Design of bridges with external prestressing: Construction and testing of a model bridge

Prepared for Quality Services Civil Engineering Division, Highways Agency

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# CONTENTS

<table>
<thead>
<tr>
<th>CONTENTS</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Executive Summary</td>
<td>1</td>
</tr>
<tr>
<td>1 Introduction</td>
<td>3</td>
</tr>
<tr>
<td>2 Design of the bridge</td>
<td>3</td>
</tr>
<tr>
<td>3 Construction of bridge</td>
<td>4</td>
</tr>
<tr>
<td>3.1 Construction sequence</td>
<td>4</td>
</tr>
<tr>
<td>3.2 Falsework</td>
<td>4</td>
</tr>
<tr>
<td>3.3 Formwork</td>
<td>4</td>
</tr>
<tr>
<td>3.4 Bearings</td>
<td>7</td>
</tr>
<tr>
<td>3.5 Reinforcement</td>
<td>7</td>
</tr>
<tr>
<td>3.6 Steel fixing</td>
<td>7</td>
</tr>
<tr>
<td>3.7 Stressing system</td>
<td>8</td>
</tr>
<tr>
<td>3.8 Deviators</td>
<td>8</td>
</tr>
<tr>
<td>3.9 Corrosion protection system</td>
<td>8</td>
</tr>
<tr>
<td>3.10 Concrete</td>
<td>8</td>
</tr>
<tr>
<td>3.11 Casting</td>
<td>9</td>
</tr>
<tr>
<td>3.11.1 Transportation</td>
<td>9</td>
</tr>
<tr>
<td>3.11.2 Placement</td>
<td>9</td>
</tr>
<tr>
<td>3.11.3 Compaction and finishing</td>
<td>9</td>
</tr>
<tr>
<td>3.11.4 Curing</td>
<td>10</td>
</tr>
<tr>
<td>4 Material properties</td>
<td>10</td>
</tr>
<tr>
<td>4.1 Concrete</td>
<td>10</td>
</tr>
<tr>
<td>4.1.1 Compressive strength</td>
<td>10</td>
</tr>
<tr>
<td>4.1.2 Tensile strength</td>
<td>10</td>
</tr>
<tr>
<td>4.2 Steel</td>
<td>10</td>
</tr>
<tr>
<td>4.2.1 Tensile strength</td>
<td>10</td>
</tr>
<tr>
<td>4.2.2 Pull-out tests</td>
<td>10</td>
</tr>
<tr>
<td>5 Instrumentation</td>
<td>11</td>
</tr>
<tr>
<td>5.1 Strain and temperature</td>
<td>11</td>
</tr>
<tr>
<td>5.2 Load</td>
<td>11</td>
</tr>
<tr>
<td>5.3 Displacement</td>
<td>11</td>
</tr>
<tr>
<td>6 Stressing the beam</td>
<td>11</td>
</tr>
<tr>
<td>7 Loading system</td>
<td>15</td>
</tr>
</tbody>
</table>
### Analysis of bridge

**9 Load test**

- 9.1 Test to cracking: 17
- 9.2 Test to failure: 18

**10 Discussion**

- 10.1 Design and construction: 19
- 10.2 Application of prestress:
  - 10.2.1 Load in tendons: 20
  - 10.2.2 Strains through section: 21
- 10.3 Application of test load:
  - 10.3.1 Cracking: 21
  - 10.3.2 Deflections: 23
  - 10.3.3 Strains: 27
  - 10.3.4 Load in tendons: 27
  - 10.3.5 Flexural strength using BD 58: 27
  - 10.3.6 Flexural strength using other codes: 27
  - 10.3.7 Shear strength: 33
  - 10.3.8 Mode of failure: 33

**11 Conclusions**: 33

**12 Acknowledgements**: 34

**13 References**: 34

**Abstract**: 35

**Related publications**: 35
Executive Summary

There has been increasing interest in recent years in the design and construction of post-tensioned concrete bridges with external unbonded tendons. To assist in the design of these structures, the Highways Agency issued a Standard and Advice Note (BD 58 and BA 58) in 1994. These documents were based on a review of existing knowledge which included recommendations on where further research was required.

To provide further information for designers TRL was commissioned to design and construct a quarter scale model of an externally post-tensioned bridge and load test it to failure in flexure. The structure chosen was a continuous two-span bridge with an overall length of 80m. It was designed at full scale by Gifford and Partners and represented a highway bridge with two spans, 32m and 48m, carrying a two lane carriageway over a motorway.

The purpose of the project was to:

- examine the procedures used to design the full scale bridge;
- monitor the construction of the model bridge, particularly positioning, fixing and aligning the deviators;
- measure the flexural capacity of the bridge and compare it with analytical predictions;
- measure the increase in tendon force and estimate the geometrical non-linearity as the beam is loaded to failure;
- measure the friction losses at the deflector and supports;
- record the behaviour of the deflectors, support diaphragms and anchorages as the beam is loaded to failure; in particular note the onset and extent of cracking.

A companion report (Daly and Jackson 1999) gives a description of the design of the full scale bridge. This report gives a detailed description of the construction and flexural load testing of the quarter scale model.

The test demonstrated that bridges of this type can be designed and built economically and that BD 58 produces a safe design in terms of flexural capacity. No damage was experienced in the deviator and anchorage zones, which are locations of particular concern when unbonded tendons are used.

As the beam was loaded to failure there was an increase in prestressing force of 15% of the initial prestress. This compares with values of between 0% and 32% for different codes.

The increase in prestressing force would have increased the capacity of the beam but it was more than offset by a loss of eccentricity as the beam was deflected; the loss of eccentricity measured at maximum load was 38.7mm which is a reduction of 21%. Thus methods which allow an increase in prestress without taking account of loss of eccentricity may lead to unconservative results. This should be taken into account in design, particularly where non-linear methods of analysis are used.

Failure was sudden and localised, and there was a lack of ductility although this may have been due, at least in part, to the form of the loading. The deflected shape of the model demonstrated the initial development of a plastic hinge at the central support. However, the extent of damage and the measured strains showed that a hinge had not fully developed so that a plastic collapse mechanism was not achieved. The limited ductility of the model has implications for designers. When re-distribution of moments is used, the designer must ensure that sufficient ductility exists.
1 Introduction

There has been increasing interest in recent years in the design and construction of post-tensioned concrete bridges with external unbonded tendons. This was given a significant boost in 1992 with the moratorium on the construction of post-tensioned bridges with internally bonded tendons as a result of problems associated with the grouting of post-tensioning ducts. It was envisaged that external post-tensioning would facilitate the inspection and replacement of the prestressing system. To assist in the design of these structures the Highways Agency issued a Standard, BD 58 (Highways Agency 1994a) and an accompanying Advice Note, BA 58 (Highways Agency 1994b) in 1994. These documents were based on existing knowledge and current practice.

To provide further information for designers TRL was commissioned to design and construct a quarter scale model of an externally post-tensioned bridge and load test it to failure in flexure. The structure chosen was a continuous two-span bridge with an overall length of 80m. It was designed by Gifford and Partners and a report describing the design process has been published separately (Daly and Jackson 1999). The objectives of the test were as follows:

- monitor the construction of the beam, particularly positioning, fixing and aligning the deviators
- measure the flexural capacity of the beam and compare it with analytical predictions
- measure the increase in tendon force and estimate the geometrical non-linearity as the beam is loaded to failure
- measure the friction losses at the deflector and supports
- record the behaviour of the deflectors, support diaphragms and anchorages as the beam is loaded to failure, in particular note the onset and extent of cracking.

Whilst it was recognised that many of the difficulties encountered during construction would be those associated with the construction of a model at reduced scale rather than a full scale bridge, it was considered that it would still be possible to obtain useful information on many aspects of the construction process.

This report presents a description of the model bridge and its construction, outlines the structural behaviour as it was loaded to failure and gives conclusions based on comparisons with theoretical analysis based on methods contained in existing design rules.

2 Design of the bridge

The bridge was designed to BD 58 (Highways Agency 1994a) and BS 5400: Part 4 (BSI 1990). A brief description of the design is given below but a more detailed description is presented by Daly and Jackson (1999).

