Performance of an integral bridge over the M1–A1 Link Road at Bramham Crossroads

Prepared for Quality Services (Civil Engineering), Highways Agency

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TRL Report TRL521
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Executive Summary

Integral bridges are designed in the main with continuous decks and abutments which are structurally connected to the deck. The advantage of this design is that maintenance costs are expected to be lower than those for conventional jointed bridges because of the elimination of mechanical expansion joints and bearings which can suffer corrosion and immobilisation primarily through the penetration of deicing salts. For this reason the Highways Agency are encouraging design for durability by advocating the use of integral bridges for spans not exceeding 60m.

Now that integral bridges are being constructed following the advice of BA42, the opportunity has arisen to install instrumentation at the construction stage on a number of highway bridges so that seasonal performance can be assessed at full scale. One of these bridges is the Bramham Crossroads North Bridge which carries the A64 over the M1-A1 Link Road constructed for Yorkshire Link Ltd. The bridge deck has two spans totalling about 50m and comprises prestressed concrete beams composite with a 200mm thick reinforced concrete slab. The deck beams are structurally connected to full height reinforced concrete abutments which are founded in Magnesian Limestone. Instrumentation was installed to measure movements of the abutments, changes in length and level of the deck, strains in the abutment and deck, deck temperatures and lateral earth pressures acting on the abutments. This report describes the findings from measurements during construction and over the first three years in service.

After three years in service, the approach of BS5400 would predict that 87% of the creep and shrinkage deformation has occurred. During this period, this deformation has produced small movements of the bridge abutments away from the retained ground so relieving any high lateral stresses in the backfill. In the longer term, creep and shrinkage effects will be negligible and the dominant effect is then likely to be that of thermal expansion and contraction of the deck. This seasonal and diurnal cyclic action may lead to densification of the backfill and an increase in the peak lateral stresses acting on the abutments in the summer months. The extent of this stress increase and its significance can only be evaluated by further long term monitoring. The study so far has provided an invaluable insight into the relative magnitudes of shrinkage deformation, creep deformation and thermal cyclic effects.
1 Introduction

Integral bridges are designed in the main with continuous decks and abutments that are structurally connected to the deck. The advantage of this design is that maintenance costs are expected to be lower than those for conventional jointed bridges because of the elimination of mechanical expansion joints and bearings, which can suffer corrosion and immobilisation primarily through the penetration of de-icing salts. For this reason the Highways Agency are encouraging design for durability (BD57, DMRB 1.3.7) by advocating the use of integral bridges for spans not exceeding 60m and skews not exceeding 30°.

With integral bridges, abutment design is however complicated by the cyclic movements that are induced by thermal expansion of the bridge deck which may result in high earth pressures developing behind the abutment during summer months. In the long term, these cyclic movements are also likely to lead to an increased stiffness of granular backfill and the possibility of stress escalation with time (England and Dunstan, 1994). BA42 (DMRB 1.3) takes these factors into account and gives advice on safe design pressures for full height portal frame structures largely based on the findings of centrifuge and analytical studies reported by Springman et al. (1996). Now that integral bridges are being constructed following the advice in BA42, the opportunity has arisen to install instrumentation at the construction stage on a number of highway bridges so that seasonal performance can be assessed at full scale.

One of these bridges is the Bramham Crossroads North Bridge which carries the A64 over the M1-A1 Link Road constructed for Yorkshire Link Ltd. A number of bridges of similar design were constructed on this scheme. In addition to having full height abutments, the integral bridge is designed such that the two span precast composite prestressed deck is fully integral with all supports, utilising no bearings at all.

Instrumentation was installed to measure movements of the abutments, changes in length and level of the deck, strains in the wall and deck, deck temperatures and lateral earth pressures acting on the abutments. This report describes the measurements during construction and over the first three years in service.

2 Location and description of the bridge

Bramham Crossroads North Bridge carries the A64 (York to Leeds road) over a section of the new M1-A1 (Lofthouse to Bramham) Link Road in West Yorkshire. Figure 1 shows the location of the bridge, which has a continuous deck with a full height integral abutment at each end and is supported by a central pier. Structurally independent wing walls of reinforced soil construction are utilised to support the road behind the abutments.

Figure 1 Location of the bridge
The bridge has two spans of 26.3m and 23.9m and is 18.1m in width. The deck comprises 12 precast prestressed concrete beams composite with a 200mm thick slab. The prestress within a beam is provided by 43 steel superstrands (18 of which are debonded near the ends of the beam) of 15.7mm diameter which were loaded to give an initial prestressing force of approximately 200kN on each superstrand. In integrating the deck with the abutments, use has been made of a reinforced concrete diaphragm, which increases the thickness of the abutments from 800mm to 900mm at the top and fully encases the ends of the deck beams and extends back into the span by a further 1.6m.

A section through the bridge giving the spans and further construction details is shown in Figure 2a.

3 Construction sequence

A summary of the sequence of construction of the bridge, which began in December 1996, is shown in Table 1.

Construction work commenced with the excavation and installation of the spread bases, which are founded on Magnesian Limestone, for the two abutments and central pier. The construction of both abutment walls, which are socketed into the spread bases (Figure 2b) and the fixed central pier, began in February 1997 and was completed by early April 1997. The concrete used for the spread bases was Class 40/20, i.e. a minimum required 28 day strength of 40N/mm² and a maximum aggregate size of 20mm.

The west abutment is 8.8m in height from the top of the deck to socket level and was cast in one section (Plate 1a). The socket in the spread bases in which the abutment wall sits is lined with rubber (Figure 2b). Construction of the east abutment is similar although its height is 9.3m. The concrete used in construction of the abutments and central pier was Class 50/20.

The prestressed deck beams (Plate 1b) were placed during the period from April to June 1997, after which the deck was made integral with the abutments and central pier using Class 50/20 concrete for the reinforced concrete diaphragm. Details of the average strengths and densities of the concrete used in the construction of the abutments, central pier and deck are summarised in Table 2.

Drainage behind the abutments was ensured by using a permeable backing of hollow porous drainage blocks filled with pea gravel to prevent the theoretical crushing of blocks due to the calculated earth pressures generated by cyclic loading. After placement of the blocks, backfilling behind the abutment walls commenced in July 1997 as shown in Plate 2a. The backfill was Class 6P material (Specification for Highway Works, MCHW1) and backfilling was completed in August 1997.

The backfill was placed at an average moisture content of 9.2% and compacted in layers to an average bulk density of 2.11Mg/m³ using a combination of a BOMAG 120 twin drum roller and a WACKER DPU2430 vibrating plate. After the compaction of each layer of fill material was complete, a series of galvanised steel reinforcing straps were attached to construct the reinforced earth wing walls. These straps were laid flat onto the compacted layers at approximate horizontal spacings of 0.25m and vertical spacings of 1m. The facing of the wing walls comprised a series of hexagonal concrete units each approximately 2m in height.

The bridge was opened to traffic in early December 1997 and Plate 2b shows the completed structure.

