The durability of bonded steel plates for strengthening concrete highway bridges - a 20 year trial

Prepared for Quality Services Civil Engineering, Highways Agency

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Executive Summary

Strengthening concrete bridges by bonding external steel plates to the soffit of the deck using structural epoxy adhesives has become a recognised practice over the last 20 years. The technique provides an economic solution where additional strengthening to concrete bridges is required. Alternatives such as the replacement or addition of structural members, or demolition and reconstruction are clearly more expensive. To date, about 20 bridges in the UK have been strengthened using this method. Various authorities have expressed concern about the long term durability of the technique. For example, the Institutions of Civil, Municipal and Structural Engineers Standing Committee on Structural Safety (1983) highlighted the lack of information on long term performance of resins and warned against their use in critical parts of a structure unless effective replacement can take place should structural deterioration occur.

TRL have carried out durability studies to investigate the long term performance of bonded steel plates. Concrete beam specimens, 0.5m and 3.5m long, strengthened with single mild steel plates were exposed to high rainfall, industrial and coastal environments and compared with specimens stored under controlled laboratory conditions. The performance of two different resins was investigated (resin I: Ciba Geigy XD800 and resin II: Colebrand CXL 83). Some of the specimens were maintained under load during exposure. Load tests on the 0.5m beams were carried out after 1, 2, 10 and 19 years exposure and on the 3.5m beams after 1, 8, and 16 years exposure. The performance of plates bonded to the soffit of a motorway bridge at Quinton in 1975 has also been monitored.

Only one of the two resins tested had adequate durability. This resin was used on the Quinton bridge. The other resin did not fulfil the requirements of the performance specification in the Department of Transport Advice Note BA30/94 (1994) for plate bonding.

The plates on all the specimens (both loaded and unloaded) assembled using resin I remained fully bonded to the concrete during exposure even after nearly 20 years. Exposure for up to nearly 20 years only had a marginal effect on the failure loads of the 0.5m and 3.5m long beams despite some corrosion of the steel at the interface with the resin. The corrosion was caused by the ingress of moisture through microcracks in the resin and the concrete. There was also evidence on some of the 0.5m long specimens that corrosion had spread from the edge of the plate by ingress of moisture at the interface between the resin and the steel. The degree of corrosion was significantly reduced by coating the steel with an epoxy primer paint prior to assembly of specimens. In this case, the primer did not adversely affect the structural performance.

The bond between resin II and the concrete failed on the specimens exposed with the resin under stress after only 3 months of exposure. After 8 years there was some debonding of the plates on beams which were exposed with resin II unstressed. After 12 years, the plates had become completely detached from the concrete. This was attributable to ingress of moisture between the steel and the resin at the edge of the plate. The structural performance of the 3.5m long beams exposed with resin II unstressed was not adversely affected by corrosion that occurred during the first year of exposure. The corrosion was caused by the ingress of moisture through the edges of the plate at the interface with the resin and after 8 years of exposure, continuing corrosion caused debonding of the plate during the loading tests and hence premature failure of the beams.

The durability of external reinforcement bonded with resin I has been satisfactory on the Quinton bridge since its installation in 1975 except where the plates have been exposed directly to leakage of water and chlorides from the carriageway above. Otherwise, only a small amount of debonding has been detected. Tapping the surface with a steel hammer has proved to be a useful method for detecting debonding.

This research programme confirms that satisfactory long term performance of bonded steel plates can be obtained provided care is taken in the choice of adhesive. The resin that was used on the Quinton bridge, which fulfilled the requirements of the Advice Note on plate bonding, was durable both in service and in the exposure trials. Steel plate bonding can therefore be recommended as a viable and economic technique for providing additional strength to concrete bridges.
1 Introduction

Strengthening concrete bridges by bonding external steel plates to the soffit of the deck using structural epoxy adhesives has become a recognised practice over the last 20 years. The technique provides an economic solution where additional strengthening to concrete bridges is required. Alternatives such as the replacement or addition of structural members, or demolition and reconstruction are clearly more expensive. To date, about 20 bridges in the UK have been strengthened using this method. Considerable research has been carried out into the effectiveness of the technique. In particular, the Transport Research Laboratory (TRL) has investigated the structural implications of strengthening both by laboratory testing (Macdonald, 1978, 1982) and by full scale loading tests on two pairs of motorway bridges before and after plating (Raithby, 1980). The results show that increases in flexural stiffness and ultimate load carrying capacity are achieved by strengthening structural members using this technique. These results have been confirmed at other centres both in the UK and abroad; for example, Swamy et al (1987), Soloman et al (1976) and Hugenschmidt (1982). In 1994, the Department of Transport published an Advice Note (BA 30/94) on strengthening of concrete highway structures using externally bonded steel plates.

Various authorities have expressed concern about the long term durability of the technique. For example, the Institutions of Civil, Municipal and Structural Engineers Standing Committee on Structural Safety (1983) highlighted the lack of information on long term performance of resins and warned against their use in critical parts of a structure unless effective replacement can take place should structural deterioration occur.

This report describes durability studies carried out by TRL to investigate the long term performance of bonded steel plates. Concrete beam specimens, 0.5m and 3.5m long, strengthened with single mild steel plates were exposed to high rainfall, industrial and coastal environments and compared with specimens stored under controlled laboratory conditions. Some of the specimens were maintained under load during exposure. Load tests on the 0.5m beams were carried out after 1, 2, 10 and 19 years exposure and on the 3.5m beams after 1, 8, and 16 years exposure. The performance of plates bonded to the soffit of a motorway bridge in 1975 has also been monitored.