The structure was a two-span continuous box girder bridge with an overall length of 80m. The bridge carried a two-lane carriageway with spans of 32m and 48m. It was designed for normal HA and 45 units of HB load to BD 37 (Highways Agency 1988). It was assumed to carry two lanes of traffic although it is actually narrower than most real two-lane bridges.

The design was constrained by the details of the test bed used for the load test on quarter scale model. This meant that some features, in particular the overall length and width of the bridge, were close to the minimum likely to be found in practice for this type of structure. It was therefore not necessarily the most realistic or economic form of construction but does illustrate the various clauses in the code. In addition, the designer was asked to avoid any arbitrary reserves of strength in the choice of dimensions, reinforcement size and spacing. Thus, the structure was designed ‘tight’ to the code. The general lay-out of the model deck is shown in Figure 1.

The cross-section of the model bridge is shown in Figure 2. The full scale section was developed from practical flange thicknesses combined with the minimum web thickness required to comply with the code requirement for the upper (web crushing) limit in shear. The depth was based on a realistic but quite shallow, span to depth ratio.

Figure 3 shows the lay-out of the tendons in elevation. The model was to be prestressed using normal commercially available tendons and anchorages and ducts. This necessitated the use of rather larger tendons than would normally be used in practice. The requirement in

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**Figure 1** General layout of the model bridge
BD 58 that the tendons should be replaceable without traffic restrictions would make the use of such large tendons impractical unless the prestressing system allowed individual strands to be de-stressed and replaced. The stressing system for the model bridge consisted of a series of six tendons, each of which consisted of three 12.9mm diameter super-strand.

3 Construction of bridge

3.1 Construction sequence
As the bridge consisted of only two spans, it was assumed in the full scale design that the whole structure would be cast on falsework in one go. This meant that the dead-weight was applied to a computer model of the completed deck and no detailed analysis of the construction stages was required. For the model, it was appropriate to carry out construction in the following sequence:
- fix all steelwork for bottom slab, deviators, diaphragms and anchorages, and vertical steel for walls: cast bottom slab;
- fix longitudinal steelwork for walls: cast walls and deviators;
- fix top steel: cast top slab and anchorages.

Figures 4 to 9 show the various stages of the construction process.

3.2 Falsework
The falsework used to support the formwork and deck during construction comprised a scaffolding frame. Four longitudinal poles were used to support the main timbers for the formwork and the frame was designed to limit the deflection of the poles to less than 2mm.

The scaffold frame also supported a walkway to give access to the beam as shown in Figure 4. This was used to provide access for steel fixing as well as for pouring the concrete.

3.3 Formwork
Formwork consisting of timber panels was erected to form the soffit of the bottom flange, the outside of the walls and the base of the flange overhangs. These were supported on 100mm by 50mm transverse timbers at 450mm centres on the scaffold frame. The inside of the walls were formed using similar panels positioned after the bottom flange had been cast. These were removed when the walls were cast.

The required tolerances were as follows:
- Distance between webs: 0 to +5mm;
- Depth of walls: -3 to 0mm;
- Width of cantilever: 0 to +3mm;
- Depth of cantilever: 0 to +3mm;
- Distance between faces of walls: -5 to 0mm;
- 25mm chamfer: 0 to 2mm.

Dimensions were carefully checked after the formwork had been constructed.

The formwork panels were sealed to prevent moisture getting in and causing them to warp. The panels were also covered in plastic sheeting as an additional protection. To prevent leakage of concrete through the joints in the formwork a conventional foam seal was applied around the edges prior to casting and trimmed. Oil was applied to the formwork to facilitate striking.

Small inspection ports (approximately 150mm square) were cut into the soffit to allow the concrete to be inspected after it had been cast. This was required to ensure that the quality of the concrete was satisfactory before continuing with the next cast.

Glass reinforced plastic (GRP) panels which had been profiled to give the correct section geometry were used for the base of the top slab. They were placed transversely across the top of the box and supported on L-shaped steel brackets fixed to the inside of the walls of the box, as shown in Figure 8. A polythene sheet was placed on top of the GRP panels to prevent them from bonding with the concrete and acting compositively with the top slab.
Figure 3 Profiles of cables in model
**Figure 4** Walkway for access to the beam

**Figure 5** Reinforcement in deviators the support diaphragms

**Figure 6** Bottom slab being cast

**Figure 7** Positioning of deviators before casting walls and diaphragms
3.4 Bearings
The model deck was supported on three concrete plinths. Each support consisted of a pair of steel plates mounted on a cylindrical bearing. The centre support was fixed horizontally but was free to rotate. Sliding bearings were used at the end supports which also allowed rotation. All six bearings incorporated a shear beam type load cell to enable reactions to be measured. Figure 10 shows the centre bearing after construction of the model.

3.5 Reinforcement
Correct scaling of the prototype deck required that all reinforcement diameters be scaled 1:4. Six bars sizes were used in the full-scale structure: 40mm, 32mm, 25mm, 20mm, 16mm and 12mm diameter. The 40mm and 32mm bars were modelled using 10mm and 8mm mild steel bars, respectively, to grade 460 high yield deformed bar. The 25mm bars were modelled using 6mm indented diameter wire, and the 16mm and 12mm diameter bars were modelled using 4mm and 3mm deformed stainless steel bars, respectively.

It was not possible to obtain 5mm steel bars to model the required 20mm diameter bars. Instead, these were modelled as follows:

- In the anchor blocks and slab pairs of 20mm bars were modelled by alternate 6mm and 4mm diameter bars.
- The main span deviators were re-detailed to use only 6mm diameter bars.

3.6 Steel fixing
The reinforcement was bent to schedule using templates. The target tolerances were to exact dimensions and when checked they were found to be generally within ± 2mm.

The bottom slab and the lower steel in the walls were fixed external to the formwork in three lengths using stainless steel tie wire. The individual cages were supported over the formwork, spliced together and finally
dropped into position (see Figure 4). The fixing was in accordance with the full-size drawings. In some cases, laps were eliminated by the use of bars longer than the full scale length. Figure 5 shows the reinforcement in one of the deviators and one of the support diaphragms.

The required concrete cover to reinforcement for the full scale deck was 30mm. For the model, 7.5mm spacers were used to provide the necessary cover to a tolerance of ±1.25mm. To maintain correct positioning of the reinforcement mats, standard 35mm spacer blocks were used between the layers of reinforcement. In general, the reinforcement details set out in the full scale drawing were followed at quarter scale. In some cases, minor variations were required to facilitate the fixing of the steel. In particular, the position of some of the laps was changed to minimise reinforcement congestion. These were not considered to have any effect on structural behaviour.

In reality, the construction of the model was a time consuming operation. Most of the problems were due to working on such a small scale. Experienced steel fixers were utilised but they were not accustomed to fixing reinforcement down to 3mm diameter. In particular the scaling of tolerances posed a few problems and required close site supervision to ensure that the model realistically represented the full scale deck. Casting also required close supervision because of the congested reinforcement and the low cover to the reinforcement.

3.7 Stressing system

VSL Type A anchorages which were designed to accommodate three 12.9mm diameter super-strands were used. The tendons were housed in 40mm diameter HDPE ducting with a wall thickness of 3.5mm: Figure 8 shows the duct in position before the top slab was cast. The equipment used to stress the beam is described in section 6.

3.8 Deviators

The deviator tubes were made up of lengths of 60mm diameter mild steel tubes with a wall thickness of 4mm. Straight tubes were used as the small radii of curvature required were such that the curvature was less than 2mm over the width of the diaphragms. The deviator tubes in the support diaphragm during final positioning are shown in Figure 7.

At some locations the deviator tubes protruded into the top and bottom slabs to a depth greater than the cover to the reinforcement. This was because exactly scaled tubing was not available and the next size up had to be chosen. To overcome this problem the deviators at these locations were sliced along their length and 3mm thick steel plates were welded to the sliced surface. This reduced their overall height and enabled them to be positioned without interfering with the reinforcement.

The ends of the deviators were machined to form a bell mouth to ensure that any slight alignment errors could be accommodated and to reduce the risk of the strands/duct system bearing on them were they entered and exited the deviators.

