THE BEHAVIOUR OF PERMANENT FORMWORK

by C Beales and D A Ives

The views expressed in this report are not necessarily those of the Department of Transport

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<td></td>
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<td></td>
<td></td>
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</tbody>
</table>

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THE BEHAVIOUR OF PERMANENT FORMWORK

ABSTRACT

This report describes an investigation and assessment of the four main types of permanent formwork. Steel formwork and ‘Omnia’ precast concrete planks were load tested in the 1970s at Imperial College; glass reinforced cement (GRC) and glass reinforced plastic (GRP) panels were load tested at TRRL in the 1980s. All types were inspected on bridges built around 10 to 20 years ago.

The results of these investigations suggest that profiled steel permanent formwork is generally strong and durable but is susceptible to corrosion at cut edges. Panels with shear connectors indented in the troughs are not recommended.

Loading tests on ‘Omnia’ planks generally gave satisfactory results though some fatigue failures of the welded lattice occurred. Cracks were observed in the ‘Omnia’ planks on some of the bridges inspected. The reason for this cracking is unclear and further investigation is recommended.

GRP formwork performed well in laboratory fatigue tests but modifications to the manufacturing process are needed to ensure the corrosion protection of the steel reinforcement in the panels.

Single skin GRC formwork panels are considered to be safe to use but problems inherent in multi-skinned panel designs have proved difficult to correct. Hence the use of multi-skinned GRC formwork is not recommended.

1 INTRODUCTION

Permanent formwork has been used on UK bridges for about twenty years to provide a convenient means of supporting the in-situ concrete during the casting of the bridge deck. The formwork spans between the main girders, requires no supporting ‘falsework’ and remains in position for the lifetime of the bridge.

The use of permanent formwork is described in BS 5400: Part 5 [BSI, 1979]. The following materials are considered suitable for use:

(i) reinforced or prestressed precast concrete;
(ii) precast concrete acting compositely with a steel lattice which is eventually embedded in the overlying in-situ concrete (normally referred to by the trade name of ‘Omnia’ planks);
(iii) profiled steel sheeting;
(iv) reinforced plastic or asbestos cement sheeting or similar.

Timber sheets (normally plywood or chipboard) have been used between ‘M’ beams or in similar construction but are not now considered acceptable as a permanent formwork material [Department of Transport, 1989]. They are not considered in this report.

The code specifies that the formwork may be considered either structurally participating with the overlying in-situ concrete slab or structurally non-participating. Materials described in (iv) may only be considered in the latter category.

Precast concrete units described in (i) and (ii) must comply with the relevant clauses of BS 5400: Part 4 [BSI, 1984], particularly with respect to cover to the reinforcement and crack widths.

Problems have occurred during construction with certain types of permanent formwork and, by 1985, there was also concern about their long term durability and compliance with current codes. The Department of Transport restricted the use of steel, glass reinforced plastic (GRP) and glass reinforced cement (GRC) permanent formwork to situations where a hazard would not exist if failure occurred.

In May 1985 TRRL was asked to conduct fatigue tests on various types of formwork and to carry out a survey of in-service behaviour. This work was underway when, in July 1986, a failure occurred at a bridge in Cwmbran, South Wales. A GRC permanent formwork panel broke away from the soffit of the bridge and fell onto the dual carriageway below, fortunately without causing injury. Another failure occurred at Fiddler’s Elbow, Mid-Glamorgan, in July 1987. Other examples of cracked formwork have since been found.

This report describes the types of permanent formwork commonly used and (for three types) the manufacturing procedures. The results of an in-service survey of 16 bridges are summarised together with four other bridge inspections, the reasons for the failures at Cwmbran and Fiddler’s Elbow are discussed. Fatigue tests carried out at TRRL on GRP and GRC panels are described together with corrosion tests on GRP specimens. The main conclusions from earlier tests at Imperial College on ‘Omnia’ planks and steel formwork are also discussed. The report concludes with a summary of each type of formwork with recommendations for their future use.
2 TYPES OF PERMANENT FORMWORK

2.1 PROFILED STEEL SHEETING

The panels are made from pressed steel sheet, often galvanised and coated with plastic on the outer surface. Some of the panel types have indentations in the webs of the trough sections to increase the shear connection between the panel and the concrete slab. Typical panels are illustrated in figure 1 and shown in use on the M23 in figure 2.

(a) Profiled steel

![Profiled steel](image)

(b) Profiled steel with indented shear connectors

![Profiled steel with indented shear connectors](image)

Fig. 1 Typical steel permanent formwork panels
2.2 GLASS REINFORCED PLASTIC (GRP) PANELS

The panels consist of a GRP/sand filled resin/GRP sandwich, typically 6 mm thick, stiffened by rectangular section steel bars. The spacing and size of the bars varies according to the span, as shown in figure 3.