2 Exposure tests

2.1 0.5m Unreinforced concrete beams

A detailed description of the experiment was given by Calder (1979) and is summarised below:

Exposure tests were started in 1976 on unreinforced concrete beams (Calder, 1979 and 1988) which were 508mm long with a square cross section of 102mm and strengthened with a mild steel plate (500mm long, 30mm wide and 3mm thick) bonded to one face using a structural epoxy adhesive (resin I: Ciba Geigy XD800). This resin was chosen because it was used for strengthening the Quinton bridge. The specimens were exposed on racks (Figure 1) at 3 exposure sites which were chosen to represent environmental conditions encountered at bridge sites in the UK. The three sites were:

- Silverdale which is a rural site with relatively high rainfall (typically 1050mm per annum) in north west England.
- Tinsley which is an industrial site situated between Sheffield and Rotherham and was characterised by high levels of atmospheric sulphur dioxide. At the start of the experiment, the concentrations of atmospheric sulphur compounds measured by lead dioxide candles were typically 2mg SO$_2$/dm$^2$/day. However the adjacent blast furnaces have since been closed down. A reduction in the atmospheric corrosivity was measured using special mild steel specimens which corroded at a rate of 59µm/year in 1979/80 and 45µm/year in 1984/5.
- Pilsey which is a marine site situated in Chichester harbour and is exposed to high levels of atmospheric chlorides (typically 0.3mg Cl/dm$^2$/day).

![Figure 1](image)

Control specimens were also kept in a laboratory environment (20°C and 65 per cent RH). After 7 years of exposure, the Silverdale site was closed and the specimens were moved 6 miles further north to a site at Milnthorpe which had similar environmental characteristics.

Half the specimens at each site were subjected to a sustained bending load during exposure and half remained unloaded. The loaded specimens were stressed in pairs, back-to-back (Figure 2), with the load initially set at 75% of that required to crack the specimen. The loads applied to these specimens during exposure were monitored periodically by measuring the strains in the tie rods using a 100mm demec gauge. The upper specimens of the loaded pairs were used only for reaction and the results of loaded specimens refer to the results of tests on the lower specimens. For these specimens, the steel plate is on the underside of the beam which replicates the situation in practice. The unloaded specimens were also exposed with their plate on the underside of the beam. Sets of 4 loaded
pairs and 4 unloaded specimens were brought back to TRL for testing after 1, 2, 10 and 19 years exposure. A four point bending test to failure was carried out on each of the specimens (Figure 3).

Supplementary tests were carried out on specimens where the bonded surface of the steel plate was protected with an epoxy primer paint prior to bonding.

2.2 3.5m Reinforced concrete beams

Larger 3.5m long reinforced concrete beams were strengthened with mild steel plates before and after being cracked in flexure. The beams were then exposed at Eastney, which is a coastal exposure site approximately 5 miles west of Pilsey. A detailed description of the experiment was given by Calder (1989) and is summarised below.

The reinforced concrete beams were 3.5m long, 250mm deep and 150mm wide. The beams were identical to those used by Macdonald (1982) to investigate the flexural performance of concrete beams with various external reinforcement geometries. The concrete had 20 mm maximum sized aggregate and a mean 28 day compressive strength of 47 N/mm$^2$. The layout of the internal reinforcement, which complied with BS 4461 (1969) and had a characteristic strength of 460 kN/mm$^2$, is given in Figure 4. The beams were designed to be under reinforced so that failure would occur by yielding of the internal reinforcement. They were plated with their tension face uppermost with a single mild steel plate, 2.8 m long, 85 mm wide and 3 mm thick, bonded centrally to the tension face of the beam.

The beams were mounted in reaction frames (Figure 5). Some were loaded to produce flexural cracks and then plated under load; the rest were plated before the beams were cracked in flexure.

![Figure 2](image1.png)

**Figure 2** Pairs of 0.5m specimens maintained under load

![Figure 3](image2.png)

**Figure 3** Four point bending tests on 0.5m beams

![Figure 4](image3.png)

**Figure 4** Layout of internal reinforcement in 3.5m beams

![Figure 5](image4.png)

**Figure 5** Method of loading 3.5m beams prior to exposure
The crack widths at the level of the reinforcement were measured using a demec gauge (Figure 6). It was assumed that there was no strain in the concrete between the cracks. The beams in their reaction frames were turned upside down after plating so that the steel plate was exposed on the underside of the beam to simulate strengthening conditions normally encountered on a bridge. The beams were then maintained under load for the duration of exposure.

Two resins were chosen for the exposure tests; 12 beams were plated using each resin. Resin I was the same as used for the 0.5m beams. A second resin (resin II: Colebrand CXL 83) was chosen as it had been recommended for plate bonding and had a much lower modulus of elasticity than resin I. Of the 12 beams, 6 were loaded before plating and 6 loaded after plating. It is important to note that for specimens plated prior to loading, the resin and steel plate were exposed under load, whereas for specimens bonded after loading, the resin and the plate remained unstressed during exposure.

The specimens were transported to the exposure site in their reaction frames. The crack widths were monitored periodically using the demec gauges. The bond between the steel and the concrete was also checked by lightly tapping the plate with a steel hammer: any debonding of the plate was identified by a hollow sound.

Beams were brought back to TRL for testing after 1, 8 and 16 years exposure. Each of the beams, mounted in its reaction frame, was placed on a box section base and loaded as shown in Figure 7. The load, which was increased in increments up to failure, was measured using load cells mounted between each jack and the beam. The end deflections were recorded using displacement transducers. Debonding of the steel plate was checked at each load increment by tapping the plate with a steel hammer. After the load test to failure, the steel plate was removed from each beam, cut up, photographed and kept in a desiccator for subsequent examination.

3 Results

3.1 0.5m Unreinforced beams

A summary of the results from the tests on the 0.5m beams is given in Table 1 and is discussed in detail below.

