TRANSPORT RESEARCH LABORATORY

Performance of the two integral bridges forming the A62 Manchester Road Overbridge

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Executive Summary

BA42 (The Design of Integral Bridges) gives advice on the design pressures relevant to three main classes of integral abutment, ie. shallow height, full height portal frames and embedded wall abutments. Field measurements on a shallow height abutment have been previously reported, however few data are available on the performance of full height abutments. This study reports on the performance of two full height integral bridges of about 40m span over the Manchester Outer Ring Road. One of the bridges (Phase 2) was constructed with contiguous bored pile abutments founded in glacial till. The other bridge (Phase 1) was a more conventional portal frame structure with the abutments retaining granular backfill.

Instrumentation was installed on both bridges to measure movements of the abutments and changes in deck length, wall and deck loads and bending moments, deck temperatures and lateral earth pressures acting on the abutments. This report describes the measurements during construction and over the first two years in service.

Measurements indicated that the coefficients of thermal expansion of the deck were $9.6\times10^{-6}/^{o}C$ and $9.25\times10^{-6}/^{o}C$ for Phase 1 and Phase 2 bridges respectively. These values were consistent with that expected when limestone aggregates are used in concrete (BD37, DMRB 1.3). Axial deck loads during construction and due to deck temperature changes were higher with the stiffer reinforced concrete abutments than with the bored pile abutments. In the design of integral abutments and where the deck is cast in place, it would appear prudent to allow for the total bridge dead load being equally shared between the supports as the assumption of simply supported spans may not be appropriate.

The findings from this project are expected to be of value in future updates of BA42 (DMRB 1.3).

1 Introduction

Integral bridge abutments are attracting increasing interest in the UK following the issue of BD57 (DMRB 1.3) encouraging their use. In principle it is recommended that all bridges of up to 60m length and skews not exceeding 30^o have continuous decks that are integral with their abutments. This policy is expected to reduce maintenance costs compared with those of conventional jointed bridges where there is a risk of de-icing salts penetrating the deck and substructure resulting in durability problems.

However, with integral bridges, design uncertainties exist because seasonal and diurnal thermal expansion of the deck causes cyclic movements of the abutments which may result in high earth pressures acting on the abutment. These earth pressures may also increase with time because of the effect of 'strain ratcheting' induced by many cycles of thermal movement (Broms and Ingleson, 1972; England and Dunstan, 1994; Card and Carder, 1993). BA42 (DMRB 1.3) gives advice on the design pressures relevant to three main classes of integral abutment, ie. shallow height, full height portal frames and embedded wall abutments. This design advice on pressures was largely based on the findings of Springman et al (1996), who undertook a series of centrifuge and analytical studies, and on the results of a field study on a shallow abutment of an integral bridge on the M74 reported by Darley et al (1996).

More recently the opportunity has arisen to complement the latter field study with performance measurements on the full height abutments of the twin decks of 40m long integral bridges under construction to carry the A62 over the M66 Manchester Outer Ring Road (Denton to Middleton). Originally both bridges were designed with contiguous bored pile abutments and a continuous reinforced concrete deck. However problems with one of the bridges necessitated remedial works which effectively changed the design to that of a conventional portal frame structure. Because of this change it became possible to compare the performance of two integral bridges with similar decks but with abutments of different stiffnesses, in one case retaining granular backfill and in the other the *in situ* glacial till.

Instrumentation was installed on both bridges to measure movements of the abutments and changes in deck length, wall and deck loads and bending moments, deck temperatures and lateral earth pressures acting on the abutments. This report describes the measurements during construction and over the first two years in service.

2 Location and description of the bridge

The Manchester Road Overbridge carries the A62 over the section of the M66 Manchester Outer Ring Road which is being constructed under Contract 2 (Medlock to Irk - Advanced Side Roads). Due to construction phasing, the scheme was constructed as two separate bridges (Phases 1 and 2) side by side with a continuous reinforced concrete deck of 1.7m thickness being used in each case. In addition to the continuous deck, the original design included integral abutments and a central pier formed by contiguous bored piles constructed under bentonite and founded in glacial till. As discussed further in Section 4, problems with the bored piling for Phase 1 (the south bridge) necessitated remedial works which effectively changed the design to that of a conventional portal frame structure with reinforced concrete abutments retaining granular backfill. Construction of Phase 2 (the north bridge) employed integral bored pile abutments as originally planned.

Sections through the Phase 1 and 2 bridges giving the spans of each deck and further details of construction are shown in Figure 1. The bridges were designed for HA loading and for HA loading combined with 45 units of HB loading (BD37, DMRB 1.3).

3 Soil properties

3.1 *In situ* ground

At the site, the ground conditions are glacial till (Boulder Clay) overlying moderately strong sandstone at about 23m depth. The nature of the till is highly variable although it is predominantly a firm to stiff brown sandy or silty clay. Undrained shear strengths on 100mm diameter triaxial specimens and Atterberg limits obtained from a TRL borehole in the instrumented area are plotted against depth in Figure 2. Generally there was poor correlation between strength and depth, with plasticity indices ranging from 10% to 26%.

Consolidated undrained triaxial tests with pore pressure measurement were carried out on 38mm diameter specimens from 4.3m and 7.1m depth. Effective stress strength parameters of $c'=0$ and $\phi'=32.5^o$ were determined at 4.3m depth and $c'=9kN/m^2$ and $\phi'=25.5^o$ at 7.1m depth.

3.2 Backfill

The backfill used against the structure for the remedial works during Phase 1 construction and for the 2.5m deep reinforced earth fill behind each abutment complied with Class P requirements in Table 6/1 of the Specification for Highway Works (MCHW1). The source of the fill was Bardon Highmoor Quarry. An investigation of the particle size distribution showed that 100% by mass passed the 37.5mm sieve and only 12% passed the 63 micron sieve. Nuclear density gauge tests on the compacted backfill gave an average dry density of $2.04Mg/m^3$ and moisture content of 8.1%. Determination of strength by direct shear using a 300mm shearbox apparatus gave effective angles of internal friction ϕ'_{peak} of 42^o and $\phi'_{residual}$ of 37^o at maximum travel of the machine.

4 Construction sequence

The construction sequence for Phase 1 (the south bridge) is summarised in Table 1 and commenced with the installation of contiguous bored piles for the two abutments and the central pier. The piles for the abutments were 1200mm in diameter and installed at 1500mm centres with pile lengths in the instrumented area of 29.4m. Pile

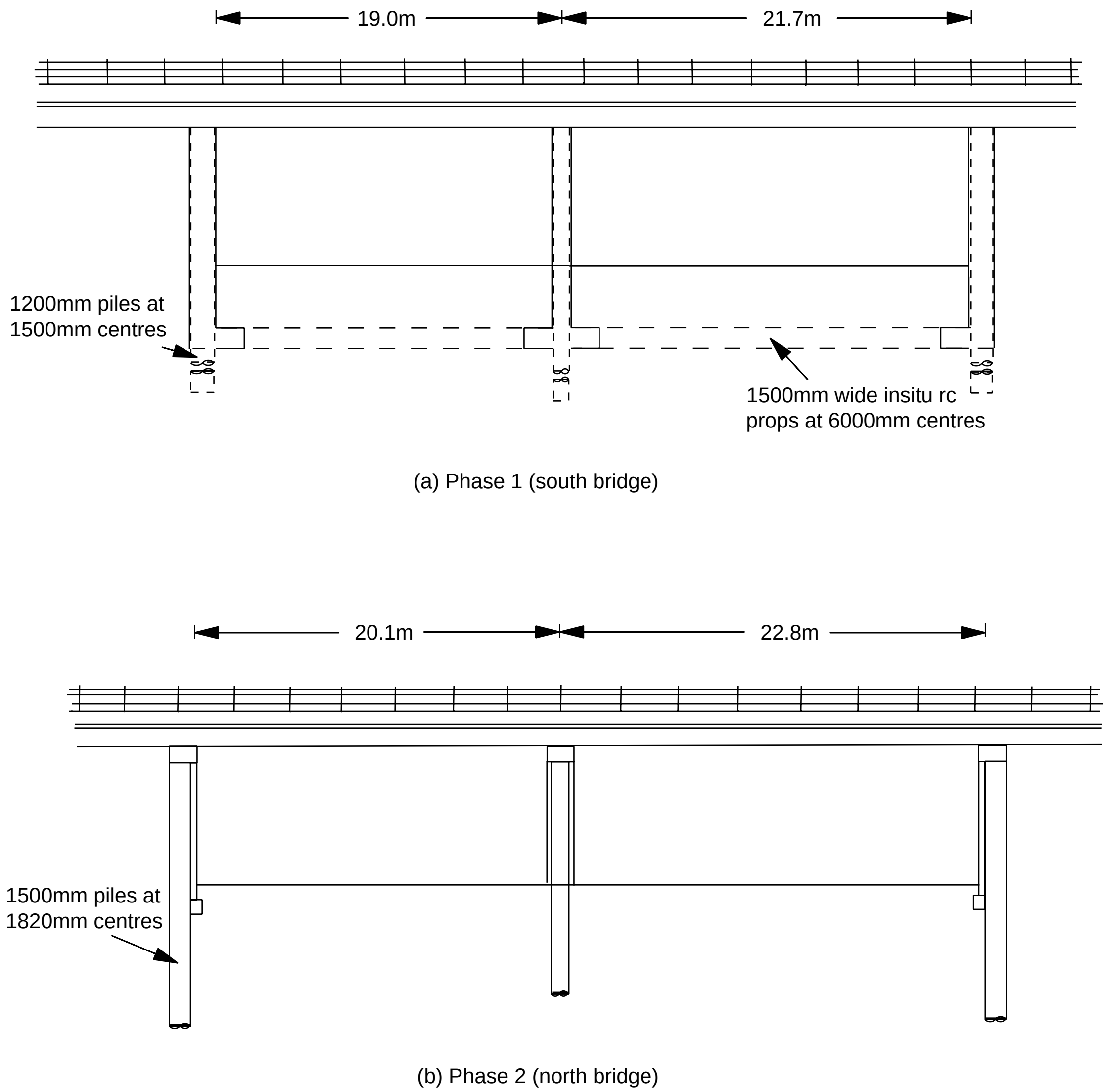


Figure 1 A62 Manchester Road Overbridge (sections)

diameters for the central pier were 750mm and these piles were installed at 1030mm centres. The pile boreholes were rotary augered using a short length of casing at the top and bentonite slurry to provide temporary ground support. After the reinforcing cage had been lowered into position, concrete was tremied to the bottom of the hole and the displaced bentonite returned to storage.

On completion of the piling, the pile tops were trimmed and the formwork placed ready for deck construction. The deck consisted of 1.7m thick reinforced concrete which was cast as a continuous length, ie. with no construction joints. Subsequent to deck construction for Phase 1, the formwork was removed and bulk excavation then took place beneath the deck. On exposure of the front face of the piles, many of them were found to be defective and remedial works were therefore necessary.

An outline section showing the remedial measures is given in Figure 3. A system of low level props comprising 1.5m wide *in situ* reinforced concrete slabs at 6m centres acting on walings cast against the piles was installed to ensure base stability of the abutment. A reinforced concrete facing of thickness 0.4m was then cast on the road face of the piles and a facing of 0.3m thickness on the earth face. The final design was therefore that of a reinforced concrete portal frame structure with the abutments propped at low level. As shown in Figure 3, infill concrete was placed in the excavation at depth on the retained side of the wall. Imported granular fill (see Section 3.2) was then used to complete the backfilling with the uppermost 2.5m being of reinforced earth construction employing high adherence steel reinforcing strips and Terratrel facing. The reinforced earth construction was part of the original design and intended to prevent settlement of the pavement construction for the bridge approaches. Drainage behind the abutments was ensured using a permeable backing of hollow porous concrete blocks. Where the reinforced earth met the abutment a 25mm layer of Aerofill joint filler was additionally used as shown in Figure 3.

Because of the problems with the Phase 1 construction, modifications were made to the pile design for the abutments of Phase 2. Pile diameters were increased to 1500mm and the distance between pile centres to 1820mm, also the pile boreholes were cased to between 6m and 8m depth. Details of the construction sequence are

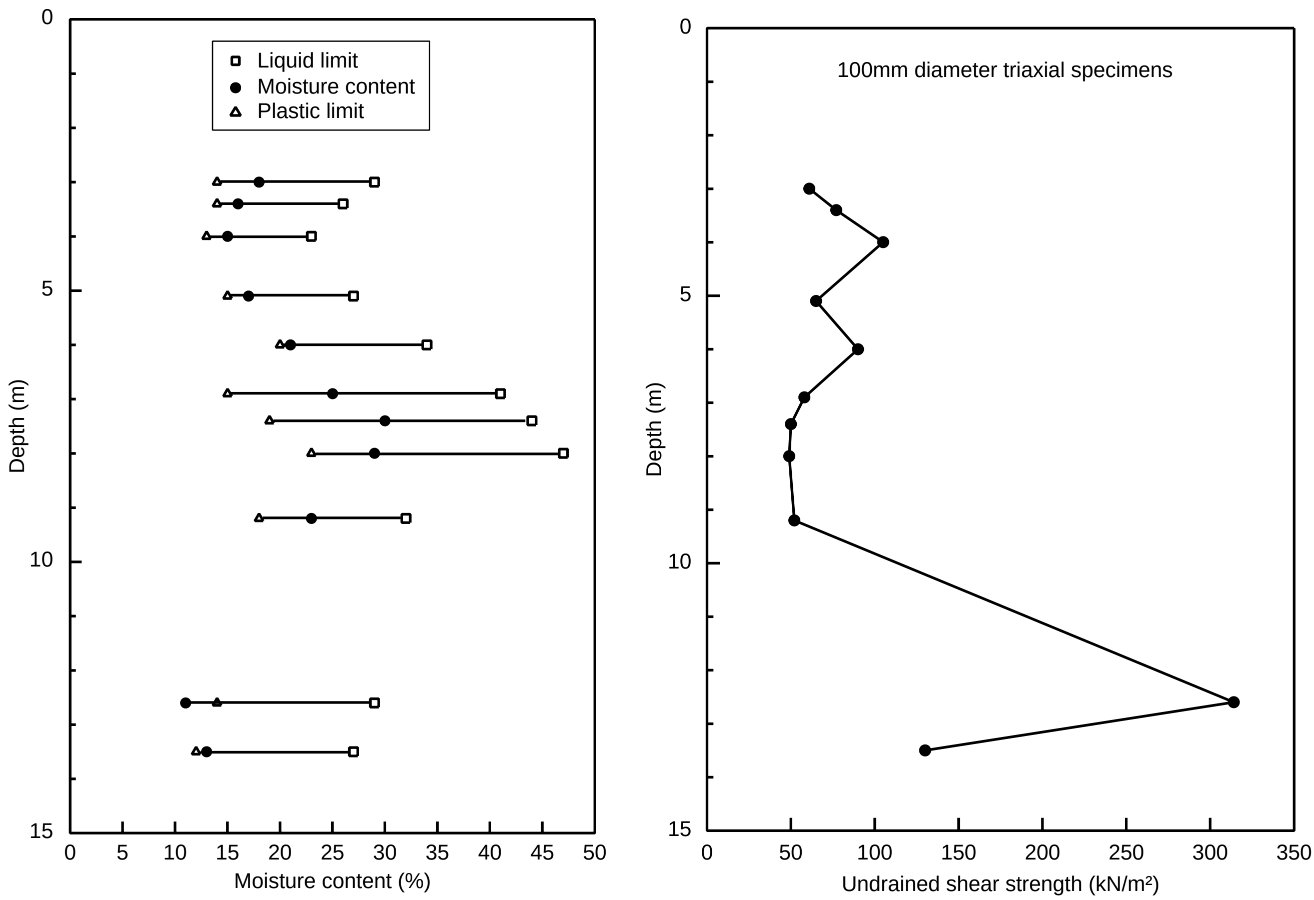


Figure 2 Properties of the glacial till

Table 1 Instrumentation and construction schedule for Phase 1

Instrumentation	*Date installed (day number)*	*Construction stage*
	1 July 1995 (0)	
Pile strain gauges	8 July 1995 (7)	
	12 July 1995 (11)	Instrumented pile installed
Inclinometer tubes	28 Nov 1995 (150)	
Deck strain gauges/thermocouples	7 Dec 1995 (159)	
Geomensor sockets	8 Dec 1995 (160)	
	12 Dec 1995 (164)	Deck cast
Geomensor pillar	26 March 1996 (269)	
	17 May 1996 (321)	Retained ground excavated to expose faulty piles
	25 October 1996 (482)	Bulk excavation completed below deck
	19 Dec 1996 (537)	Excavation behind both abutments to prop level
	28 January 1997 (577)	*In situ* concrete props below carriageway completed
	19 March 1997 (627)	Cast *in situ* facing to piles completed
Pressure cells	18 April 1997 (657)	Backfilling completed behind north abutment
	24 May 1997 (693)	Blacktop laid
	14 June 1997 (714)	Road opened to traffic