### STANDARD PANEL TYPES

<table>
<thead>
<tr>
<th></th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
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<th>8</th>
<th>9</th>
<th>10</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>t</strong> = thickness of grp</td>
<td>mm</td>
<td>6</td>
<td>6</td>
<td>6</td>
<td>6</td>
<td>6</td>
<td>6</td>
<td>6</td>
<td>6</td>
<td>6</td>
</tr>
<tr>
<td><strong>h</strong> = overall height including rib</td>
<td>mm</td>
<td>38</td>
<td>38</td>
<td>38</td>
<td>38</td>
<td>50</td>
<td>50</td>
<td>63</td>
<td>63</td>
<td>63</td>
</tr>
<tr>
<td><strong>l</strong> = length (maximum)</td>
<td>mm</td>
<td>762</td>
<td>914</td>
<td>1220</td>
<td>38</td>
<td>1371</td>
<td>1600</td>
<td>1905</td>
<td>2210</td>
<td>2615</td>
</tr>
<tr>
<td><strong>cs</strong> = clear span (maximum)</td>
<td>mm</td>
<td>686</td>
<td>838</td>
<td>1143</td>
<td>1295</td>
<td>1524</td>
<td>1830</td>
<td>2134</td>
<td>2438</td>
<td>2744</td>
</tr>
<tr>
<td><strong>b</strong> = breadth</td>
<td>mm</td>
<td>1220</td>
<td>1220</td>
<td>914</td>
<td>838</td>
<td>838</td>
<td>838</td>
<td>838</td>
<td>838</td>
<td>686</td>
</tr>
<tr>
<td><strong>w</strong> = weight</td>
<td>kg/m²</td>
<td>15</td>
<td>16</td>
<td>17</td>
<td>19</td>
<td>24</td>
<td>27</td>
<td>30</td>
<td>33</td>
<td>36</td>
</tr>
<tr>
<td><strong>c</strong> = centres of ribs</td>
<td>mm</td>
<td>229</td>
<td>190</td>
<td>165</td>
<td>152</td>
<td>152</td>
<td>127</td>
<td>127</td>
<td>102</td>
<td>102</td>
</tr>
<tr>
<td><strong>p</strong> = position of end ribs</td>
<td>mm</td>
<td>38</td>
<td>38</td>
<td>45</td>
<td>38</td>
<td>38</td>
<td>38</td>
<td>38</td>
<td>38</td>
<td>38</td>
</tr>
<tr>
<td><strong>n</strong> = number of ribs</td>
<td>No.</td>
<td>6</td>
<td>7</td>
<td>7</td>
<td>6</td>
<td>6</td>
<td>7</td>
<td>7</td>
<td>7</td>
<td>7</td>
</tr>
</tbody>
</table>

Fig. 2 Profiled steel permanent formwork in use on Cooper's Hill Viaduct, M23

Fig. 3 Typical GRP permanent formwork panels
2.3 GLASS REINFORCED CEMENT (GRC) PANELS

The panels are made from a sprayed matrix of sand/cement and chopped glass fibres. They range from simple flat sheets for spans up to 0.5 m to more complex triple skinned trough sections for longer spans up to 4 m. Polystyrene void formers are often employed in the larger panels, which may also contain steel reinforcement in the bottom (tension) skin. Some examples of the types of panel are given in figure 4.

<table>
<thead>
<tr>
<th>Type</th>
<th>Economic span range — m</th>
<th>Typical panel cross section (mm)</th>
<th>Typical dimensions (for a 175mm slab)</th>
<th>Longitudinal section showing beam/panel details (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Flat sheet up to 0.5</td>
<td><img src="image1.png" alt="Diagram" /></td>
<td>Span: 0.5, A: 15, B: 25, C: 40</td>
<td><img src="image2.png" alt="Diagram" /></td>
</tr>
<tr>
<td>2</td>
<td>Single corrugated 0.5—1.5</td>
<td><img src="image3.png" alt="Diagram" /></td>
<td>Span: 0.5, 1.5, A: 6, B: 8, C: 31</td>
<td><img src="image4.png" alt="Diagram" /></td>
</tr>
<tr>
<td>3a</td>
<td>Double skin flat soffit 1.5—2.5</td>
<td><img src="image5.png" alt="Diagram" /></td>
<td>Span: 1.5, 2.5, A: 14, B: 18, C: 30</td>
<td><img src="image6.png" alt="Diagram" /></td>
</tr>
<tr>
<td>3b</td>
<td>Double skin ribbed soffit (steel reinforced) 1.5—4.5</td>
<td><img src="image7.png" alt="Diagram" /></td>
<td>Span: 1.5, 4.5, A: 18, B: 18, C: 50</td>
<td><img src="image8.png" alt="Diagram" /></td>
</tr>
<tr>
<td>4</td>
<td>Triple skin 2.5—4.0</td>
<td><img src="image9.png" alt="Diagram" /></td>
<td>Span: 2.5, 4.0, A: 16, B: 20, C: 36</td>
<td><img src="image10.png" alt="Diagram" /></td>
</tr>
</tbody>
</table>

Fig. 4 Typical GRC permanent formwork panels
2.4 PRECAST CONCRETE PLANKS

Ordinary reinforced or prestressed precast concrete slabs were not considered in this research programme.

‘Omnia’ planks are precast concrete panels reinforced by a welded lattice which projects into, and provides a mechanical linkage with, the concrete deck so that the precast and in-situ elements combine in composite action to form a solid slab. A typical ‘Omnia’ plank is shown in figure 5.

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Fig. 5 Typical ‘Omnia’ plank
3 MANUFACTURING PROCEDURES

The authors observed the manufacture of GRC, GRP and 'Omnia' permanent formwork panels. The following are brief descriptions of and comments on the manufacture of typical panels.

3.1 GRC FORMWORK

The panel being manufactured was similar to that shown in figure 4, type 3b.

A smooth sand/cement slurry was sprayed into a steel mould together with chopped glass fibres sprayed from a separate nozzle. Both materials impacted the mould together and formed a layer of glass fibre reinforced cement approximately 8 mm thick. The GRC was rolled and compacted with hand rollers.