3.1.1 Site observations

In 1986 the Tinsley exposure site was broken into and vandalised. All the beams that had been under load were unloaded, and the stainless steel tie rods, cross beams and rollers stolen. The test beams had also been scattered around the exposure site. As it was difficult to assess the damage that had been done and it would have been impossible to reload the specimens to recreate the loading conditions; the trial at this site was terminated and the results up to 1986 reported (Calder, 1988).

The loads applied to each of the 0.5m beams were monitored by measuring the strains in the tie rods. The loads on the beams exposed at Milnthorpe and Pilsey reduced rapidly at the start of exposure and since then have varied between about 50 and 80 per cent of their original value (Figure 8). The loads measured on beams kept under controlled laboratory conditions have been consistently higher than the loads measured at Milnthorpe and Pilsey.

Seven of the reaction specimens cracked during the first year of exposure. These were replaced and the beams reloaded to the mean load measured on the uncracked pairs at that site at the time of replacement (Calder, 1979). The load applied to Beam 44.4 at Milnthorpe steadily reduced during exposure and by 1981 was about 20% of the original value. This was caused by cracking of the reaction beam and subsequent debonding of the plate; in this instance, the cracked reaction beam was not replaced.
3.1.2 Loading tests to failure

For each beam, the concrete strain at mid span was plotted against the total applied load (Figure 9). At the onset of cracking there was a reduction in stiffness of the beam which was reflected in a change in slope of the curve. The load at which this change in slope occurs has been defined as the cracking load; this load was slightly less than the load at which cracking became visible to the naked eye. The load at failure of each beam was also recorded. The results from the loading tests are given in Tables 2 and 3.

For one of the unloaded specimens recovered from Milnthorpe after 19 years, the plate had become debonded over a length of 135mm from one end resulting in severe corrosion. As the loads at onset of cracking (6.6kN) and failure (15.8kN) were much less than for the other

Table 1 Summary of results for the 0.5m plated beams

<table>
<thead>
<tr>
<th>Site</th>
<th>Period of exposure (years)</th>
<th>Mean load at cracking (kN)</th>
<th>Mean load at failure (kN)</th>
<th>Mean area of steel plate debonded (%)</th>
<th>Mean area of steel plate corroded (%)</th>
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<tr>
<td></td>
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<td>Loaded  Unloaded</td>
<td>Loaded  Unloaded</td>
<td>Loaded  Unloaded</td>
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<td>3.8  0.8</td>
<td>6.1  5.4</td>
</tr>
</tbody>
</table>

1 The area of corrosion of the steel plates removed after 2 years of exposure was estimated using the graticule method. The photographic image analysis method was used for plates removed from specimens after 10 and 19 years of exposure.

3.1.2 Loading tests to failure

For each beam, the concrete strain at mid span was plotted against the total applied load (Figure 9). At the onset of cracking there was a reduction in stiffness of the beam which was reflected in a change in slope of the curve. The load at which this change in slope occurs has been defined as the cracking load; this load was slightly less than the load at which cracking became visible to the naked eye. The load at failure of each beam was also recorded. The results from the loading tests are given in Tables 2 and 3.

For one of the unloaded specimens recovered from Milnthorpe after 19 years, the plate had become debonded over a length of 135mm from one end resulting in severe corrosion. As the loads at onset of cracking (6.6kN) and failure (15.8kN) were much less than for the other

Table 2 Tests on 0.5m beams: load at onset of cracking for each specimen

<table>
<thead>
<tr>
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<th>Period of exposure (years)</th>
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1 Steel plate debonded from concrete during exposure

Table 3 Tests on 0.5m beams: load at failure for each specimen

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<td>51.6</td>
</tr>
<tr>
<td></td>
<td>Unloaded</td>
<td>50.6</td>
<td>47.3</td>
<td>48.7</td>
<td>45.1</td>
</tr>
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<td></td>
<td>44.4</td>
<td>46.4</td>
<td>45.7</td>
<td>47.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>40.0</td>
<td>43.4</td>
<td>46.2</td>
<td>44.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>37.8</td>
<td>53.3</td>
<td>55.6</td>
<td>45.0</td>
</tr>
</tbody>
</table>

1 Steel plate debonded from concrete during exposure

* Missing value
specimens from that set, it was decided to exclude the results from this beam from the subsequent analysis.

The mean and range of cracking loads measured at each site for each period of exposure are plotted in Figure 10. An analysis of variance of the results showed that there were significant differences between the results from specimens tested after different periods of exposure and specimens which had been loaded and unloaded during exposure. On average, the specimens cracked at lower loads after 19 years of exposure (11.7kN) compared with 1 year (14.8kN), 2 years (14.6kN) and 10 years (16.0kN).

The mean cracking load of the specimens which were loaded during exposure (15.2kN) was higher than the mean of the specimens which were unloaded during exposure (13.3kN).

The mean and range of loads at failure are shown in Figure 11. On average the beams that had been exposed failed at lower loads (41.2 and 41.5kN for Milnthorpe and Pilsey respectively) than the beams which had been stored under controlled laboratory conditions at TRL (46.0kN). An analysis of variance showed that these differences were statistically significant. The analysis also showed significant differences between periods of exposure. Overall, the failure loads after 10 years of exposure were on average (45.9kN) significantly higher than the failure loads after 1 year (40.6kN), 2 years (42.6kN) and 19 years (42.5kN) of exposure.

Although the differences found in the cracking and failure loads were statistically significant, they were in fact small and would be unlikely to effect the long term structural performance.

The failure loads of the specimens where the plate had been treated with a primer and exposed at Milnthorpe for 17 years (47.1 and 43.7kN) were similar to the failure loads of the specimens which were not primed. This confirms earlier results reported by Calder (1988).