4 Properties of the backfill

The backfill material used against the structure complied with Class 6P requirements in Table 6/1 of the
Table 1 Bridge construction schedule

<table>
<thead>
<tr>
<th>Construction sequence</th>
<th>Date</th>
<th>Day number</th>
</tr>
</thead>
<tbody>
<tr>
<td>West abutment reinforcing cage constructed</td>
<td>25/02/97 to 11/03/97</td>
<td>55 to 69</td>
</tr>
<tr>
<td>West abutment cast</td>
<td>17/03/97</td>
<td>75</td>
</tr>
<tr>
<td>Central pier reinforcing cage constructed</td>
<td>19/02/97 to 17/03/97</td>
<td>49 to 75</td>
</tr>
<tr>
<td>Central pier cast</td>
<td>01/04/97</td>
<td>90</td>
</tr>
<tr>
<td>East abutment reinforcing cage constructed</td>
<td>18/03/97 to 26/03/97</td>
<td>76 to 84</td>
</tr>
<tr>
<td>East abutment cast</td>
<td>04/04/97</td>
<td>93</td>
</tr>
<tr>
<td>Deck beams placed</td>
<td>16/04/97</td>
<td>105</td>
</tr>
<tr>
<td>Deck steelwork placed</td>
<td>06/05/97 to 16/06/97</td>
<td>125 to 166</td>
</tr>
<tr>
<td>East diaphragm and deck cast to make deck integral with abutment</td>
<td>05/06/97</td>
<td>155</td>
</tr>
<tr>
<td>West diaphragm and deck cast to make deck integral with abutment</td>
<td>10/06/97</td>
<td>160</td>
</tr>
<tr>
<td>Backfilling started</td>
<td>24/07/97</td>
<td>204</td>
</tr>
<tr>
<td>Backfilling finished</td>
<td>06/08/97</td>
<td>217</td>
</tr>
<tr>
<td>Deck waterproofed</td>
<td>11/08/97</td>
<td>222</td>
</tr>
<tr>
<td>Sand carpet laid</td>
<td>16/08/97</td>
<td>227</td>
</tr>
<tr>
<td>Approach roads constructed</td>
<td>23/08/97</td>
<td>234</td>
</tr>
<tr>
<td>Basecourse laid</td>
<td>29/10/97</td>
<td>301</td>
</tr>
<tr>
<td>Bridge furniture placed</td>
<td>03/11/97</td>
<td>306</td>
</tr>
<tr>
<td>Footpath constructed</td>
<td>25/11/97</td>
<td>328</td>
</tr>
<tr>
<td>Wearing course laid</td>
<td>28/11/97</td>
<td>331</td>
</tr>
<tr>
<td>Bridge opened</td>
<td>01/12/97</td>
<td>334</td>
</tr>
</tbody>
</table>

Table 2 Properties of concrete used during construction

<table>
<thead>
<tr>
<th>Part of structure</th>
<th>Concrete class</th>
<th>Average 28 day strength (N/mm²)</th>
<th>Average density (Mg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>West abutment</td>
<td>C50/20</td>
<td>69.2</td>
<td>2.44</td>
</tr>
<tr>
<td>East abutment</td>
<td>C50/20</td>
<td>62.8</td>
<td>2.40</td>
</tr>
<tr>
<td>Central pier</td>
<td>C50/20</td>
<td>61.7</td>
<td>2.43</td>
</tr>
<tr>
<td>Deck</td>
<td>C50/20</td>
<td>63.9</td>
<td>2.43</td>
</tr>
</tbody>
</table>

5 Instrumentation

The instrumentation used on the bridge was designed to monitor:

i movement of the abutments and deck;

ii strains in the west abutment;

iii strains in the deck;

iv deck temperatures;

v earth pressures on the west abutment.

A plan view of the deck instrumentation is shown in Figure 4. Details of the instrumentation in and behind the west abutment and on the bridge beams are shown in Figures 5 and 6 respectively.

5.1 Movement of the abutments and deck

A proprietary sleeve jointed plastic inclinometer tube was installed in the reinforcing cage for each abutment, between beam positions 4 and 5 near the traffic face of the abutment, before the concrete was cast. Prior to the casting of the deck and making the bridge integral with the abutments, each tube was extended through the abutment and concrete deck to just below the surface of the footpath of the bridge where they are protected by small manhole covers. Inclinometer surveys at various stages during construction and in the longer term enabled lateral movements of each abutment to be determined assuming base fixity of the inclinometer tubes.

Although base fixity of the inclinometer tubes is a reasonable assumption, its appropriateness was validated by electronic distance measurements. The electronic distance measurements were taken using a high precision Geomensor system, which is capable of measuring changes in distance to better than ±0.3mm over the ranges employed. Measurements were taken on target reflectors inserted into machined stainless steel sockets installed next
Plate 1a Construction of the west abutment

Plate 1b Placement of deck beams
Plate 2a Backfill behind the west abutment

Plate 2b The completed bridge
Figure 3 Particle size distribution of abutment backfill

Figure 4 Plan view of Bramham Crossroads North Bridge
to the top of each inclinometer tube and also at deck level above the central pier. These enabled changes in span of the west and east side of the bridge deck to be calculated and correlated with deck temperature changes.

Changes in level of the bridge deck were monitored using a line of precise levelling points installed along the length of the deck at the locations shown in Figure 4. Levelling points were incorporated in the stainless steel sockets used for the Geomensor and supplemented by stainless steel road nails installed in the footpath. Measurements were determined relative to remote temporary benchmarks using precise levelling techniques employing an invar staff and a parallel plate micrometer. The precision of these measurements was considered to be better than ±0.25mm.

5.2 Strain measurements in the west abutment

Two vertical profiles using a total of 40 vibrating wire embedment type strain gauges were installed in the west abutment. Each profile comprised 10 pairs of gauges which were wired to the horizontal reinforcing bars such that one gauge of each pair was on the inside of the traffic face steel mat of the abutment and the other gauge was on the inside of the soil face steel mat. The pairs of gauges were installed at depth intervals of approximately 0.6m and both axial loads and bending moments induced in the abutment were derived from strain measurements. A layout of one of the profiles is shown in Figure 5; the other profile is nearly identical.

All vibrating wire strain gauges are affected by a change of temperature as the steel of the wire expands with a coefficient of thermal expansion of $11 \times 10^{-6}/\degree C$. Generally reinforced concrete will have a similar coefficient to that of the steel wire and the effects are then self-compensating. However, at this site, limestone aggregate was used in the concrete of the abutment and this is known to have a lower coefficient. Browne (1972) measured the coefficients of thermal expansion of various concrete mixes and found an average value of $7.4 \times 10^{-6}/\degree C$ for those containing limestone aggregate: this value was subsequently adopted for CIRIA Report 91 (1981). For the purpose of this study, strains from the vibrating wire gauges have therefore been corrected using the difference in coefficients of thermal expansion and the temperature change (measured using the internal thermistor incorporated in the strain gauge body). In this way the gauges are effectively temperature compensated for concrete containing limestone aggregate and strain changes will reflect changes in applied loading and movements arising from creep and shrinkage. It is worth noting that, after this correction, changes in thermal load in a restrained structural member will still be monitored.

5.3 Strain measurements in the deck

Profiles of vibrating wire strain gauges were attached to beams 4, 7 and 10 respectively near each of the abutments as shown in Figure 6. One profile, consisting of seven gauges was installed 3m from the west abutment with a second profile of six gauges being installed at 3m from the east abutment.