The deviators were fixed in the correct transverse position by measuring from the web formwork. The vertical positions were fixed relative to a string-line joining the tops of the web formwork. A second string-line was then threaded along the line of each tendon in turn. It was fixed at the anchorages at the position of the top of the duct and pulled taut (see Figure 7). After positioning, the deviators were checked to ensure that the clearance between the string-line and both ends of each tube were equal. Where this was not the case the alignment of the deviators was adjusted. The deviators were fixed in their final position using tie wire and resin to prevent them from moving when the formwork was fixed. The deviators were filled with foam to prevent leakage of concrete into them.

The length of the tubes exceeded the width of the deviators so that they protruded into the formwork. The formwork on each side of the diaphragms was cut along the line of the deviators and slightly oversized holes drilled to accommodate the tubes. After the tubes had been set in position the formwork was placed around them and they were fixed in place using wedges. The gap between the tubes and the formwork was then sealed with foam.

3.9 Corrosion protection system

It had been planned to inject at least one of the polyethylene ducts with wax and examine the affect of the wax on the properties of the ducting, particularly where it was under compressive stress at the deviators. However, when a coil of ducted tendon was being manoeuvred into position ready for insertion into the bridge it toppled over and snapped in a brittle manner. This was attributed to the fact that the ducting was stored outside for a considerable period before the test which may have caused the HDPE material to embrittle. The brittleness was accentuated by the low temperature (-2°C) during installation. The breaks in the duct were taped together before it was threaded through the bridge. This was not considered important for the model as long term protection against corrosion was not required.

The brittleness of the ducting cast some doubt on the validity of trying to determine whether the wax would cause the ducting to embrittle as it had already appeared to have lost some of its ductility. It was therefore decided not to proceed with wax injection.

3.10 Concrete

The requirements for the concrete were as follows:

- 28 day strength 60N/mm²;
- high workability, able to flow through the dense reinforcing mesh without segregating;
- retention of workability for at least 30 minutes;
- maximum aggregate size 6mm.

A series of trial mixes was carried out using Thames Valley aggregate. The mix was used to cast the bottom slab and walls. However an examination of the soffit showed that the concrete had not flowed around the bottom layer of reinforcement and was badly honeycombed. The concrete was consequently removed by water jetting. This damaged
both the formwork which had to be replaced and the reinforcing cage which had to be removed from the beam and repaired. The height of the timbers used to replace those damaged by the water jetting was 5mm less than those used initially. This had the affect of increasing the cover to the bottom reinforcement by 5mm, thus increasing the overall depth of the section and the second moment of inertia. It also provided partial dead weight compensation (discussed in section 7) and made it easier to compact the concrete around the reinforcement.

After the unsuccessful attempt to cast the bottom slab further trials were undertaken to develop a mix with the required properties. In addition a rigorous procedure was adopted for mixing, transporting and placing the concrete (Section 3.11). The mix selected was as follows:

- 480kg/m³ cement;
- 977kg/m³ aggregate (maximum size 6mm);
- 737kg/m³ sand;
- 192 litres of water (w/c ratio 0.4);
- 3.84 litres (0.8% by weight of cement) Superplasticiser (Sikament 10).

The concrete was batched in six bag mixes and the water content of the mix was controlled by adding water until a target slump was achieved. Superplasticiser was then added and slump measured again. The target slumps were as follows:

- Target slump (unplasticised) 60mm (± 10mm)
- Target slump (plasticised) collapse (> 200mm)

The actual amount of water required depended on the moisture content of the aggregate. To reduce any variability during mixing, aggregate from the bottom of the hoppers was discharged and discarded at the start of batching. This material was likely to have suffered water loss during transport. The cylinders were minimised.

3.11 Casting
Casting was scheduled according to the availability of resources and weather conditions, temperatures below 0°C having to be avoided during the casting and curing period to prevent damage. The beam was cast in four sections over a period of 5 weeks in the following sequence:

- bottom slab;
- webs, pier diaphragms, and back-span deviators;
- main-span deviators;
- top slab and anchorages.

Stressing began just under three weeks after the top slab and anchorages had been cast and the beam was load tested five weeks after stressing was completed.

3.11.1 Transportation
The mixing plant was about three hundred metres from the beam and there was concern that transporting the concrete from the mixer to the beam would affect its properties by causing it to segregate or by loss of water from the concrete bucket used to carry the mix. It would also increase the time between mixing and placing the concrete.

A transportation trial was undertaken using a 0.26m³ batch with an unplasticised slump of 50mm. Plasticiser was added (0.75% Sikament 10) to give a cohesive mix which produced a collapse slump with no evidence of segregation. The plasticised mix was then discharged from the mixer directly into a concrete bucket and transported to the model bridge site. Water seepage from the bucket was monitored and care was taken to ensure that any losses were minimised.

It arrived at the beam 22 minutes after mixing and the lower layer of concrete was discharged into a concrete cylinder mould and the upper layer of concrete was discharged into a second mould. The concrete in the first cylinder was noticeably stiffer than the second presumably due to loss of water during transportation. The cylinders were sliced after three days but there was no evidence of segregation. Slump tests 27 minutes and 42 minutes after mixing produced a collapse slump.

Four 150mm cubes were cast, two at the mixer and two at the beam sites and the 28 day compressive strengths were 63.9 and 64.5N/mm², and 63.6 and 62.6N/mm², respectively.

The trial was considered a success and the same procedure for transporting the concrete was used when the beam was cast. The dosage of superplasticiser was increased slightly to 0.8% as that was the minimum dosage recommended by the supplier.

3.11.2 Placement
On arrival at the beam site the first 20 litres of concrete from the bottom of the bucket were discarded as this material was likely to have suffered water loss during transportation. A slump test was performed before placement of concrete in the beam was allowed. If the material failed to achieve a collapse slump (<200mm) it was discarded. When the required slump was achieved the remaining concrete was discharged directly from the bucket to which it was required.

The turnaround was quicker than in the trial and concrete was normally available for placement within 10 to 15 minutes of batching. This was acceptable as trials had shown that the mix would not lose workability within half an hour. For concrete that was placed after 30 minutes additional slump tests were taken to check that the slump was still >200mm.

The concrete was spread out over the area of slab such that any variations in a batch were dispersed within the beam (see Figure 6). Each batch covered approximately 3 metres of bottom slab.

3.11.3 Compaction and finishing
The slab was surcharged with uncompacted concrete and then vibrated with one inch vibrating pokers at a maximum of 400mm spacing. Particular attention was given to areas where access was difficult such as the base of walls and deviators, and areas where the reinforcement was heavily congested such as the anchorage zones and the deviators (Figures 5 and 6).
The concrete was batched in a continuous operation: this ensured that subsequent batches were available for placement immediately after placement of the previous batch. Where a new batch was placed against a previous batch vibration of the overlap zone was carried out to ensure that cold joints were not created.

The intention had been to screed the bottom slab using a drop screed from the level rails and then float it. In the event it was levelled using battens with spacings of the correct height resting on the reinforcement cage. The method of levelling the top slab is shown in Figure 9.

3.11.4 Curing
Care was taken to ensure that casting was scheduled to avoid freezing temperatures during the curing period. Sub-zero temperatures did occur within 24 hours of casting the walls of the model, but the temperature gauges within the concrete section indicated that temperatures here did not fall below 0°C. The bridge was covered with polythene sheet for 7 days after each cast. Ambient conditions were such that humidity remained high throughout the curing period.

4 Material properties

4.1 Concrete

4.1.1 Compressive strength
Four 100mm cubes were taken from each batch of concrete for compressive strength tests at 7 and 28 days (2 at each interval). Pairs of cubes were also left to cure under polythene at the beam location. The strengths of the cubes are summarised in Table 1.