3.1.3 Observations of the fractured surfaces

The plates were removed from each beam after the loading test to failure (Figure 12). Generally, a layer of resin and concrete adhered to the steel plates along part of their length. This suggests that failure had occurred by horizontal shear in the concrete close to the concrete/resin interface (Calder, 1979). In a few cases the failure had occurred at the steel/resin interface.

On average, the area of steel plate that had become debonded from the concrete during the load test to failure increased from 27% for beams exposed for 2 years at Milnthorpe and Pilsey to 47% for beams that had been exposed for 10 years (Table 4). After 19 years of exposure the average value was 45%. The results for each site and
Figure 10 Cracking loads of 0.5m plated beams
Figure 11 Failure loads of 0.5m plated beams
each exposure period were very variable; for example, after 19 years of exposure under load at Milnthorpe, the debonded areas for the 4 duplicate specimens varied between 1% and 100%.

There was some corrosion on the debonded areas of the steel plates on all the exposed beams. Corrosion usually consisted of spots distributed over the surface of the plates, occasionally merging to form continuous areas of corrosion on some of the plates. Lloyd and Calder (1982) reported that this corrosion was caused by migration of moisture through microcracks in the concrete and the resin. There was evidence on some specimens that corrosion had also spread from the edge of the plates at the interface between the steel and the resin (for example, specimen 17.2; Figure 12).

The area of corrosion of the debonded steel on plates removed from specimens after 2 and 10 years of exposure was estimated with the aid of a transparent graticule ruled with 1mm squares (Table 5). Calder (1988) reported that for the plates removed from specimens exposed at Milnthorpe, Tinsley and Pilsey, the corroded area was 21% after 2 years of exposure compared with 55% after 10 years. The percentage of the complete plate found to be corroding, after the resin had been removed by soaking in methylene chloride, was similar to the percentage of the steel exposed during the loading tests.

For the plates removed from specimens after 19 years of exposure, the area of corrosion of the steel plates after the resin had been chemically removed was estimated with the aid of the graticule. On average, 37% of the area had corroded, compared with 55% for plates removed from specimens exposed at Milnthorpe and Pilsey for 10 years. The measurements after different periods of exposure were carried out by different people on different specimens and may not have been consistent due to the subjective nature of the technique. In order to make objective comparisons

<p>| Figure 12 Plates removed from 0.5m specimens recovered from exposure sites after 10 years exposure |
| Table 4 Tests on 0.5m beams area of debonding of steel plate after load test to failure |</p>
<table>
<thead>
<tr>
<th>Site</th>
<th>Period of exposure (years)</th>
<th>2</th>
<th>10</th>
<th>19</th>
</tr>
</thead>
<tbody>
<tr>
<td>Percentage of area debonded</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Milnethorpe</td>
<td>Loaded</td>
<td>38</td>
<td>100</td>
<td>44</td>
</tr>
<tr>
<td></td>
<td></td>
<td>26</td>
<td>99</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td></td>
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<td>100</td>
<td>1</td>
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<tr>
<td></td>
<td></td>
<td>31</td>
<td>15</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>Unloaded</td>
<td>19</td>
<td>19</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>30</td>
<td>21</td>
<td>44</td>
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<tr>
<td></td>
<td></td>
<td>2</td>
<td>18</td>
<td>16</td>
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<td></td>
<td></td>
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<td>73</td>
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<td>Pilsey</td>
<td>Loaded</td>
<td>12</td>
<td>27</td>
<td>16</td>
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<td></td>
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<td>34</td>
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<td>12</td>
<td>67</td>
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<td></td>
<td></td>
<td>*</td>
<td>100</td>
<td>*</td>
</tr>
<tr>
<td></td>
<td>Unloaded</td>
<td>43</td>
<td>34</td>
<td>52</td>
</tr>
<tr>
<td></td>
<td></td>
<td>98</td>
<td>*</td>
<td>89</td>
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<td></td>
<td></td>
<td>23</td>
<td>7</td>
<td>82</td>
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<tr>
<td></td>
<td></td>
<td>50</td>
<td>16</td>
<td>*</td>
</tr>
<tr>
<td>TRL</td>
<td>Loaded</td>
<td>0</td>
<td>0</td>
<td>16</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0</td>
<td>7</td>
<td>12</td>
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<tr>
<td></td>
<td></td>
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<td>22</td>
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<td>0</td>
<td>7</td>
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<tr>
<td></td>
<td>Unloaded</td>
<td>0</td>
<td>10</td>
<td>12</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0</td>
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<tr>
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<td></td>
<td>0</td>
<td>7</td>
<td>22</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0</td>
<td>5</td>
<td>4</td>
</tr>
</tbody>
</table>

* Missing value

| Table 5 Tests on 0.5m beams area of corrosion on plate removed from each specimen |
| Site | Period of exposure (years) | 2 | 10 | 19 |
| Percentage area of corrosion |
| Milnethorpe | Loaded | 6 | 72 | 22 |
| | | 5 | 79 | 25 |
| | | * | 62 | 31 |
| | Unloaded | 5 | 5 | 10 |
| | | 45 | 88 | 24 |
| | | 30 | 89 | 32 |
| | | 35 | 31 | 52 |
| Pilsey | Loaded | 22 | 38 | 45 |
| | | 44 | 3 | 52 |
| | | 9 | 16 | 26 |
| | | 38 | 50 | 5 |
| | Unloaded | 5 | 63 | 30 |
| | | 31 | 9 | 45 |
| | | 5 | 46 | 6 |
| TRL | Loaded | 0 | 0 | 5 |
| | | 0 | 0 | 0 |
| | | 0 | 0 | 0 |
| | | 0 | 0 | 10 |
| | Unloaded | 0 | 0 | 0 |
| | | 0 | 0 | 3 |
| | | 0 | 0 | 0 |
| | | 0 | 0 | 0 |