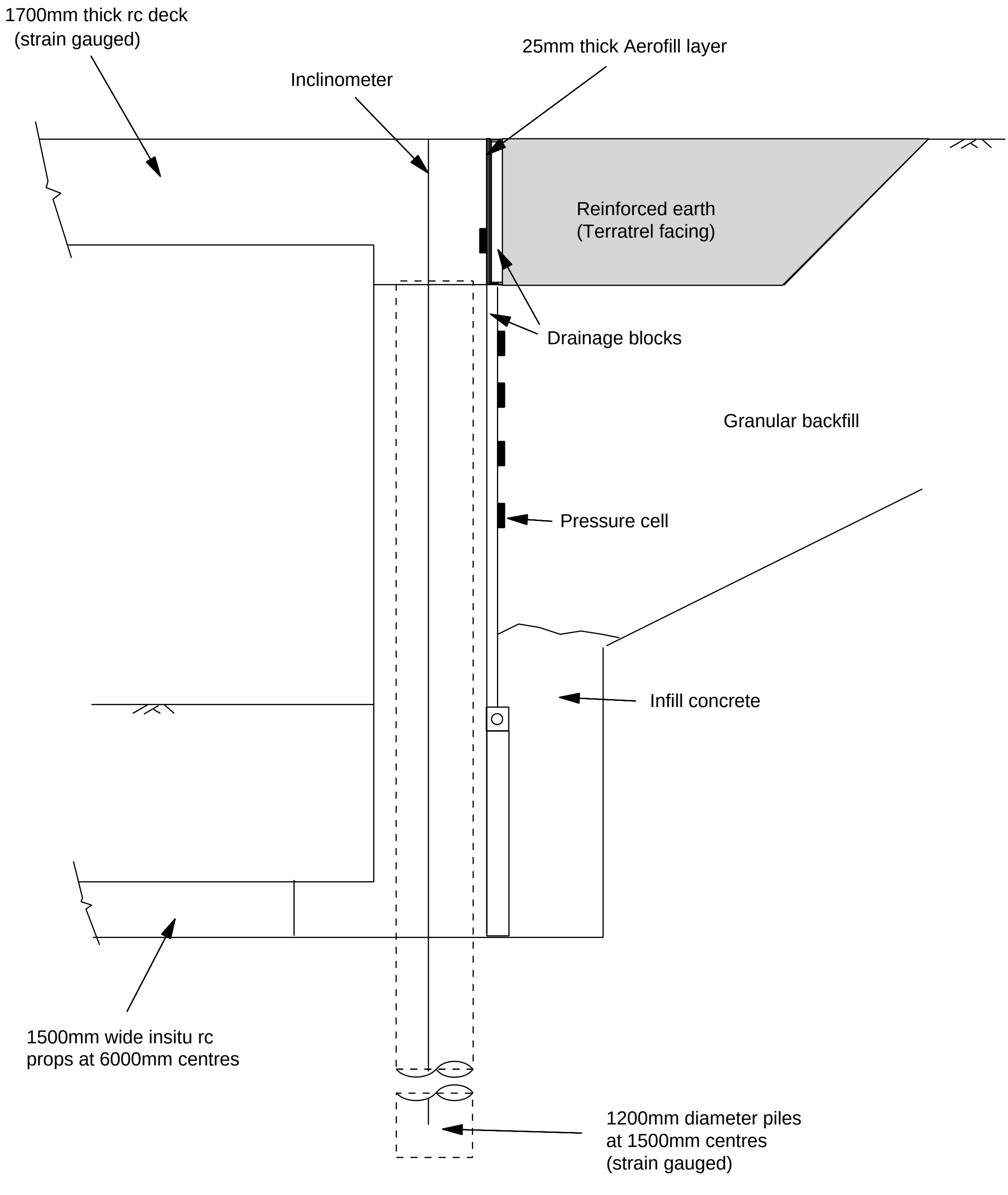


Figure 3 Section showing instrumentation of Phase 1 abutment

shown in Table 2. After pile installation, construction of the deck followed the same procedure as for Phase 1. Pile condition was found to be good on subsequent bulk excavation below the deck and work was therefore able to commence immediately on constructing the reinforced earth fill to 2.5m depth over the *in situ* glacial till on the retained side of the abutments. Construction details at the top of the abutment were similar to those shown in Figure 3 and incorporated drainage blocks and Aerofill joint filler.

The bridges for Phase 1 and Phase 2 (Plate 1) were opened to traffic in June 1997 and in September 1997 respectively.

5 Instrumentation

Instrumentation on both Phase 1 and Phase 2 bridges was essentially similar although minor differences occurred as necessitated by the remedial works on the Phase 1 abutments. Plan views of the deck instrumentation are shown in Figures 4 and 5 for Phase 1 and Phase 2 respectively. The sequences of instrument installation during construction of the two bridges are given in Tables 1 and 2.

The instrumentation was designed to monitor the lateral movement of the deck and abutments; the lateral earth pressures acting on, and the bending moment developed in, the abutments; and the deck strains and temperatures.

Table 2 Instrumentation and construction schedule for Phase 2

Instrumentation	*Date installed (day number)*	*Construction stage*
Pile strain gauges	5 February 1997 (585)	
	6 February 1997 (586)	Pile cage installed, concrete poured
	24 March 1997 (632)	Excavation completed to 3m below deck soffit level prior to installation of falsework
Inclinometer tube	17 April 1997 (656)	
Deck strain gauges/thermocouples	29 April 1997 (668)	
	30 April 1997 (669)	Deck cast
Inclinometer tube	8 May 1997 (677)	
Pressure cells	9 May 1997 (678)	
Geomensor sockets	19 May 1997 (688)	
	3 June 1997 (703)	Reinforced earth fill completed behind north abutment
	7 August 1997 (768)	Bulk excavation completed below deck
	9 Sept 1997 (801)	Blacktop laid
	27 Sept 1997 (819)	Road opened to traffic

Plate 1 View of the near completed bridge

5.1 Measurement of lateral movement of the deck and abutments

During installation of the bored piles, four steel tubes of nominally 100mm diameter were attached vertically to the reinforcing cages prior to pouring the concrete of the piles. These tubes were positioned in piles 29 and 69 of the south and north abutments of Phase 1 and in piles 9 and 25 of the south and north abutments of Phase 2. After pile installation, inclinometer access tubes (I1 to I4) were grouted into the steel tubes using a cement grout containing a non-shrink additive. At the appropriate stage of construction the inclinometer tubes were extended vertically through the deck up to the surface of the new carriageway where they were protected by small manhole covers.

Although the inclinometer tubes extended to within about 0.5m of the pile toes (ie. depths of between 25m and 29m) and base fixity of the inclinometer tubes was a reasonable assumption, machined stainless steel sockets for electronic distance measurements were installed at the top of each inclinometer tube to verify this assumption. The electronic distance measurements were taken using a high precision Geomensor system which is capable of measuring changes in length to better than ±0.5mm over the ranges employed. Measurements were also taken on Geomensor sockets installed over both the central piers. This enabled changes in span of the south and north sides of both bridge decks to be calculated and correlated with deck temperature changes.

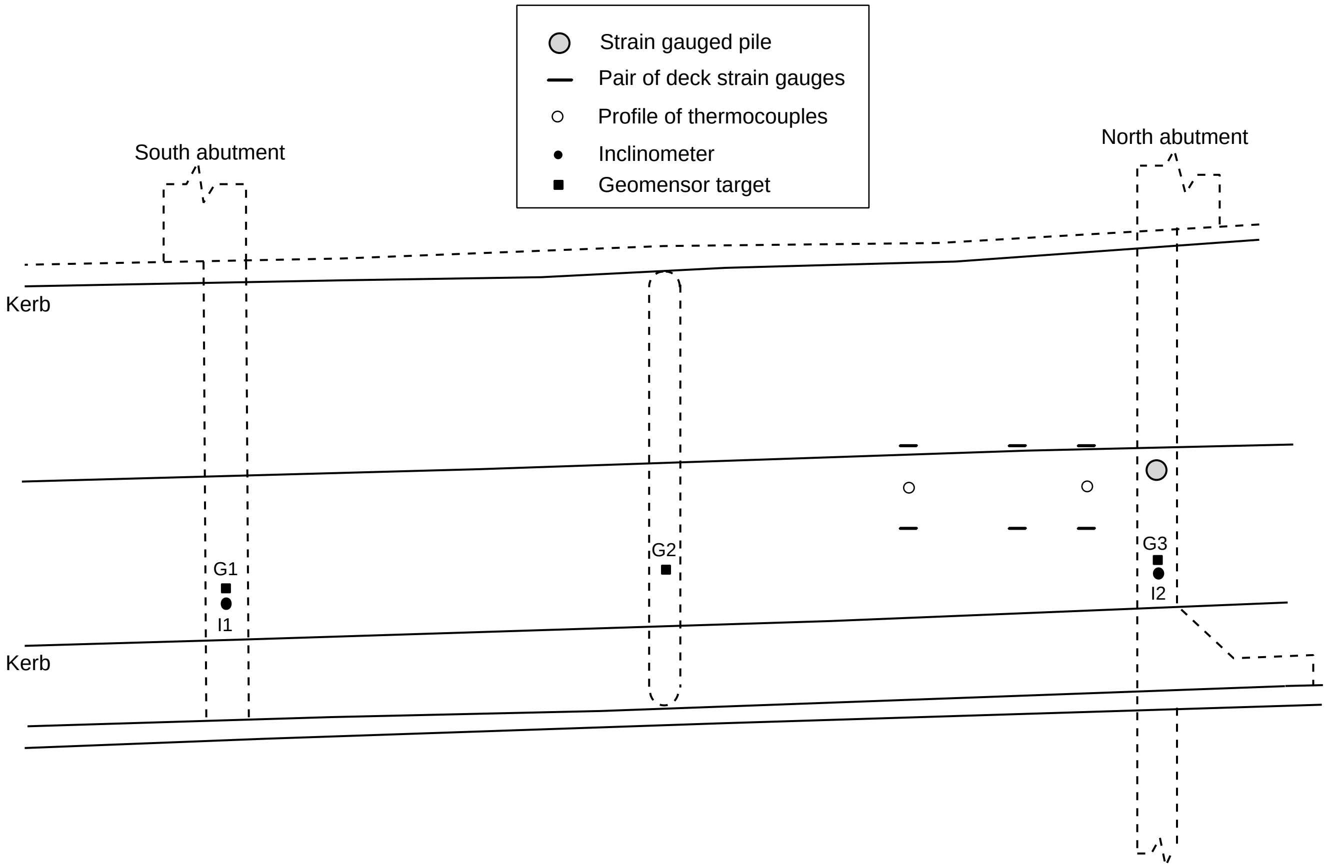


Figure 4 Plan view of Phase 1 deck showing instrument locations

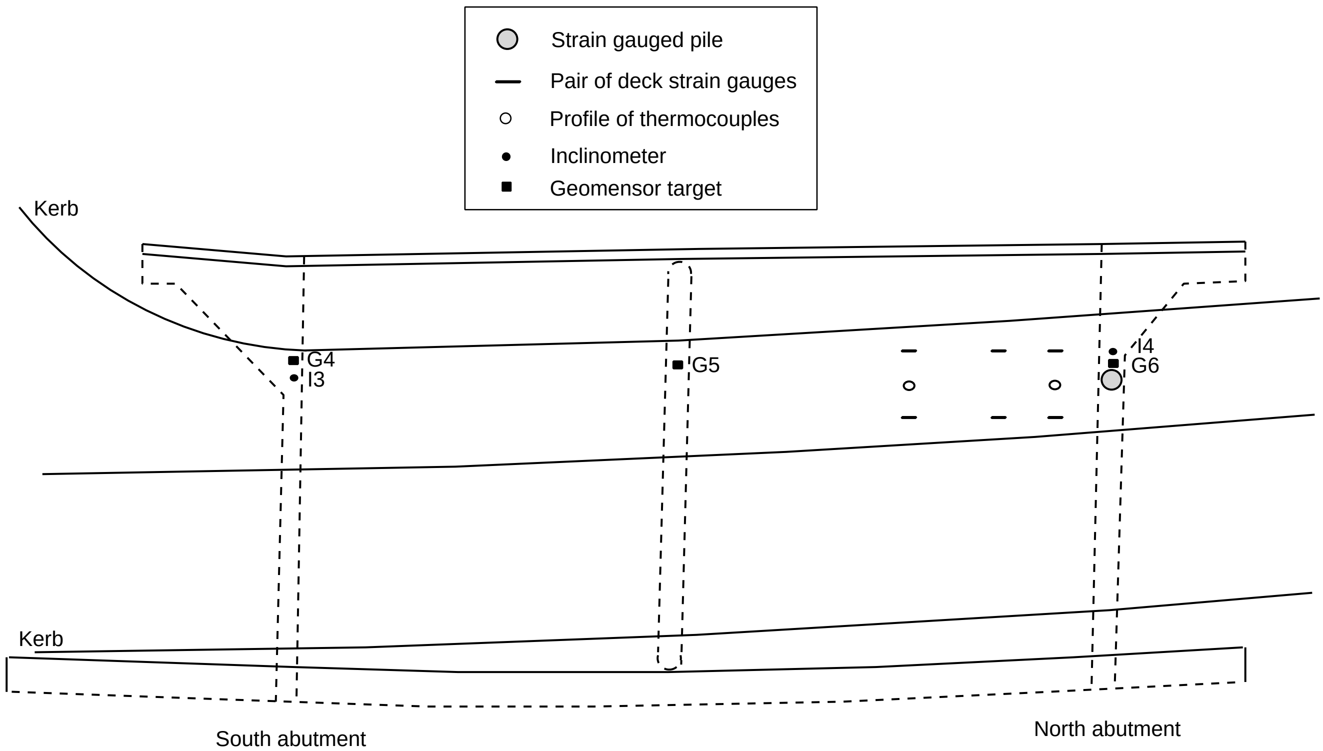


Figure 5 Plan view of Phase 2 deck showing instrument locations

5.2 Measurement of strains in the abutments

Thirty pairs of vibrating wire embedment strain gauges were wired to the vertical reinforcing bars so that one gauge of each pair was positioned at the back and one towards the front of the reinforcing cage of piles 66 and 26 in the north abutments of Phases 1 and 2 respectively. Plate 2 shows the strain gauges being installed on one of the bored pile reinforcing cages. The interval of depth between pairs of gauges was approximately 1m over the upper 11m of each pile and then progressively increased to a maximum spacing of 4m towards the pile toe.

Measured axial strains in the piles were converted to loads by multiplying by the modulus (E) and the cross-sectional area (A) per metre run of the abutment. For the Phase 1 abutment appropriate correction was made to allow for the concrete facings which were part of the remedial works. An E value of 31×10^6 kN/m^2 was used for the pile concrete which had a 28 day strength of 40N/mm^2 (Table 3 of BS5400: Part 4: 1990).