A welded steel mesh was laid in the soffit of the troughs and covered by a second spraying of GRC material. A vibrating trowel was used to compact the GRC around the mesh.

Polystyrene spacer blocks (about 100 mm cube) were laid in the bottom of the troughs. A 50 mm thick sheet of polystyrene was placed on the blocks to enclose the troughs (this panel did not have the full-depth void former illustrated in figure 4).

A further layer of GRC was sprayed over the polystyrene to form the top skin of the panel.

The following comments are offered:

(i) Many of the operations and quality control procedures were unsophisticated and heavily dependent on the diligence of the operators. This is not to say that they were ineffective.

(ii) There was some indication of slumping as the GRC was rolled up the webs of the troughs. This could be caused by the cement slurry being too fluid.

(iii) The spacer blocks used to position the polystyrene sheets were a recent introduction. Previously it had been possible to force the polystyrene too far down into the trough thereby indenting the GRC webs of the trough. Despite the introduction of spacers, it is considered that this possibility remains.

3.2 GRP FORMWORK

A glass fibre mat was laid over a mahogany mould and saturated with pure resin. Resin bulked out with fine sand was poured over the mat and roughly screeded to a depth of 6 mm.

Steel bars were pushed into slots in the mould pulling the resin soaked mat and the sand filled resin with them.

The sand filled resin was screeded to the required thickness. A second glass fibre mat was laid over the panel and pure resin was applied by a roller to give a tough finish to the surface of the panel.

The following comments are offered:

(i) The steel stock was kept in a storehouse in good corrosion free condition. The steel was not cleaned before use.

(ii) There was no positive means of locating the steel bar in the slot in the mould. This could result in low cover at the top, bottom, sides or ends of the steel.

3.3 'OMNIA' FORMWORK

A preformed welded steel lattice formed the basis of the reinforcing cage; additional longitudinal reinforcement specified by the customer (the bridge designer) was attached to the lattice using stainless steel spacers and a locating jig.

Concrete spacer blocks were tied to the reinforcement to ensure adequate concrete cover to the bars. The reinforcement cage was placed in a steel mould the top side of which was hinged. This produced the upper sloping face of the plank and could be hinged open to allow the extraction of the completed unit. The moulds were placed on a vibrating table to compact the concrete.

Overall, the manufacturing procedures and quality control were good.

4 INSPECTION OF BRIDGES

At the start of the research programme inspecting engineers were commissioned to inspect twelve bridges. Permanent formwork had been used in the construction of all the bridges, four each with steel, GRC and GRP panels. The bridges were selected where possible, to cover a range of panel spans, bridge types (motorway viaduct, minor road overbridge) and geographical locations. In some cases the choice was restricted by a limited number of examples and site access considerations. Most of the bridges were between ten and twenty years old.

The inspectors were to search for signs of cracking or debonding of the formwork and separation at the joints. They were also to look for evidence of corrosion and mechanical damage. They were to measure, at selected places, the bond strength, the thickness of the formwork and the size of areas with lack of adhesion. Samples
of the in-situ concrete were to be taken to
determine chloride levels.

Later, a further survey was carried out on four
bridges with 'Omnia' formwork. The specification
for the inspection was amended to include
measurement of cover to the reinforcement and
coring at the joint between planks to determine
whether cracking had occurred in the in-situ slab
at this point.

The sixteen bridges examined are listed in Table 1.
The results of the inspections are summarised
below.

### 4.1 STEEL FORMWORK

The steel panels were typically just over 1 mm
thick and had a profiled section (without indented
shear connectors) similar to that illustrated in
figure 1a. All appeared to have been galvanised.
The exposed side was coated with a polyester
material about 100–200 micrometres thick. The
coatings had a ripple finish and, overall, were in
good condition.

Some physical damage to the formwork was
observed including some minor denting on bridge
S1. There were isolated areas of corrosion

### TABLE 1

<table>
<thead>
<tr>
<th>Reference number &amp; formwork type</th>
<th>Bridge name &amp; location</th>
<th>Approximate date of construction</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1 COOPER’S HILL VIADUCT, REDHILL M23 over rail/minor road</td>
<td>1974</td>
<td></td>
</tr>
<tr>
<td>S2 WARDLEY HALL VIADUCT, SWINTON M63 over A580</td>
<td>1967</td>
<td></td>
</tr>
<tr>
<td>S3 BIRCH MILLS ROAD, ROCHDALE M62 over A6045</td>
<td>1969</td>
<td></td>
</tr>
<tr>
<td>S4 BEURSIL HEAD BRIDGE, ROCHDALE M62 over A671</td>
<td>1969</td>
<td></td>
</tr>
<tr>
<td>GRP1 MERSTHAM INTERCHANGE, REDHILL M23 over M25</td>
<td>1974</td>
<td></td>
</tr>
<tr>
<td>GRP2 ALDERBURY INTERCHANGE, SALISBURY Minor road over A36</td>
<td>1976</td>
<td></td>
</tr>
<tr>
<td>GRP3 BROAD GREEN RAIL BRIDGE, LIVERPOOL A5080 over rail</td>
<td>1975</td>
<td></td>
</tr>
<tr>
<td>GRP4 ROBIN HOOD RAIL BRIDGE, SWINTON M62 over rail/minor road</td>
<td>1968</td>
<td></td>
</tr>
<tr>
<td>GRC1 (double skin type 3a)* KENFIG VIADUCT, PORT TALBOT M4 over fields/rail</td>
<td>1975</td>
<td></td>
</tr>
<tr>
<td>GRC2 (single skin type 1)* ARUN BRIDGE, LITTLEHAMPTON A259 over River Arun</td>
<td>1973</td>
<td></td>
</tr>
<tr>
<td>GRC3 (double skin type 3a)* NEW INN BRIDGE, CWMBRAN Minor road over A4042</td>
<td>1973</td>
<td></td>
</tr>
<tr>
<td>GRC4 (triple skin type 4)* BROOK STREET VIADUCT, BRENTWOOD M25 over A12</td>
<td>1982</td>
<td></td>
</tr>
<tr>
<td>OM1 QUEENS DRIVE VIADUCT, LIVERPOOL Junction of A5058 and A59</td>
<td>1969</td>
<td></td>
</tr>
<tr>
<td>OM2 KIRKLEES VIADUCT, HUDDERSFIELD Junction 25 on M62</td>
<td>1973</td>
<td></td>
</tr>
<tr>
<td>OM3 RED LANE DYKE BRIDGE, HUDDERSFIELD Carries A640 over M62</td>
<td>1970</td>
<td></td>
</tr>
<tr>
<td>OM4 PONT-NEATH-VAUGHAN VIADUCT, NEATH Carries A465 over valley</td>
<td>1972</td>
<td></td>
</tr>
</tbody>
</table>