* Missing value
between plates removed from specimens that had been exposed for 10 and 19 years, the corroded areas were estimated using photographic image analysis. For plates removed from specimens exposed for 10 years, the area of corrosion was estimated from the steel that had become debonded during the load tests. For plates removed from specimens exposed for 19 years, the corroded area over the whole plate was estimated. The results using this technique were compared with results obtained by manual inspection with aid of the graticule (Figure 13). For each period of exposure, there was good correlation between the results from the two techniques. However the lines of best fit had markedly different slopes for 10 years and 19 years. Using these relationships, an area of corrosion of 40% measured by image analysis corresponded to areas of 78% and 38% corrosion obtained by graticule observation by the individuals after 10 and 19 years of exposure respectively. These differences clearly confirm that the technique using the graticule is subjective, but was consistent for each of the two individuals.

3.2 3.5m Reinforced beams

A summary of the results from the tests on the 3.5m beams is given in Table 6 and is discussed in detail below.

3.2.1 Site observations

All the beams bonded with resin I performed satisfactorily on site with no debonding after 16 years of exposure. By contrast, all the beams bonded with resin II that had been loaded after plating failed during the first 3 months of exposure (Calder, 1988). For the beams which had been loaded prior to plating, some debonding of the plates was identified after 8 years of exposure by lightly tapping the plate with a steel hammer. After 12 years of exposure, the debonding had extended along the complete length of the plate which had become completely detached from the beam.

Typical crack widths measured during exposure are given in Figure 15. There was an increase in crack width during the first year for some specimens caused by creep of the concrete. The widths subsequently remained more or less constant for the rest of the exposure period.

3.2.2 Loading tests to failure

Loading tests to failure were carried out on all the beams plated with resin I after up to 16 years of exposure. The deflection of the beam under each jack was measured and the mean value plotted against the applied load. Figure 16 shows the results for the three plated beams recovered after 16 years of exposure and for an unplated beam. For both plated and unplated beams there was a linear relationship between load and deflection at low loads. As the load was increased further, failure occurred by yielding of the internal reinforcement and the steel plate; this is illustrated by the increase in deflection that occurred with no significant increase in load. After 16 years, the ultimate strength of the plated beams was higher than that of the unplated beam.

Calder (1988) reported that the modes of failure and failure loads were similar to those observed by Macdonald.
(1982) on plated beams with the same geometry but bonded using different resins. Table 7 gives results for comparison. In 10 of the 11 tests on beams plated with resin I failure occurred as a result of yielding of the internal reinforcement and the steel plate (Failure Mode: F1). In the case of Beam 7, the plate also lifted away from the concrete over the centre span of the beam during yield (Figure 17). The beams with plates bonded with resin II and loaded prior to plating were satisfactory after 1 year of exposure whereas after 8 years premature failure occurred due to debonding of the steel plate (Failure Mode F3).

3.2.3 Observations of the fractured surfaces
After the loading tests to failure, the steel plates were prised from the surface of each beam using a hammer and chisel. The appearance of the plates from beams bonded with resin I was different from those bonded with resin II (Calder, 1988). A layer of resin and concrete remained on the steel on all the plates removed from beams bonded with resin I. By contrast, separation of the plates which had been bonded with resin II was between the steel and the resin. There were indications that this was caused by the ingress of moisture from the edge of the plate at the interface between the steel and the resin.

All the resin and concrete remaining on the steel plates from 16 year old beams after load testing was removed by soaking the plates in methylene chloride. For the two beams that had been loaded after plating, severe bands of corrosion were visible on the steel plates at the position of the cracks in the beams (Figure 18). By contrast, for beams plated after loading, there was light corrosion similar to that observed on the plates removed from the 0.5m beams uniformly distributed over the surface of the steel (Figure 19). These observations are consistent with observations made on beams recovered after 1 and 8 years of exposure.

<table>
<thead>
<tr>
<th>Period of exposure (years)</th>
<th>Beams loaded before or after plating</th>
<th>Performance during exposure</th>
<th>Mean load at failure (kN)</th>
<th>Mode of failure during load test to failure</th>
<th>Corrosion of the steel plate</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Before</td>
<td>Satisfactory</td>
<td>45.0</td>
<td>By yielding of internal reinforcement and steel plate or by shear in concrete</td>
<td>Light corrosion</td>
</tr>
<tr>
<td></td>
<td>After</td>
<td>Satisfactory</td>
<td>44.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>Before</td>
<td>Satisfactory</td>
<td>47.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>After</td>
<td>Satisfactory</td>
<td>44.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>Before</td>
<td>Satisfactory</td>
<td>49.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>After</td>
<td>Satisfactory</td>
<td>46.5</td>
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</table>

Resin II

<table>
<thead>
<tr>
<th>Period of exposure (years)</th>
<th>Beams loaded before or after plating</th>
<th>Performance during exposure</th>
<th>Mean load at failure (kN)</th>
<th>Mode of failure during load test to failure</th>
<th>Corrosion of the steel plate</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Before</td>
<td>Plates became debonded from beams during first 3 months of exposure</td>
<td>-</td>
<td>By yielding of internal reinforcement and steel plate or by shear in concrete</td>
<td>Corrosion resulting from ingress of moisture from edge of plates</td>
</tr>
<tr>
<td></td>
<td>After</td>
<td>Plates remained in place during exposure, but hammer survey showed some areas of debonding</td>
<td>28.5</td>
<td>Premature failure by debonding of the steel plate</td>
<td>Severe corrosion of whole area of plate</td>
</tr>
<tr>
<td>8</td>
<td>Before</td>
<td>Plates became debonded from beams during first 3 months of exposure</td>
<td>-</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>After</td>
<td>Plates became completely debonded after 12 years of exposure</td>
<td>-</td>
<td></td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>Before</td>
<td>Plates became debonded from beams during first 3 months of exposure</td>
<td>-</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>After</td>
<td>Plates became completely debonded after 12 years of exposure</td>
<td>-</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Figure 15 Variation of crack width with time for 3.5m beams