<table>
<thead>
<tr>
<th>Location</th>
<th>Age at test (days)</th>
<th>Strength (N/mm²)</th>
<th>Average</th>
<th>Characteristic</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bottom slab</td>
<td>28</td>
<td>70.5</td>
<td>64.0</td>
<td></td>
</tr>
<tr>
<td>Webs, pier diaphragms &amp;</td>
<td></td>
<td>**</td>
<td></td>
<td></td>
</tr>
<tr>
<td>back-span deviators</td>
<td>28</td>
<td>74.0</td>
<td>71.5</td>
<td></td>
</tr>
<tr>
<td>Main-span deviators</td>
<td>28</td>
<td>60.0</td>
<td>59.5</td>
<td></td>
</tr>
<tr>
<td>Top slab</td>
<td>57¹</td>
<td>76.5</td>
<td>68.5</td>
<td></td>
</tr>
<tr>
<td>All</td>
<td></td>
<td>73.5</td>
<td>64.5</td>
<td></td>
</tr>
<tr>
<td>Cured adjacent to beam</td>
<td>2</td>
<td>76.0</td>
<td>69.5</td>
<td></td>
</tr>
</tbody>
</table>

¹ Cubes tested on day of load test
² Cubes from all components, tested on day of load test

4.1.2 Tensile strength
Six concrete cylinders 200mm long by 150mm diameter were cast and tested in accordance with BS 1881: Part 117: 1983, Testing concrete Part 117, Method for determination of tensile splitting strength. In addition four cubes were also cast and their compressive strengths measured. The results obtained are given in Table 2.

<table>
<thead>
<tr>
<th>Age when tested (days)</th>
<th>Average (N/mm²)</th>
<th>Characteristic (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cylinder splitting</td>
<td>28</td>
<td>4.3</td>
</tr>
<tr>
<td>Compressive strength</td>
<td>28</td>
<td>64.5</td>
</tr>
</tbody>
</table>

4.2 Steel

4.2.1 Tensile strength
The reinforcing bars used had properties which were significantly different than conventional reinforcement. It was important to determine the actual strengths, so that the subsequent analysis would take account of these differences. Tests were carried out to determine the ultimate tensile strengths and elongation in accordance with EN 10 002-1: 1990. The properties are shown in Table 3.

Also shown in Table 3 are the results of tensile tests carried out on individual wires of the prestressing strand.

4.2.2 Pull-out tests
Pull-out strength tests were carried out to confirm that the bond between the smaller diameter reinforcing bars and the concrete used in the beam met the code requirements. Tests were undertaken on 3mm, 4mm and 6mm diameter bars embedded in 150mm by 150mm by 500mm concrete prisms. The prisms were reinforced with 6mm diameter ribbed bars in each corner with stirrups at 100mm centres along the length of the prism. The bars to be tested were positioned along the centre of each prism.

For each bar size a specimen was cast with a bond length calculated from the formula given in BS 4449: 1998 and corrected to allow for the increase in compressive strength of the concrete. The correction was required because the code requires a concrete strength of between 40N/mm² and 45N/mm² whereas the concrete used was the same as that for the beam and had a compressive strength over 60N/mm². Two further specimens were cast with half this bond length. The bars were debonded for the remainder of their length by inserting them inside a plastic tube. Table 4 gives details of the tests carried out.

*Bars tested had a strength of 569N/mm² and 575N/mm²*
The slip at the free end of the bar was measured using a dial gauge as load was applied to the end that was bonded to the concrete. Slip was recorded on three of the bars tested; the two 6mm bars and one of the 4mm bars, all with the shorter bond lengths. Plots of slip versus applied load are shown in Figure 11. It was difficult to grip the 3mm bars and the force applied only reached about 80% of their UTS. The remaining bars fractured before any slip was recorded at the free end.

The results demonstrate that the bond strengths of the bars met the code requirements.

5 Instrumentation

An instrumentation system was devised which would demonstrate the behaviour of the model bridge deck as it was loaded to failure. The instrumentation consisted of displacement gauges, vibrating wire strain gauges, load cells and temperature sensors placed at various points on the model and support system to define the structural behaviour. The gauges were logged automatically using a Scorpio data logging system and two Gage Technique Geologgers. All the data were stored in a computer for later processing and presentation. In addition, dial gauges were used to indicate movement at the bearings. To assist in visual examination of the model, and in particular to identify the presence of cracks, all surfaces were painted white. The main span deviators were examined for cracks using a borescope inserted through 25mm diameter holes which were positioned on both sides of the diaphragms and incorporated into the top slab during construction. A crack gauge was used to determine crack widths at each increment of load. These were read manually as the test progressed.

5.1 Strain and temperature

To measure strain in the concrete section sixteen vibrating wire strain gauges were attached to the reinforcing cage before the concrete was cast. Gauges were installed at two sections, one in the main span 1.0m from the centre support and the other between the centre support and the load, 1.0m from the load point. At each of these sections, four gauges were fixed in the top flange, one in each of the walls, at approximately half depth, and two in the bottom flange. Figure 12 shows the position of these gauges. Vibrating wire gauges 4, 5, 6, 7 and 8 contained integral thermistors which were used to monitor the temperature within the concrete section. The ambient temperature was recorded periodically during the test using a hand-held K-type thermocouple.

Changes in strain and temperature during the load test were logged using one of the Geologgers controlled by a personal computer. At each load increment, a series of four scans were taken. Data was stored on hard disk and subsequently imported into a spreadsheet for further processing.

5.2 Load

A series of load cells of various types were used to monitor load before and during the load test. Two shear beam load cells were positioned at each support to measure reactions. These were incorporated into the bearings and were used to determine the way in which the load was carried by the model.

Compression, through-hole type, load cells were incorporated into the cable anchorages to indicate the actual prestress load in the bridge. Because the section was symmetric, only three cables were instrumented. Two load cells were used, one at the live end and one at the dead end anchorage. This was so that overall friction losses could be determined during stressing and during the application of test load. Cables 4, 5 and 6 had load cells installed (see Figure 3 for location of these cables). The load cells were positioned between the collet block and anchorage bearing plate at both ends of the instrumented tendons. Figure 13 shows the bearings and load cells in position at the live end of the model.

The load applied during the load test was measured using load cells built into the stressing jacks (see Section 6). These, along with the reaction load cells and two of the cable load cells (electrical resistance strain gauge load cells), were monitored using the Scorpio data logging system. The other four cable load cells (vibrating wire strain gauge load cells) were logged using one of the Geologgers.

5.3 Displacement

Vertical displacement of the beam was measured using HSI rotary type variable resistance displacement gauges. These gauges had a stroke of 300mm and were connected to the Scorpio data logger. The position of the gauges is shown in Figure 14. In general, the displacement gauges were positioned along the centre line of the model. At the load point, two gauges, D4 and D5, were used: these were positioned on the sides of the beam to detect any twisting of the model due to unevenly applied load. The displacement gauges were logged using the Scorpio logger.

6 Stressing the beam

The tendons were stressed using a pair of Enerpac hollow ram jacks, each of which had a capacity of 300kN, acting between the collet block adjacent to the anchorage and a second collet block. A manual hydraulic pump was used to apply the tension force in the cables. Load was measured using a pressure gauge. A cylindrical collar with three longitudinal slots was placed between the jack and the anchorage collet block. The slots were positioned in line with the individual strands to allow access for insertion or removal of the collets. Figure 15 shows the stressing system used.

The stressing procedure consisted of applying load by reacting against the second collet block. When the required load was reached (either by reading the load cell or a pressure gauge on the pump), the collets at the anchorage plate were pushed into place and the jack unloaded and removed. A small allowance was made to take account of the loss of cable force as the collets were pulled into the anchor block. The stressing system allowed the beam to be de-stressed or ‘topped-up’, if required.
Figure 11 Pull-out tests on reinforcing bars
Figure 12 Location of strain gauges and temperature sensors
Figure 13 Load cells in tendon anchorages at live end of model

Figure 14 Position of the displacement gauges

Figure 15 Stressing system used to stress the beam
The tendons were stressed in two stages. Initially each strand was stressed individually to remove the slack. The load in all three strands was then taken up to about 30% of the required prestress load. This was to allow the formwork and falsework to be removed. Once the weight of the beam was taken by the supports, the deck was fully stressed. During this final stressing operation, the reactions were monitored to determine how the dead load was being re-distributed. Stressing was completed about five weeks before the load test. During this time the prestress load was monitored to determine any changes due to temperature, creep, relaxation, etc.