*see figure 4
particularly on panel edges and where cutting to size on site had damaged the protective treatment. Corrosion was apparent over an extensive area of bridge S2 and was severe in places. A higher chloride content (about 0.6% chloride ions by weight of cement) was found in the concrete deck above a selected corroded area. In the other three structures the chloride content of the in-situ concrete was very low (about 0.07% chloride ions by weight of cement).

Discs were cut from the formwork using a tank cutter. If they had remained bonded to the concrete the intention was to measure the bond strength. None remained bonded, but some very small areas of mortar were found adhering to some of the discs.

In general, apart from a few instances of physical damage, the steel formwork appeared to be in good condition. The exception was bridge S2, which had many corroded areas. This is thought to be due to a breakdown of the bridge deck waterproofing membrane allowing water to leak through the deck to the steel formwork.

4.2 GRP FORMWORK

Except where corrosion of the steel ribs was apparent, the formwork seemed to be in good condition. The panels were typically 3–5 mm thick, although one bridge (GRP1) had a panel thickness of 9–16 mm.

The steel reinforcing bars in the panels are potentially vulnerable to corrosion. Where the GRP cover was very low or permeable, as at bridge GRP2, corrosion had taken place. In this case it was probably exacerbated by road salt spray, because there was a greater incidence of corrosion above the inside lane of the dual carriageway. Corrosion of a lesser degree was noted on bridge GRP4.

There were low levels of chloride in the in-situ concrete of the deck (up to 0.14% chloride ions by weight of cement) in all the samples.

There was no bond between the concrete slab and the permanent formwork in any of the locations examined. However there is evidence that where the stiffening ribs extend upwards into the concrete, they serve to provide a mechanical key.

Some panels had been damaged and broken. There was evidence of punching damage through and into the panels by spacer blocks (bridge GRP3), and localised damage by impact from above (bridge GRP1). Instances of panels being slightly displaced, and of considerable voidage and honeycombing of the in-situ concrete, possibly due to undercompaction, were identified in several locations on bridge GRP3.

4.3 GRC FORMWORK

Multi-skinned panels were used on bridges GRC1, GRC3 and GRC4, the trough sections containing expanded polystyrene inserts. At bridge GRC3 (figure 4, type 3a) a grouted hessian fabric had been applied to the top surface of the panel. At bridge GRC4 the construction type (figure 4, type 4) did not allow a detailed examination of the upper GRC layer which delaminated during pull-off testing; there was possibly only a thin layer present.

All the panel types exhibited a degree of surface crazing on the exposed soffit. In addition, many of the multi-skinned panels were cracked, generally across the panel at mid-span. On bridge GRC3 nearly all the panels examined were cracked in this way (crack widths up to 2 mm), as were the majority of the panels in the inspected areas near the abutments at bridge GRC4.

In the survey areas the multi-skin panels, particularly on bridges GRC1 and GRC3, were found to be bowed downwards (typically 5 mm in 0.5 m) with differential edge displacements (measured from one panel to the adjacent panel) of up to 24 mm.

In most cases there was a bond between the GRC panels and the in-situ concrete. The exception was bridge GRC3 where the upper layer of the panel comprises grouted hessian. One of the panels on this bridge had fallen off and the remainder were being removed for safety. This bridge is discussed further in Section 5.1. In all other cases the GRC/concrete bond strength exceeded the internal tensile strength of the GRC panel and in one case (bridge GRC2) the bond exceeded the strength of the in-situ concrete above the panel. There were no bond failures at the GRC/concrete interface in the five tests successfully carried out.

Minor levels of construction damage were found on most of the bridges.

The chloride contents of the in-situ concrete were all at very low levels.

4.4 ‘OMNIA’ FORMWORK

The spans of the planks examined ranged from 1.32 m to 2.8 m; the widths were all about 300 mm. The planks on bridges OM2 and OM4 appeared similar to that represented in figure 5. The planks on bridge OM3 had a slightly different upper chamfer detail and on bridge OM1 they appeared to be of a significantly different section and were possibly slightly thicker. The edge thickness of the planks on bridge OM1 was about 57 mm whereas the cores from the other bridges showed thicknesses ranging from 29 mm to 40 mm with an average of about 33 mm.
The soffits of some of the planks showed surface blemishes, efflorescence or water marks. Others exhibited cracking, crazing or rust staining.