Four of the gauges in each profile were located on the neutral axis of the composite unit formed by the beam and the section of deck supported by that beam, so that axial strain could be directly measured. In the case of Bramham Crossroads North Bridge, the neutral axis has been identified as being 0.858m from the bottom of the deck beam. The remaining gauges in each profile were located on the flanges of the beams so that bending strain could be determined.

As both the precast beams and the deck were formed from a concrete mix containing limestone aggregate, the same approach to that described in Section 5.2 could have

Figure 5 Layout of instrumentation in and behind the bridge abutment
been adopted to eliminate the effects of thermal expansion from the strain gauge measurements. However, whereas changes in applied load were important in the case of the abutment, thermal movements were important for the deck. Strain gauge readings were therefore corrected to absolute strains using the coefficient of $11 \times 10^{-6}$/°C for the steel wire of the gauge together with the temperature as measured by the internal thermistor incorporated within the gauge body. Measured changes in absolute strain in the deck were therefore due to a combination of factors including creep and shrinkage deformation, thermal effects and change in the applied loadings.

Where axial loads and bending moments were determined for the composite unit formed by the beams and 200mm thick reinforced concrete deck, a cross-sectional area of 0.552m² per metre width and a second moment of area (I) of 0.143m⁴ per metre width were used respectively. The modulus used in these calculations was appropriate to whether short or long term loading was being considered and is specified in the text.

5.4 Deck temperatures

Temperatures within the 200mm thick deck were established from two profiles of thermocouples installed in pairs located above beams 4, 7 and 10 at a distance of 3m from each abutment. For each pair, installation was at heights of 50mm and 150mm above the precast beams, with the thermocouple cables routed through the deck to a cabinet near the north west end of the wing wall. Surface temperatures of the deck beams were monitored using thermistors incorporated in the strain gauges units, which were protected from changes in air temperature by a thermally insulated box.

\begin{figure}
\centering
\includegraphics[width=\textwidth]{figure6.png}
\caption{Arrangement of strain gauges on bridge beams}
\end{figure}
5.5 Earth pressures on the west abutment

The earth pressures acting on the abutment were monitored using pressure cells installed vertically on the drainage blockwork against the retained face of the west abutment. Two profiles of six cells were installed to measure the variation of lateral pressure with depth. The locations of the cells are shown in Figure 5. The cells were positioned at 1m intervals of depth with one profile (PC1 to PC6) starting at 0.5m above the drainage filter and the second profile (PC7 to PC12) starting at 1m above the drainage filter. A further two cells (PC13 and PC14) were installed within the backfill to measure vertical stresses at distances of 0.5m and 1.5m from the drainage blockwork behind the abutment.

The active diaphragm of each cell was 180mm in diameter and the cells were oil filled with the oil pressure being monitored by a vibrating wire transducer. Thermocouples were also attached to six of the pressure cells so that the temperature of the backfill was recorded. The instrument cables were again routed to a cabinet near the end of the wing wall.

When the backfilling reached a cell location, hand compaction was used against the face of the cells to prevent damage with the risk of point loading being minimised by sieving off all material greater than 10mm from the fill placed in contact with the cell face.

6 Observations during construction

During the construction period, monitoring of the instrumentation took place at regular intervals, so that changes associated with the various activities could be identified. On completion of the construction of the bridge, seasonal monitoring of performance commenced and these results are reported in Section 7. Initially readings were taken manually but from day 348 vibrating wire and electrical instruments were logged by computer.

6.1 Movement of the abutments and deck

To establish lateral movement profiles during the construction period, the inclinometer tubes in each abutment, were surveyed at regular intervals. Figure 7 shows the lateral movement measured on both abutments during this period. At the top of the abutments, the difference between the extremes of movement was of the order of 4mm. Inclinometer surveys carried out immediately before backfilling (14/7/97 in Figure 7) and shortly after completion of backfilling (13/8/97) showed only small movements towards the central pier of about 1mm at both west and east abutments.

The mean deck temperatures measured using the thermistors are shown in Figure 8a. The data in Figure 8b compare the lateral movements at deck level for each

![Figure 7 Lateral movements during construction of the bridge](image-url)
abutment during the construction of the bridge. These lateral movements are determined from the inclinometer tubes in the abutments assuming base fixity of the tubes. In general, the measured movements of the east and west abutments followed a very similar trend. Generally this trend up until opening of the bridge indicated a shortening of the deck and associated lateral movement of both abutments towards the central pier. This movement is probably time dependent and associated mainly with creep deformation of the prestressed beams which is discussed in Section 6.3, however temperatures were falling over the latter part of this period and thermal contraction of the deck would have also contributed to this behaviour.

6.2 Strain measurements in the west abutment

Figure 9 shows the axial strains and temperatures recorded by one of the profiles of vibrating wire strain gauges and thermistors located at various depths within the west abutment. The other profile of gauges and thermistors gave nearly identical results and is therefore not reproduced. The measured strains in Figure 9b have been corrected for the difference in the coefficients of thermal expansion of the concrete and the steel wire of the gauge as described in Section 5.2. A datum for the strain gauges was established on day 92 prior to the deck beams being placed and about 17 days after the abutment was cast. At this time the high abutment temperatures caused by the concrete hydration
process (Figure 9a) had nearly reduced to the ambient temperature although the continuing increase in strain with time indicated that drying shrinkage was still occurring over the construction period.

An assessment of the magnitude of the shrinkage deformation that would have been anticipated over 100 days and long term was obtained following the procedure given in Appendix C of BS5400: Part 4 (1990). From this text:

Shrinkage deformation = \( k_l \cdot k_c \cdot k_e \cdot k_j \)

where \( k_l \) is a partial coefficient which depends on the environment, \( k_c \) depends on the composition of the concrete, \( k_e \) depends on the effective thickness of the member and \( k_j \) defines the development of shrinkage as a function of time.

In the above equation, \( k_l \) was taken as \( 275 \times 10^{-6} \) assuming relative humidity for normal air and \( k_c \) was taken as 1.0 for the free water/cement ratio of 0.44 and total binder content of 395kg/m³. Normally cement content is used for the determination of \( k_c \), but total binder content was
considered appropriate in this case as the concrete mix included an equal proportion of cement and ggbs (ground granulated blast furnace slag). The value of $k_\text{r}$ depends on the effective thickness of the member, which is the ratio of its area to the semi-perimeter in contact with the air. As backfilling was towards the end of the construction, both faces of the abutment were exposed until then and the effective wall thickness is therefore 800mm. Extrapolation of the plot in Figure 15 of BS5400 indicates that $k_\text{r}$ is approximately 0.5. The term $k_\text{r}$ defines the time dependence of shrinkage and values of 0.1 and 0.5 were used at 100 days and 1000 days from which shrinkage deformations of 14 and 70 microstrain were calculated respectively. A key factor in the calculation of shrinkage strains in the abutment is its thickness of 800mm which is sufficiently large to limit the ultimate shrinkage and the proportion of the ultimate shrinkage that occurs in the first 100 to 1000 days.

