Corrosion of Reinforcement in Relation to Presence of Defects at the Interface between Steel and Concrete

T. A. Söylev¹ and R. François²

Abstract: In this study, steel-concrete interface defects were analyzed in order to define their potential to induce corrosion. Various types of steel-concrete interface defects were classified into two main groups: macrodefects and microdefects. Gaps formed beneath horizontal reinforcement as a result of bleeding and settlement of fresh concrete were analyzed for macrodefects. Microdefects presented no signs that could be identified by visual inspection and resulted not only from controlled pull-out of the steel bar but also from bleeding and settlement (but without the production of macrodefects as found with gap formation). Apart from interface defects, cover concrete porosity was defined as an intrinsic defect. The effect of these defects on reinforcement corrosion was investigated. Macrodefects have a direct effect on corrosion, whereas microdefects have no significant effect on corrosion. Where the level of intrinsic defects was high, these had a greater effect on corrosion than interface macrodefects. The behaviors of conventional and self-compacting concrete (SCC) were compared. SCC was found to have better interface quality.

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CE Database subject headings: Interfaces; Reinforcement; Concrete; Steel; Compaction; Corrosion.

Introduction

In a chloride-containing environment, chloride levels must be higher than a critical value (known as the chloride threshold value) before active corrosion can occur. This threshold value is a function of the pH of the concrete. Research performed in a simulated solution where the pH is equivalent to that of concrete pore solutions shows, however, that steel covered by concrete is better protected (Page 1975; Page and Treadaway 1982; Lambert et al. 1991; Yonezawa et al. 1988; Suryavanshi et al. 1988). In a good quality concrete, the high degree of contact between steel and concrete provides physicochemical protection, which involves both a buffering effect (which counteracts the lowering in pH caused by the action of Ca(OH)₂ crystals deposited at the interface) as well as a physical barrier that blocks dissolution of iron. This barrier limits the movement of chloride and prevents oxygen from gaining access to the steel (Yonezawa et al. 1988). The formation of defects at the interface is therefore required for initiation of corrosion (Page 1975; Page and Treadaway 1982; Lambert et al. 1991; Yonezawa et al. 1988; Suryavanshi et al. 1988; Mohammed et al. 1999, 2002; Mohammed and Hamada 2001). These defects are generally caused by air bubbles and bleeding, but mechanical degradation of the interface has recently been implicated (Castel et al. 2000).

In the present research, the role of steel-concrete interface defects in reinforcement corrosion was studied. The types of defects considered were as follows:

- Defects that may arise as a result of inadequate compaction or similar commonly occurring practices. These are gross defects that are essentially the result of segregation, settlement, and bleeding of fresh concrete. These occur in the form of gaps beneath the reinforcement (relative to the concrete casting position).
- Defects that are formed as the result of shear loading of the reinforcement.

The nature of the transition zone at the steel-concrete interface affects both durability and bond strength. This zone contains hydration products that have been precipitated during the early stages of hydration (Page 1975; Page and Treadaway 1982; Lambert et al. 1991; Yonezawa et al. 1988; Suryavanshi et al. 1988; Page et al. 1978). This layer, comprised largely of portlandite, is relatively porous and heterogeneous when compared with the bulk concrete mix (Hamad and Itani 1998; Gjorv et al. 1990). The buffer effect that is a result of the dissolution of the portlandite crystals was found to be effective even for high w/c ratios (Suryavanshi et al. 1988), but the higher porosity of both the transition zone (Page et al. 1978) and the bulk paste results in increased mobility of chloride ions (Suryavanshi et al. 1988). Another effect of higher w/c ratio is the formation of gaps beneath horizontal bars, formed as a result of bleeding (Mohammed et al. 1999, 2002; Mohammed and Hamada 2001; Kosmatka 1994). This type of interface defect is a macro-defect which disrupts the integrity of the cement hydration-product layer that is deposited at the steel-concrete interface (Lambert et al. 1991).