Table 7 Results of loading tests on 3.5m beams

<table>
<thead>
<tr>
<th>Beam number</th>
<th>Resin</th>
<th>Beams loaded</th>
<th>Load at Failure (kN)</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>before or after plating</td>
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<td></td>
</tr>
<tr>
<td>Specimens tested soon after plating (Macdonald, 1982)</td>
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<tr>
<td>4</td>
<td>A</td>
<td>After</td>
<td>42</td>
<td>F2</td>
</tr>
<tr>
<td>6</td>
<td>B</td>
<td>After</td>
<td>45</td>
<td>F1</td>
</tr>
<tr>
<td>3</td>
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<tr>
<td>37</td>
<td>B</td>
<td>After</td>
<td>42</td>
<td>F1</td>
</tr>
<tr>
<td>38</td>
<td>C</td>
<td>After</td>
<td>42</td>
<td>F2</td>
</tr>
<tr>
<td>39</td>
<td>C</td>
<td>After</td>
<td>42</td>
<td>F1</td>
</tr>
<tr>
<td>Specimens tested after 1 year of exposure</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>I</td>
<td>After</td>
<td>43</td>
<td>F1</td>
</tr>
<tr>
<td>9</td>
<td>I</td>
<td>After</td>
<td>45</td>
<td>F1</td>
</tr>
<tr>
<td>14</td>
<td>I</td>
<td>Before</td>
<td>45</td>
<td>F1</td>
</tr>
<tr>
<td>15</td>
<td>I</td>
<td>Before</td>
<td>46</td>
<td>F1</td>
</tr>
<tr>
<td>31</td>
<td>II</td>
<td>Before</td>
<td>41</td>
<td>F2</td>
</tr>
<tr>
<td>33</td>
<td>II</td>
<td>Before</td>
<td>51</td>
<td>F1</td>
</tr>
<tr>
<td>Specimens tested after 8 years of exposure</td>
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<td>After</td>
<td>43</td>
<td>F1</td>
</tr>
<tr>
<td>16</td>
<td>I</td>
<td>Before</td>
<td>47</td>
<td>F2</td>
</tr>
<tr>
<td>17</td>
<td>I</td>
<td>After</td>
<td>48</td>
<td>F1</td>
</tr>
<tr>
<td>34</td>
<td>II</td>
<td>After</td>
<td>25</td>
<td>F3</td>
</tr>
<tr>
<td>35</td>
<td>II</td>
<td>After</td>
<td>32</td>
<td>F3</td>
</tr>
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<td>Specimens tested after 16 years of exposure</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>I</td>
<td>Before</td>
<td>46</td>
<td>F1</td>
</tr>
<tr>
<td>11</td>
<td>I</td>
<td>Before</td>
<td>47</td>
<td>F1</td>
</tr>
<tr>
<td>18</td>
<td>I</td>
<td>After</td>
<td>49</td>
<td>F1</td>
</tr>
</tbody>
</table>

Failure Modes:
F1: Ductile failure due to yield of the internal reinforcement and steel plate
F2: Brittle Failure associated with shear in the concrete
F3: Premature failure by debonding of the steel plate
The service history of Quinton bridge, the first to be strengthened with bonded steel plates in the UK, has been monitored by TRL. Quinton interchange comprises of four pairs of bridges at Junction 3 on the M5 south west of Birmingham where the motorway crosses the A456 at a large roundabout. Each bridge carries a two-lane carriageway and hard shoulder for one half of the motorway, and there is a narrow central reservation suspended between each pair of bridges. The deck of each bridge is a voided concrete slab of variable depth which is continuous across the three spans. The length of the centre span is 27.4m and the length of the sidespans is 17m.

The bridges were built in 1966, opened to traffic in 1970 and strengthened in 1975 after some cracking was observed on the underside of each span in critical regions. A total of 1376 plates were bonded to the underside of the four bridges in 1975. The procedure adopted was very similar to that used for bonding the 3.5m beams used for the exposure trials, except that the plates were held in position during cure by wedging against jacking beams hung from suspender rods beneath the bridge (Raithby, 1980). Resin I was used for most of the bonding although some of the plates were bonded using a different resin.

Full scale load tests carried out before and after strengthening showed that the flexural stiffness of the deck had increased by about 11% and crack widths were reduced by 35-40% (Raithby, 1980).

Calder (1989) reported that the performance of the plates in service had been checked by tapping the steel plates to locate any areas of debonding. Up until 1987 only about 30 areas of debonding were found on the whole structure, each affecting less than a 1m length of plate. These were unlikely to have caused any appreciable loss in structural performance. One exception occurred adjacent to the joint between the two carriageways (Figure 20), where visible signs of corrosion on the external surface of one plate were observed. In this location water and chloride de-icing salts flowed down the edge of the slab and onto the plate. Tapping surveys suggested that this plate had partially debonded.