Similar calculations were carried out using Appendix C of BS5400 to evaluate creep of the concrete section under the dead load from the bridge beams, the 200mm thick reinforced concrete deck and the abutment itself. As the abutment was a relatively thick member, vertical stresses ($f_v$) calculated from these dead loads were small as were the corresponding creep deformations. About 1 and 7 microstrain were calculated at 100 days and 1000 days after load application respectively.

The calculations of both the shrinkage and creep strains are likely to provide upper bound values because both the steel reinforcement and the cement replacement used for the concrete mix have not been taken into account. Both of these factors are expected to reduce the magnitude of shrinkage and creep that can be realised.

The magnitudes of the vertical load expected on the top of the west abutment were determined from the dead loads of the components of the deck. Since the bridge was not achieved until casting of the diaphragms, a dead load of 118kN/m from the precast beams was calculated for simply supported conditions. An additional 120kN/m was then imposed due to the dead load from the 200mm thick reinforced concrete deck and the in situ diaphragm, this load was assumed to be simply supported as the concrete load was imposed in a ‘wet’ state, that is before the bridge could be considered to be integral. Finally a further abutment load of about 48kN/m was expected from the surfacing and footways of the integral bridge. On this basis the total dead load on the west abutment was 286kN/m which, assuming a short term bridge. On this basis the total dead load on the west expected from the surfacing and footways of the integral bridge was not achieved until casting of the diaphragms, a further abutment load of about 48kN/m was therefore suggested that either (a) shrinkage strains in the abutment concrete were higher than estimated, (b) the strain correction for thermal expansion effects described in Section 5.2 was insufficient, or (c) some creep was occurring in the wire of the vibrating wire gauge. Of these, (a) is likely to be dominant particularly as high air temperatures and low humidity were experienced over part of the construction period, with some contribution possibly from (b): effect (c) is unlikely. It is interesting to note that closure agreement is achieved if short term changes are evaluated to minimise error from these various effects. For example, measurements between day 156 and day 162 during which time the west deck was cast, gave a mean change of 5 microstrain compared with the predicted change of about 3 microstrain. These strain levels were however so small that the comparison is very tentative.

As shrinkage and creep effects slow down with time, improved accuracy for the seasonal monitoring was obtained by using a new datum for the strain gauges in the abutment established at day 348.

### 6.3 Strain measurements in the deck

The variation with time of the strains measured using the deck neutral axis gauges is shown in Figure 10. Although there was continuing creep deformation of the prestressed beams throughout the construction period, there were increases in strain coinciding with the deck being cast, subsequent compression of the beams induced by drying shrinkage of the deck slab and the backfilling operation behind the abutments.

An estimate of the magnitudes of shrinkage and creep movement during the construction period was obtained following the procedures given in Appendix C of BS5400: Part 4 (1990). The concrete mix design of the deck was similar to that of the abutment and the same equation for shrinkage deformation (Section 6.2) was used with a $k_\text{e}$ value of 0.8 for an effective thickness of member of 0.2m. The term $k_\text{e}$ also varied because of the different thickness and values of 0.4 and 0.85 were used in calculating shrinkage deformations of 88 and 187 microstrain at 100 days and 1000 days respectively.

Information from the supplier indicated that the beams were cast about 100 days prior to the casting of the reinforced concrete deck and allowance for this was made in the calculation of their shrinkage deformation. The effective thickness of each beam was calculated as 0.3m by dividing the area of its section by the semi-perimeter and this gives a value for $k_\text{e}$ of 0.65. The coefficient of $k_\text{e}$ which depends on the composition of the concrete was about 0.9 for the cement content of 454kg/m$^3$ and free water/cement ratio of 0.4. The development of shrinkage with time took account of the initial 100 day period with $k_\text{e}$ values of 0.5 and 0.8 and additional shrinkage deformations of 32 and 81 microstrain being calculated at 200 days and 1100 days respectively (i.e. at 100 days and 1000 days after the slab was cast). Based on this simplistic evaluation there would be some compression of the beams as differential shrinkage between the deck and the precast beams of 56 and 106 microstrain would be expected at 100 days and 1000 days after casting of the deck. In practice differential shrinkage is not likely because the slab would be subjected to loads imposed by creep of the prestressed beams.
Generally only a minor part of the strain development shown in Figure 10 is accounted for by shrinkage effects and the major part is due to the significant creep deformation which develops in the prestressed beams due to their high compressive loading. This creep deformation was evaluated using the following formula from BS5400 (Part 4, 1990):

\[
\text{Creep deformation} = f_c \phi / E_{28}
\]

where \( f_c \) is the constant stress and \( E_{28} \) is the elastic modulus of the concrete at 28 days. The coefficient \( \phi \) is determined from the product of the partial coefficients \( k_l, k_m, k_c, k_e \) and \( k_j \). These are defined as in Section 6.2 with the addition of \( k_m \) which depends on the hardening of the concrete at the age of loading. It should be noted that some of the partial coefficients vary in magnitude from those used for shrinkage calculations.

The beams were assumed to have a \( k_l \) of 2.3 (for normal air), a \( k_m \) of 1.6 for load transfer at 3 days, \( k_c \) and \( k_j \) values as used for the shrinkage calculations, and a \( k_e \) of 0.75 for an effective thickness of 0.3m. Each beam at this site included 43 steel superstrands (18 of which were debonded near the ends of the beam) which were prestressed to an initial force of about 200kN. As the strain gauges on the beams were at 3m distance from their ends in the region where some superstrands were debonded, the initial prestressing force in the beam was calculated as 5MN at this location by only considering the fully bonded superstrands. However prestress losses on load transfer will occur not only because of creep but also because of relaxation of the steel and elastic deformation of concrete so the initial prestressing force was accordingly reduced by 10% to account for these latter effects. For the purpose of comparing with strain gauge data at the neutral axis, this prestressing force was assumed to be spread uniformly throughout the area of the beam. On this basis the calculated creep deformations were as shown in Table 3.

Overall the mean axial strains developed during the construction period were 336 microstrain for the west deck and 307 microstrain for the east deck as shown in Figure 10. The mean strain in both decks at day 260, that is 100 days after casting of the deck, was 256 microstrain. This
Table 3 Evaluation of creep deformation of prestressed beam

<table>
<thead>
<tr>
<th>Time since loading (days)</th>
<th>Creep deformation (microstrain)</th>
<th>Change in deformation from 100 days (microstrain)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>184</td>
<td>n/a</td>
</tr>
<tr>
<td>200 (ie. 100 after casting deck)</td>
<td>304</td>
<td>120</td>
</tr>
<tr>
<td>1100 (ie. 1000 after casting deck)</td>
<td>488</td>
<td>304</td>
</tr>
</tbody>
</table>

can be compared with the predicted shrinkage and creep of the beams of 32 microstrain and 120 microstrain respectively, that is a total of 152 microstrain. On this basis, a major part of the axial strain measured by the vibrating wire gauges during the construction period can be attributed to shrinkage and creep deformation with smaller compressive strains arising from backfill compaction and thermal expansion of the deck itself.

A new datum was established at day 348 (opening of the bridge) for evaluation of the seasonal monitoring data presented in Section 7.3.