Self-compacting concrete (SCC) has the advantage of producing a high-flow concrete without the need for an increased water/cement (w/c) ratio, and its use therefore avoids the undesirable effects of high w/c ratios. Special formulations of SCC that incorporate chemical and mineral admixtures will help to reduce bleeding and so improve the quality of the interface (Khayat 1998a,b, 1999; Petrov 1998). As well as reducing bleeding, most
commonly used mineral and chemical admixtures will directly affect the interface. The use of silica fume, for example, will produce an increased density as a result of the pozzolanic reaction (Hamad and Itani 1998; Gjørv et al. 1990), but this reaction may cause a diminution of the buffering effect (Nilsson et al. 1996; Glass et al. 2000; Page and Vennesland 1983; Arya and Xu 1995; Cao and Sirivivathanon 1991; Lorentz and French 1995). The very fine silica fume particles act as a filler and result in a less porous transition zone being produced (Gjørv et al. 1990). The use of superplasticizer results in better dispersion of cementitious materials at the interface and therefore increases interface homogeneity (Khayat 1999). Another advantage of the use of superplasticizer [or a high-range water reducer (HRWR)] is to reduce the water content. This keeps the w/c ratio constant and therefore reduces bleeding. When used at the same w/c ratio, it increases bleeding by resulting in a greater fluidity. The incorporation of viscosity-enhancing admixtures (VMAs), however, overcomes this problem, because it increases the cohesiveness of the concrete. This means that SCC becomes a high-stability concrete, because bleeding (already reduced due to the high fines cement + silica fume content) is eliminated (Khayat et al. 1997; Khayat 1998a,b, 1999; Petrov 1998).

**Experimental Program**

In the first part of the study, defects due to settlement and bleeding of fresh concrete were analyzed. The formation of gaps beneath the horizontal reinforcement was examined as a function of the casting position and as a function of the nature of the concrete involved. In the second part, defects due to pull-out loading were analyzed.

**Concrete Mixture Proportions, Casting, and Curing**

Five different concrete mixes were used in the first part of the study (the analysis of the effect of bleeding). These were referred to as C20, C40, SCC40, C50, and SCC50 (Table 1). The mixture name refers to its compressive strength class (20, 40, or 50 MPa), with the prefix C indicating that the concrete has been vibrated and the prefix SCC denoting self-compacting concrete. In the second part (studies on the effect of shear loading), C40 and SCC50 were used. Concrete members were removed from their molds on day 6 and cured at 100% relative humidity up until day 28. They were then sawed to produce 13 samples per member (column or beam), with the steel bar at the center of each sample.

<table>
<thead>
<tr>
<th>Table 1. Mixture Proportions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Materials (kg/m³)</td>
</tr>
<tr>
<td>CPA CEM I 52.5 PM ES CP2</td>
</tr>
<tr>
<td>CPA CEM I 52.5 CP2</td>
</tr>
<tr>
<td>Silica fume</td>
</tr>
<tr>
<td>Total water</td>
</tr>
<tr>
<td>Sand 3.15 R</td>
</tr>
<tr>
<td>Sand 0/4 C</td>
</tr>
<tr>
<td>Sand 0/4 R</td>
</tr>
<tr>
<td>Sand 0/5 R</td>
</tr>
<tr>
<td>Coarse aggregate 4/10 C</td>
</tr>
<tr>
<td>Coarse aggregate 4/10 R</td>
</tr>
<tr>
<td>Coarse aggregate 10/14 C</td>
</tr>
<tr>
<td>Coarse aggregate 5/15 R</td>
</tr>
<tr>
<td>Limestone filler</td>
</tr>
<tr>
<td>Viscocrete 2100 (plasticizer)</td>
</tr>
<tr>
<td>Viscocrete 3010 (VMA-HRWR)</td>
</tr>
<tr>
<td>Plastiment HP (plasticizer)</td>
</tr>
<tr>
<td>Sikatell 200 (VMA)</td>
</tr>
<tr>
<td>Glénum 27 (superplasticizer)</td>
</tr>
<tr>
<td>Water/cementitious materials</td>
</tr>
</tbody>
</table>

Note: SCC=self-compacting concrete; VMA=viscosity-enhancing admixture; and HRWR=high-range water reducer.

**Defects due to Shear Loading**

The same concrete members were used for this test, but they were placed horizontally, and different types of steel bar were used (deformed steel bars were used for SCC50). Video-microscopic examination of the 3-cm-high cores was carried out after pull-out loading. This is in contrast to the first part of the study, in which core sampling was undertaken before pull-out loading. These cores were also used for corrosion studies.