Bending moments per metre run of the abutment were determined from pile bending strains based on an equivalent flexural rigidity (EI) of 3.8×10^6 kN/m^2, assuming that the concrete would remain uncracked at the small strain levels involved and making allowance for the contiguous nature of the piles (ie. 1500mm diameter piles at 1820mm centres). In the case of Phase 1 piles, where a concrete facing was added to both sides of the piles as part of the remedial works, the equivalent flexural rigidity increased to 1.7×10^7 kN/m^2 per metre run of the abutment.

5.3 Measurement of deck strains and temperatures

Three pairs of vibrating wire embedment strain gauges were attached at two locations on the reinforcing cages for the decks of both Phases 1 and 2. The pairs of gauges were positioned at 2m, 5m and 10m from the north abutment in all cases. One gauge of each pair was fixed to the top reinforcing steel and one to the bottom steel, thus enabling both axial and bending strains to be determined.

Axial loads were determined for the 1700mm thick reinforced concrete deck assuming a modulus (E) of 31×10^6 kN/m^2 for the concrete which had a 28 day strength of 40N/mm^2 (Table 3 of BS5400: Part 4: 1990). Bending moments were determined using a second moment of area (I) of 0.41m^4.

Deck temperatures were established both from thermistors incorporated in the arrays of strain gauges described above and also from two profiles of thermocouples installed in each deck. Each profile comprised six thermocouples at depths of 40mm, 100mm, 250mm, 500mm, 900mm and 1450mm below the top of the concrete deck. One profile was located at 2m and the other at 10m from the north abutment.

5.4 Measurement of earth pressure acting on the abutments

The initial intention had been to measure lateral pressures in the *in situ* clay behind the bored pile wall for Phase 1. However, because of the necessity for remedial works, lateral pressures on the abutment during backfilling were measured instead. For this purpose, nine vibrating wire

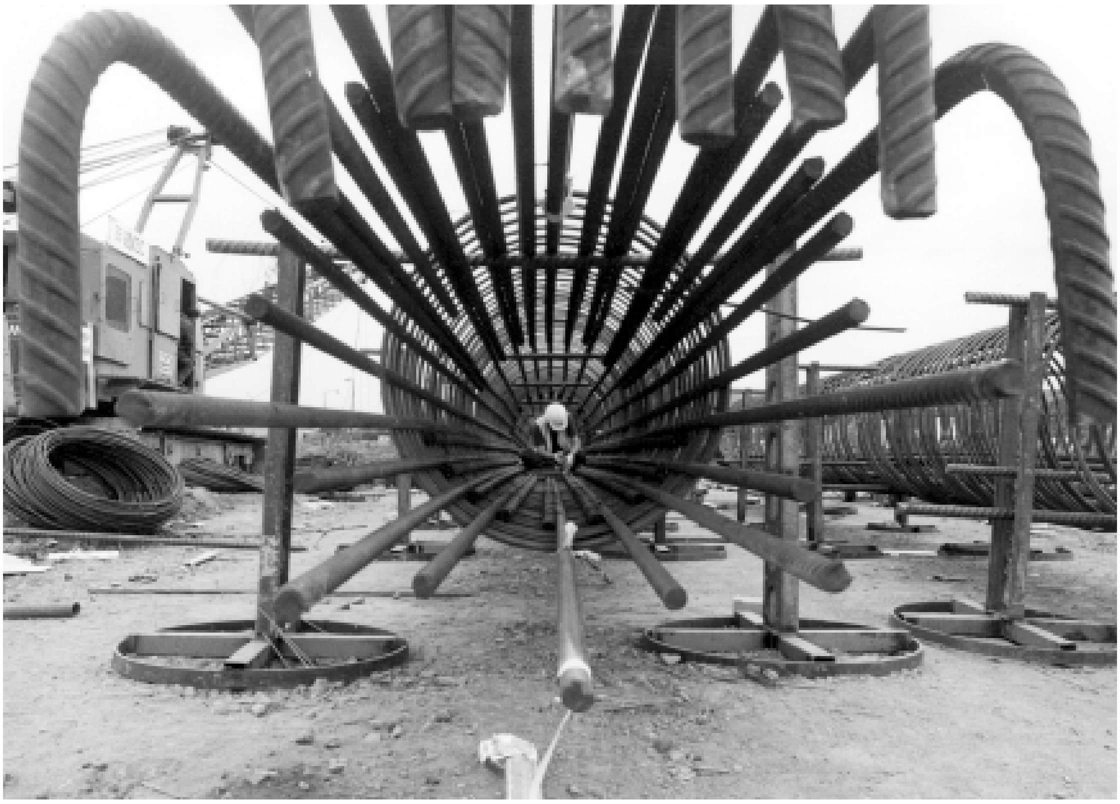


Plate 2 Installation of strain gauges on the bored pile reinforcing cage

pressure cells were installed in recesses cast into the retained face of the abutment. The diameter of the active face of each cell was 180mm and the cells were oil filled. The pressure in the oil was measured by a vibrating wire transducer connected to the cell.

The locations of the cells are shown in Figure 3 for the abutment of Phase 1. As pressures develop near the top of the abutment due to thermal expansion of the deck, this level was of particular importance. Four cells were placed at the uppermost location behind the 25mm thick Aerofill layer to monitor pressures from the 2.5m deep reinforced earth fill. The remaining five cells measured backfill pressures directly with two cells being placed at the next depth and one at each of the lower depths. During backfilling behind the abutment, fill material passing a 6.3mm sieve was compacted by hand against the faces of these cells to prevent damage to the diaphragm from coarser particles. The hand compacted material was separated from the backfill by a permeable geotextile membrane to prevent fines being washed out and a void forming over the cell face.

Only a limited investigation of the lateral pressures acting on the bored pile abutment of Phase 2 was possible. Four pressure cells were again placed to monitor pressures from the reinforced earth backfill in a similar manner to that described above for Phase 1.

6 Performance of Phase 1 bridge

Monitoring of instrumentation during construction took place at regular intervals and the changes associated with the various construction activities (Table 1) were identified. On completion of construction, seasonal monitoring of performance commenced. From day 888 the instrument cables were installed in cabinets and results from electrical and vibrating wire instruments were then computer logged. The changes from manual to logged data are apparent in some of the time plots which follow. Graphs of data during construction and the first winter in service are presented in the main body of the text. For convenience similar graphs of results during seasonal monitoring during the following year in service are presented in Appendix A.

6.1 Lateral movement of the deck and abutments

Surveys of the inclinometer tubes (I1 and I2) in the abutments for Phase 1 were carried out at regular intervals during construction to establish lateral movement profiles. Because of the complicated nature of the remedial works, considerable variation in the results was obtained. Figure 6 shows the extremes of lateral movement measured on both

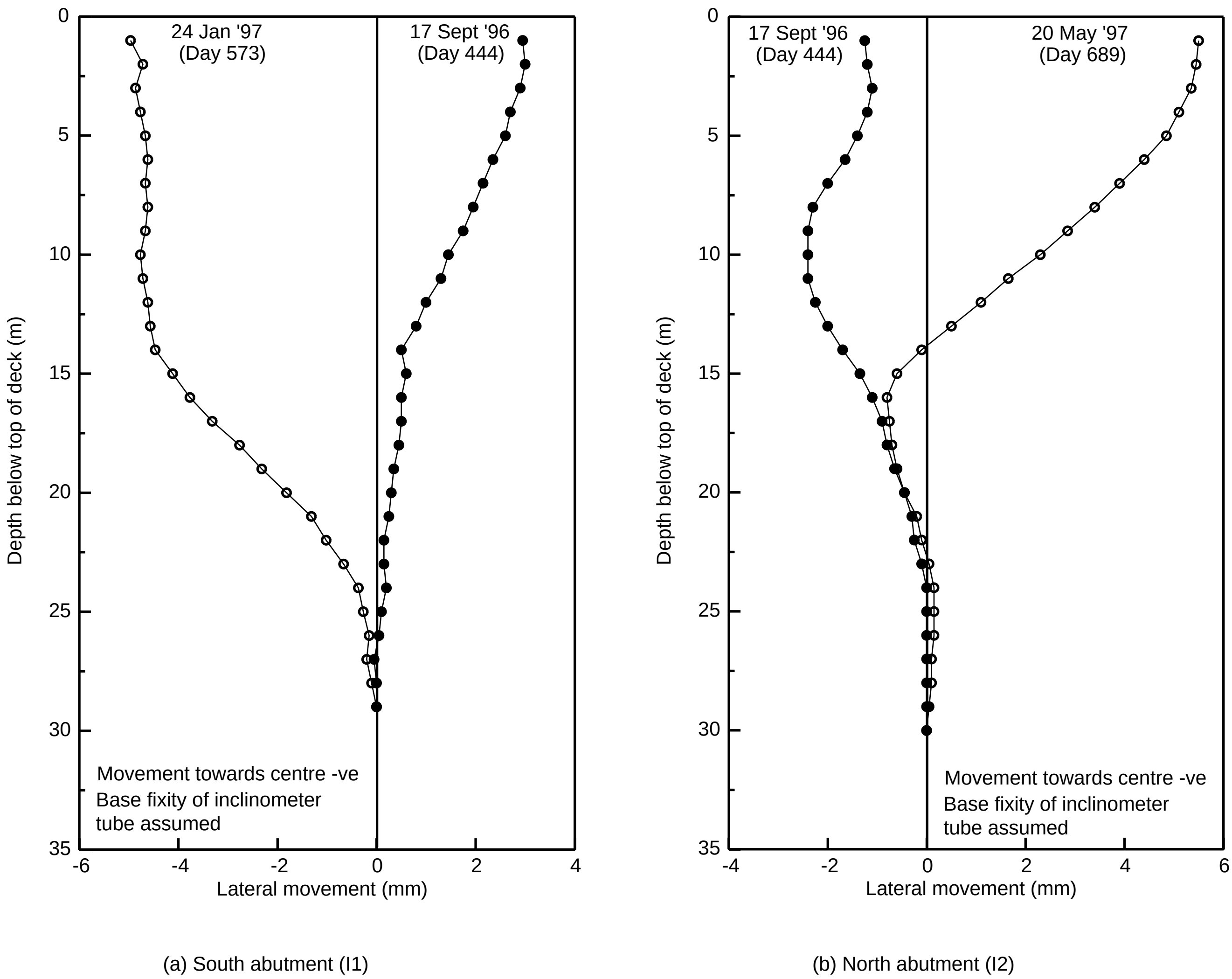


Figure 6 Extremes of lateral movement measured during construction (Phase 1)

abutments during this period. Generally, at the top of the abutments, the difference between the extremes of movement was about 8mm. Excavation on both sides of the abutments for the remedial works meant that significant movement occurred over at least the upper 15m of each abutment and with the south abutment to a greater depth as shown in Figure 6.

The lateral movements measured on both abutments during the seasonal monitoring are shown in Figure A1 of Appendix A. At the top of the abutments, the difference between the extremes of movement was about 2mm for the south abutment (I1) and about 3mm for the north abutment (I2).

Figure 7c compares the surface movements of the deck determined from the inclinometer tubes in the abutments assuming base fixity with those measured from a remote pillar using the Geomensor electronic distance measuring system. Generally movements measured using the two techniques were in close agreement. It must be noted however that the south and north abutments did not move identically whilst the remedial works were underway. Some lateral movement (4mm) of the bridge towards the north occurred as is confirmed by the Geomensor measurements at the central pier position (Figure 7b).

The mean deck temperatures measured using the thermocouple profiles are also shown in Figure 7a. The underlying trends of movement developed by thermal expansion and contraction of the bridge deck were to some extent masked by construction effects until the bridge was nearing completion. The results in Figure 8 are for the period of seasonal monitoring following completion of the blacktop over the bridge. A correlation was obtained between the lateral movement of the top of the abutments measured by inclinometer and the mean deck temperature. The slopes of the lines in Figure 8b used in conjunction with the respective span between the abutment and central pier gave equivalent coefficients of thermal expansion of the deck of $9.59\times10^{-6}/^{o}C$ and $9.58\times10^{-6}/^{o}C$ for the south (I1) and north abutment (I2) respectively. Changes in deck span over the same period were measured using the Geomensor and the data in Figure 8a give an equivalent coefficient of thermal expansion of the deck of $8.43\times10^{-6}/^{o}C$. These values were lower than the $12\times10^{-6}/^{o}C$ that would normally be encountered for reinforced concrete. However as the aggregate used in the concrete was carboniferous limestone (from Tunstead Quarry, Buxton), a lower coefficient of about $9\times10^{-6}/^{o}C$ would be expected (BD37, DMRB 1.3).

6.2 Axial loads and bending moments developed in the north abutment

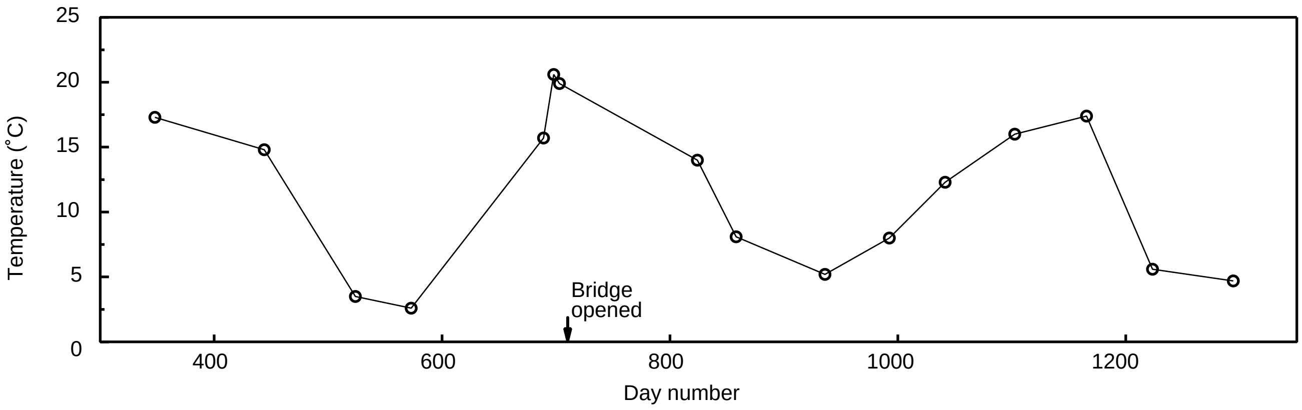
Figure 9 shows the axial loads measured using strain gauges at various depths in the north abutment of Phase 1. At this stage, the loads per metre run of the abutment were calculated from strains using the cross-sectional area of the piles rather than the increased area after the remedial works. Shortly after the deck was cast, axial loads in the wall increased to about 500kN/m which corresponded with the value calculated from the dead load of the deck assuming each span was simply supported between the central pier and abutment. Some increase in dead load from this calculated value would however be expected because of the integral nature of the bridge. Loads increased and were generally between 500 and 1000kN/m on removal of the formwork supporting the deck and excavation on both sides of the abutment for the remedial works. A sharp increase in the apparent axial load occurred around day 627 (March 1997) when the concrete facings were cast onto the piles. This additional compressive load was probably induced by shrinkage as the concrete of the facings cured.

Because of this effect, a new datum for the strain gauges in the abutment was established at day 647 after completion of the remedial works and before the major part of the backfilling had taken place. From this time the revised cross-sectional area and second moment of area were used in calculating the axial loads and bending moments in the abutment. The subsequent variation of loads and moments with time and temperature is shown in Figure 10 and Figure A2 of Appendix A. Generally over the period from day 887 to day 938 (Figure 10) and day 936 to day 1294 (Figure A2) when logged data were available, the vertical axial loads in the abutment were reasonably stable with time and temperature. However, as would be anticipated, bending moments in the abutment varied as changes in temperature caused deck expansion and contraction. These changes were more significant at the upper gauge locations as shown in Figure 11 for eight arbitrary dates. The data in Figure 11 have been calculated by adding the bending moment profile immediately prior to casting the concrete facing to changes from the new datum established on day 647. In this way an assessment could be made of the overall development of bending moment which ignored the shrinkage effects as the concrete of the facing cured. A negative bending moment was observed at the top of the abutment due to the moment from the deck dead load being transferred to the integral abutment.