6.4 Earth pressures on the west abutment

The variation of lateral stress from the backfill acting on the abutment with time and mean deck temperature is shown in Figure 11. Generally lateral stresses reduced over the period following backfilling as the deck contracted due to creep and shrinkage deformation and also because deck temperatures were falling as winter approached.

Figure 12 shows the distributions with depth of the lateral earth pressure acting on the west abutment after completion of the backfilling on day 225 and up until day 299. After backfilling, pressures on all of the cells (with the exception of cells 4 and 10) were consistent in magnitude to those that would be predicted from the coefficient of earth pressure at rest, \( K_o \), of 0.34 calculated from \( (1-\sin \phi') \) using \( \phi'_{peak} \) of 41°. Cells 4 and 10 were installed in particularly hot weather when the mean deck temperature was about 25°C and some arching of the backfill possibly developed over the cell face leading to an under-registration in stress. This arch is likely to break down under the seasonal cyclic loading and it is anticipated that cells 4 and 10 will then perform more typically.

Two cells were installed in the backfill at the same level (72.5mAOD) but at different distances of 0.5m and 1.5m from the drainage blockwork behind the abutment to measure vertical stresses. These cells were intended to give an indication of the effect of cyclic loading on wall friction and hence vertical stresses in the vicinity of the wall. During the construction period, measurements shown in Figure 13 from these cells gave near identical readings with vertical pressures of about 40kPa to 50kPa. A vertical stress of 47kPa was calculated from overburden considerations ignoring wall friction effects and this agreed well with the measurements.

7 Observations with time

Monitoring of the instrumentation continued during the first three years of the bridge being in-service. All vibrating wire and electrical instruments were logged by computer at six hourly intervals. Manual readings of abutment movements, vertical deflections of the bridge deck and changes in deck length were taken at regular intervals.

7.1 Movement of the abutments and deck

The lateral movement surveys, carried out during the first three years in-service, using inclinometer tubes I1 and I2 in the west and east abutments respectively are shown in Figure 14. The changes in movement of the top of the abutment, that is at deck level, during this period were of the order of 6mm for both abutments.

Figure 15 shows the variation in deck temperature and the corresponding movements at deck level calculated from the inclinometer data assuming base fixity of the abutments. The movements obtained on inclinometer tubes I1 and I2 were relatively similar which indicated that, during this period, there was little or no movement of the central pier. Generally a trend of shortening of the deck can be identified due to creep and shrinkage effects coupled with expansion and contraction of the deck due to temperature changes. Also shown in Figure 15 is the change in the half deck length recorded by electronic distance measurements using the Geomensor which were commenced on day 680 and for this purpose the initial readings on that day were arbitrarily assumed as the datum values. On a few occasions distance measurements could not be taken during the site visit because of inclement weather conditions. Changes in half deck length measured in this manner generally confirmed the trends in abutment movement recorded by inclinometer. The close agreement between the two techniques also validated the assumption of base fixity of the inclinometer tubes for the post-construction monitoring.

As discussed above, the movement results in Figure 15 show both time dependence due to creep and shrinkage effects, and temperature dependence due to thermal expansion and contraction of the deck. A crude evaluation of the magnitude of creep and shrinkage effects was obtained using a best fit regression analysis of the movement data against time. The mean of the inclinometer data for the west and east abutments gave a slope of 28 microstrain/year. This value can be compared with the creep and shrinkage of the prestressed beams of 233 microstrain between 100 days and 1000 days, ie. a rate of 94 microstrain/year, predicted using the approach of BS5400 and given in Section 6.3. This comparison is however particularly tentative as creep and shrinkage rates are expected to reduce with time, and creep rate may also reduce due to increased humidity in the winter months. A more detailed comparison of measured and predicted creep and shrinkage rates is given in Section 8.1.

The time dependence of the movements was then eliminated from the data given in Figure 15 using the equations which had been determined. For this purpose each movement data point was corrected by subtracting the
Figure 11 Variations in lateral pressure on the abutment with temperature during construction of the bridge
Figure 12 Variation of lateral pressure with depth during construction of the bridge
Figure 13 Variations in vertical earth pressure with temperature during construction of the bridge
Abutment cast: 17th March 1997
Deck cast: 10th June 1997
Datum: 24th June 1997

Abutment cast: 14th April 1997
Deck cast: 5th June 1997
Datum: 24th June 1997

Movement towards centre of bridge is negative
Base fixity of tube assumed

Figure 14 Lateral movements after opening of the bridge
Figure 15 Lateral movements at deck level after opening of the bridge
movement predicted to have occurred because of creep and shrinkage at that time. Following this procedure, best fit regression analyses of the corrected movements at the top of the abutments with deck temperature are shown in Figure 16. The slopes for each abutment were similar and gave an overall change of 0.351 mm/°C for the 50.2m long deck which corresponded to an apparent coefficient of thermal expansion of 7.0 × 10⁻⁶/°C. As previously discussed, a coefficient of about 7.4 × 10⁻⁶/°C might be expected for both the 200mm thick reinforced concrete deck and the precast beams as limestone aggregate was used in both cases.

The variations in level across the bridge deck measured at selected dates by precise levelling are shown in Figure 17. Generally little or no change in level was measured at the support points, ie. the abutments and central pier. Some hogging movement was recorded (relative to the datum taken in April 1999) at the midpoints between supports although this was generally less than 3mm and could not readily be correlated with changes in temperature of the deck. The magnitudes of the changes in level were small throughout and not viewed as particularly significant.

### 7.2 Strain measurements in the west abutment

Mean deck temperatures ranged from 0°C to +28°C during the first three years of seasonal monitoring as is shown in Figure 18a. From a datum established after the opening of the bridge, the changes with time in the bending and axial strains determined using vibrating wire gauges in the west abutment are shown in Figures 18b and 18c respectively. Although there may be some shrinkage (and a small amount of creep) of the abutment concrete still occurring, about 1000 days had elapsed by January 2000 from the concrete pour and long term deformation of the relatively thick section with time was expected to be small. This was confirmed by comparing the mean measured strains in Figure 18c at day 350, day 715 and day 1080 when deck temperatures were nearly identical, this suggested that a change of about 30 microstrain had occurred probably due to shrinkage by day 715. Little further change occurred in the mean strain measured a year later on day 1080.

The results in Figure 18c indicate that tensile axial strains in the abutment tended to develop during the summer with compressive strains occurring during the winter months. Although wall movements were insufficient to develop large lateral pressure changes in the backfill behind the abutment, reversal in the direction of wall friction may well have occurred which could partly account for these strain changes. When the abutment is moving away from the retained ground, wall friction will produce more compressive strain, and vice-versa. If the magnitude of wall friction is roughly estimated from \( \frac{1}{2}K \gamma h \tan \delta \) where \( h \) is the depth of backfill and full wall friction (\( \delta \)) is assumed to be mobilised, a frictional force of ±220kN is determined. If a short term E value of 34×10⁶ kN/m² is then assumed for the concrete this corresponds to an axial strain change of ±9 microstrain. After allowing for 30 microstrain caused by shrinkage effects, the measured seasonal change of about ±15 microstrain from Figure 18c is then of the same order of magnitude as that predicted for wall friction.