In September 1988, a length of 3.5m of corroding plate was removed from the soffit adjacent to the edge of the slab by prising it away from the concrete. There was a layer of resin still adhering to the plate along much of its length. In those areas where the plate had debonded from the resin there was heavy corrosion with a layer of corrosion product adhering to the steel surface (Figure 21). The resin still adhering to the plate was removed by soaking in methylene chloride exposing further though less severe corrosion (Figure 22). In these areas the corrosion had not developed sufficiently for the resin to become detached from the plate during removal from the bridge. The corroding areas broadly corresponded to the debonded regions. The corrosion was initiated from the edge of the plate exposed to the leakage between the carriageways. These results suggest that the water and chloride solutions had penetrated the interface between the steel and the resin and caused corrosion of the plate.

4 Performance in service

The service history of Quinton bridge, the first to be strengthened with bonded steel plates in the UK, has been monitored by TRL. Quinton interchange comprises of four pairs of bridges at Junction 3 on the M5 south west of Birmingham where the motorway crosses the A456 at a large roundabout. Each bridge carries a two-lane carriageway and hard shoulder for one half of the motorway, and there is a narrow central reservation suspended between each pair of bridges. The deck of each bridge is a voided concrete slab of variable depth which is continuous across the three spans. The length of the centre span is 27.4m and the length of the sidespans is 17m.

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Full scale load tests carried out before and after strengthening showed that the flexural stiffness of the deck had increased by about 11% and crack widths were reduced by 35-40% (Raithby, 1980).

Calder (1989) reported that the performance of the plates in service had been checked by tapping the steel plates to locate any areas of debonding. Up until 1987 only about 30 areas of debonding were found on the whole structure, each affecting less than a 1m length of plate. These were unlikely to have caused any appreciable loss in structural performance. One exception occurred adjacent to the joint between the two carriageways (Figure 20), where visible signs of corrosion on the external surface of one plate were observed. In this location water and chloride de-icing salts flowed down the edge of the slab and onto the plate. Tapping surveys suggested that this plate had partially debonded.

In September 1988, a length of 3.5m of corroding plate was removed from the soffit adjacent to the edge of the slab by prising it away from the concrete. There was a layer of resin still adhering to the plate along much of its length. In those areas where the plate had debonded from the resin there was heavy corrosion with a layer of corrosion product adhering to the steel surface (Figure 21). The resin still adhering to the plate was removed by soaking in methylene chloride exposing further though less severe corrosion (Figure 22). In these areas the corrosion had not developed sufficiently for the resin to become detached from the plate during removal from the bridge. The corroding areas broadly corresponded to the debonded regions. The corrosion was initiated from the edge of the plate exposed to the leakage between the carriageways. These results suggest that the water and chloride solutions had penetrated the interface between the steel and the resin and caused corrosion of the plate.
A short length of plate which showed no evidence of external corrosion or debonding was removed from the end of the adjacent plate at the same time for comparison with the corroding plate. The resin was firmly bonded to the steel and after removal by soaking in methylene chloride only very slight isolated speckles of corrosion were observed. These were similar to those seen on a section of plate removed from the Quinton bridges in 1979 (Lloyd and Calder, 1982).

Hutchinson (1996) has reported that tapping surveys have shown that the total area of the plates that were considered to be suspect had increased from 0.5% of the whole area initially to about 1.5% in 1993. This included the area which was replaced in 1988 where debonding had been caused by salt laden water washing over the steel plate (see above). This same area exhibited further corrosion in January 1995, necessitating replating.

Hutchinson (1996) analysed 16 cores through the steel plates removed from two side spans of the North bridge in 1995 (after 19 years in service). Eight of the cores were 100mm in diameter and were taken through a double layer of plate. These were subsequently sawn into thick adherend shear (TAST) test specimens 25mm long with a 10mm overlap. The other 8 cores were 50mm in diameter were taken through a single layer of plate. These were used for visible light and scanning electron microscopy. X ray photoelectron microscopy was also used to examine some of the steel surfaces. Cores with both the adhesives used for the plating were included in the analysis.

The analysis showed that the bulk mechanical properties of both adhesives were satisfactory although there were some mixing and application problems. The bondline thicknesses were very variable (steel/steel and steel/concrete thicknesses varied between 0 and 1.5mm, and 0.5 and 5mm respectively). In some areas the adhesive was completely missing. The bond between the concrete and the adhesive was satisfactory; all failures at the adhesive/concrete interfaces were within the concrete itself. Some light brown speckling of the surface of the steel was found in places although this did not seem to affect the shear strength of the joint.

5 Discussion

There was virtually no corrosion on the 0.5m long control beams plated with resin I and stored in the dry laboratory environment. Even after 19 years, the failure mode was such that the plates broke away from the concrete leaving the resin-steel and resin-concrete interfaces almost intact. This demonstrates that the shear strength of interfaces between the steel and the resin, and the concrete and the resin was greater than that of the concrete. After natural exposure, separation occurred to some extent at the steel-resin interface and in all cases the debonded steel had corroded. This indicates that corrosion had reduced the strength of the resin-steel interface and explains why the control specimens failed, on average, at slightly higher loads. However, this difference was small and had little effect on the overall structural performance of the specimens as tested.
In the case of the 3.5m beams plated with resin I, the adhesion between the concrete and resin was sufficient to maintain the bond during exposure under load. By contrast, there was loss of adhesion between resin II and the concrete after only 3 months of exposure under load. This may be related to the lower modulus of resin II (2.3 kN/mm²) compared with resin I (8.4 kN/mm²) which would have generated higher strains at the ends of the plate, with increased risk of subsequent debonding. After 8 years exposure, there was some debonding of plates exposed with resin II unstressed; this is attributed to moisture penetrating the interface of the steel and the resin and causing corrosion of the steel. After a further 4 years, these plates had become completely detached from the concrete and their surfaces were completely corroded.