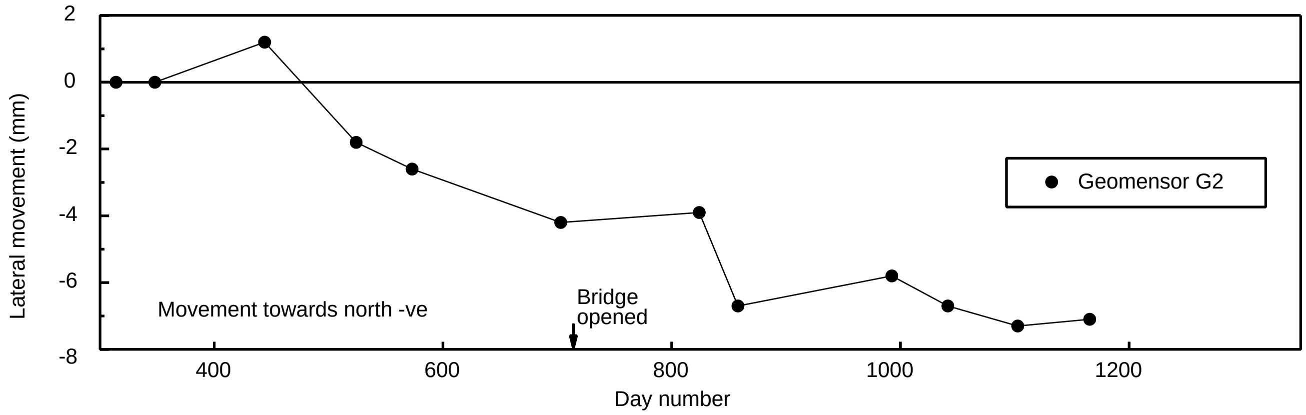
6.3 Axial loads and bending moments developed in the deck

The variation of axial loads and bending moments measured using embedment strain gauges in the deck with time and temperature is shown in Figure 12 and Figure A3 of Appendix A. In both the axial load and bending moment cases a sharp increase occurred due to the rise in surface temperature of the deck when the blacktop was laid, although values soon reverted when the deck cooled. Subsequent to completion of construction, both axial loads and bending moments showed a strong dependence on deck temperature as would be expected. Although two sets of gauges were installed at distances of 2m, 5m and 10m from the abutment, faults occurred on the gauges at 5m so that the readings were not available at this distance from the abutment.

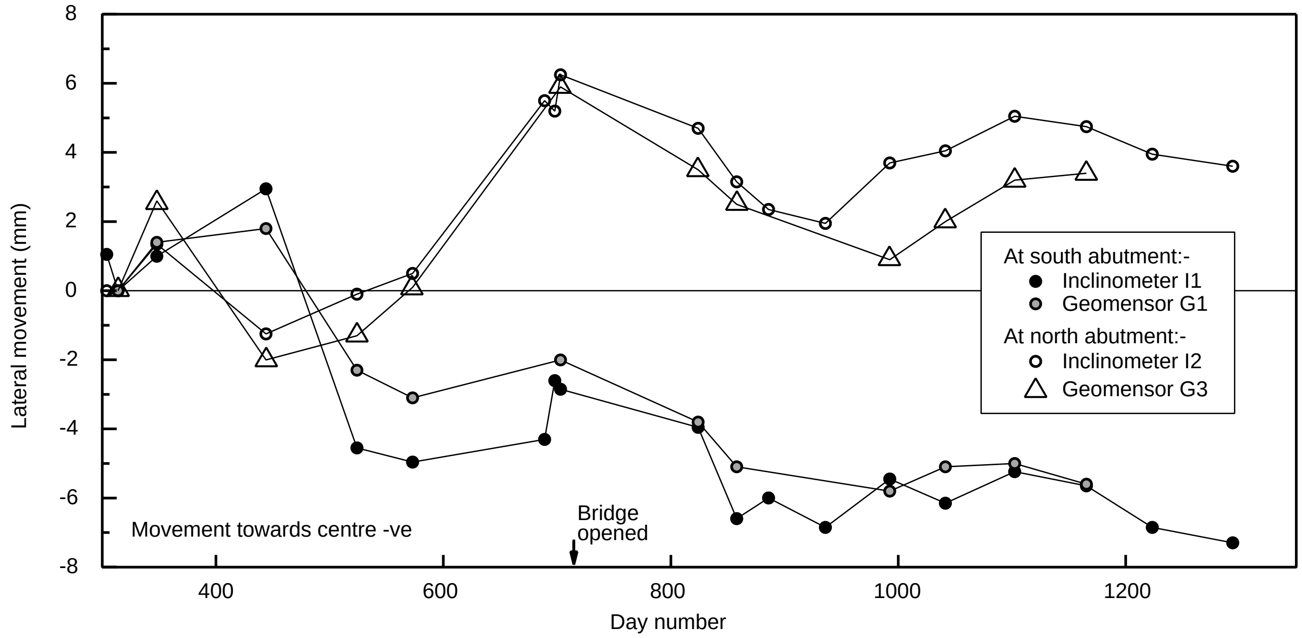
Evaluation of the likely shrinkage and creep deformation of the 1.7m thick concrete deck was carried out following the procedure given in Appendix C of BS5400: Part 4 (1990). Over the period of monitoring covered by the results in Figure 12 and Figure A3, calculated upper bounds of shrinkage and creep may have accounted for 4% and 10% respectively of the measured loads.



(a) Mean deck temperature (thermocouples)

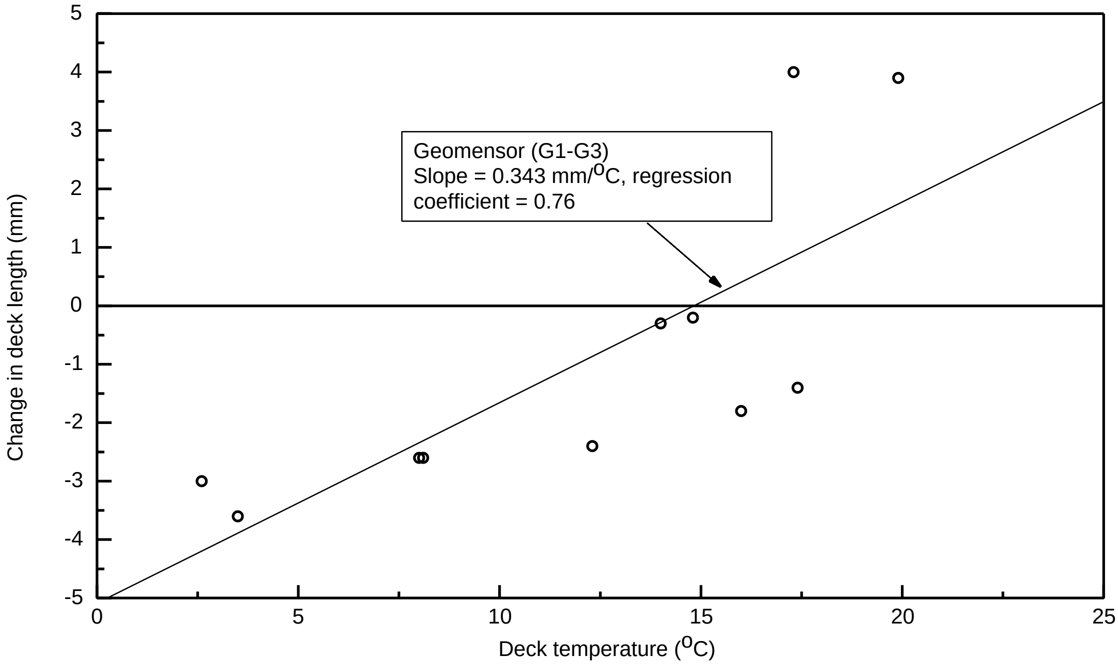


(b) Movement at central pier

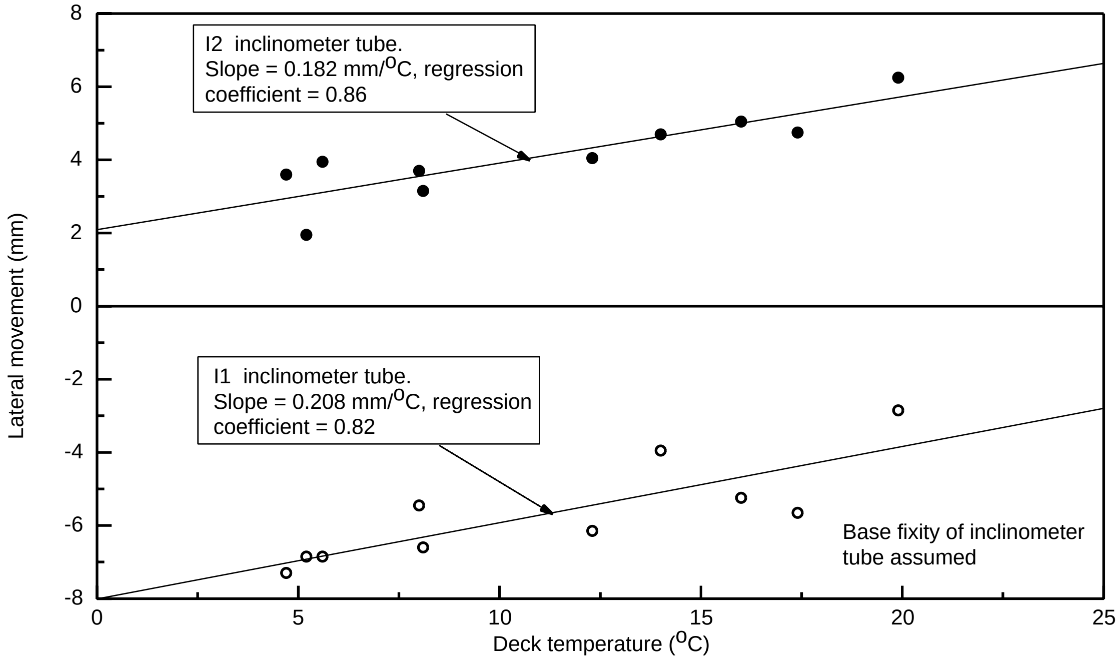


(c) Movement at abutments

Figure 7 Lateral movements at deck level (Phase 1)



a) Deck span



b) Abutment movements

Figure 8 Variation in lateral movements with deck temperature (Phase 1)

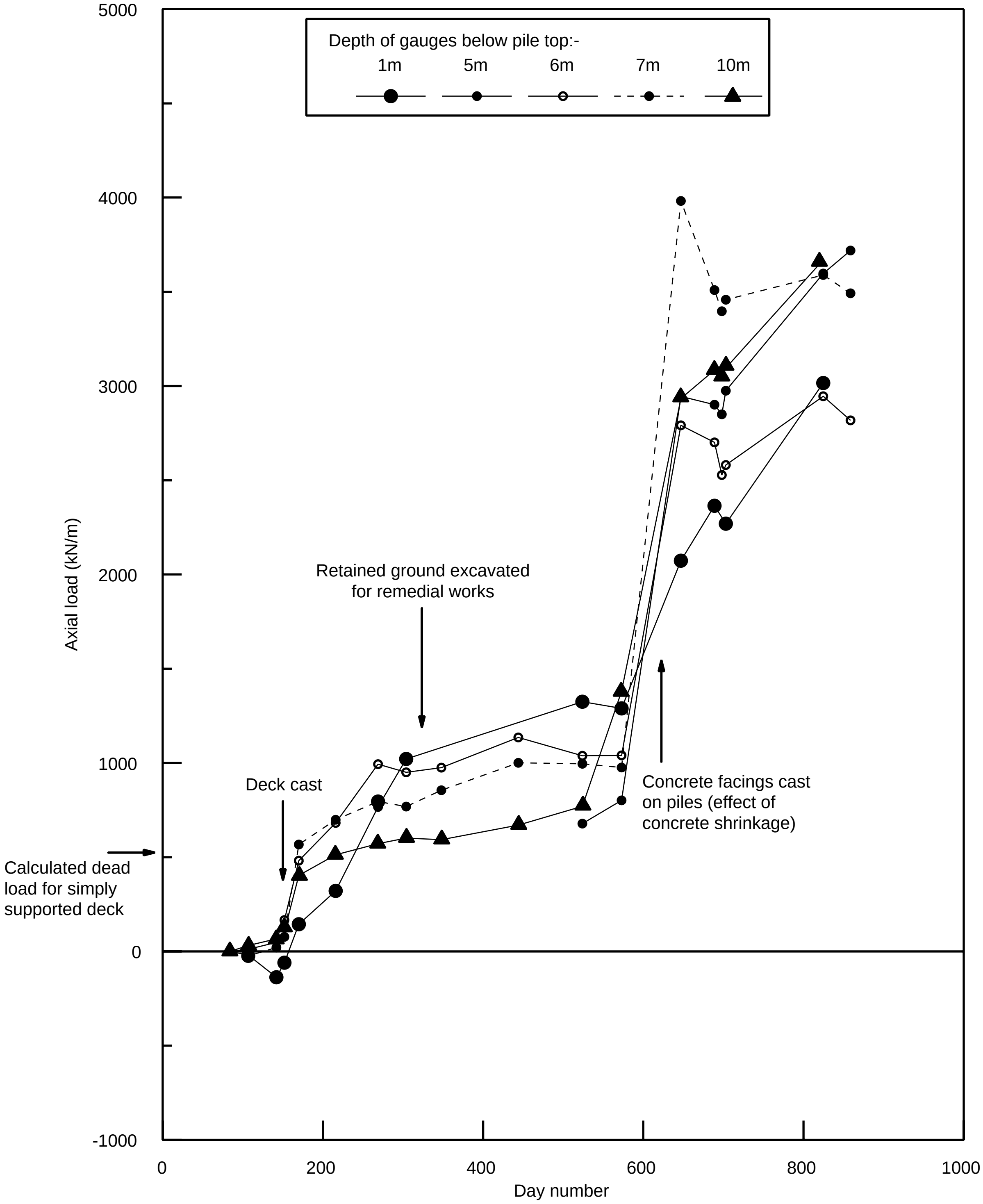


Figure 9 Variation of axial load in the north abutment (Phase 1)