Bending strain measurements in Figure 18b show only small fluctuations with time and temperature. It is worth noting that whereas axial strain measurements in the abutment are influenced by shrinkage and creep effects, bending strains are likely to be less affected. A correlation between wall bending moments and changes in lateral earth pressures and deck axial loads is attempted in Section 8.3.

Results from the second profile of strain gauges in the west abutment are very similar to those of the first profile and are therefore not presented in this report.

### 7.3 Strain measurements in the deck

Measurements during the first three years in service of the axial strains developed in the deck beams are compared with temperature changes in Figure 19. All the gauges at the neutral axis performed similarly in showing a trend of increasing compressive strain with time which was broadly consistent with that expected mainly from creep with some shrinkage of the prestressed beams. Changes ranging from 100 to 200 microstrain were measured at the start of 2000 when the deck temperatures were similar to those recorded when the strain gauges were zeroed at the end of 1997. These measured changes over a two year period can be compared with the predicted creep and shrinkage of 233 microstrain over 900 days (ie. between 100 and 1000 days) as determined in Section 6.3. A more detailed comparison of measured and predicted creep and shrinkage rates is discussed in Section 8.1.

In addition to creep and shrinkage deformation, the results in Figure 19 show a seasonal effect consistent with that expected from thermal expansion and contraction of the deck. An increase in compressive axial strain occurs in the winter when deck contraction occurs and the converse in the summer. The peaks and troughs in the strain plots correlated well with the deck temperature changes. These results are broadly consistent with the seasonal changes in lateral movement of the abutments and deck already described in Section 7.1.

Figure 20 shows the time variation of deck bending strains measured at 0.558m below the neutral axis and compares the results with the deck temperature changes. There was considerable scatter in the results possibly due to localised thermal gradients, although generally some sagging strains developed over the first three years in service. It must be noted that the deck gauges were located at only 3m away from the abutments and that the sagging strains may therefore be due to the fixity provided by the integral abutments. Additional gauges would have been useful if placed midway between the abutment and central pier where hogging strains may have developed, however access to fix gauges to the beams at this location was not available at the time of instrumentation. With conventionally articulated prestressed beams, a combination of the spatial distribution of prestressing strands and the development of creep in the concrete would generally induce hogging strains due to the higher compressive strains towards the base of the beams.
Figure 16 Variation in lateral movement of the top of the abutments with deck temperature after opening of the bridge
Figure 17 Variation in deck level after opening of the bridge
Strains are temperature compensated

Figure 18 Variations in abutment strains with temperature after opening of the bridge (Profile 1)
Figure 19 Strains measured in the deck neutral axis gauges after opening of the bridge
Figure 20 Variations in deck bending strains with temperature after opening of the bridge
7.4 Earth pressures on the west abutment

Plots of the variation of lateral pressure with time over the bridge’s first three years in service are compared with the variation in mean deck temperature in Figure 21. During this period the deck temperatures, as previously mentioned, range from 0°C to +24°C and the pressures developed behind the abutment rose and fell with mean deck temperature. Over the monitoring period the maximum measured stresses generally did not exceed 75kPa, with the exception of cell 5 (located just below the base of the deck) which peaked at approximately 130kPa.

The distributions with depth of the lateral earth pressure acting on the west abutment for selected dates during the period of seasonal monitoring are shown in Figure 22. During the first three years in service, creep with some shrinkage of the prestressed beams induced abutment movement away from the retained ground which acted to limit the development of lateral earth pressures against the abutment. Nevertheless during the summers, pressures exceeded those calculated from the earth pressure at rest coefficient \(K_a\) using a \(\phi'_{peak}\) of 41° (see Section 4). In the winters, pressures reduced and were, with some exceptions, between \(K_c\) (active earth pressures calculated from \(1-\sin\phi'/\sin\phi\)) ignoring the effects of wall friction) and \(K_o\) pressures. The low pressures measured by cells 4 and 10 at around 71mAOD were the main anomalies probably because of arching of the backfill over the cell face at the time of installation (see Section 6.4). This arch is likely to break down under the seasonal cyclic loading, and there are signs of this already happening with the latest data for cell 10. Although it is anticipated that the same behaviour will be exhibited by cell 4, it has not yet responded in this way. In the longer term it is possible that, densification of, and stress escalation in, the backfill will eventually occur because of thermal cyclic loading as the effects of creep slowly diminish.

Figure 23 shows a time plot of the vertical stresses in the backfill measured at 72.5mAOD and at distances of 0.5m and 1.5m from the drainage blockwork behind the abutment. Little seasonal variation in stress is evident on cell 14 at 1.5m from the drainage blocks. However cell 13 nearer to the blocks shows a distinct seasonal variation with lower stresses being recorded in the winter than the summer. This is generally consistent with the reversal in direction of wall friction that would be expected from earth pressure theory, and also indicates that the effect of wall friction is only discernible very close to the wall. A further discussion of wall friction effects is given in Section 8.4.

8 Discussion

8.1 Creep and shrinkage effects

An overview comparing the absolute strains measured in the deck with those predicted for creep and shrinkage is given in Figure 24. In this figure the mean measured strains were determined both from the vibrating wire strain gauges located at the neutral axes of the east and west decks and also from the inclinometer surveys of the tubes in the east and west abutments. The strain gauge results are therefore essentially those already presented in Figure 10 for the construction period with added data from Figure 19 for the long term monitoring. All these results are therefore calculated from datum values established at the time the deck was cast. Equivalent strains were calculated from the inclinometer data in Figures 8 and 15 by summation of the lateral movements at the top of the east and west abutment and dividing by the deck length of 50.2m. On this basis, strains from the inclinometer data were lower than those measured using the strain gauges. This discrepancy however developed during the construction phase and was accounted for by two factors. Firstly, a slight difference in the datum dates adopted for the two techniques but, more importantly, the assumption of base fixity of the inclinometer tubes in the abutment walls was probably invalid during the backfilling stage. This activity is likely to have resulted in compression of the rubber liner to the socket of the spread bases into which each abutment wall is seated. For this reason, the equivalent strains determined from the inclinometer results are also plotted from day 348 (opening of the bridge) with appropriate correction to the strain gauge data at this time. From this time onwards reasonably good agreement was found in Figure 24 between the deck strains determined using the two different measurement techniques.

The predicted strains due to shrinkage and creep were determined following the procedures given in Appendix C of BS5400: Part 4 (1990) and detailed in Sections 6.2 and 6.3 of this report. In these plots, allowance has been made for the 100 day age of the prestressed beams prior to casting of the deck on day 160. As anticipated, creep deformation is a larger effect than that of shrinkage.

In general a similar trend of a continuing small shortening of the deck was determined both by measurement and prediction. If anything the predicted creep and shrinkage deformation from BS5400 slightly exceeded that which was measured. The measured strain values also show the seasonal effects of thermal expansion and contraction of the deck in the summer and winter respectively. As Figure 24 shows absolute values of compressive strains, deck expansion in the summer months induces a shortening of the deck and hence an apparent reduction in compressive strain. This expansion of the deck acts to produce an increased load on the abutments and also an increased compressive load in the deck when allowance is made for the thermal coefficient of the concrete of the deck.