A high proportion (possibly 80%) of the apparently thicker type of 'Omnia' planks on bridge OM1 had transverse cracks. Some longitudinal cracking was also observed. In the survey areas all the planks were cracked and these cracks ranged in width from hairline to 0.3 mm. About 20% of the planks inspected on bridges OM3 and OM4 showed hairline cracking; no cracking was perceptible on bridge OM2.

The maximum observed crack widths (0.3 mm) exceed slightly the maximum permitted design crack widths (0.25 mm) for bridges under moderate exposure conditions given in BS 5400: Part 4 [BSI, 1984].

The cover to the reinforcement in the 'Omnia' planks was somewhat variable ranging from 15–25 mm (typically 20 mm) on bridge OM2 and from 12–30 mm (typically 25 mm) on bridge OM4. The cover was slightly greater, 25–40 mm (typically 30 mm) in the thicker planks of bridge OM1. It was lower (typically 15 mm) and more variable (2–20 mm) in bridge OM3 and the concrete had spalled in the area of lowest cover.

Examination of the cores showed no cracks in the in-situ concrete above the edges of the planks. The transverse cracks visible on the surface of the planks on bridges OM1 and OM3 also did not appear to penetrate the in-situ concrete.

The chloride content of both the planks and the in-situ concrete was generally low; up to 0.10% chloride ions by weight of cement. Higher chloride contents (up to 0.30%) were found in bridge OM4 which could lead to a risk of corrosion in the future.

4.5 OTHER BRIDGE INSPECTIONS

4.5.1 Gilbey Road Viaduct, Grimsby (GRC Formwork)

Gilbey Road viaduct was built in 1983 with triple skin formwork (figure 4 type 4). During construction of the deck, a panel in each of two bays collapsed during concreting. There was evidence of cracking and water retention in other panels. Some panels were subsequently drilled to release the trapped water. There was an indication that the GRC skins were thinner than designed. All remaining bays were temporarily propped to prevent further problems.

An inspection carried out in February 1987 found signs of cracking of the outer skin and water penetration in approximately 6% of the panels. The cracks extended for nearly the full length of the panels, ran parallel to the main girder at either mid-span or quarter-span, and were about 0.1–0.2 mm wide. There were also a number of panels with short, isolated cracks.

4.5.2 Cromarty Firth Bridge, Cromarty (GRP Formwork)

Cromarty Bridge carries the A9 trunk road over the Cromarty Firth. Built in 1981, it is a multi-span structure with prestressed concrete main beams and a reinforced concrete deck. Inspecting engineers were commissioned to determine the condition of the GRP permanent formwork.

A visual survey discovered extensive rust staining on the GRP surrounding the steel reinforcing bars. The use of the permanent formwork on this bridge was unusual in that the stiffening ribs protruded downwards.

Samples of the formwork revealed corrosion of the steel bars, with especially heavy rust deposits on their lower edges. Scrapings taken from the surface of the bars had an average chloride ion content of 0.04%.

The position of the bars within the GRP varied considerably. The minimum measured cover was 1 mm.

There was evidence of debonding between the formwork and the concrete deck. The small amount of vibration used to remove samples was enough to debond extensive areas of GRP.

4.5.3 Howgate Viaduct, Dumbarton (GRP Formwork)

Howgate Viaduct, built in 1975, is a four span composite bridge carrying the A82 near Dumbarton. GRP permanent formwork was used for the centre span. The bridge was inspected in May 1987 by TRRL Scottish Branch and was found to be in very good condition, with no visible sign of rust staining.

5 FAILURES IN SERVICE

5.1 NEW INN BRIDGE, CWMBRAN (GRC FORMWORK)

The bridge carries a minor road over a railway line and the A4042. In July 1986 a GRC panel fell from the bridge onto the road below. Initial examination by Gwent County Council showed that many of the panels were cracked. As a precaution, the panels over the road were removed. The panels were of the ribbed twin
spray, flat soffit type (figure 4, type 3a) with expanded polystyrene void formers in both the enclosed and open trough sections. A coat of grouted hessian had been applied at the formwork/concrete interface, presumably to restrain the polystyrene in the open trough sections. The panels spanned transversely between longitudinal steel I beams. Failure of the panel occurred through cracking at mid-span; the grouted hessian did not provide an adequate bond to the concrete deck and so the cracked panels were free to fall.

Inspecting engineers were appointed by the Welsh Office in September 1986 to carry out materials testing. It was concluded that the properties of the GRC panel samples were generally in agreement with the published values for this material although the limit of proportionality and the modulus of rupture appeared to be lower than expected.

The grouted hessian layer did not conform to panel designs given in the Concrete Society Current Practice Sheet No. 97 [True, 1985]. Although the cause of the cracking has not been confirmed it is likely that the addition of the hessian contributed to the loss of the panel from the bridge.

The cracks occurred on a level with the bottom of the polystyrene former inserted in the top of the trough during the manufacture of the panel. Measurements showed that the webs of the trough were particularly thin at this point, only 2–3 mm in places. This is consistent with observations made on one of the sectioned laboratory test specimens (see Section 6.1.3.2) and substantiates the comments made on the manufacture of similar panels (see Section 3.1). Where failure had occurred, the bottom section of the trough had fallen away leaving the stubs of the trough webs and the polystyrene in place.