The total design prestress force in the full scale deck was 37,500kN equivalent to 70% of the characteristic load of the tendons. An allowance for losses reduced this to 33,500kN. Thus for the model, the load per cable was 349kN (using a scale factor of 16). In addition, a load factor of 0.87 is required on the prestress force which reduced the target load per cable to 304kN. The stressing of each cable was halted when the pressure applied to the pair cylinders was 8000psi. Previous calibration suggested that this corresponded to a load of 375kN. However, friction in the system made this method of measuring load unreliable and the readings in the live end load cells were used to determine the cable load.

The average of the instrumented cables immediately after stressing all cables was 326kN measured at the live end. This had fallen to 321kN after 12 days, and was 320kN just prior to the first load test. This is 55% of the characteristic load. The load cells in the dead end anchorages gave a reading on average 10% lower than that measured at the live end. This was attributed primarily to friction at the deviators. This is discussed in more detail in section 10.2.1.

7 Loading system

The objective of the load test was to determine the behaviour of the model deck as load was applied in a controlled way until failure was achieved. Load was applied in small increments using a cross-head placed transversely across the model at the designated section. The bearing surface consisted of a 200mm wide knife-edge welded to the bottom of the cross-head which spanned between the webs of the box. The cross-head was lifted into place using a crane and epoxy resin was used to bed the knife-edge in place to eliminate any irregularities in the surface of the concrete. After placing the cross-head, a pair of cables were inserted through it. These passed through holes in the flange cantilevers and were attached to two anchors installed in the base of the test bed. Loading was effected by placing two CCL stressing jacks (capacity 3000kN) on top of the cross-head through which the cables were threaded. Each of the cables comprised four 12.7mm diameter strands which were sufficient to give a load capacity of over 1000kN. This was sufficient to ensure that failure would be achieved. Figure 16 shows the loading rig prior to the start of the test.

Load was applied using a single hydraulic pump which delivered equal pressure to the two jacks. During loading, the magnitude of the applied load was approximated using a pressure gauge at the pump. Actual load was determined using the output from the load cells in the jacks. For the initial part of the test, load was applied in increments of about 50kN. Later on, increments giving a deflection of about 15mm at the load point were applied. After applying a load increment, the pump was locked off so that a series of scans could be made for a constant value of applied load.

Because of the configuration of the model and loading system, it was clear that the end support on the short span would tend to lift off its bearings. This was accentuated because of the scaling of the model which reduced the sectional area by a factor of sixteen but the dead load by a factor of sixty four. To provide exact scaling of dead load it would be necessary to provide dead load compensation, ie, increase the dead load by a factor of four. The preliminary analysis showed, however, that the prestress was unlikely to cause cracking in the beam and the need for dead load compensation was removed.

The analysis also showed that, with no dead load compensation, the beam would lift off the bearings at a relatively low value of applied load (about 280kN). To prevent this, kentledge was placed over the support so that lift off would not occur. The kentledge consisted of 12 steel blocks each of which weighed 10kN. This was sufficient for a test load of over 1000kN. Figure 17 shows the kentledge in position prior to the load test.

8 Analysis of bridge

Prior to the load test a series of analyses were carried out to supplement the design analysis so that the behaviour of the model as it was loaded to collapse could be predicted. This was used for the following purposes:

- to determine the design parameters for the loading rig, including load capacity and required deflection;
- to ensure safety of personnel during the stressing of the beam and during the load test;
to ensure that the model was not damaged during stressing and erection of the loading rig;

• to identify appropriate locations for the instrumentation system;

• to estimate the increase in tendon force during the load test;

• to predict how the model would behave under load and to identify the various stages of damage so that these can be anticipated during the load test.

Analysis was carried out using the commercial analysis package STAAD III. The deck was modelled as a prismatic beam with elements of length 1.0m. The cables were entered as tension elements with appropriate vertical eccentricity. Horizontal profiles were ignored. Thus the resulting bending moments and shear forces include the parasitic effects of the external cables. A limitation of STAAD was that each straight length of cable had to be entered as a separate element. In addition, the cable forces entered changed as the model deformed under load. This was overcome by changing the initial tendon forces until the final forces (under dead load and prestress only) were approximately equal to the design values.

Analysis was first carried out to determine the concrete stresses due to the permanent loads, ie, dead load and prestress load. This was important since the reduced dead load of the model increased the likelihood of cracking due to prestress. The critical locations were hogging at the central support and sagging at the test point. Steel curtailment points in the back span were also checked. The analysis showed tensile stresses due to the prestress only (taken as 60% of the characteristic strength of the cables) of 2.8N/mm² 2.3N/mm² at the test section and central support respectively. Theoretically this was below the cracking capacity of about 3.6N/mm² but to ensure that cracking did not occur, it was decided to partially prestress the model before the falsework was removed. Stresses due to dead load and prestress together were close to zero.

The model was then analysed to determine the combined effects of dead load, prestress and live load using the methods for flexure and shear given in BS 5400 and BD 58 (see Table 5). The overall conclusion of the analysis was that a flexural failure would occur in the main span at the load point. The magnitude of the calculated load to failure depended on the assumptions made in the analysis. Using BS 5400 in conjunction with BD 58, flexural failure was indicated at an applied load of 448kN. Design sagging moment was achieved at 445kN. Removing the partial safety factors for material strength (ie, setting them to unity) and using actual material strengths determined from tests, gave a failure load of 524kN. Using SECT, a TRL program which performs a strain compatibility section analysis, flexural failure was indicated at 645kN. Allowing the tendon force to increase by 10% gave a failure load of 675kN. However, no allowance for reductions in eccentricity were made in calculating these values.

### Table 5 Summary of conclusions of preliminary STAAD analysis

<table>
<thead>
<tr>
<th>Load effect/Location</th>
<th>Capacity Equivalent test load</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Cracking:</strong></td>
<td></td>
</tr>
<tr>
<td>Zero strain on soffit at test point</td>
<td>0 N/mm²</td>
</tr>
<tr>
<td>Cracking on soffit at test point</td>
<td>3.6N/mm²</td>
</tr>
<tr>
<td>Zero strain on top at pier</td>
<td>0 N/mm²</td>
</tr>
<tr>
<td>Cracking on top at pier</td>
<td>3.6N/mm²</td>
</tr>
<tr>
<td><strong>Flexure in main span:</strong></td>
<td></td>
</tr>
<tr>
<td>Design sagging moment</td>
<td>821kNm</td>
</tr>
<tr>
<td>Resistance, using BD 58</td>
<td>830kNm</td>
</tr>
<tr>
<td>Resistance, no partial safety factors¹</td>
<td>1007kNm</td>
</tr>
<tr>
<td>Resistance from SECT, no partial factors¹</td>
<td>1291kNm</td>
</tr>
<tr>
<td>Resistance from SECT, with 10% increase in tendon force</td>
<td>1360kN</td>
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<tr>
<td><strong>Flexure in back span:</strong></td>
<td></td>
</tr>
<tr>
<td>Design hogging moment at support</td>
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<tr>
<td>Resistance, using BD 58</td>
<td>775kNm</td>
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<tr>
<td>Resistance, no partial safety factors¹</td>
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</tr>
<tr>
<td>Resistance from SECT, no partial factors¹</td>
<td>1055kNm</td>
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<tr>
<td><strong>Shear at main span abutment:</strong></td>
<td></td>
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<tr>
<td>Design ultimate</td>
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</tr>
<tr>
<td>Design resistance²</td>
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</tr>
<tr>
<td>Design resistance, no partial factors¹</td>
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<tr>
<td>Resistance, ignoring shear steel³</td>
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<tr>
<td><strong>Shear at pier:</strong></td>
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<tr>
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<tr>
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<tr>
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<tr>
<td>Lift at back span abutment</td>
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</table>

¹ Actual material properties used
² Lower than design shear since model scale depth factor used in calculations
³ Used to estimate load at which shear cracking occurs

Initial flexural cracking was calculated to take place in the soffit of the main span at a load of 241kN. This assumed a tensile capacity of 3.6N/mm². Cracking over the central pier was calculated to take place at 600kN. Similar values for hogging moment over the central pier are presented in Table 6.
The design shear at the main span abutment was achieved at an applied load of 508kN. Using the unfactored shear capacity, shear failure was indicated at 1146kN. Ignoring the shear steel gave an equivalent load magnitude of 250kN: this indicated the possible load at which shear cracking might occur. Table 6 includes corresponding values for shear at the central pier.