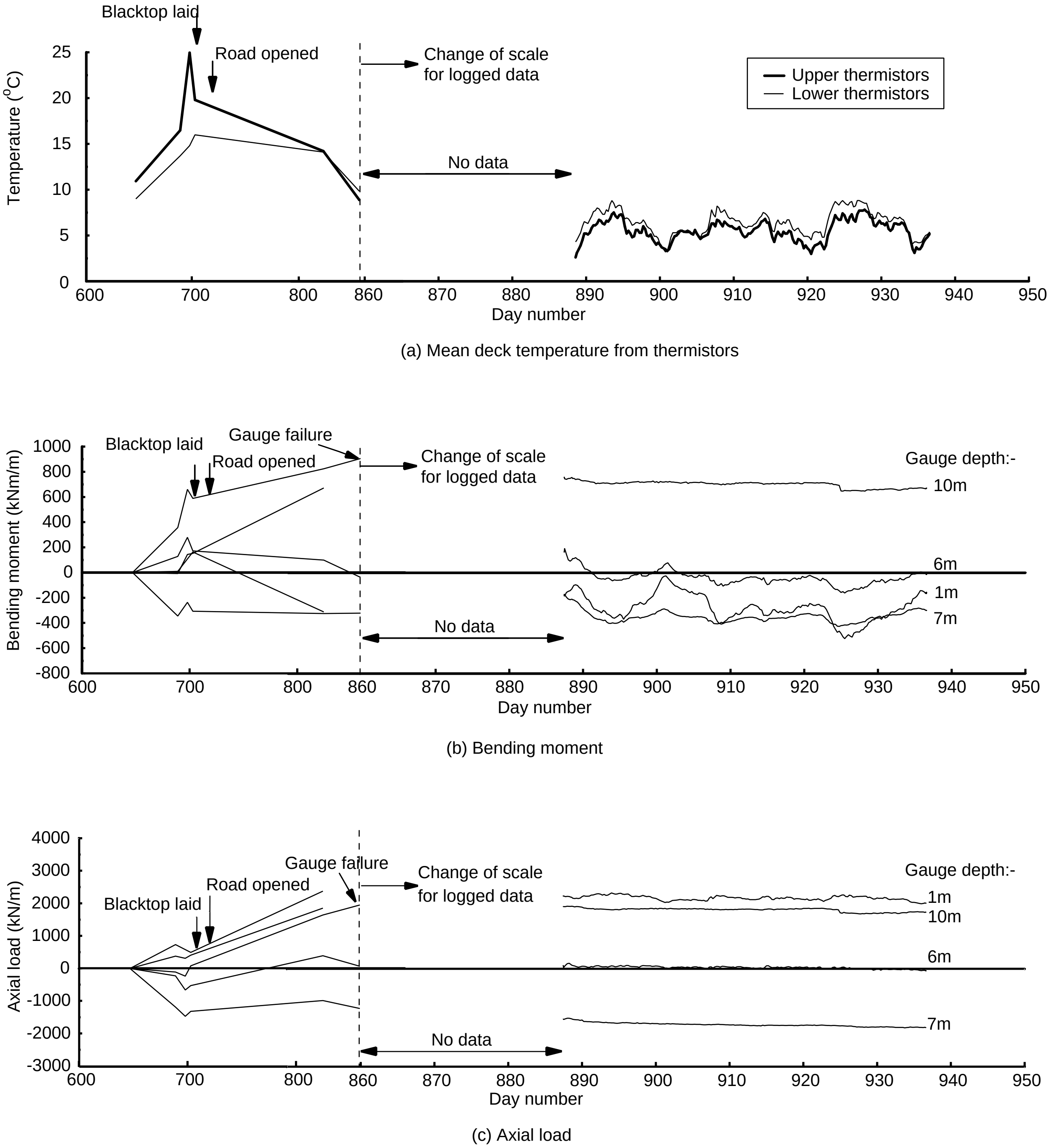


Figure 10 Variations in abutment loads and moments with temperature relative to new datum on day 647 (Phase 1)

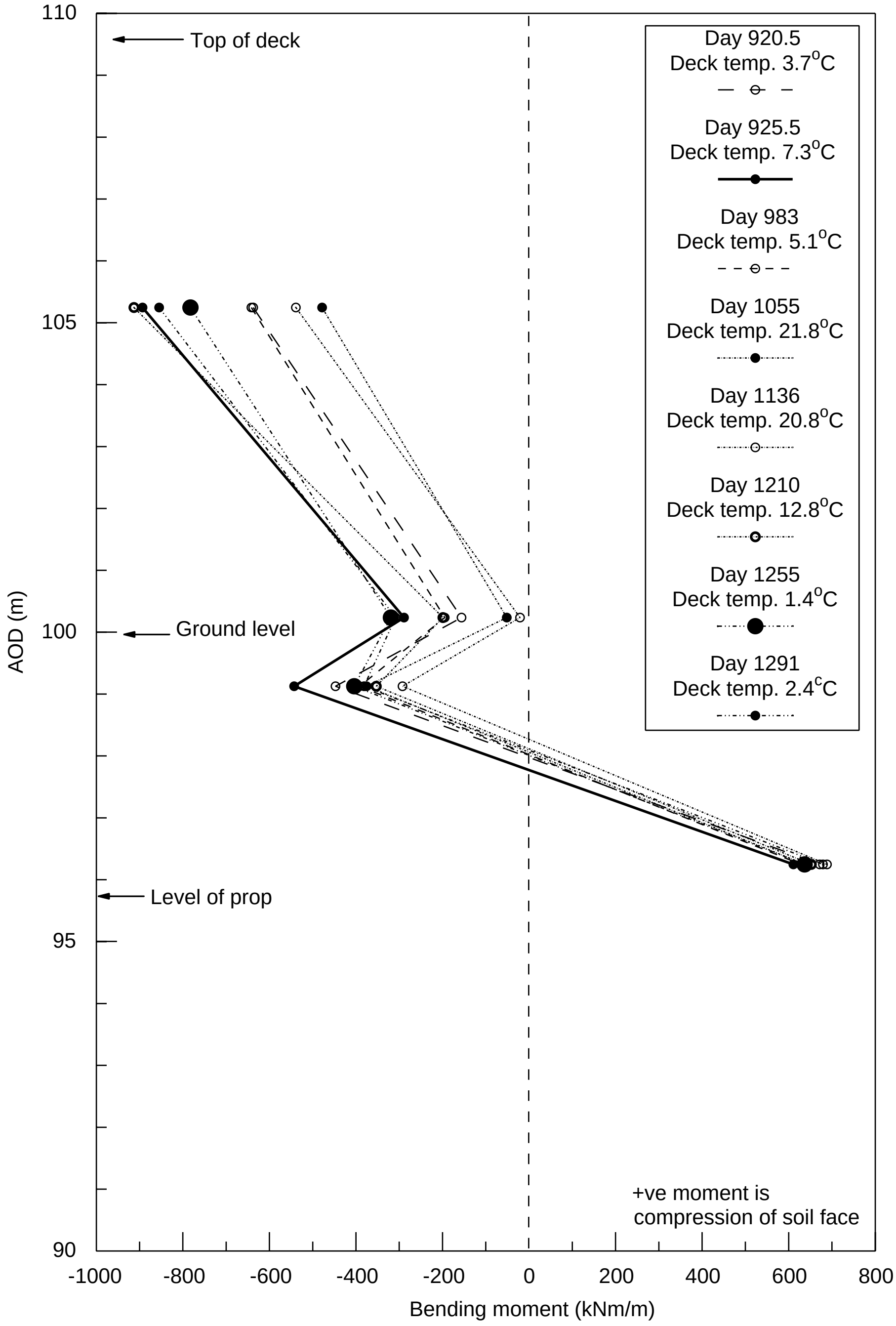


Figure 11 Variation of wall bending moment with depth (Phase 1)

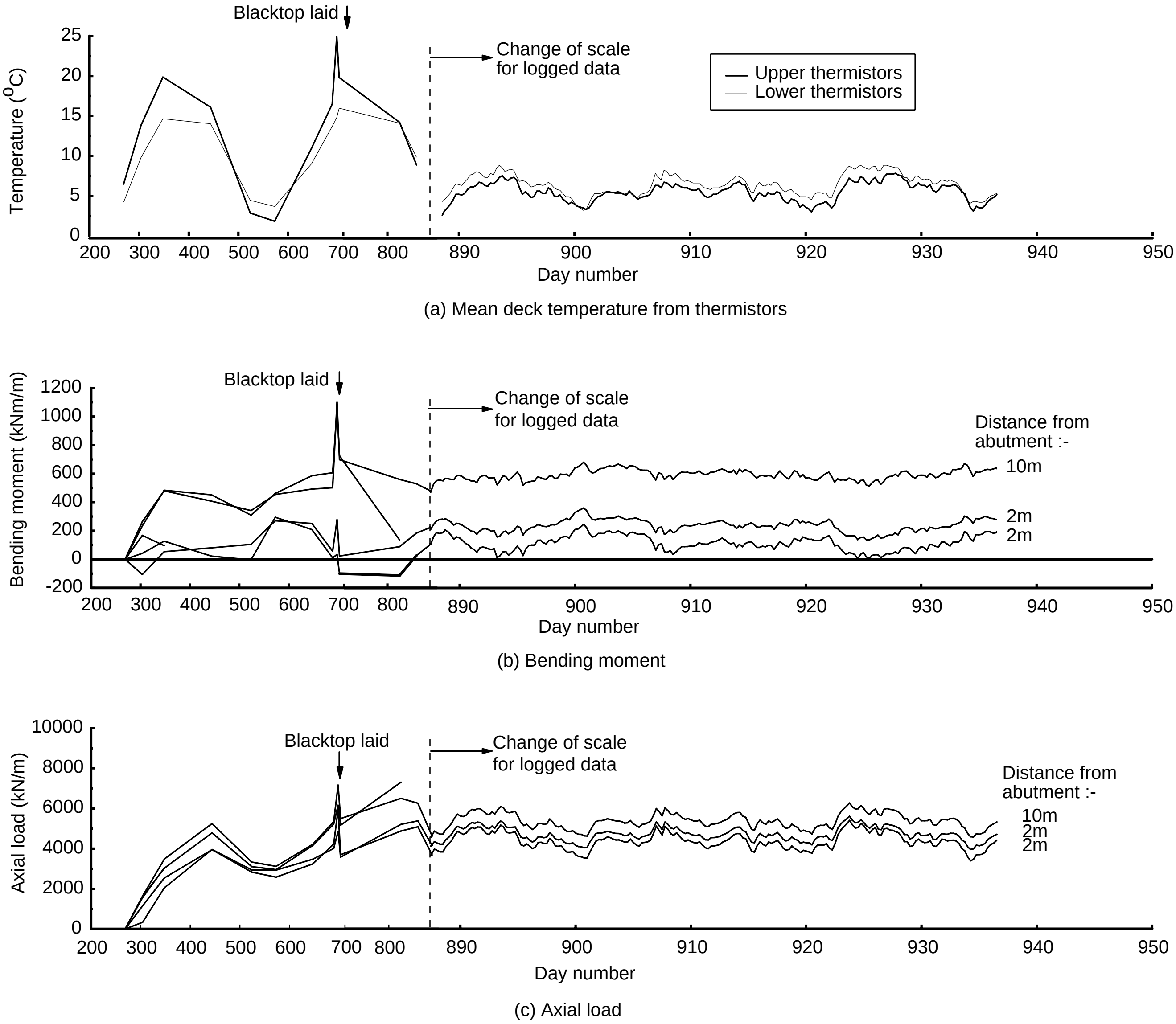


Figure 12 Variations in deck loads and moments with temperature (Phase 1)

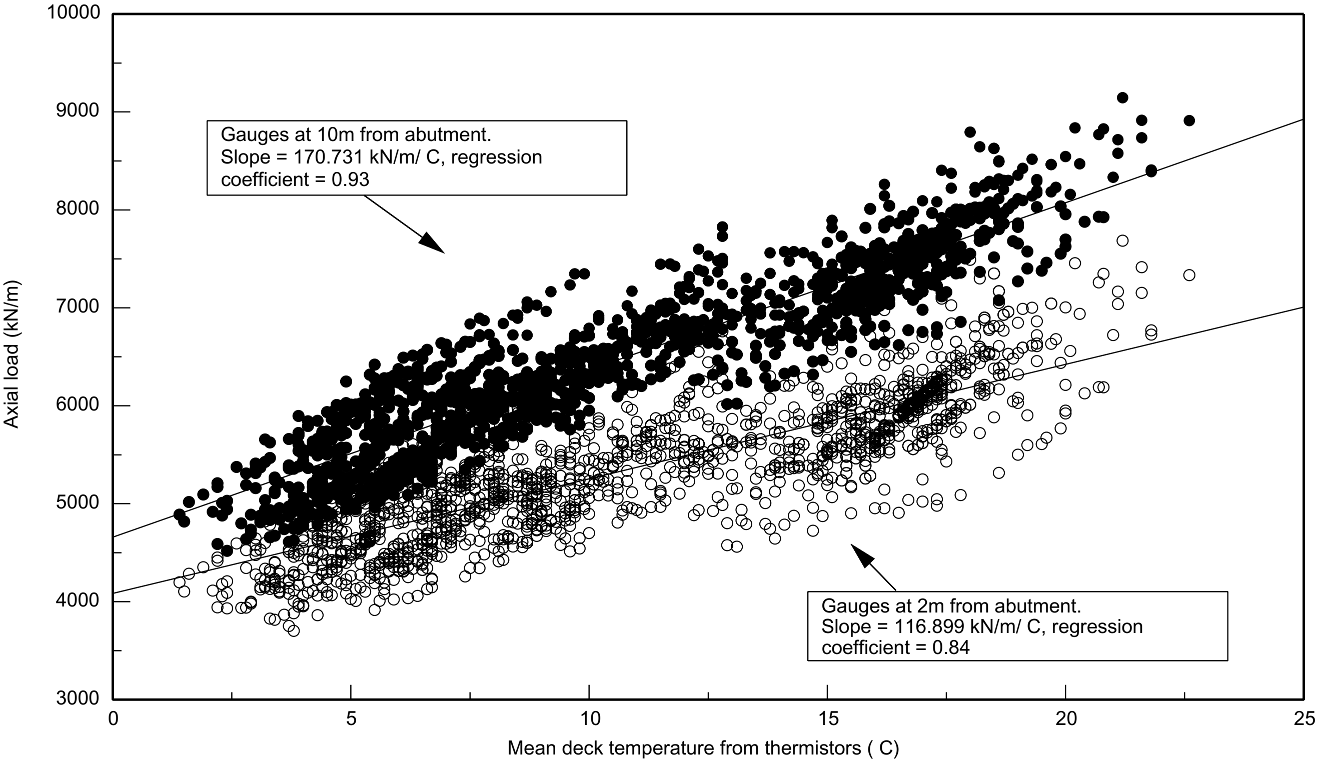
The results in Figure 13 indicate the variation of axial load and bending moment with temperature for the post-construction period. Figure 13a demonstrates that axial load increases fairly linearly with deck temperature at a mean rate of 144kN/m/°C. If the abutments of the bridge were fully constrained and assuming the measured coefficient of thermal expansion (from Section 6.1) together with the same deck modulus as used for the load calculations, a load increase of 505kN/m/°C would be anticipated. However as movement of the abutments occurs, these findings confirm that only a minor part of the expansive load is taken by the deck itself. The sharing of load between the deck, abutments and backfill will depend on the relative stiffnesses of each. The results for the reinforced concrete abutment of Phase 1 generally showed much higher mean deck loads and a higher thermal dependence than those for the bored pile abutments of Phase 2 (Section 7.3): this was accounted for by the increased flexural stiffness of the reinforced concrete abutment and the different method of construction.

Figure 13b shows a best fit regression of bending moment against deck temperature. The slope of -15kNm/m/°C measured at 10m from the abutment indicates that a small hogging moment develops as the deck expands thermally. At 2m from the abutment, bending moment reversal occurs with a measured slope of +11kNm/m/°C, ie. sagging.