Vertical cracking was also present in the webs of the troughs but few of the soffits were cracked.

The soffits of the troughs were considerably thicker than specified, up to 110 mm in one place. The design thickness was 15 mm for both the webs and the soffit of the troughs.

It is concluded that the failure of the panels was precipitated by the loss of section in the webs caused by the insertion of the polystyrene formers.

As a result of these failures, the trough sections of the panels were subsequently removed from the bridge together with the polystyrene formers and the remaining top skin pinned to the concrete deck using ‘Hilti’ nails with large washers.

6 LABORATORY TESTS

Manufacturers of steel, GRP and GRC permanent formwork were invited to supply panels for assessment at TRRL under repeated loading. A reinforced concrete slab was to be cast on the panels with span lengths covering the intermediate and maximum ranges for the type of panel.

The GRP and GRC manufacturers each originally supplied only three specimens of equal span length. Two modified GRC panels were subsequently supplied for testing following the poor performance of the first specimens.

Steel panels were not supplied and consequently this type of formwork was not tested at TRRL.

Tests were carried out at Imperial College in 1978 [Gorf and Dowling, 1978] and are summarised in section 6.2.

Tests carried out at Imperial College on ‘Omnia’ planks [Dowling and Labib, 1972] are summarised in section 6.3.

TRRL conducted corrosion trials in the laboratory on GRP panels which are described in section 6.4.

6.1 LOADING TESTS ON GRC AND GRP SPECIMENS

6.1.1 Test Specimens

The concrete slabs were designed for a stress of 235 N/mm² in the longitudinal tension
Fig. 6 GRC and GRP test specimens
reinforcement under a 100 kN load applied at the mid-span of the simply supported specimen.

The following types of specimen were supplied:

**TYPE A** Triple skinned GRC with polystyrene void formers. See figure 6a.
3 specimens at 2.5 m span.

**TYPE C** Double skinned ribbed soffit GRC with polystyrene void formers. Stainless steel reinforcing mesh in the soffits of the troughs. See figure 6b.
3 specimens at 3.5 m span.

**TYPE J** Double skinned ribbed soffit GRC with polystyrene void formers (like Type C) incorporating a high modulus polypropylene mesh in the body of the GRC material.
1 specimen at 3.5 m span.

**TYPE K** Double skinned ribbed soffit GRC with polystyrene void formers (like Type C) but with ‘Tensar’ SS20 mesh instead of stainless steel mesh in the soffit of the troughs.
1 specimen at 3.5 m span.

**TYPE D** GRP panel with 6 steel ribs. See figure 6c.
3 specimens at 1.5 m span.

6.1.2 Test Procedure

The test specimens were simply supported at the ends and loaded at mid-span through two 200 mm square rubber pads by a hydraulic actuator. The loading arrangement is shown in figure 7.

Cyclic loads of 42.5 kN were to be applied to produce a stress of 100 N/mm² in the reinforcement. However, it was necessary to maintain a minimum load of 15 kN in the hydraulic testing machine and a load cycle from 15–57.5 kN was therefore selected.

During the static incremental loading of the first type A specimen, the load-deflection curve became non-linear at a load of 50 kN and the specimen cracked. The load cycle was therefore reduced to 15–42.5 kN for subsequent tests.

Loads were cycled at between 3.0 and 7.5 Hz; the target endurance was 10 million cycles. The behaviour of the panel was monitored at regular intervals and the test stopped if extensive cracking occurred. Loads, deflections and strains were measured, together with crack widths and propagation where applicable.

Most of the GRC panels were artificially aged by immersing them for 19 days in a tank of water heated to 60°C to simulate 15 years normal life.

6.1.3 Test Results

6.1.3.1 Type A Specimens (GRC)

All three specimens were artificially aged before testing. The surfaces of all the panels were crazed before test loads were applied.

The first specimen cracked during the initial static test at a load of 50 kN. The second specimen was overloaded during the dynamic tests (maximum load 58 kN) and it cracked at 1.25 million cycles. The third specimen was cycled between

**Fig. 7** Monitoring GRC specimen during load test
15–42.5 kN for 5 million cycles. The first significant crack developed after 0.34 million cycles; at the end of the test the crack width was 0.7 mm.

None of the specimens were tested to the target endurance of 10 million cycles because the crack widths were considered to be excessive. There was one single main crack in each of the panels at the end of the test, the crack running across the soffit of the panel and up the webs at approximately mid-span (figure 8a). Additional hairline cracking occurred in the in-situ slab.

There was some debonding between the GRC and the concrete on the sides of all the specimens but no pieces of GRC or concrete broke away during testing.

One of the specimens was sectioned after testing and the following observations were made:

(i) The thickness of the panel ranged from 9 to 15 mm in the soffit and from 13 to 23 mm in the webs.

(ii) There was extensive debonding between the panel and the slab along the edge of the specimen. Towards the centre the bond was good and showed no sign of deterioration.

6.1.3.2 Type C Specimens (GRC)

Two out of the three specimens were artificially aged before testing. Stainless steel reinforcement in the soffits of the specimens comprised 3 mm wires welded to form a 50 mm square mesh.

There was extensive surface crazing and cracks up to 0.12 mm wide in the soffits of the artificially aged specimens before any load was applied. The specimen which had not been artificially aged cracked after 0.23 million cycles.