Department of Transport Advice Note BA 30/94 (1994) gives a performance specification for the adhesive which is based on a series of performance tests developed by Mays and Hutchinson (1988). Adhesives that fulfil the requirements are deemed to be suitable for steel plate bonding applications. Requirements for mixing, placing and curing the adhesive as well as moisture resistance and mechanical properties are specified. The specification requires the flexural modulus to be within the range 4-10kN/mm². The modulus of resin II (2.3kN/mm²) is outside this range and would not now be considered suitable for plate bonding applications. There is no information available on the performance of resin II in the other tests, but it is reasonable to assume from the results of the tests on the 3.5m beams exposed at Eastney that its moisture resistance was also inadequate. Resin I fully met all the requirements of the specification. The results of these long term exposure tests therefore support the performance tests specified in BA 30/94.

The loading tests to failure showed that the mild corrosion at the interface of resin I with the steel did not adversely affect the structural performance of the beams and that the bond was sufficient to utilise the full resistance of the external reinforcement. Beams strengthened using this resin failed either by yielding of the reinforcement or by brittle shear failure of the concrete rather than failure of the adhesive bond.

Resin II only provided adequate structural performance for 1 year of exposure when the plate and resin were not under stress. Although there was some corrosion of the plate at the interface with the resin after 1 year there was still sufficient adhesion to give as good a structural performance as obtained for the beams plated with resin I. After 8 years of exposure however, the corrosion had developed to such an extent that the bond was insufficient to transmit appreciable load into the plate during the loading tests. This resulted in ductile failure at loads only marginally higher than the failure loads of the unplated beams. Exposure under load proved to be a much more rigorous test; bond failure between resin II and the concrete occurred after only 3 months exposure.

Observations of the plates on the Quinton bridges over a period of nearly 20 years confirm that resin I appears to be performing satisfactorily in service except where it was exposed to de-icing salts. This was supported by the

detailed examination of 16 cores removed from the bridge in 1995.

6 Conclusions

1 Only one of the two resins tested had adequate durability. This resin was used on the Quinton bridge. The other resin did not fulfil the requirements of the performance specification in the Department of Transport Advice Note for plate bonding.

2 The plates on all the specimens (both loaded and unloaded) assembled using resin I remained fully bonded to the concrete during exposure even after nearly 20 years.

3 Exposure for up to nearly 20 years only had a marginal effect on the failure loads of the 0.5m and 3.5m long beams plated with resin I, despite some corrosion of the steel at the interface with the resin. The corrosion was caused by the ingress of moisture through microcracks in the resin and the concrete. There was also evidence on some of the 0.5m long specimens that corrosion had spread from the edge of the plate by ingress of moisture at the interface between the resin and the steel.

4 The degree of corrosion was significantly reduced by coating the steel with an epoxy primer paint prior to assembly of specimens. In this case, the primer did not adversely affect the structural performance.

5 The bond between resin II and the concrete failed on the specimens exposed with the resin under stress after only 3 months of exposure. After 8 years there was some debonding of the plates on beams which were exposed with resin II unstressed. After 12 years, the plates had become completely detached from the concrete. This was attributable to ingress of moisture between the steel and the resin at the edge of the plate.

6 The structural performance of the 3.5m long beams exposed with resin II unstressed was not adversely affected by corrosion that occurred during the first year of exposure. The corrosion was caused by the ingress of moisture through the edges of the plate at the interface with the resin and after 8 years of exposure, continuing corrosion caused debonding of the plate during the loading tests and hence premature failure of the beams.

7 The durability of external reinforcement bonded with resin I has been satisfactory on the Quinton bridge since its installation in 1975 except where the plates have been exposed directly to leakage of water and chlorides from the carriageway above. Otherwise, only a small amount of debonding has been detected. Tapping the surface with a steel hammer has proved to be a useful method for detecting debonding.

This research programme confirms that satisfactory long term performance of bonded steel plates can be obtained provided care is taken in the choice of adhesive. The resin that was used on the Quinton bridge, which fulfilled the requirements of the Advice Note on plate bonding, was durable both in service and in the exposure trials. Steel
plate bonding can therefore be recommended as a viable and economic technique for providing additional strength to concrete bridges.

7 References


Abstract

External steel plates bonded to the concrete surface using structural epoxy adhesives are sometimes used to strengthen existing structures. The durability of such systems has been investigated by TRL, both by exposure trials and by monitoring the performance of a strengthened bridge. Unreinforced 500mm long concrete beams and reinforced 3.5m long concrete beams were externally strengthened with mild steel plates, and exposed to high rainfall, industrial and coastal environments. The performance of two different resins was investigated (resin I: Ciba Geigy XD800 and resin II: Colebrand CXL 83). The beams were maintained under load during exposure. Load tests on the 0.5m beams were carried out after 1, 2, 10 and 19 years exposure and on the 3.5m beams after 1, 8, and 16 years exposure. The plates were then removed from the beams and examined. Some light corrosion was observed on the plates bonded with resin I but this did not adversely affect the structural performance of the plated beams. There was less corrosion on the plates which had been coated with an epoxy primer paint prior to assembly of specimens. The performance of beams plated with resin II was less satisfactory. Corrosion at the interface between the resin and the steel resulted either in failure during exposure or premature failure during the load tests. The durability of the bond with resin I has proved to be satisfactory on the Quinton bridge since plate bonding was carried out in 1975 except where the plates have been exposed directly to leakage of water and chlorides from the carriageway above. Tapping the surface with a steel hammer has proved to be a useful method for detecting debonding. At Quinton little debonding was identified during the most recent survey in 1993.

Related publications

RR191  Exposure tests on 3.5m externally reinforced concrete beams. The first 8 years  
by A J J Calder. 1989 (price code B)

RR129  Exposure tests on externally reinforced concrete beams - performance after 10 years  
by A J J Calder. 1988 (price code B)

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