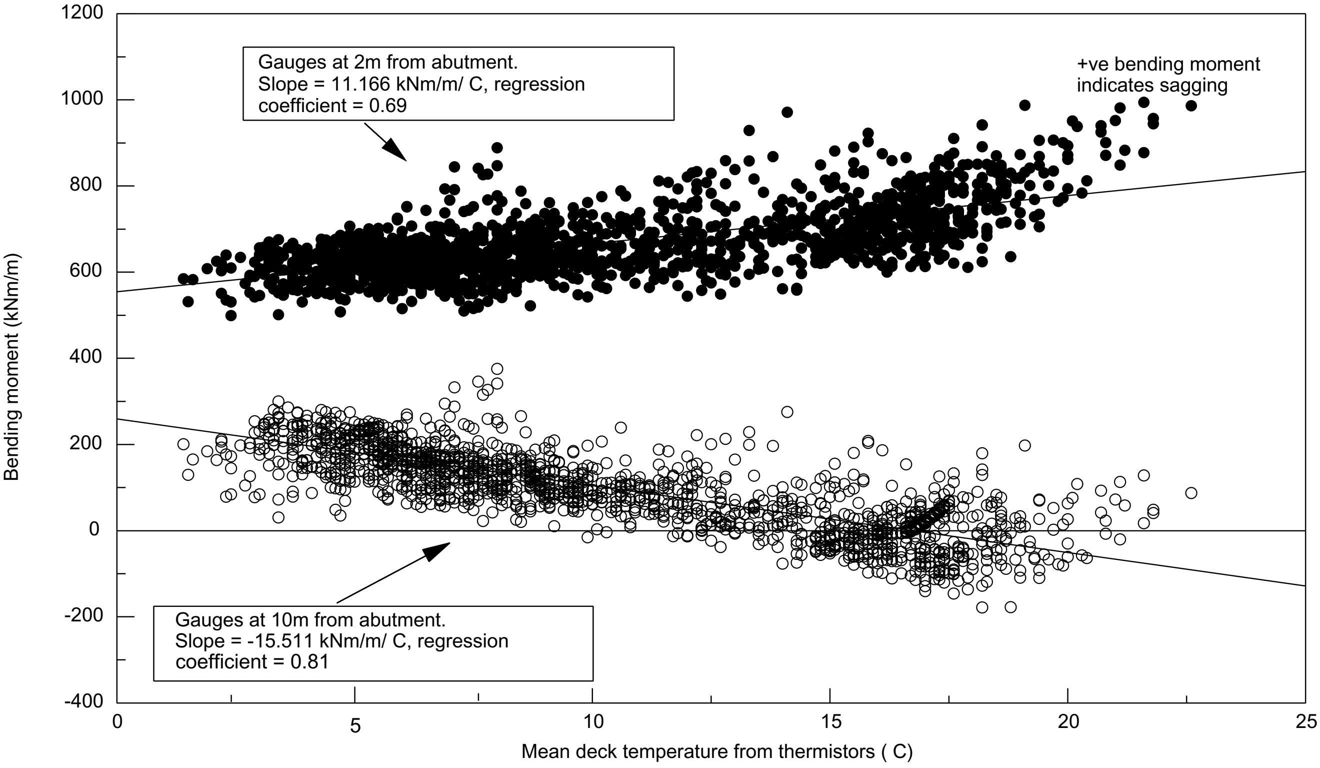
6.4 Lateral stresses acting on the north abutment

The variation of lateral stress of the backfill acting on the abutment with time and mean deck temperature is shown in Figure 14 and Figure A4 of Appendix A. A strong correlation existed with temperature and as the bridge deck expanded lateral stress increases were observed.

Figure 15 shows the distributions with depth of the lateral earth pressure acting on the north abutment shortly after backfilling was completed on day 657 and up until day 1291. After backfilling, pressures on cells 2, 3 and 4 were consistent in magnitude to those that would be predicted using the coefficient of earth pressure at rest (K_o=1-sinϕ') of 0.33 calculated from the ϕ'_{peak} of 42°. As



a) Axial load



b) Bending moment

Figure 13 Variation of deck axial load and bending moment with temperature (Phase 1)

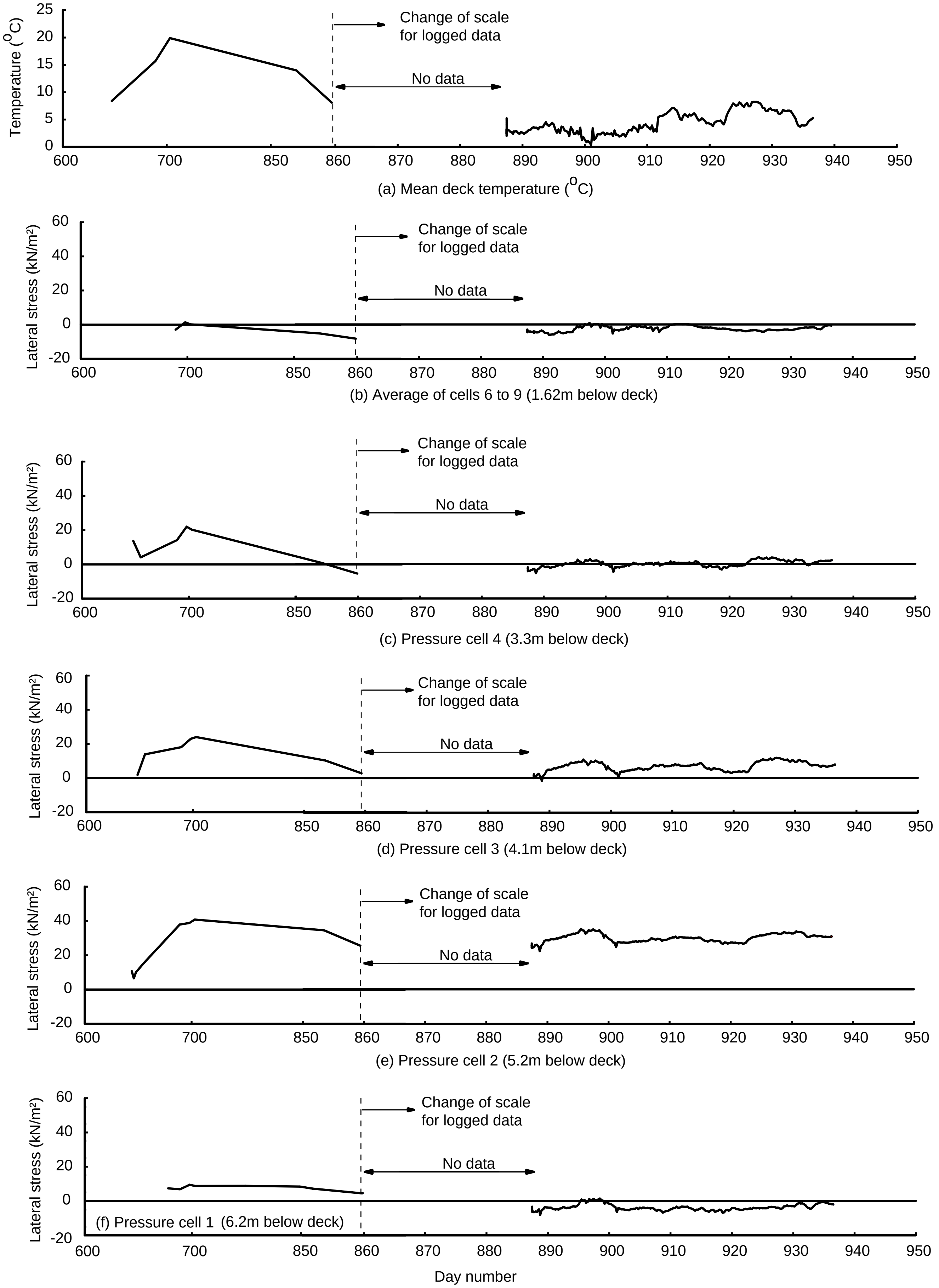


Figure 14 Variations in lateral stress with temperature (Phase 1)

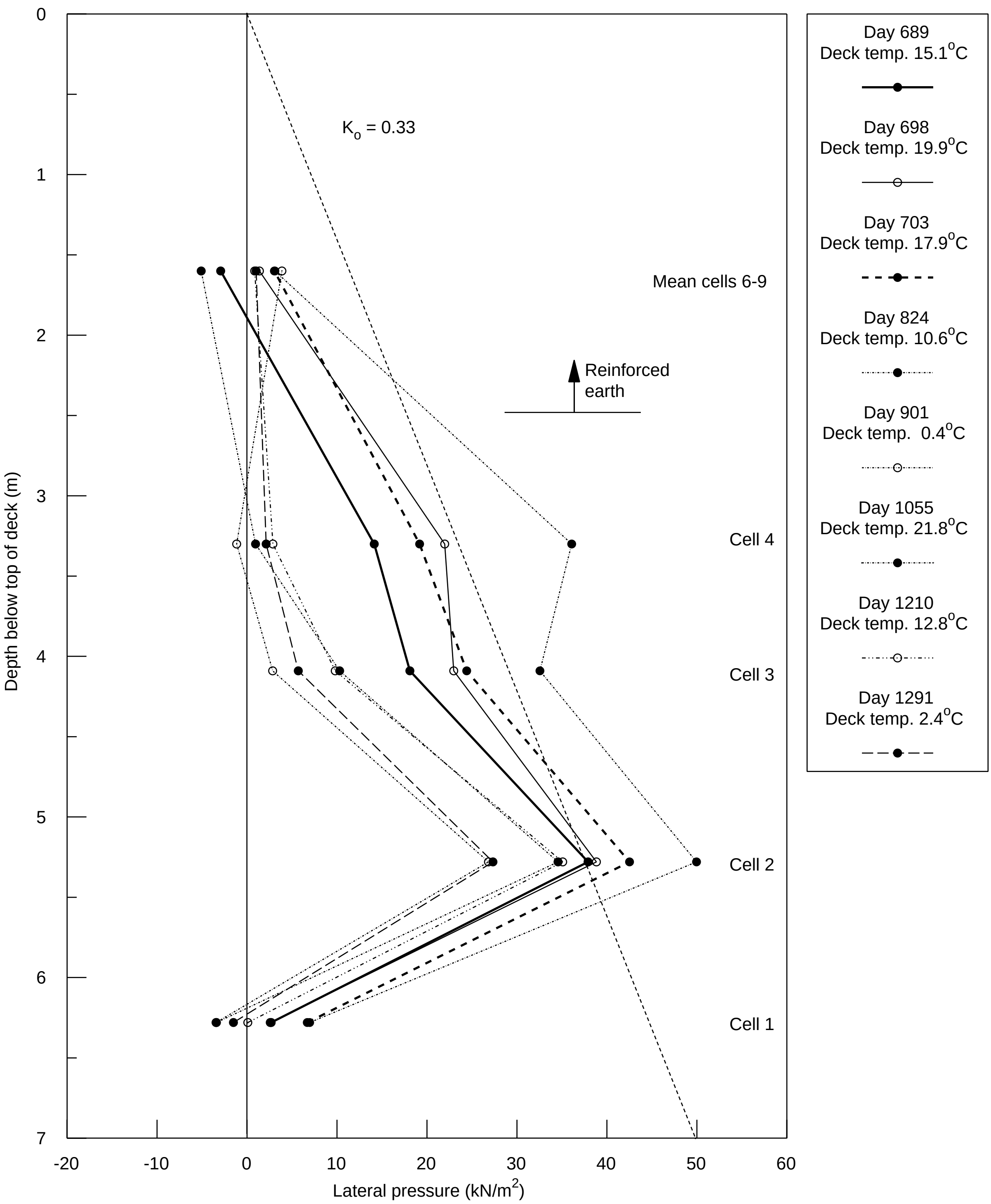


Figure 15 Lateral pressures acting on the abutment (Phase 1)

backfilling took place in the summer months, these stresses soon started to reduce as the bridge deck contracted with the approach of winter. During the following summer, maximum lateral pressures were recorded on day 1055 when the mean deck temperature reached 21.8°C. As shown in Figure 15, the pressures on cells 2, 3 and 4 were then slightly above those calculated from the coefficient of earth pressure at rest. The mean pressures on cells 6 to 9 were small as these cells measured the contact stresses behind the 25mm thick Aerofill packing layer which separated the reinforced earth fill from the abutment wall.

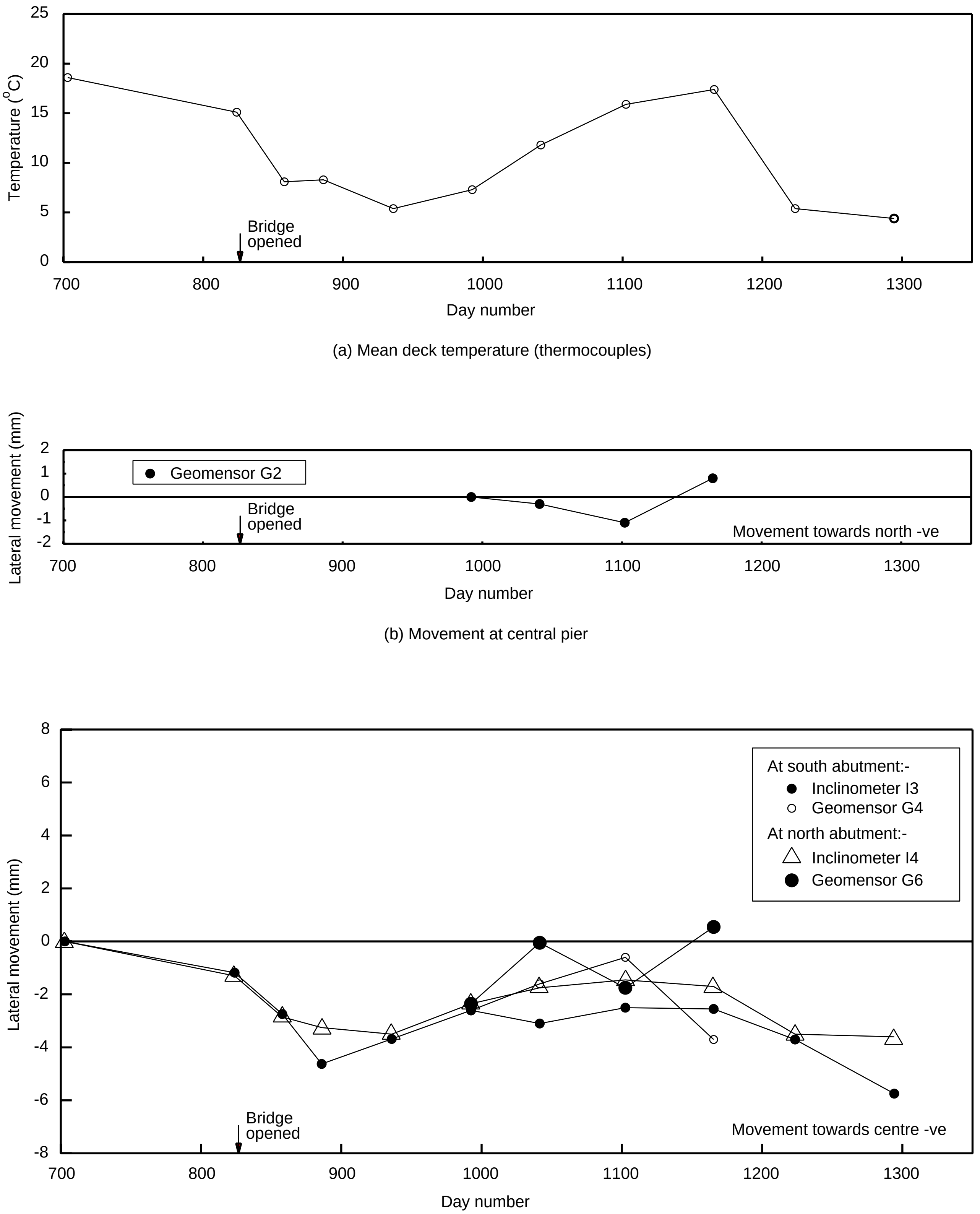
As the reinforced earth fill was constructed during the summer, the first real indication of any stress escalation towards the top of the abutment might be expected to take many seasonal cycles of movement to develop.

7 Performance of Phase 2 bridge

The timetable of construction activities is given in Table 2. On completion of construction, seasonal monitoring of performance commenced. From day 859 computer logging of the electrical and vibrating wire instruments was introduced and the changes from manual to logged data are evident in some of the time plots. As for Phase 1, graphs of data during construction and the first winter in service are presented in the main body of the text. For convenience similar graphs of seasonal monitoring results during the following year in service are presented in Appendix B.

7.1 Lateral movement of the deck and abutments

The lateral movements of the surface of the deck established using inclinometer tubes I3 and I4 in the south and north abutments respectively are shown in Figure 16c and Figure B1 of Appendix B. Figure 16a shows the variation in mean deck temperature and Figure 16b shows the movement at the central pier over the same period.