Two of the tests were stopped before they had completed 10 million cycles because of excessive crack width in the panel (3.0 and 1.88 mm). One of the artificially aged specimens was tested beyond the target endurance to 11.35 million cycles. At the end of this test each trough had three main cracks with a maximum crack width of 0.82 mm.

At the end of each test there were between one and three cracks across the soffit of the troughs which continued vertically up the webs. At the top of the web, the cracking progressed horizontally (figure 8b).

There was no evidence of any debonding between the GRC and the concrete. No pieces of GRC or concrete broke away from the specimens during testing.

There was no apparent difference in performance between the specimens that had been artificially aged and the one that had not been aged.

One of the specimens was sectioned after testing and the following observations were made:

(i) The thickness of the panel ranged from 24 to 32 mm in the soffits of the troughs and from 9 to 23 mm in the webs.

(ii) There was a significant change in the web thickness at one location where the polystyrene former had been inserted (figure 9).

(iii) Cracks in the soffits of the troughs occurred at the location of the transverse wires of the stainless steel reinforcement mesh.

(iv) Ten failures were observed at the welds between the transverse and longitudinal wires of the stainless steel mesh. Cracks in the GRC usually also occurred at the failure sites.

(v) There were corrosion deposits on some of the exposed stainless steel mesh.

6.1.3.3 Type J Specimen (GRC)

This was similar to the type C specimens but incorporated a high modulus polypropylene mesh in the body of the GRC material in the soffit and webs of the troughs. The mesh was made up of 0.25 mm diameter strands of polypropylene forming 5 mm squares. This modification was proposed by the manufacturer to reduce the likelihood of pieces of formwork falling from the panel in the event of cracking.

The overall performance of this panel was similar to the type C specimens. The first significant crack was recorded at 4.6 million cycles and a
maximum crack width of 0.9 mm was measured at 10 million cycles.

It is not clear whether the polypropylene mesh would reduce the possibility of pieces of formwork falling from the panel since this did not happen in the tests on either the standard or the modified panels. However, such improvement is considered unlikely since the mesh broke at the location of the largest cracks.

6.1.3.4 Type K Specimen (GRC)
This was also similar to the type C specimens but with a ‘Tensar’ SS20 mesh instead of stainless steel. This mesh was made from high density polyethylene, approximately 5 mm wide by 1.5 mm thick forming 50 mm squares.

The fatigue performance of this specimen was generally poor. The first significant crack was found after only 0.075 million cycles and by the end of the test (11.3 million cycles) the crack width was 1.51 mm. Nevertheless, the Tensar mesh itself did not fracture.

6.1.3.5 Type D Specimens (GRP)
The concrete slabs of all the specimens were poorly compacted with extensive voids.

All three specimens achieved the target endurance of 10 million load cycles with no perceptible damage to the panel. Debonding occurred between the concrete slab and the panel and flexure cracks in the concrete were observed.

One test specimen was tested for an additional 1.88 million load cycles at over twice the previous maximum load with no noticeable damage to the panel.

6.2 LOADING TESTS ON STEEL SPECIMENS
Tests were carried out at Imperial College on steel permanent formwork panels with indented shear connectors in the webs of the trough sections, similar to the type shown in figure 1b. Static tests [Gorf and Dowling, 1976] on panels acting compositely with a concrete slab indicated that the static performance of the formwork was satisfactory.

In a second series of tests under dynamic loading, some fatigue failures occurred in the steel panels [Gorf and Dowling, 1978]. The fatigue cracks initiated at the lower end of the shear connectors and propagated rapidly across the bottom flange.
and up in to the adjacent web. By the end of the test the crack extended across two thirds of the width of one panel.

Insufficient data were available to classify accurately the detail or estimate the fatigue life of the formwork in service with any degree of certainty. However, conservative estimates were made which assessed the detail as Class D with a corresponding fatigue life of approximately 75 years (based on British Standard B116: Part 10 [BSI, 1975]).

6.3 LOADING TESTS ON ‘OMNIA’ PLANKS

A series of static and dynamic tests were carried out at Imperial College on single ‘Omnia’ planks [Farrell and Dowling, 1971]. Further tests were carried out on a model of a full scale bridge deck incorporating 100 ‘Omnia’ planks [Dowling and Labib, 1972].

The tests confirmed that the formwork was able to withstand construction loads and the units behaved compositely with the in-situ slab. No breakdown in bond occurred after cyclic loading, representing more than a life time of live load, had been applied.

Some fatigue failures of the welded reinforcement occurred at the connection between the diagonal stirrups and the longitudinal bars in the plank. In all cases breaks occurred in the longitudinal plank reinforcement and not in the stirrups. It was concluded that the shear connection and composite action between the plank and the in-situ slab would therefore be unaffected by these failures.

The fatigue data from these tests indicated a weld classification between Class E and F. Imperial College concluded that a possibility of fatigue failure occurring within the design life of a bridge (120 years) existed. Since the contribution of such reinforcement to the overall load carrying capacity of the slab is generally small it was suggested that it could be disregarded in design for in-service dead and live loading with no significant loss of economy.

6.4 CORROSION TESTS ON GRP SPECIMENS

Corrosion of the steel reinforcing bars of GRP formwork panels was observed on a number of the bridges inspected. Tests were carried out in the laboratory to establish the extent of the problem on standard panels and to investigate alternative systems for protecting the steel.

Two panels were tested, each containing six ribs. Some of the ribs contained untreated steel bars while others were protected with paint primer or epoxy resin.