It is worth noting that after three years in service, the approach of BS5400 would predict that 87% of the creep and shrinkage deformation of the deck has occurred. During this period, creep and shrinkage has produced small movements of the bridge abutments away from the retained ground so relieving any high lateral stresses in the backfill. In the longer term, creep and shrinkage effects will be negligible and the dominant effect is then likely to be that of thermal expansion and contraction of the deck. This seasonal and diurnal cyclic action may lead to densification of the backfill and an increase in the peak lateral stresses acting on the abutments in the summer months.
Figure 21 Variations in lateral pressure with temperature after opening of the bridge
Figure 22 Variation of lateral pressure with depth after opening of the bridge
Figure 23 Variations in vertical earth pressure with temperature after opening of the bridge
Figure 24 Comparison of measured and predicted deck strains
8.2 Effective temperature of the bridge deck

Longitudinal movement of the deck of a bridge with bearings and expansion joints is directly related via the coefficient of linear expansion to temperature, but the effective temperature of the bridge is difficult to measure. Because thermal radiation, wind speed and shade temperature are variable quantities the distribution of temperature through the depth of a bridge is complex. Emerson (1976) reported on the summer and winter distributions of temperature for a concrete bridge of a similar design to that at Bramham with the deck slab supported by concrete beams. Emerson found that typically the daytime temperature of the deck slab peaked at about 16.00 hours (GMT) in the summer and at this time may exceed the temperature of the deck beams by 5°C or more. Generally variation between slab and beam temperatures was only of the order of 2°C on a typical winter day. Temperature measurements from the bridge at Bramham followed a similar pattern (Figure 19). Daytime temperatures from the thermocouples in the deck slab usually slightly exceeded those measured by the thermistors on the deck beams particularly in the summer months.

In an integral bridge situation where there are no bearings or joints, longitudinal expansion of the deck caused by an increase in temperature is subject to the restraints offered by the abutment backfill and the flexural rigidity of the abutment itself. In this case therefore, longitudinal movement may be slightly reduced from that calculated using the coefficient of thermal expansion although some build-up in compressive strain in the deck and lateral stresses in the backfill would be expected.

8.3 Forces and moments acting on the west abutment

The performance of integral bridges is a complex soil-structure interaction problem, however the normal laws of static equilibrium of horizontal forces and moments still apply at any one time. For this reason, the magnitudes of the changes in lateral earth pressures, deck axial loads and abutment bending moments were further investigated. Two periods were studied, one from day 708 to day 945 and the other from day 1182 to day 1322, that is from winter to summer 1999 and 2000 respectively. Figure 25 compares the changes in lateral earth pressure and the associated changes in bending moment developed in the abutment over the two periods. The changes in deck temperature were 16°C and 9.3°C over the two periods. The results in Figure 25 clearly demonstrate that higher earth pressures and wall bending moments developed towards the top of the abutment for the higher temperature change.

If the total horizontal loads corresponding to the changes in lateral stress are determined from Figure 25, changes in abutment load of 205kN/m and 133kN/m are found for the two periods. Changes in apparent deck load, calculated from the mean strains determined using the inclinometer surveys and vibrating wire gauges, are much higher. Values of 2073kN/m and 1845kN/m are calculated for the changes in deck temperature of 16°C and 9.3°C respectively. Although these latter values conform well with those predicted using a coefficient of thermal expansion of $7 \times 10^{-6}$/°C for the deck, this apparent and large incompatibility in load changes on the abutment and in the deck arises for a number of reasons as follows:

i. Worst case loads in the deck have been determined from measured deck strains by assuming full restraint at the ends of the deck. There is evidence from the pressure

![Figure 25 Changes in lateral stress and bending moment on the west abutment](image-url)
cell plots in Figure 22 that lateral stresses acting on the abutments fall below their active values in a cold winter. If there is slight tension cracking behind the abutment, expansion of the deck will occur to close the cracks before the deck load will increase. Actual deck loads will therefore be well below those calculated from apparent strain.

ii The resisting load over the top 1.5m of the abutment has not been taken into account. Lateral stresses are likely to be highest in this area and the resistance of the pavement structure may be significant.

iii The load transferred by the abutment into the socket of the spread base has not been considered.

In the context of (i), it is worth noting that cracks with widths of up to 4mm were evident in the asphaltic pluggoints in the carriageway surface above the line of both east and west abutments during the winter of 2000/2001. Although this may be associated with the seasonal thermal deck movements, it is expected that the material of the joints would creep and tolerate these movements. In general, it is reported that the viscoelastic material of the joints is more susceptible to movements at high frequency such as those induced by traffic (Barnard and Cunninghame, 1997).

### 8.4 Wall friction effects

Wall friction is an important consideration in the design of an integral bridge abutment especially because of its considerable influence on the magnitude of the lateral pressure developed when the abutment is pushing into the retained ground due to thermal expansion of the bridge deck. In addition to the pressure cells measuring lateral stress acting on the abutment, two cells (PC13 and PC14 in Figure 5) were installed to measure vertical stress in the backfill close to the abutment. The variation of the measured vertical stresses with time has already been presented in Figure 23 and showed lower stresses when the bridge deck is contracting and lateral stresses are approaching the active condition.

A more detailed evaluation of wall friction was undertaken by considering the relative magnitudes of the lateral stress on cell PC6 and the vertical stress on cell PC13 at 0.5m away from the abutment. Both of these cells were installed at about 2.3m below the carriageway surface. The frictional stress acting vertically at 0.5m away in the backfill was then calculated by deducting the measured stress on cell PC6 from the vertical overburden. This stress equates to $K \gamma h \tan \delta$ where $K \gamma h$ is measured directly by cell PC6, in this way the angle of internal friction can be calculated using the following equation.

\[
\text{Friction angle (} \delta \text{)} = \tan^{-1} \left( \frac{\text{overburden-reading on PC13}}{\text{reading on PC6}} \right)
\]

Figure 26 shows the seasonal variation of the friction angle calculated in this way and compares it with the changes in deck temperature. In this figure, a positive angle of friction represents stresses tending towards the earth active condition as the abutment moves away from the retained ground. Conversely, a negative angle of friction develops when passive movements occur.

In Figure 26, the pattern of the development of wall friction is much as expected from earth pressure theory with the direction of wall friction tending to revert from negative values (ie. upward force on the wall) when the deck temperature is increasing to positive values when the temperature is decreasing. In theory, the absolute value of the angle of wall friction would not be expected to exceed the angle of internal friction ($\phi_{int}$) of 41° for the backfill (see Section 4). The maximum absolute value is only about 20° when deck temperatures are increasing and this is consistent with the lateral pressures which exceed $K_o$ but fall well short of their full passive values. Apparent values of friction angle exceeding 41° and typically as high as 60° are determined when the abutment is in ‘active mode’. Plots of lateral pressure given in Figure 22 confirm that winter values are in the fully active state. However the anomaly of wall friction values exceeding 41° is not considered real as the calculation is very sensitive when only small pressures are recorded on pressure cell PC6 which is in the denominator of the equation.