**Test Program**

Slump (for conventional concrete), slump flow (for SCC), density, and the amount of air trapped in the fresh concrete were measured. Compressive and tensile strengths (splitting test) at day 28 were measured using cylindrical samples (220 mm high by 110 mm diameter; see Tables 2 and 3).

The ultimate bond strength was calculated using

$$\tau = \frac{P}{\pi dl}$$

where $P$, $d$, and $l$ correspond to the applied load (measured by pull-out test), the bar diameter, and the anchored length, respectively. The bonded length of the steel bar was only 5 cm, with the rest of the steel bar being covered away from the loaded end in order to reduce confinement.

Shear load damage was created at different load levels by controlled pull-out of the reinforcing bars (round or deformed) as shown in Figs. 2 and 3. The damage points were identified on these pull-out force versus slip curves relative to the failure in bonding, both prior to it and after it. As a result, five samples that were damaged or loaded in different ways were used, as well as the control sample.

The steel-concrete interface was examined using a video-microscope at ×25 and ×175 magnification. The defect zones...
were quantified in terms of the length of steel that was not bonded to the concrete at the lower part of the bar as a result of bleeding (defect factor) (Fig. 4).

The concrete cores used to measure corrosion were dried completely at 50°C before the first immersion in order to enhance the capillary suction effect. The steel areas that were not embedded in concrete, as well as the upper and lower surfaces of the concrete cores, were covered with an epoxy coating.

Seven wetting-drying cycles were carried out. In each cycle, the samples were immersed in a 35 g/l NaCl solution for one week and then stored at ambient conditions for four weeks. The corrosion rate of the reinforcing steel was measured as the polarization resistance ($R_p$) at the end of each wetting cycle over a period of 55 weeks. At the end of the 55 weeks, the samples were crushed and the corroded surface area was calculated as the percentage of total area of the steel embedded in concrete.

### Results and Discussions

**Investigation of Defects at Steel-Concrete Interface**

**Defects due to Bleeding and Settlement of Fresh Concrete**

These are the defects formed as a result of gap formation beneath the horizontal reinforcing bar placed perpendicularly to the casting position. These defects were observed using video-microscopy and the defect factor recorded for each sample. The defect factor increases over the height of the column and the rate of this increase depends on the nature of the concrete (Fig. 5).

For C20, there are no defects below a distance of 40 cm from the base of the column. Near the top of the column (above 160 cm) a total loss of steel-concrete bond was observed because of high external bleeding water accumulation. For the two 40 MPa strength class concrete mixtures, defects are larger for conventional concrete than for SCC. Defects were observed above a height of 25 cm for C40 and above 40 cm for SCC40. For the 50 MPa strength class, there were no defects at the interface.

The decrease in pull-out resistance along the height of a vertical concrete section is known as the top-bar effect and is related to the deterioration in quality of the steel-concrete interface as height increases (Khayat 1998a,b; Petrov 1998). Investigation of the steel-concrete bond strength provides an additional option for assessing the quality of the interface.

The variation of percentage pull-out strength in the bottom part of column as a function of the depth of concrete beneath the bar (known as the bond efficiency ratio) is shown in Fig. 6 and Table 4.

### Table 2. Tests on Fresh Concrete

<table>
<thead>
<tr>
<th>Parameter</th>
<th>C20</th>
<th>C40</th>
<th>SCC40</th>
<th>C50</th>
<th>SCC50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slump (cm)</td>
<td>15.8</td>
<td>7.6</td>
<td>23</td>
<td>63</td>
<td>60</td>
</tr>
<tr>
<td>Slump flow (cm)</td>
<td>63</td>
<td>60</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Entrapped air (%)</td>
<td>2</td>
<td>1.4</td>
<td>2.2</td>
<td>1.4</td>
<td>1.8</td>
</tr>
<tr>
<td>Density</td>
<td>2.38</td>
<td>2.43</td>
<td>2.33</td>
<td>2.48</td>
<td>2.41</td>
</tr>
</tbody>
</table>

Note: SCC=sself-compacting concrete.