The panels were first loaded with sand bags to represent the construction loading of a 380 mm thick concrete slab. This caused a mid-span deflection of the panels of around 2 mm; there were no visible cracks after loading.

The mid-span deflection to span ratios for the two panels were 1/285 and 1/339 immediately after the load was applied. Although these values were obtained in simulating a particularly thick bridge deck slab, it should be noted that one panel exceeded the specified limit of 1/300 four hours after the concrete is poured [Department of Transport, 1989].

The panels were sawn into strips, each containing one rib, and placed in a salt spray chamber along with a number of steel ‘control’ bars. The test pieces were exposed to a 3% sodium chloride solution at 25°C (on a 4 hour on, 20 hour off cycle) for up to eleven weeks. They were then removed and examined.

The ends of some of the steel bars were corroded where the GRP cover was inadequate. The corrosion extended only a millimetre or two along the bar. Otherwise the bars broken out from the GRP test pieces were corrosion free.

All the ‘control’ bars were corroded, the protective coatings offering only limited protection.

There was considerable variation in the GRP cover to the steel. In the worst case the thickness of GRP was only about 1 mm on one side of the bar and 12 mm on the other. A trial use of spacers, to centralise the steel bar in the rib, was unsuccessful.

Although the steel bar with only 1 mm of GRP cover was not corroded, such a thickness is considered unacceptable for use on site where rough handling could cause damage to the GRP and expose the steel. However, it may be concluded that the corrosion protection offered by the GRP is significantly greater than that from the paint primer or resin.

7 SUMMARY AND CONCLUSIONS

7.1 STEEL FORMWORK

(i) On the evidence of the four bridges inspected the steel formwork was generally in good condition. There were areas of heavy corrosion on one bridge but these are thought to be due to a breakdown of the bridge deck waterproofing membrane.
In previous work it was found that indented shear connectors, pressed into the webs of the troughs in some types of panel, initiated fatigue cracks in panels subjected to repeated loading. Calculations showed that failures could occur within the design life of a bridge. This type of panel is not recommended.

7.2 GRP FORMWORK

(i) One panel tested in the laboratory under a static load representing a 380 mm thick concrete slab exceeded the recommended mid-span deflection to span ratio limit.

(ii) Panels tested in the laboratory under repeated loading fully met the required criteria, that is, there was no apparent deterioration of the panels after 10 million load cycles.

(iii) The inspection of bridges in service showed that the panels were generally in good condition. There were, however, instances of corrosion in the steel reinforcing bars.

(iv) Corrosion tests in the laboratory showed that although resin and paint systems offered some protection to the steel bars, they may be regarded as insignificant when compared with the protection offered by the GRP itself. It is considered that untreated steel bar in clean rust-free condition is acceptable for use in these panels.

(v) The observed manufacturing methods were unacceptable with respect to the positioning of the steel bar within the rib. This could result in low GRP cover to the steel and increase the risk of corrosion. To obtain acceptable serviceability it is necessary to positively locate the steel centrally and at the required depth within the rib.

7.3 GRC FORMWORK

(i) In the case of single skin panels (flat sheet or corrugated panels not containing polystyrene) there is contact between the concrete and the GRC over the total area of the panel. The concrete/GRC bond has been found to be good. Generally, it exceeds the internal strength of the GRC panel. Although panels of this type were not tested it is considered that, even if these panels cracked in service, there would be little danger of pieces of the GRC falling from the structure.

(ii) Multi-skinned panels are deep in section (over 350 mm deep in some cases) and in service may experience large tensile strains in the bottom skin. Laboratory tests confirm that these panels have a poor fatigue performance. They crack readily across the soffit and the cracks grow vertically up the webs of the trough sections. Panels of this type have collapsed in service. Other examples of serious cracking have been found.

(iii) Welded steel mesh, inserted in the soffit of the troughs, fractured in laboratory fatigue tests.

(iv) High modulus polypropylene mesh embedded in the trough sections broke in the vicinity of the GRC cracks. It is therefore unlikely to prevent cracked pieces of panel from breaking away.

(v) Replacing the steel reinforcement with a ‘Tensar’ mesh is not considered worthwhile. The fatigue behaviour of the panel was poor although the ‘Tensar’ mesh itself did not fracture. However, this would not prevent the collapse of a trough section caused by horizontal cracking at the top of the trough webs (as at Fiddler’s Elbow).

(vi) The use of multi-skinned GRC panels is not recommended.

7.4 ‘OMNIA’ FORMWORK

(i) Previous laboratory load tests on ‘Omnia’ planks confirmed that they were able to withstand construction loading and that they behaved compositely with the in-situ slab under service live loads.

(ii) The inspection of bridges revealed instances of cracking and crazing of the ‘Omnia’ planks. Cracks up to 0.3 mm were found which exceed the maximum permitted design crack widths given in BS 5400: Part 4. Further research is needed to determine the reason for the cracking and whether the cracks stabilise or continue to propagate.

(iii) Instances of low cover to the reinforcement in the ‘Omnia’ planks were also found. Quality control in the factory producing ‘Omnia’ planks visited during this investigation was good and it is not clear how such faults could have occurred.

(iv) Fatigue failure of the welded lattice reinforcement occurred in the tests; calculations suggest that they could occur within the design life of a bridge. Such failures are not expected to affect the shear connection between the plank and the concrete slab. However, the effect of the loss of longitudinal reinforcement in the plank must be considered.

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9 REFERENCES


