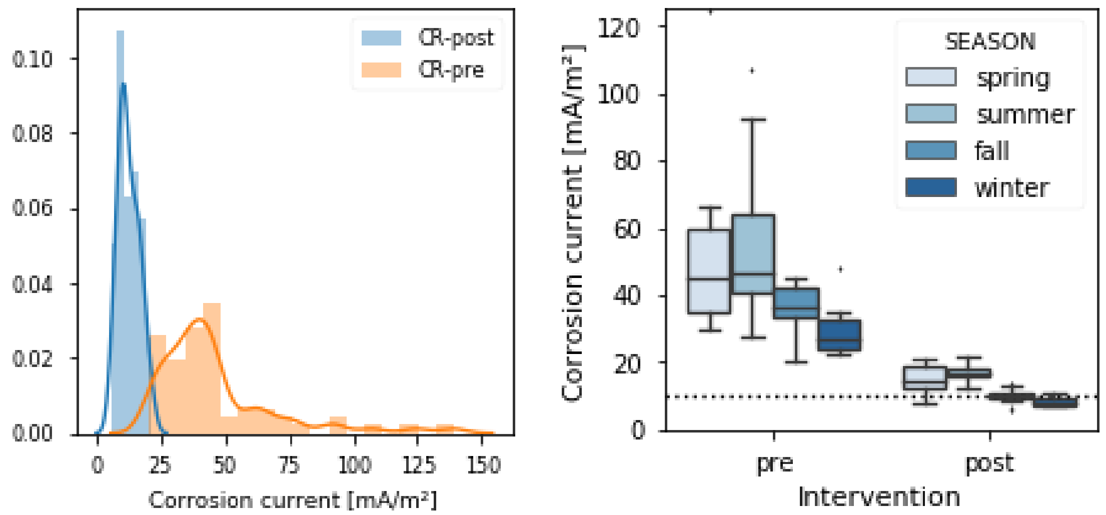

Figure 7: Density plot (left) and boxplot categorized into seasons (right) before (pre) and after (post) hydrophobic treatment of column #17.

During the observation period the measured corrosion current density was always high, regardless of the maintenance technique. Only during the autumn and winter season the corrosion current density measured at the column with hydrophobic treatment has been moderate ($i_{corr} < 10$ mA/m²). But it has to be kept in mind that the load of the structure with chlorides was very high and visible deterioration due to corrosion from the beginning of the observation period. However, both treatments led to a significant reduction of the corrosion rate. This is clearly visualized in figures 6 and 7. After the interventions (repair measures) have taken place, the variances of the observed corrosion rates became smaller. This effect is clearly shown in figure 7. The treatments obviously have a corrosion rate controlling influence which is especially shown by the statistical analyses of the measurements carried out in the columns with both hydrophobic treatment and coating application. An additional observation was that the corrosion current density is increasing and decreasing with the temperature.

5 CONCLUSION

The paper deals with the corrosion conditions of a real concrete structure before and after repair. While the corrosion potential is easily and directly measured with reference electrodes the determination of the corrosion current density is more sophisticated and prone to measurement errors. However, installation of sensors for determining the corrosion rate offers an adequate possibility to measure the corrosion activity of repaired components. Even more, the measurement technique allows to make qualitative statements about the performance of maintenance measures.

Both repair measures, coating application and hydrophobic treatment were reducing the corrosion rate significantly at the investigated structure. The development of the corrosion potential also indicated ennoblement of the reinforcement. The ennoblement was however more pronounced for the column with hydrophobic treatment than for the coated column.

The corrosion rate fluctuates considerable due to the environmental impacts, like rain and temperature. In summer the corrosion current density can be more than twice as high than in winter. However, this apparently high scatter disappears when the accumulated corrosion rate is calculated. The observations indicates that the corrosion rate seems to develop differently after the intervention, independent of the previously measured corrosion current.

It is difficult to establish a performance index based only on the corrosion rate measurements. Several factors affect the course of the corrosion rate. There are on the one hand, the environmental influences, but also the initially measured corrosion intensity has influence. Further research has to be done to establish a solid performance index that can objectively assess the effectiveness of repair measures. Therefore several factors have to be taken into account, like the electrical resistance or relative humidity.

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