### Table 3. Compressive and Tensile Strength at Day 28

<table>
<thead>
<tr>
<th>Strength</th>
<th>C20</th>
<th>C40</th>
<th>SCC40</th>
<th>C50</th>
<th>SCC50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive strength (MPa)</td>
<td>27.4</td>
<td>45.8</td>
<td>43.9</td>
<td>55.4</td>
<td>57.1</td>
</tr>
<tr>
<td>Tensile strength (MPa)</td>
<td>3.0</td>
<td>3.8</td>
<td>3.4</td>
<td>4.4</td>
<td>5.1</td>
</tr>
</tbody>
</table>

Note: SCC=sself-compacting concrete.
The largest decrease in bond strength is observed in C20. Up to a height of 25 cm, there is no fall in bond strength. At a height of 40 cm, a loss of 54% of bond strength is observed. The loss of bond strength varies between 56 and 64% between the heights of 55 and 145 cm, while the interface defects (corresponding to 31 and 47%) exhibit a tendency to decrease. At 160 cm, a loss of 53% in bond strength is observed, which corresponds to an interface defect of 46% at this height. For the two top bars, at 175 and 190 cm, the bond strength is zero, which corresponds to complete steel-concrete debonding.

For C40 and SCC40, the loss of bond strength starts at a height of 25 cm, with values of 19 and 64%, respectively. In spite of the 0% steel-concrete debonding, this loss of bond strength can be attributed to the fact that the 25 cm height is above the densification zone. For C40 there is considerable loss of bond strength at 40 cm (reaching 59%), which corresponds to an interface defect of 31%. Between the heights of 55 and 190 cm, the loss of bond strength varies between 59 and 79%, with a tendency for the bond strength to decrease. The difference between the bottom and the top of the column is 77%. For the same height interval, steel-concrete debonding goes from 22 to 41%. For SCC40, the loss of bond strength is 53% at 40 cm, and between 55 and 190 cm it changes from 39 to 64% with a tendency for the bond strength to decrease. Steel-concrete debonding changes from 6 to 34% with a trend towards increasing debonding over the same height interval.

Changes in bond strength in the 50 MPa strength class with height are more uniform than for the other concretes. For C50, values up to 55 cm are 14–19% higher than the values for the base. Up to 75 cm the values continue to be 3–4% greater than those for the base. Above 90 cm, the bond strength is lower than the base value and changes from 13 to 58% (at 175 cm) with increasing height. There is a 26% difference between the base and the top of the column. The difference between the maximum and minimum values is 64%. The unexpected increase in bond efficiency as the depth of concrete below the rebar increased is probably due to segregation of the concrete at the bottom of the column from the free fall of fresh concrete. This proneness to segregation must be due to the presence of a high amount of HRWR, which may decrease the stability of the concrete. For SCC50, the fall in bond strength with increasing height goes from 21 to 47% (175 cm). The difference between the top and the base of the column is 42%.

The steel-concrete bond strength is a function of the interface between the bar and concrete, so increasing interface defects result in decreased pull-out strength. Despite the fact that defects only start at 25 cm for 40 MPa strength class concretes and at 40 cm for C20, there is, nevertheless, a significant fall decrease in pull-out strength before these heights are reached. Above these heights, the falls in pull-out strength correspond to interface defects, except for some points of irregularity. But the average loss of bond strength is 50–60% and defect levels are 20–30% at the middle section of the column. The difference may be the result of changes taking place in the quality of the concrete as a function of concrete height, as well as due to changes in the quality of the steel-concrete interface (such as increases in porosity at both the interface and in bulk due to a higher w/c ratio). These factors could not be detected by video-microscopy, but because they increase the porosity in bulk and at the interfacial zone, they could affect the rate of corrosion of reinforcing bars. Before entering the base value and changes from 13 to 58% (at 175 cm) with increasing height. There is a 26% difference between the base and the top of the column. The difference between the maximum and minimum values is 64%. The unexpected increase in bond efficiency as the depth of concrete below the rebar increased is probably due to segregation of the concrete at the bottom of the column from the free fall of fresh concrete. This proneness to segregation must be due to the presence of a high amount of HRWR, which may decrease the stability of the concrete. For SCC50, the fall in bond strength with increasing height goes from 21 to 47% (175 cm). The difference between the top and the base of the column is 42%.