(a) Mean deck temperature (thermocouples)

(b) Movement at central pier

(c) Movement at abutments

Figure 16 Lateral movements at deck level (Phase 2)

Generally a fall in temperature produces a contraction of the deck and Figure 17 shows the regressions of top of abutment movements against deck temperature. Movements obtained on inclinometer tubes I3 and I4 were similar which indicated that little or no movement of the central pier was occurring. On the basis of the limited number of readings, some of which were in the construction period, an overall change of 0.397mm/^{o}C for the 42.9m long deck was obtained. This corresponded to a coefficient of deck thermal expansion of 9.25×10^{-6}/^{o}C which is consistent with the expected value when limestone aggregates are used in the concrete (BD37, DMRB 1.3). The Geomensor results, shown in Figure 16c, confirm that there was little or no movement of the central pier.

7.2 Axial loads and bending moments developed in the north abutment

Figure 18 and Figure B2 of Appendix B show the variation with time of the axial loads and bending moments determined from measurements using the vibrating wire strain gauges in the piled abutment. Soon after the deck was cast, loads were approximately those calculated from the dead load assuming the deck was simply supported. As has been previously discussed, the simply supported case is likely to predict the minimum load taken by the abutment. Over the following 150 days, some increase in load occurred, with values generally stabilising between 500 and 800kN/m although there were some exceptions to this.

After day 819 when the road was opened to traffic and during the period of seasonal monitoring, the deck temperatures ranged from approximately +2^{o}C to +25^{o}C and only small changes in axial load in the piled abutment were measured. Slightly more fluctuation in the bending moments occurred particularly with the measurements on gauges nearer to the top of the abutment. Figure 19 shows the distribution of bending moment with depth for seven dates. As with Phase 1, a negative bending moment was recorded at the top of the integral abutment due to the moment from the deck dead load. A peak bending moment of about 450kNm/m was recorded at about 99m AOD which was just below finished ground level beneath the bridge. This bending moment reversal and the general shape of the distribution is typical of that which would be expected for an embedded retaining wall propped at the top and founded in stiff clay (Padfield and Mair, 1984).

7.3 Axial loads and bending moments developed in the deck

As construction of the bridge for Phase 2 took place over a much shorter time than Phase 1, fewer readings were available from the instruments installed in the deck. The measurements of temperature, bending moment and axial load in the deck that were available during this construction stage are shown in Figure 20 whilst Figure B3 of Appendix B shows the measurements available during the seasonal monitoring. Generally the axial load developed during construction was no more than 3000kN/m whereas comparable values for Phase 1 (Figure 12) were about double this.

Calculated shrinkage and creep magnitudes over the period of monitoring suggested that, in theory, upper bound values of 8% and 10% respectively of the measured deck load could be due to these effects.

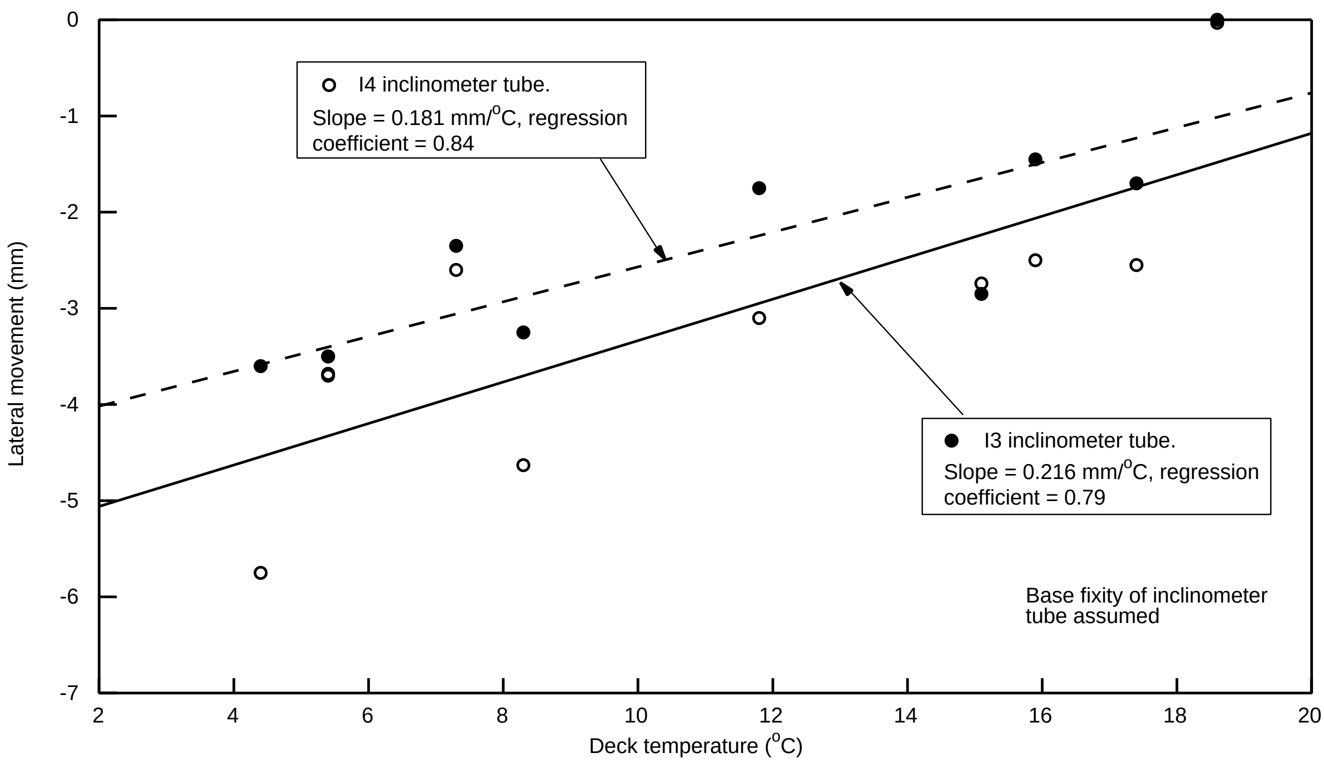
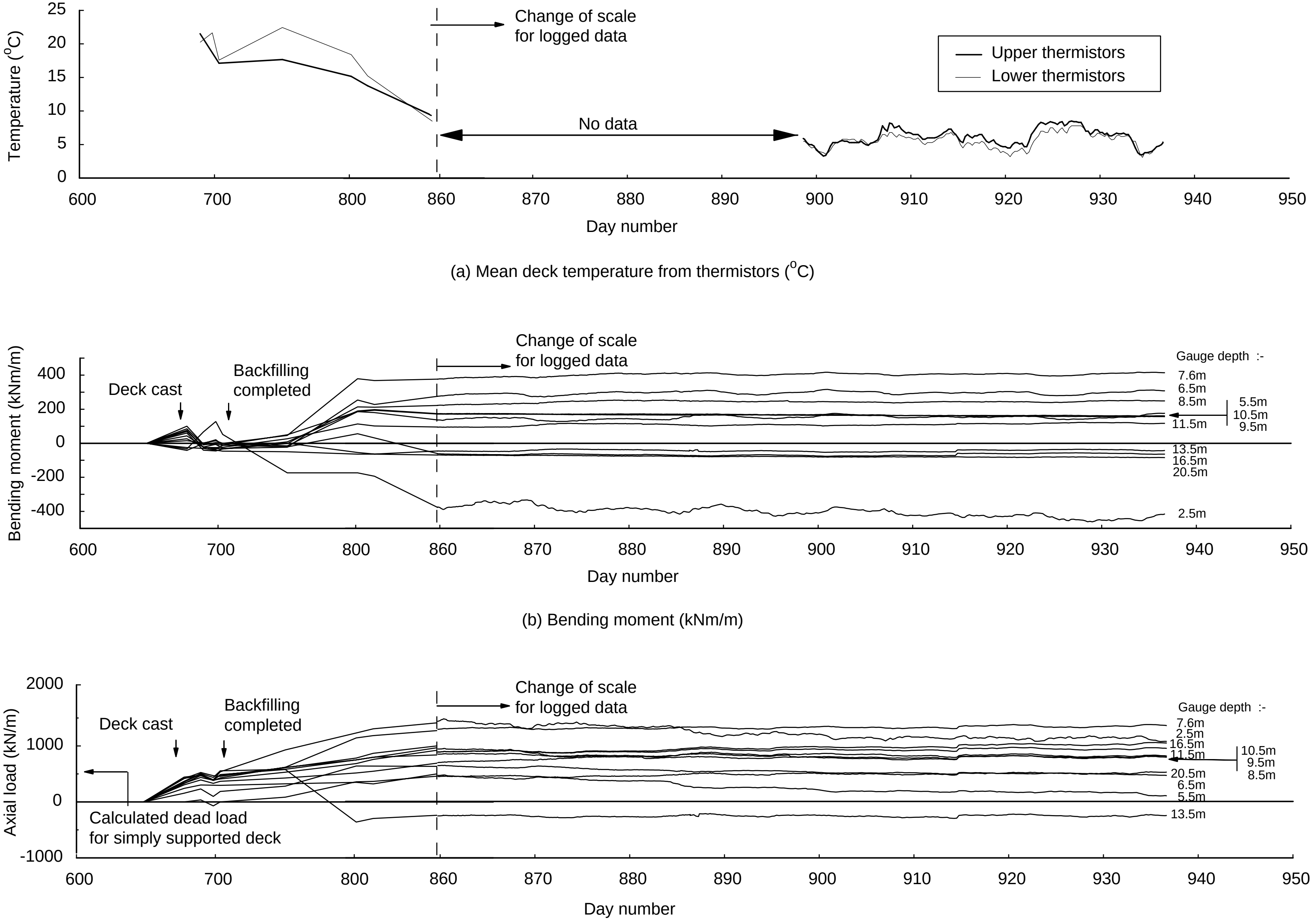


Figure 17 Variation in lateral movement of the top of the abutments with deck temperature (Phase 2)



(a) Mean deck temperature from thermistors (°C)

(b) Bending moment (kNm/m)

(c) Axial load (kN/m)

Figure 18 Variations in pile loads and moments with temperature (Phase 2)

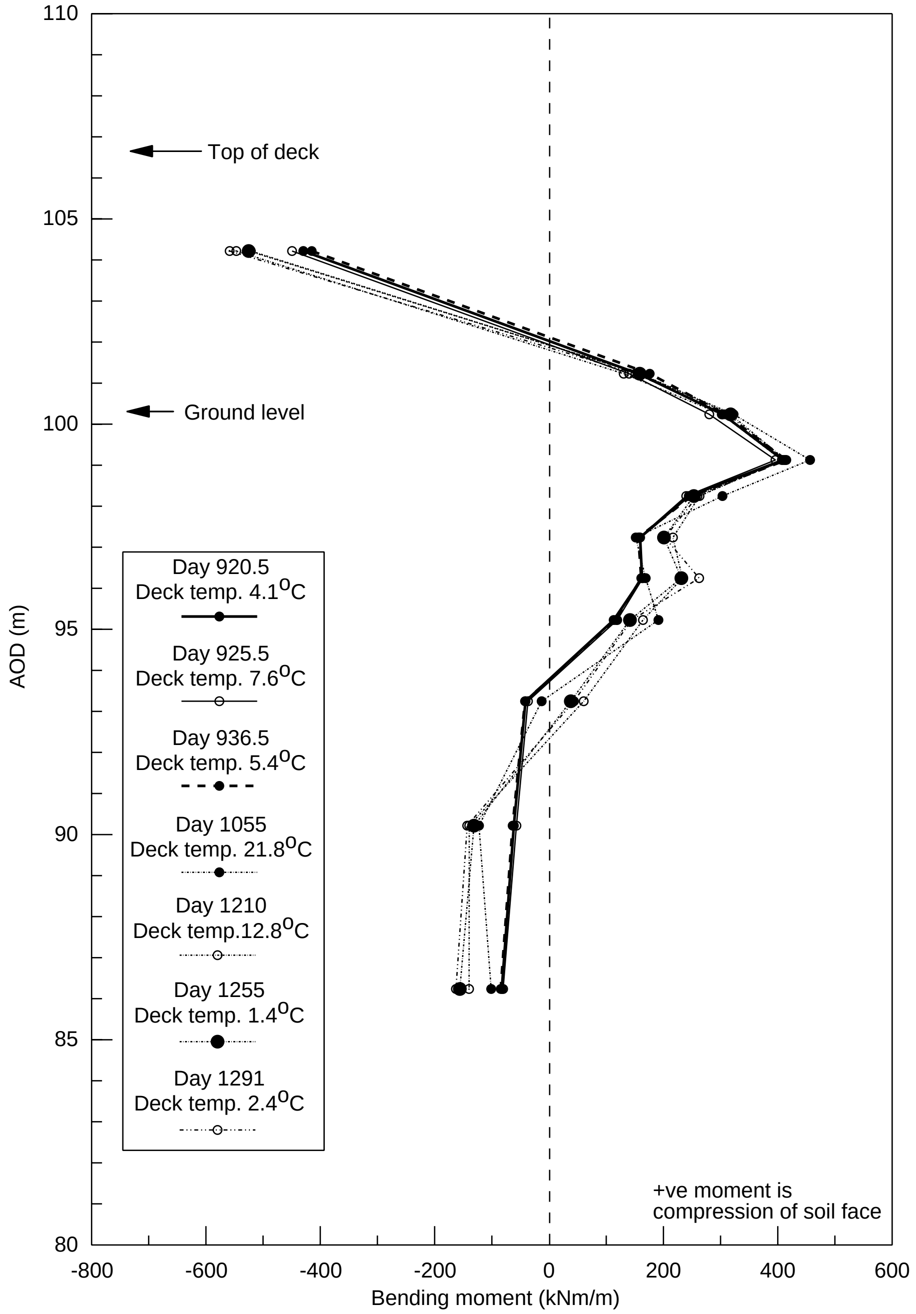
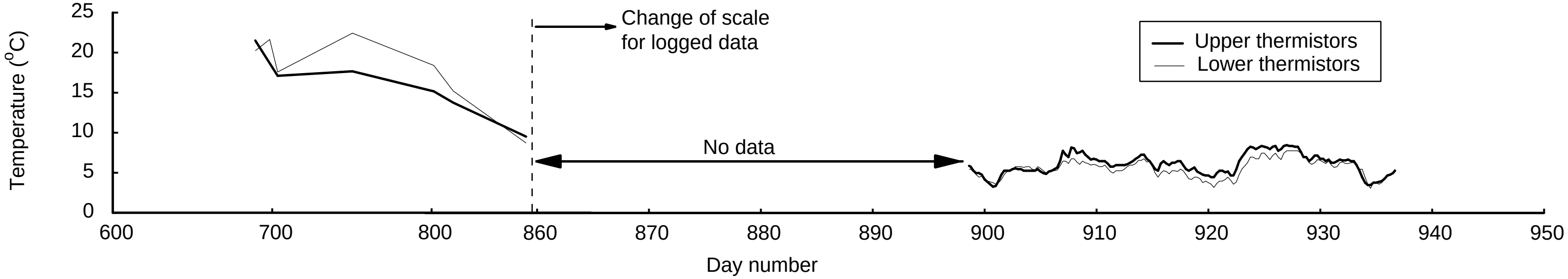
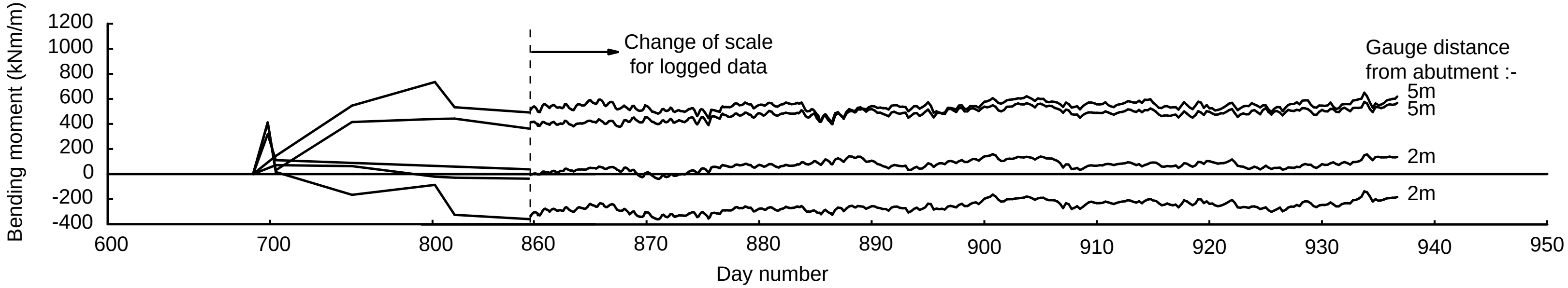