Finally, Figure 18 shows the bending moment obtained from the STAAD analysis for dead load plus prestress plus an applied test load of 500kN.

<table>
<thead>
<tr>
<th>Code used (See references for full title of standard used)</th>
<th>Design Capacity (kNm)</th>
<th>Design Load (kN)</th>
<th>Assessment Capacity (kNm)</th>
<th>Assessment Load (kN)</th>
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<td>497.3</td>
<td>1189.5</td>
<td>589.4</td>
</tr>
<tr>
<td>BS 8110 (BSI 1985)</td>
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<td>510.2</td>
<td>1079.7</td>
<td>542.4</td>
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<tr>
<td>OHBDC (OMTC 1985)</td>
<td>1182.9</td>
<td>586.6</td>
<td>1396.3</td>
<td>677.7</td>
</tr>
<tr>
<td>ACI (ACI 1994)</td>
<td>1263.1</td>
<td>620.8</td>
<td>1395.6</td>
<td>677.4</td>
</tr>
<tr>
<td>CAN3-A23 (CSA 1984)</td>
<td>1135.8</td>
<td>566.5</td>
<td>1246.1</td>
<td>613.6</td>
</tr>
<tr>
<td>AASHTO (1991)</td>
<td>1293.6</td>
<td>633.9</td>
<td>1350.6</td>
<td>658.2</td>
</tr>
<tr>
<td>NZS (SANZ 1982)</td>
<td>1213.4</td>
<td>599.6</td>
<td>1269.0</td>
<td>623.4</td>
</tr>
<tr>
<td>AUSTROADS (1992)</td>
<td>1323.1</td>
<td>646.5</td>
<td>1396.3</td>
<td>677.7</td>
</tr>
</tbody>
</table>

9 Load test

The load test was carried out over two days. On the first day load was applied until the beam was cracked at the load point and over the support. On the second day the loading was continued until failure was achieved. The following sections describe the behaviour of the beam in terms of the applied load which were obtained from the jack load cell readings. The load values quoted include the additional load due to the cross-head (weight 12.8kN) and the stressing jacks (7.2kN each). Reference is made to the results of the preliminary analysis presented in Table 6 to compare the actual behaviour of the model to that indicated by the analysis.

9.1 Test to cracking

The test was carried out in January in dry slightly overcast conditions with very little wind. The ambient temperature remained constant at around 5.5°C for the duration of the test.

The load was applied in nine increments of approximately 50kN up to a maximum load of 477kN (including the weight of the loading rig). All the instrumentation was scanned and any cracking marked on the model between each loading increment.

The first tensile cracks appeared on the soffit of the beam at the load point during the application of load increment 5. During this increment, the load was increased.
from 230kN to 286kN and the resulting deflection was 21.0mm. This compares well with the theoretical cracking load of 241kN. A series of cracks were observed, the longest of which extended half way up the side of the beam. These occurred on both sides and were all within about 1.0m of the load point. As the load was increased further the initial cracks approached the top flange and intermediate cracks were observed. Further cracks appeared as the load test progressed. The cracks were approximately symmetrical about the load point and extended to within about 400mm and 180mm of the deviators nearest the centre support and end supports, respectively, at the maximum load of 477kN.

The first tensile cracks over the centre support appeared at increment 8, when the applied load increased from 393kN to 440kN. The theoretical cracking load at this point was considerably higher and possible explanations for the low load at which cracks occurred will be discussed later in section 11. A series of cracks were observed and they extended over a distance of approximately 200mm from the support in the main span to 550mm from the support in the back span. The cracks were visible on the top of the beam, on the edge of the flange and the longest one extended about 100mm into side of the box. When the load was increased further cracking occurred, extending from 375mm from the support in the main span to 1500mm from the support in the back span.

The ends of the beams and anchorages were carefully examined for cracks but none were observed. The deviators nearest the support were examined using a borescope inserted through the top slab but again no cracks were observed.

The load was removed in approximately 50kN steps and readings were taken at each of the load increments. When all the load was removed from the model, it was examined closely for any other damage. None was found: indeed all the cracks closed due to the prestressing force and were not visibly without close examination with a crack gauge. Figures 19 and 20 show the cracking in the model at the test point and at the central pier at the conclusion of test 1. The numbers written on the model indicate the increment number.

Figure 19 Cracking in main span at conclusion of test 1

Figure 20 Cracking in central pier at conclusion of test 1

9.2 Test to failure

The test to failure was carried out under similar conditions to the previous day although a little colder: the ambient temperature increased from 1.9°C when the test started to 3.4°C by the end of the test.

As for the first test, the load was applied in increments of approximately 50kN. Once the cracking was well progressed and non-linear behaviour was apparent from the deflection readings, further increments were applied by increasing deflection rather than load. Initially steps of 15mm were used. During the latter part of the load test, this was increased to 20mm. As in the first test, all the instrumentation was scanned and cracks marked at each increment.

The flexural cracks in the main span started to re-open under the load point at mid-span at a load of 139kN (increment 2). Extension of these cracks began at a load of 404kN. Cracks at the centre support did not start to open until the load reached 343kN (increment 6). Further cracking ensued at a load of 518kN.

When the load was increased to 517kN (increment 9) the cracks in main span started to extend into the flange but were not visible on the side of the flange. Also the first crack outside the deviators appeared on the end support side of the main span and a number of 100mm long transverse cracks were observed along the centre line of the soffit extending from deviator towards the end support.

Water leakage through cracks clearly highlighted their position. The water actually came from rainwater which had collected inside the box as the ends of the model were not protected from the weather. Under the load point the cracks were straight across the beam but cracks away from the load point were curved towards the support, the curvature increasing with distance from the load point. This can be seen in Figure 21.

As the load was increased beyond 548kN (increment 10) a series of ‘clunks’ could be heard in the bridge: these were attributed to the tendons slipping at the deviators although no significant variations in tendon load were recorded. The maximum deflection at this stage was 87.3mm. As the deflection was increased to 133.7mm
(load 627kN, increment 13) there was compressive failure of the concrete in the top flange under the load point on one side of the bridge. As the next increment was applied compressive failure occurred in the top flange on the other side of the bridge, as shown in Figure 22. As the deflection was increased further spalling of the concrete occurred on the underside of the flange on one side of the beam (increment 16). When the next increment was applied there was a similar failure on the other side of the beam. The maximum load carried by the model was 627kN which occurred at a deflection of 134mm and the load in the prestressing cables had increased by 15.5% (about 50kN per cable).

At the conclusion of the test, further cracks were noted extending from the top flange between the first main span deviator and the support. They extended from 1m from the support to 1.3m from mid-span deviator nearest the support.

10 Discussion

10.1 Design and construction

The design of the full scale bridge was carried out by an experienced consultant and posed few technical problems. Some variations in the design were imposed since it was intended to construct and test a quarter scale model. For example, it was required that the eventual design should have no arbitrary reserves of strength so that the model satisfied, as closely as possible, the minimum requirements of the code. In fact, this was not possible particularly due to the much higher strength of reinforcement which had to be used. However, these variations were taken into account in the analysis.
The main difficulties encountered were in the construction of the model. The small size of reinforcement, down to 3mm diameter bars, while easier to bend and position, were time-consuming to fix and produced a very flexible reinforcement cage which had to be handled carefully. This posed particular problems for the steel fixers, who were experienced in fixing full scale reinforcement but were unfamiliar with working at a small scale with correspondingly reduced tolerances. The positioning and aligning of the deviator tubes also posed some problems. Again this was exacerbated by working at such a small scale. Eventually a tolerance of ±5mm was accepted.

Compromises had to be made with regard to the material properties of the reinforcement used. Because the small size bars (3mm and 4mm diameter) were not available, stainless steel bars had to be used. Tensile tests on this material showed that the bars had significantly greater ultimate strength than conventional reinforcement which increased the strength of the model particularly in shear. However, the test was devised so that flexure was critical and the effect on flexural strength was minor. In the analyses which were carried out, the actual material strengths were used to determine the capacity of the model.