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into a detailed corrosion analysis, therefore, the variation of parameters such as porosity, density, and compressive strength along the height should also be examined.

Figs. 7 and 8 show the variation of density and porosity, respectively, along the column for the five concrete mixtures tested. Porosity increases as a function of the water-cement ratio, with the average over the column being greatest for C20 and lowest for SCC50 and C50. At the bottom of the column (at a height of 10 cm) the weight of the concrete makes it denser, which might explain the fall in pull-out strength in the absence of interface defects. The confinement provided by the concrete cover is higher with denser concretes. The porosity of C20 shows a slight increase with increasing height, due to its higher bleeding capacity (higher w/c ratio).

Fig. 9 shows the variation of compressive strength along the height of column. Despite showing a slight tendency to decrease as height increases, the compressive strength values show a high degree of variation. This dispersion of values could be due to the axis of the core samples taken (which was perpendicular to the casting direction). Because of this high degree of variation, the compressive strength values obtained were not used to normalize the bond strength values.

The porosity, density, and compressive strength results indicate that there is a decrease in concrete quality with increasing height. Despite the fact that the variation of these factors does not correlate exactly with levels of defects and pull-out strength, the tendencies to decrease that they exhibit may explain the fall in pull-out strength where there is no defect beneath the bar, as well as the greater magnitudes of decreases in pull-out strength relative to increases in interface defects. In addition to variations in gap formation at the interface, therefore, there is a variation in the quality of the concrete with height. The poorer quality confinement provided by the concrete surrounding the reinforcing steel decreases the pull-out strength. The increase in porosity in the transition zone is another factor that lowers pull-out strength and is not detectable using a video-microscope. Porosity, however, has a relatively minor effect on pull-out strength when compared with the effects of gross defects (such as gaps beneath the reinforcement).

Defects due to Pull-Out Loading

Even at a magnification of \( \times 175 \), video-microscope analysis showed no evidence of damage at the steel-concrete interface of the plain and deformed bars (Figs. 10 and 11) under shear loading. A study (Cao and Chung 2001) involving analysis of steel-concrete damage caused by shear loading using electrical contact resistance measurements indicated that there was a sudden increase in contact resistance at two points—the first point being at adhesion failure, and the second at bond failure. The first increase was significantly smaller when compared with the second, and no visible damage was observed.

### Table 4. Variation of Bond Efficiency Ratio with Concrete Depth

<table>
<thead>
<tr>
<th>Height (cm)</th>
<th>C20</th>
<th>C40</th>
<th>SCC40</th>
<th>C50</th>
<th>SCC50</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>25</td>
<td>1.1</td>
<td>0.81</td>
<td>0.68</td>
<td>1.19</td>
<td>0.79</td>
</tr>
<tr>
<td>40</td>
<td>0.46</td>
<td>0.41</td>
<td>0.53</td>
<td>1.14</td>
<td>0.72</td>
</tr>
<tr>
<td>55</td>
<td>0.44</td>
<td>0.41</td>
<td>0.58</td>
<td>1.19</td>
<td>0.6</td>
</tr>
<tr>
<td>70</td>
<td>0.37</td>
<td>0.21</td>
<td>0.53</td>
<td>1.04</td>
<td>0.77</td>
</tr>
<tr>
<td>85</td>
<td>0.35</td>
<td>0.33</td>
<td>0.44</td>
<td>1.03</td>
<td>0.73</td>
</tr>
<tr>
<td>100</td>
<td>0.46</td>
<td>0.39</td>
<td>0.4</td>
<td>0.86</td>
<td>0.92</td>
</tr>
<tr>
<td>115</td>
<td>0.34</td>
<td>0.4</td>
<td>0.38</td>
<td>0.87</td>
<td>0.86</td>
</tr>
<tr>
<td>130</td>
<td>0.33</td>
<td>0.34</td>
<td>0.61</td>
<td>0.87</td>
<td>0.72</td>
</tr>
<tr>
<td>145</td>
<td>0.36</td>
<td>0.24</td>
<td>0.51</td>
<td>0.66</td>
<td>0.59</td>
</tr>
<tr>
<td>160</td>
<td>0.28</td>
<td>0.28</td>
<td>0.41</td>
<td>0.85</td>
<td>0.69</td>
</tr>
<tr>
<td>175</td>
<td>0.02</td>
<td>0.34</td>
<td>0.47</td>
<td>0.42</td>
<td>0.53</td>
</tr>
<tr>
<td>190</td>
<td>0.01</td>
<td>0.23</td>
<td>0.36</td>
<td>0.74</td>
<td>0.58</td>
</tr>
</tbody>
</table>