The above calculations must be deemed as tentative. Firstly, because cell PC6 is 0.5m away from the abutment and may not reflect wall friction accurately and secondly, because minor differences in cell performance may occur if the density of the soil over the two pressure cells is slightly different. Furthermore no allowance in the calculations has been made for wall adhesion; even though the backfill is granular, laboratory tests suggest that the soil has a small cohesion.

### 9 Conclusions

The field performance of an integral bridge over the new M1-A1 Link Road at Bramham Crossroads in North Yorkshire has been evaluated. The bridge deck has two spans totalling about 50m and comprises prestressed concrete beams composite with a 200mm thick reinforced concrete slab. The deck beams are structurally connected to full height reinforced concrete abutments (800mm thick) which are founded in Magnesian Limestone. Field measurements were obtained during construction and over the first three years in service and the following conclusions reached:

i Inclinometer measurements of lateral movements of the east and west abutments during the construction period generally indicated a shortening of the deck associated with creep and shrinkage deformation of the prestressed beams coupled with expansion and contraction of the deck due to temperature changes. Creep and some shrinkage deformation of the beams continued to be very significant effects over the first three years in service, although their overall deformation rate was marginally less than that estimated from BS5400. When the time dependence of the movements was eliminated from the data, the measurements indicated an apparent coefficient of thermal expansion of $7.0 \times 10^{-6}/°C$ for the deck. This value is slightly less than the $7.4 \times 10^{-6}/°C$ that would be expected for the concrete mixes using limestone aggregate which were used for both the deck and beams.
Figure 26 Variation of wall friction on west abutment
ii Because of the speed of construction, datum readings on the strain gauges in the west abutment had to be taken before the major part of the drying shrinkage of the concrete was complete. For this reason, measured strains over the construction period were in excess of those calculated from the dead load of the structural members of the integral bridge: however short term strain changes, although small, showed better correlation with applied loading. From a new datum at the start of the first year in service, tensile axial strains in the abutment tended to develop during the summer with compressive strains occurring in the winter months. These changes were consistent with a seasonal reversal in the direction of wall friction.

iii Measurements, together with predictions using BS5400, of creep and shrinkage strains in the west abutment indicated that these effects were small and of no great significance in abutment design. This was primarily because the reinforced concrete abutment was a relatively thick member.

iv The major part of the axial strain measured by the vibrating wire gauges on the prestressed deck beams during the construction period can be attributed to shrinkage and creep deformation with only small compressive strains arising from backfill compaction and thermal expansion of the deck itself. Measurements during the first three years in service showed a seasonal thermal effect together with a continuing trend of increasing compressive strain with time which was broadly consistent with that expected mainly from creep of the prestressed beams. The results after construction showed reasonably good agreement with strains calculated from inclinometer data. In general a similar trend of continuing shortening of the deck was determined both by measurement and predictions of creep and shrinkage deformation based on BS5400.

v After backfilling, measured lateral earth pressures acting against the abutment were consistent in magnitude to those that would be predicted from the coefficient of earth pressure at rest ($K_a$) calculated using $\phi'$ peak of 41°. During the first three years in service, creep with some shrinkage of the prestressed beams induced abutment movement away from the retained ground which acted to limit the development of lateral earth pressures against the abutment. Nevertheless during each summer, pressures rose to slightly above the $K_a$ values although, in the following winters, they reduced to values generally between $K_a$ and $K_s$. More longer term monitoring is needed to ascertain whether or not densification of, and stress escalation in, the backfill eventually occurs because of thermal cyclic loading as the effects of creep slowly diminish.

vi A correlation between changes in lateral earth pressures, deck axial loads and abutment moments was carried out for two winter to summer periods. Measured changes in deck load calculated from strains significantly exceeded those calculated from lateral stress measurements on the abutments. This apparent incompatibility probably occurred because the resisting load from the backfill and pavement structure towards the top of the abutment and the load transferred to the spread base footing were unaccounted for. Any slight tension cracking between the backfill and the abutment in the winter would also contribute to the discrepancy.

vii An evaluation of wall friction based on pressure cell measurements indicated that the development of wall friction is much as expected from earth pressure theory. The direction of wall friction tended to revert from negative values when the deck temperature was increasing to positive values when the temperature was decreasing. In theory, the absolute value of the angle of wall friction would not be expected to exceed the angle of internal friction ($\phi'_{peak}$) of 41° for the backfill. The maximum value is only about 20° when deck temperatures are increasing and this is consistent with the lateral stresses which exceed $K_s$, but fall well short of their full passive values. Stress measurements confirm that winter values are in the fully active state and apparent values of friction angle exceeded 41°. However these calculations were generally deemed tentative for a number of reasons detailed earlier.

viii After three years in service, the approach of BS5400 would predict that 87% of the creep and shrinkage deformation has occurred. During this period, this deformation has produced small movements of the bridge abutments away from the retained ground so relieving any high lateral stresses in the backfill. In the longer term, creep and shrinkage effects will be negligible and the dominant effect is then likely to be that of thermal expansion and contraction of the deck. This seasonal and diurnal cyclic action may lead to densification of the backfill and an increase in the peak lateral stresses acting on the abutments in the summer months. The extent of this stress increase and its significance can only be evaluated by further long term monitoring. The study so far has provided an invaluable insight into the relative magnitudes of shrinkage deformation, creep deformation and thermal cyclic effects. However, the major investment in instrumentation at this site is such that it is strongly recommended that a further three year period of monitoring would provide a unique case history study of the fundamental behaviour of integral bridges of this type.

10 Acknowledgements

The work described in this report forms part of the research programme of the Structures Department of TRL and was funded by Quality Services, Civil Engineering (Project Manager: Dr D I Bush). The authors are indebted to Mr P Darley, Mr M R Easton, Mr D P Steele, Mr G H Alderman and Mr M D Ryley of TRL for their assistance with instrumentation and monitoring. The advice of Dr D W Cullington (TRL) on prestressed beams was also much appreciated.

Thanks are due to Yorkshire Link Ltd and the Highways Agency for permission to undertake the study at this site.
and for the co-operation and advice of Mr C Howard, Mr A R Ball, Mr A Baldwin (Yorkshire Link Ltd), Mr G R Campling, Mr G Smith, Mr A Causer and Mr R Atha (Babtie Group), Mr A R Green (Pell Frischmann Consultants Ltd) and Mr A Briggs (Highways Agency).

The co-operation and assistance of the site staff of the Construction Joint Venture (Kvaerner/ Balfour Beatty) in particular Mr P Long, Mr J Corvin, Mr M Wood and Mr S Phipps was appreciated and is gratefully acknowledged.

11 References


Volume 1: Section 3 General Design

BA42 The design of integral bridges. (DMRB 1.3)

BD57 Design for durability. (DMRB 1.3.7)


Volume 1: Specification for Highway Works. (MCHW1)

Abstract

This report describes the construction of an integral bridge on the M1-A1 Link Road. The bridge deck has two spans totalling approximately 50m and comprises prestressed concrete beams composite with a 200mm thick reinforced concrete slab. The deck is structurally connected to full height reinforced concrete abutments. Field instrumentation and measurements during construction are described together with seasonal effects arising from thermal expansion of the deck during its first three years in service. The findings are relevant to further updating of BA42 (DMRB 1.3) on the design of integral bridges.

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