The biggest problem encountered was in producing a concrete mix which represented the full scale design mix but which was workable enough to produce a good quality 7.5mm cover to the reinforcement. Great care was required during casting and close site supervision was imposed at the mixing and placing of the concrete.

10.2 Application of prestress
The stressing system was devised to stress each three-strand cable as a unit, although individual strands could also be stressed to remove slack. The stress was applied with a manual hydraulic pump which provided good control. The stressing system was also capable of de-stressing and re-stressing the cables.

10.2.1 Load in tendons
The preliminary analysis indicated that, even without dead load compensation, the likelihood of cracking during the stressing of the model bridge was low (see section 6). Therefore the sequence of stressing did not pose any problems. It was intended to examine prestress losses due to friction at the deviators as the beam was prestressed. This was investigated during the stressing of cable 2. A series of readings from the load cells at both anchorages were taken as the prestress load was increased from 160kN to 180kN in steps of about 0.1mm extension. These are shown in Figure 24 plotted as a function of cable extension. Also plotted (on the right Y-axis) is the difference in load between the live end and the dead end.

Figure 24 Load measured in cable during stressing
anchorage. It can be seen that the loss along the tendon increases with prestress load (although this reduces if it is expressed as a percentage of prestress load). The difference is linear, suggesting that there is no definite tendency for the tendons to ‘stick and slip’ at the deviators as the load is applied. At this range, the difference in load was about 21% of the applied prestress, falling to 10% when the cable was fully stressed. It was observed that this difference reduced to 8% at the end of the first load test. For cable 1, the difference between the live end and the dead end load cell was higher: 19% on application of final prestress falling to 15% just before the load test. The average value was 12% (readings from one of the load cells in cable 3 were unreliable so were not used). These losses compare with the value of $\mu$ of 0.15 recommended by BD 58 for a non-lubricated strand in an HDPE duct.

Other prestress losses due to creep in the concrete and relaxation of the steel might also occur. As the load test was carried out just over a month after stressing it was possible to examine the change of prestress with time. Figure 25 shows the load measured at both ends of cables 1 and 2. The plots show a gradual reduction of prestress force over the first 12 days after stressing. Expressed as a percentage, the losses were 2% for cable 1 and 3% for cable 2. On the day of the load test, these losses had stabilised at about 2%. The plots also show daily variations (of about 1%) the most likely cause being temperature changes.

### 10.2.2 Strains through section

The effect of the prestress on the beam was examined by monitoring the readings from the strain gauges embedded in the concrete section. Figure 26 shows the average strain measured near the test section and near the central support over the two days of final stressing. The load in each cable was increased from an average of 106kN to 326kN. Stressing was carried out in three stages on consecutive days, as can be seen from the steps in the figure. First, cables 3 and 4 were stressed, followed by cables 1 and 6, and finally cables 2 and 5. Figure 27 shows the average strain in the top slab near the test section over a period of 12 days after stressing. This indicates that daily fluctuations of about 10 microstrain were occurring due primarily to temperature variations. The figure also indicates a steady drift in the strain readings (in compression) probably due to creep although gauge drift cannot be eliminated completely. It is evident from Figure 25 that the load measured in the cables is not fluctuating in the same manner. This is understandable, since the coefficient of expansion of steel and concrete are very similar.

### 10.3 Application of test load

#### 10.3.1 Cracking

The behaviour of the model during the load test was more or less as expected. Cracking at the central pier occurred when the load increased from 393kN to 440kN: the

![Figure 25 Load in cables 1 and 2 after final stressing](image)
Figure 26 Average strain in concrete sections
calculated value was 600kN. The early cracking may be due to the prestress being lower than assumed or incorrect eccentricity of the tendons due to inaccuracies in the installation of the deviator tubes. It should be noted that all cables are high in the beam at this point but the profile is such that eccentricity reduces on both sides of the support. The early cracking may also be the result of the fact that this is a top concrete surface. In general, the tensile capacity of top cast concrete is reduced due to the presence of micro-cracks from concrete shrinkage. The load at which the strain is zero at this point is 450kN which is still higher than the load at which the cracks actually appeared. Making an allowance of about 12% for friction losses along the tendons reduces the theoretical cracking load to 420kN which is in line with the actual cracking load.

10.3.2 Deflections

Figure 28 presents the load deflection curve obtained during the load test to failure. The plot is annotated to indicate the sequence of damage occurring in the model. It should be noted that the load data in this figure (and in the following figures) have not been corrected to include the weight of the loading rig which was 27.2kN. The values of load used in the previous section include this allowance. Figure 29 presents all the load deflection curves for each of the gauges (see Figure 14 for gauge location).

The plot of deflection against applied load indicates clearly the behaviour of the model. The first sign of non-linearity is when the cracks re-open (or on first cracking during the first load test) which is clearly visible on Figure 28. The stiffness of the beam reduces gradually as the cracking extends under the applied load and over the central pier. As expected, the maximum load occurs just prior to the onset of concrete crushing in the top flange. This occurs at a deflection of about 134mm. Sudden failure occurred at a deflection of 200mm.

Figure 30 shows the deflection profile at various levels of applied load. The change in shape of the beam as the damage at the load point becomes progressively worse demonstrates the development of a plastic hinge as the maximum load is approached. This figure suggests that a plastic hinge is also forming at the central support.

One implication of the deflected shape shown in Figure 30 is the loss of eccentricity of the prestressing cables which reduces the effectiveness of the prestress force. The change in eccentricity as the model deflects under load can be measured by subtracting the actual deflection under the load from the average deflection at the deviator. Deflection gauges D3 and D6 were positioned for this purpose. Figure 31 shows how the eccentricity varied with applied load. The change in eccentricity at maximum load was measured at 38.7mm which is significant bearing in mind that the design eccentricity is 186mm. The effect this has on flexural capacity is discussed later in section 10.3.6.
Figure 28 Deflection of model at load point (gauge D4)
Figure 29 Load deflection curves for load test to failure
Figure 30 Deflected shape of deck for various loads

Figure 31 Change of eccentricity with applied load
10.3.3 Strains
Figure 32 presents the strain gauge readings near the load point as a function of applied load (refer to Figure 12 for location of gauges). The 4 gauges in the top flange (VW9, VW10, VW11, VW12) showed that the flange was in uniform compression with no indications of any shear lag effects. When the beam was unloaded after the test, the strains almost fully recovered. The residual tension strain at the end of the test is due to the reduction in cable force. The gauges in the tensile zone (VW13, VW14, VW15, VW16) also behaved as expected, with similar tension values in corresponding gauges. The onset of cracking is evident by a rapid increase in the measured tensile strains. Again the strains recovered on unloading the model.

Figure 33 shows the corresponding strain readings near the central pier. In this case, the gauges in the top flange (VW1, VW2, VW3, VW4) were in tension. Cracking is evident by discontinuities in the load-strain plot. The tensile strains were uniform across the flange. The strains in the web (VW5, VW6) indicated compression in the early part of the test but then went into tension. This was due to the shift in neutral axis position as the model started to crack. The compression strains in the bottom slab (VW7, VW8) indicated that crushing of the concrete was imminent.

10.3.4 Load in tendons
Figure 34 shows the readings from the load cells in the cable anchorages as a function of applied load. The total prestress just before the start of the load test was estimated to be 1918kN, based on the average value of the instrumented cables. The plots in Figure 34 show that the load in the cables increased steadily as the load was applied. The rate increased as the beam cracked. The maximum prestress load, measured when the maximum load was applied to the beam, was 2216kN, an increase of 16%.

10.3.5 Flexural strength using BD 58
The failure load achieved during the test, 627kN, compared well with the values calculated during the preliminary analysis, the results of which are summarised in Table 6. The design moment was achieved at a load of 445kN. The unfactored flexural resistance was achieved at 524kN. This indicated that the method of determining flexural capacity is conservative by about 20% in addition to allowances for partial factors for material properties and load. To determine how much of this was due to variations in the prestress and eccentricity as the beam deformed, a further analysis was carried out using the values of prestress and eccentricity measured as the model was carrying the maximum load. The values used were 2216kN for the total prestress force and 38.7mm loss of eccentricity which were the values measured at maximum applied load as discussed in section 10.3.4 above. This resulted in a decrease in load carrying capacity of about 4%, i.e., the increase in tension load was more than compensated for by the reduction in tendon eccentricity. Thus the approach adopted by BD 58 in assuming that there is no increase in cable load is appropriate in this case.