Note: SCC = self-compacting concrete.
Correlation between Corrosion and Defects

Fig. 12 shows the results obtained from $R_p$ measurements carried out at the end of seven drying-wetting cycles, corresponding to 55 weeks of chloride exposure. Two different behavior classifications are apparent in Fig. 12: that of high-strength concretes (C50 and SCC50) with a high polarization resistance, and the rest. In addition to the $R_p$ measurements, which give an instantaneous rate of corrosion, corroded surface areas after 55 weeks were also measured by breaking the samples to examine the total extent of corrosion. Fig. 13 show corrected corrosion areas, which take into account the presence of systematic accidental corrosion at the ends of all specimens. When specimens were crushed, crevice corrosion was observed on the electrode surface beneath the epoxy coating. The corroded surface area due to crevice corrosion attack was approximately 25–30% of the total electrode surface area. This area was then subtracted in order to calculate the corrected corroded surface area. The average value of the $R_p$ and the corrected corroded surface area results for all heights shows that the corrosion rate is a function of concrete quality and that corrosion increases as the class of concrete strength decreases. The corrected corroded surface area and $R_p$ for C20 range between 28 and 67% and between 4 and 2 kohm·cm², respectively, and that for C40 ranges between 4 and 47% and between 12 and 4 kohm·cm². For SCC40 the corrosion occurs only on some column heights with a maximum corroded area of 29% and a minimum $R_p$ of 2 kohm·cm². The $R_p$ values for C50 and SCC50 correspond to a moderate corrosion rate, which reflects only the crevice corrosion.

The $R_p$ method measures corrosion rates directly and is a very popular and effective nondestructive technique for the evaluation...
defects at the interface, detectable even by the naked eye; and
3. Invisible defects at the interface (defects at the steel-concrete interface that are not detectable even at \( \times 175 \) video-microscopic magnification).

**Intrinsic Defects**

Fig. 14 shows the relative variation of \( R_p \) and ultimate bond strength as a function of concrete depth for C20. It can be seen that, unlike ultimate bond strength, \( R_p \) is nearly constant over the entire height of the column. The lack of correlation between these two sets of measurements shows that corrosion development must be attributable to the higher porosity of the concrete and therefore to that of the interface; there is corrosion with or without the macroscopic defects.
height of the concrete, but this variation is small relative to $R_p$. There is thus no correlation between $R_p$ variation and the small decrease in bond strength with concrete depth.

The large values of $R_p$ for C50 and SCC50 correspond to an absence of corrected corroded area (Fig. 13). Nevertheless, the fact that there was a drop in pull-out strength even without gap formation beneath the reinforcing bar implies that defects exist at the interface, as in the case of C50 and SCC50. These defects are called invisible defects. For other columns, these defects have a minor effect on corrosion in comparison with intrinsic and visible defects.

In the study of shear loading of the steel bar, the rupture of bond between the plain round steel bar and the concrete corresponds to an adhesion failure. The steel-concrete interface examined under a video-microscope displays no signs of damage and is as described in the Cao and Chung (2001). Another question is the effect of this invisible defect on corrosion. The results show that there is no correlation between steel-concrete damage caused by controlled pull-out loading and corrosion.

However, even in the case of the rupture of the bond between the deformed bar and the concrete, contact at the steel-concrete interface was observed to be intact. Corrosion measurements show no correlation between bond failure (invisible defect) and corrosion. For controlled pull-out, the interfacial zone is still in contact with the steel and so can continue to carry out the buffering action, whereas gross defects (such as in the case of gap formation) completely prevent steel-to-concrete adhesion.