10.3.6 Flexural strength using other codes
Various methods for determining the flexural capacity of beams with unbonded tendons are presented in different national codes. All of these allow for an increase in tendon force at failure to a greater or lesser extent. The application of these codes to the model bridge is summarised in Table 6. Two sets of calculations are shown in the table. The first, labelled Design as it used the basic information available at the design stage, was carried out using the following assumptions:
- partial safety factors appropriate to design;
- $f_y = 460\text{N/mm}^2$;
- $f_{cu} = 60\text{N/mm}^2$;
- initial prestress value of 1918kN, increased according to elastic properties of members;
- increase in prestress for determining capacity, as appropriate to code;
- no change in eccentricity.

The second set of calculations, labelled Assessment, was carried out to determine which code gives the best estimate of flexural capacity when compared with the actual capacity determined during the test. The assumptions were as above except:
- partial safety factors set to unity;
- $f_y = 500\text{N/mm}^2$;
- $f_{cu} = 70\text{N/mm}^2$.

The STAAD analysis used to determine the corresponding applied load assumed an initial cable force of 1918kN (as measured just prior to the start of the load test) and a constant cable eccentricity. The codes used are listed in the first column. In all cases the longitudinal reinforcement was included in the calculations as appropriate to each code.

The main conclusion derived from Table 6 is that when the partial safety factors are removed and the actual material strengths are used, all the codes are within 6% of the capacity determined from the load test except for BS 8110. The main reason why the BS 8110 value is lower than the other codes is due to the requirement to include the area of unstressed steel according to the ratio $f_f/f_{pu}$, rather than $f_f/f_{ps}$, as it is for the other codes. The next lowest value is given by BD 58: this is because no increase in cable load is allowed and in addition, there is a requirement to impose a load factor of 0.87 on the cable force. All the other codes give similar results: this is not surprising since they are all based on a similar method. These all allow for an increase in tendon stress at failure of between 14% and 32%. In fact four of the codes marginally over-estimated the flexural capacity of the model. These are OHBDC, ACI, AASHTO, AUSTROADS. Even with partial factors included in the calculations, AASHTO and AUSTROADS over-estimated the flexural capacity. These codes allowed an increase in tendon force of up to 32%. In addition, no consideration is given to potential loss of eccentricity.

In conclusion, BD 58 can be seen to produce a safe value of flexural capacity. It is suggested that an increase...
Figure 32 Strain readings near test point
Figure 32 Strain readings near test point (continued)
Figure 33 Strain readings near centre pier
Figure 33 Strain readings near centre pier (continued)
Figure 34 Load as measured by load cells in cables
in tendon force can be allowed, but that careful consideration should be given to lay-out of the tendons and deviators and potential loss of eccentricity be taken into account. BD 58 suggests that the strain in the tendon can be calculated using non-linear analysis provided ‘conservative’ assumptions are adopted. This should include an appropriate allowance for loss of eccentricity.

10.3.7 Shear strength
The calculation of shear capacity according to BD 58 is taken from the shear strength of axially loaded columns in BS 5400: Part 4. The correction factor is dependent only on magnitude of axial load. The load test showed that the overall effect of increasing axial load and decreasing eccentricity reduced the flexural capacity by about 4%. As shear was not critical in this test, it was not possible to draw any conclusions on how this would affect shear strength. However, it is unlikely that an allowance for reduced eccentricity would be required for shear, since sections requiring shear checks would always be close to deviators or diaphragms where loss of eccentricity would be negligible. An alternative correction factor for the effect of axial load is given in BS 8110: Part 1. In this case, the correction factor is a function of the ratio of applied bending moment and shear force. Again no conclusions can be drawn.

10.3.8 Mode of failure
The sudden failure, while not unexpected, may have implications for design and requires further investigation. Previous laboratory testing of beams with unbonded tendons in flexure suggested that, while failure was invariably provoked by crushing of the concrete, the disruption was not enough to produce a catastrophic failure mode and there was considerable residual strength after the maximum load had been achieved. In addition, the prestressing force in the tendons was maintained after the test was concluded. With the quarter scale model test described here, failure occurred at a similar maximum deflection to the beams but there was more disruption to the section with consequent lower residual strength. In particular, the loss of prestress force due to the shortening of the beam after failure was not expected. The level of disruption was sufficient to cause a loss of prestress force in the tendons of about 80%. The sudden disruption was also responsible for causing the beam to lift off its bearings and move sideways. As mentioned previously, this may be attributed to the upward force applied by the cables to the deviator, which was enhanced because of the use of the knife edge load to achieve failure.

The deflection of 134mm (0.3 times the beam depth) at the maximum load, and the extent of the cracking prior to failure, affirms that there was considerable warning of failure prior to the sudden collapse, in spite of the fact that it was precipitated by concrete crushing.

The measured crack widths at the support both as the beam approached failure and after failure did not exceed 0.16mm. Except for the extensive cracking, there was little damage at the central pier and it was clear that a hinge had not fully formed. The mode of failure and the disruption to the concrete section at the load point suggested that ductility was limited, which has important implications in terms of appropriate methods of analysis and moment redistribution in particular.

11 Conclusions

a Construction of the quarter scale model of an 80m bridge caused a number of problems, particularly in regard to fixing the steel and designing a suitable concrete mix. However the problems were related to construction of the model and would not apply full scale construction.

b The method used to aligned the deviators was taken from current practice for full-scale structures and worked well. Difficulties were encountered in achieving perfect alignment but these were considered to have little effect on performance.

c Prestress losses of about 12% were measured along the tendon, due to friction at the deviators. This compares well with the coefficient of friction of 0.15 recommended by BD 58.

d The load test to failure was satisfactory and the maximum load achieved compared well with the flexural capacity calculated during the preliminary analysis. The strains, deflections and loads recorded before and during the test showed that the behaviour of the beam was as expected in general terms.

e The increase in prestress measured at failure of the model was 15% of the initial prestress. This compares well with values of between 0% and 32% for different codes.

f The loss of eccentricity measured at maximum load was 38.7mm. This along with the measured increase in prestress force resulted in an overall loss of flexural strength of 4%. Thus methods which allow an increase in prestress without taking account of loss of eccentricity may lead to unconservative results. This should be taken into account in design, particularly where non-linear methods of analysis are used.

g The deflected shape of the model demonstrated the initial development of a plastic hinge at the central support. However, the extent of damage and the measured strains showed that a hinge had not fully developed. Thus the ductility was not sufficient to allow the development of a plastic collapse mechanism.

h The limited ductility of the model has implications for designers. When re-distribution of moments is used, the designer must ensure that sufficient ductility exists.

i The anchorages and deviators performed in a satisfactory way throughout the load test. Visual examination after the test showed that no damage was sustained.
12 Acknowledgements

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13 References


ACI (1994). ACI manual of concrete practice. Part 3. ACI Committee 318: Building Code Requirements for Reinforced Concrete, American Concrete Institute, Detroit, USA.


Abstract

There has been increasing interest in recent years in the design and construction of post-tensioned concrete bridges with external unbonded tendons. To assist in the design of these structures, the Highways Agency issued a Standard and Advice Note (BD 58 and BA 58) in 1994. These documents were based on a review of existing knowledge that included recommendations on where further research was required.

To provide further information for designers a continuous two span bridge with an overall length of 80m was designed and a quarter scale model of the bridge was constructed and load tested to failure. The objectives were to monitor the construction of the beam and measure its flexural capacity. In addition its behaviour as it was load tested was carefully monitored to determine the increase in tendon force, estimate the geometrical non-linearity, measure the friction losses at the deflector and supports, and record the behaviour of the deflectors, support diaphragms and anchorages.

This report presents a description of the model bridge and its construction, and describes the structural behaviour as it was loaded to failure. The design of the bridge is presented in a separate report.

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