**Conclusion**

The interface defects analyzed can be grouped into categories as follows:

1. **Intrinsic defects**: High porosity of the covering concrete that surrounds the reinforcing bar;
2. **Visible defects**: Gaps beneath horizontal reinforcements, perpendicularly to the casting direction and formed by bleeding and settlement of fresh concrete; and
3. **Invisible defects** (not detectable with a video-microscope):
   - Porosity and heterogeneity of the interfacial transition zone, which deteriorates as a function of concrete depth under reinforcement but which is not detectable by video-microscope;
   - Rupture of adhesion between plain round steel bars and concrete in the case of a pull-out loading that causes high or low degrees of slip of the bar; and
   - Rupture of the bond between deformed steel bars and concrete.

**Intrinsic defects** are evident and significant in corrosion of high w/c ratio concrete such as C20. Indeed, for C20, corrosion is generalized whatever the location of the reinforcement and irrespective of the presence (or otherwise) and type of defect found at the steel-concrete interface.

**Visible defects** are gross defects that prevent there being contact between steel and concrete and therefore prevent the formation of the specific protection provided by the interfacial layer. As expected, corrosion varies directly with the level of these defects, as in the case for C40 and SCC40.

**Invisible defects**, which are caused by bleeding, seem to have little effect on corrosion. For C40 and SCC40, they do not influence corrosion, although they do decrease pull-out strength. For C50 and SCC50, where the invisible defects were the only type of damage present (no gap beneath the horizontal bar), they were responsible for the fall in pull-out strength, together with the decrease in quality of confinement provided by the cover concrete. They did not, however, cause corrosion.

**Invisible defects** due to mechanical degradation (as for pull-out of plain and deformed bars) did not cause corrosion. Adhesion failure with plain bars appears not to disturb the interfacial layer; this remains in contact with the steel to provide the protection against corrosion. The rupture of the bond between the deformed steel bar and the concrete did not cause corrosion, and this is explained by the contact between steel and concrete still continuing to provide friction.

From these findings, it can be concluded that the beneficial aspects in terms of protection of the embedded steel that is provided by alkaline solutions from the surrounding concrete disappears:
- If the concrete is highly porous; or
- If there is bleeding (a benefit of using SCC).

It is, however, maintained in the case of low levels of damage, such as an increase in the porosity over the height of concrete or...
rupture of the adhesion and bond between steel and concrete.

In addition to these findings, some other findings on the nature of the concrete may be summarized as follows:

1. The size of the gaps beneath the lower part of the horizontal reinforcing bars increases with the depth of concrete below the bar. The deterioration of the steel-concrete interface with height was detected by the fall in pull-out strength and is related to the formation of gaps beneath the bar (measured using a video-microscope). This was in addition to a decrease in the quality of the concrete detected through the fall in density and compressive strength of the concrete.

2. The rate of decrease in the quality of the steel-concrete interface along the height depends on the bleeding capacity of the concrete mixture. The use of HRWR allows the w/c ratio to be lowered, and VMA increases the stability of concrete. The addition of supplementary fines materials (such as silica fume and limestone filler) is another way of increasing the stability of concrete. As a result, SCC provides better protection against corrosion because of its less porous and more homogeneous interfacial zone. SCC reduces bleeding and consequently has better stability along the height of the vertical concrete section. It therefore reduces the fall in pull-out strength and the reduction in corrosion protection because of its more homogeneous interface. It provides a more homogeneous concrete cover in surrounding the steel. This affects both the pull-out strength and corrosion resistance.

The bleeding, settlement, and segregation of concrete may cause weakening of the bond between the steel reinforcement and concrete by producing some defects. The better protection against corrosion provided by a better steel-concrete bond seems not to be altered significantly in the presence of smaller defects in comparison to gap formation, despite a reduction in bond strength. The bigger defects such as gap formation under reinforcement are of major importance in the drop of bond strength and the corrosion of steel. The importance of those phenomena increases with the height of a vertical concrete section as the amount of concrete available for segregation increases. For the prevention of such a problem, in addition to the requirement for an adequate consolidation in a solid formwork, the quality of concrete is very important. Concrete with higher fluidity is prone to cause this undesirable case, and when the use of such a concrete is necessary, the homogeneity of concrete may be guaranteed by the use of admixtures. SCC is a specially formulated concrete using chemical and homogeneity of concrete may be guaranteed by the use of admixtures. However, it should be borne in mind that a very high w/c would cause high permeability and corrosion even if bleeding is avoided.

References