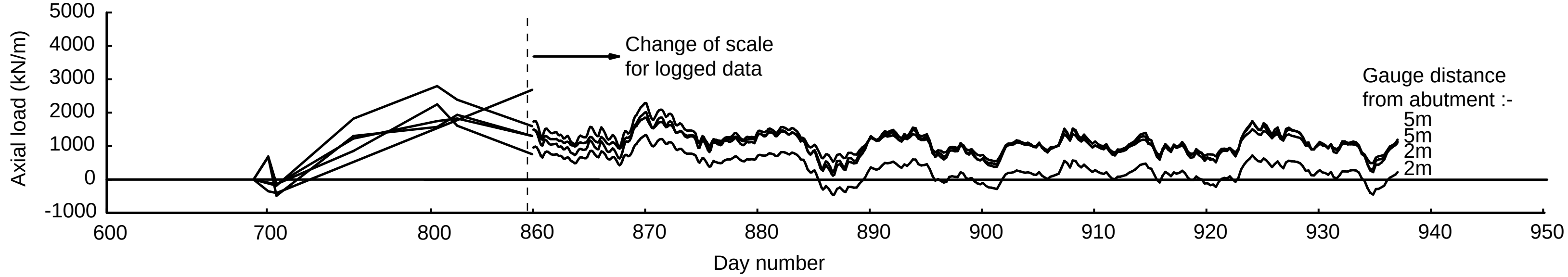
Figure 19 Variation of wall bending moment with depth (Phase 2)



(a) Mean deck temperature from thermistors (°C)



(b) Bending moment (kNm/m)



(c) Axial load (kN/m)

Figure 20 Variations in deck loads and moments with temperature (Phase 2)

The results in Figure B3 showed that the axial loads developed during the hottest period of the seasonal monitoring, when the expansion of the bridge was at a maximum, were almost treble those developed during the coldest spells when the bridge had contracted to a minimum deck length. A comparison of the seasonal changes with those for Phase 1 (as shown in Figure A3) suggested that axial loads in the latter case were again about double those measured for Phase 2.

After opening of the road to traffic on day 819, thermal fluctuations in deck axial loads and bending moments occurred. These effects are investigated in Figure 21. Figure 21a shows that axial loads in the deck changed with temperature at a mean rate of 113kN/m/^{o}C. This value can be compared with that of 144kN/m/^{o}C for the stiffer abutment of Phase 1 (Section 6.3) where deck expansion is more constrained. If the abutments of the bridge were fully constrained and assuming the measured coefficient of thermal expansion (from Section 7.1), a load increase of 487kN/m/^{o}C would be anticipated.

The bending moment data in Figure 21b show that a small sagging moment of 10kNm/m/^{o}C developed at 5m from the abutment as the deck temperature increased. At 2m from the abutment, there was little evidence of any significant thermal hogging or sagging of the deck as indicated by the low regression coefficient for this data.

7.4 Lateral stresses acting on the north abutment

As for Phase 1, the lateral stresses measured behind the Aerofill packing between the reinforced earth fill and the abutment wall were again small. Plots of the variation of lateral stress with time are compared with the variation of mean deck temperature in Figure 22 and Figure B4 of Appendix B. During the initial two months after completion of the reinforced earth, pressures on the four cells at about 1.5m depth were in the range 5 to 18kN/m^2. In the following period of about 80 days when computer logging of the data was available, deck temperatures were below 10^{o}C and the maximum lateral stress at this depth remained below 15kN/m^2. During the period of seasonal monitoring, the deck temperatures, as mentioned earlier, ranged from +2^{o}C to +25^{o}C, and the resulting lateral stresses measured behind the Aerofill packing were in the range from zero to +15kN/m^2. The reinforced earth fill extended to a depth of 2.5m and measurements of lateral stresses in the underlying *in situ* ground in which the bored piles were founded were not available.

8 Conclusions

The field performance has been evaluated of two integral bridges of 40m span over the M66 Manchester Outer Ring Road. One of the bridges (Phase 2) was constructed with contiguous bored pile abutments founded in glacial till. The other bridge (Phase 1) was a more conventional portal frame structure with the abutments retaining granular backfill. Field measurements were obtained during construction and over the first two years in service and the following conclusions reached.

i For both bridges, expansion and contraction of the deck took place with temperature changes. Measurements indicated that the coefficients of thermal expansion of the deck were 9.6×10^{-6}/^{o}C and 9.25×10^{-6}/^{o}C for Phase 1 and Phase 2 bridges respectively. These values were consistent with those expected when limestone aggregates are used in concrete (BD37, DMRB 1.3). During remedial work on the Phase 1 bridge, the south and north abutments did not move identically and some lateral movement (4mm) of the bridge towards the north occurred.

ii Shortly after both decks were cast, vertical axial loads on the abutments of about 500kN/m were measured which corresponded with the value calculated from the dead load of the deck assuming each span between the central pier and the abutment was simply supported. Loads increased to between 500 and 1000kN/m on removal of the formwork. These results suggest that, in the design of integral abutments and where the deck is cast in place, it would be prudent to allow for the total bridge dead load being equally shared between the supports as the assumption of simply supported spans may not be appropriate.

iii Bending moments in the abutments of both bridges varied as changes in temperature caused deck expansion and contraction. The largest changes occurred near to the top of the abutments. In both Phases 1 and 2, bending moments at the abutment tops were negative due to the moment from the deck dead load being transferred to the integral abutment. With the bored pile abutment, a peak bending moment of about 450kNm/m was recorded just below finished ground level beneath the bridge: bending moment reversal then occurred at depth. The results with the portal frame structure were complicated by the construction sequence and the presence of a lower permanent prop.

iv Generally the axial load developed in the integral deck during construction was no more than 3000kN/m for Phase 2 whereas comparable values for Phase 1 were about double this. This may have been caused by the complicated construction sequence involving remedial works for Phase 1 or the higher flexural rigidity (about 4 times) of the Phase 1 reinforced concrete abutments compared with the Phase 2 bored pile abutments. Results over the first two years in service indicate that the thermal load change in the deck developed against the stiffer Phase 1 abutment was 144kN/m/^{o}C compared with 113kN/m/^{o}C for Phase 2. These values can be compared with that of about 500kN/m/^{o}C calculated for a bridge with fully constrained abutments. Only small changes in deck bending moment with temperature were measured for both bridges.

v Lateral stresses shortly after compaction of the backfill against the north abutment of Phase 1 were consistent with those predicted using the coefficient of earth pressure at rest (K_o). As backfilling took place in the summer months, these stresses soon started to reduce as the bridge deck contracted with the approach of winter. During the following summer, maximum

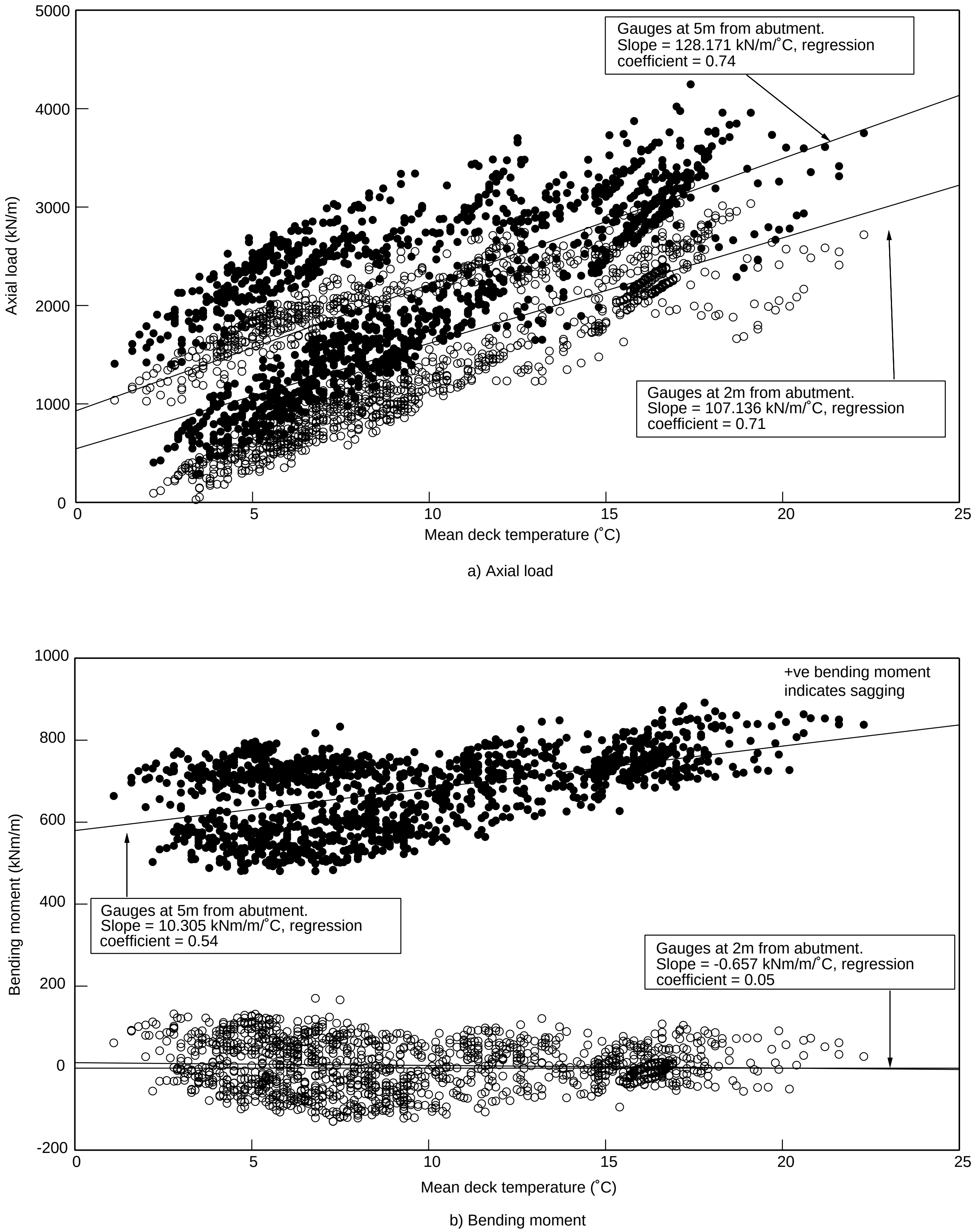


Figure 21 Variation of axial load and bending moment with deck temperature (Phase 2)

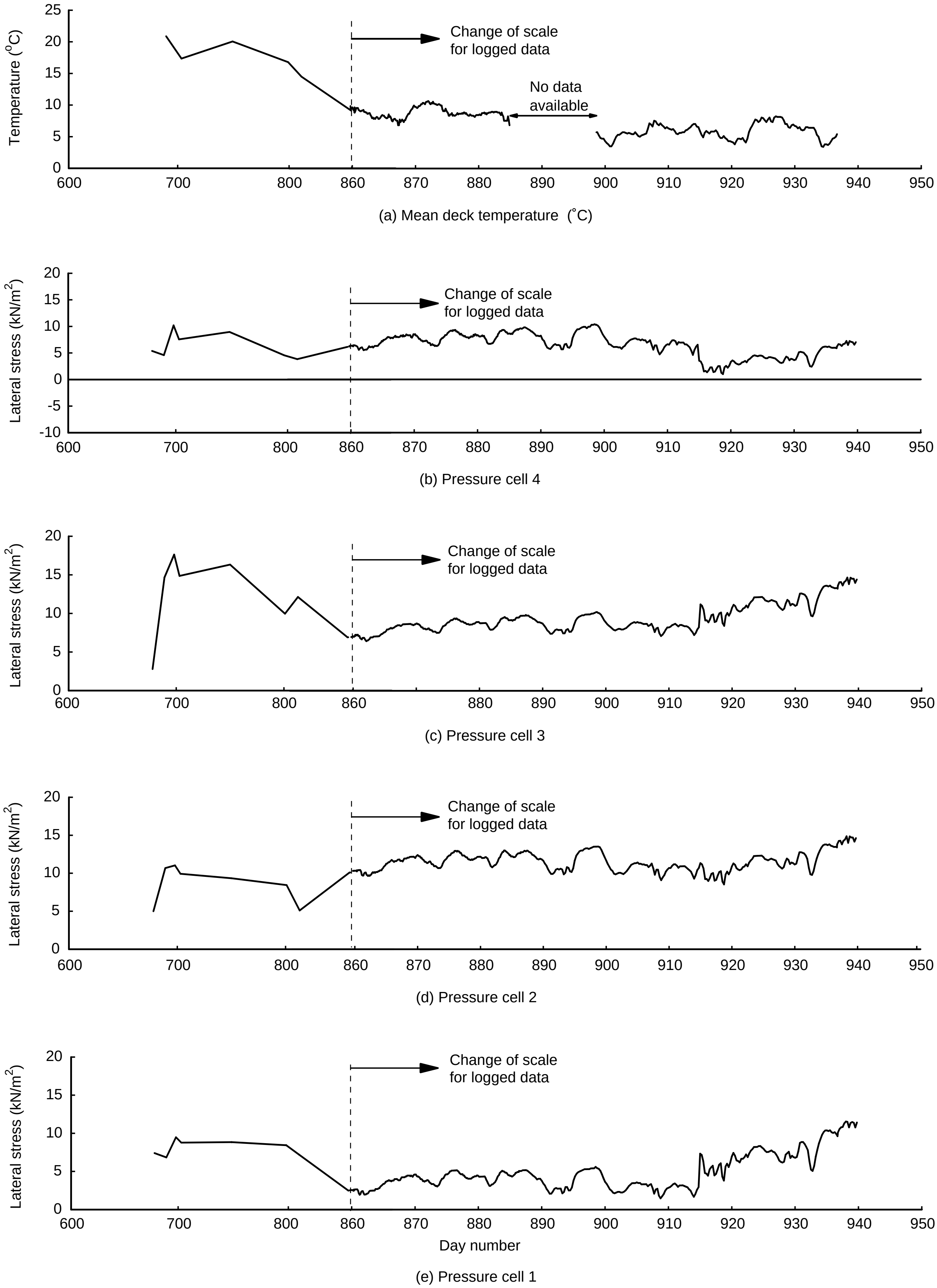


Figure 22 Variations in lateral stress with temperature at 1.53m depth (Phase 2)

lateral pressures, which were then slightly above those calculated from K_o, were recorded when the mean deck temperature reached 21.8°C. The contact stresses behind the 25mm thick compressible packing layer which separated the reinforced earth fill from the top of the abutment wall were generally small. As backfilling and reinforced earth construction took place during the summer, the first real indication of significant lateral stress escalation behind the abutment might be expected to take many seasonal cycles of movement to develop. Further long term monitoring is required to assess this effect.

9 Acknowledgements

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Thanks are due to Mr G Biswas of Highways Agency for permission to carry out the study at this site. The co-operation and assistance of the site staff of Mouchel Consulting Partners (Mr C C Sargent, Mr G Williams, Mr A White, Mr J M Heaton and Mr P Roddis) and of the Miller Kier Joint Venture is gratefully acknowledged.

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Appendix A: Seasonal monitoring results for Phase 1

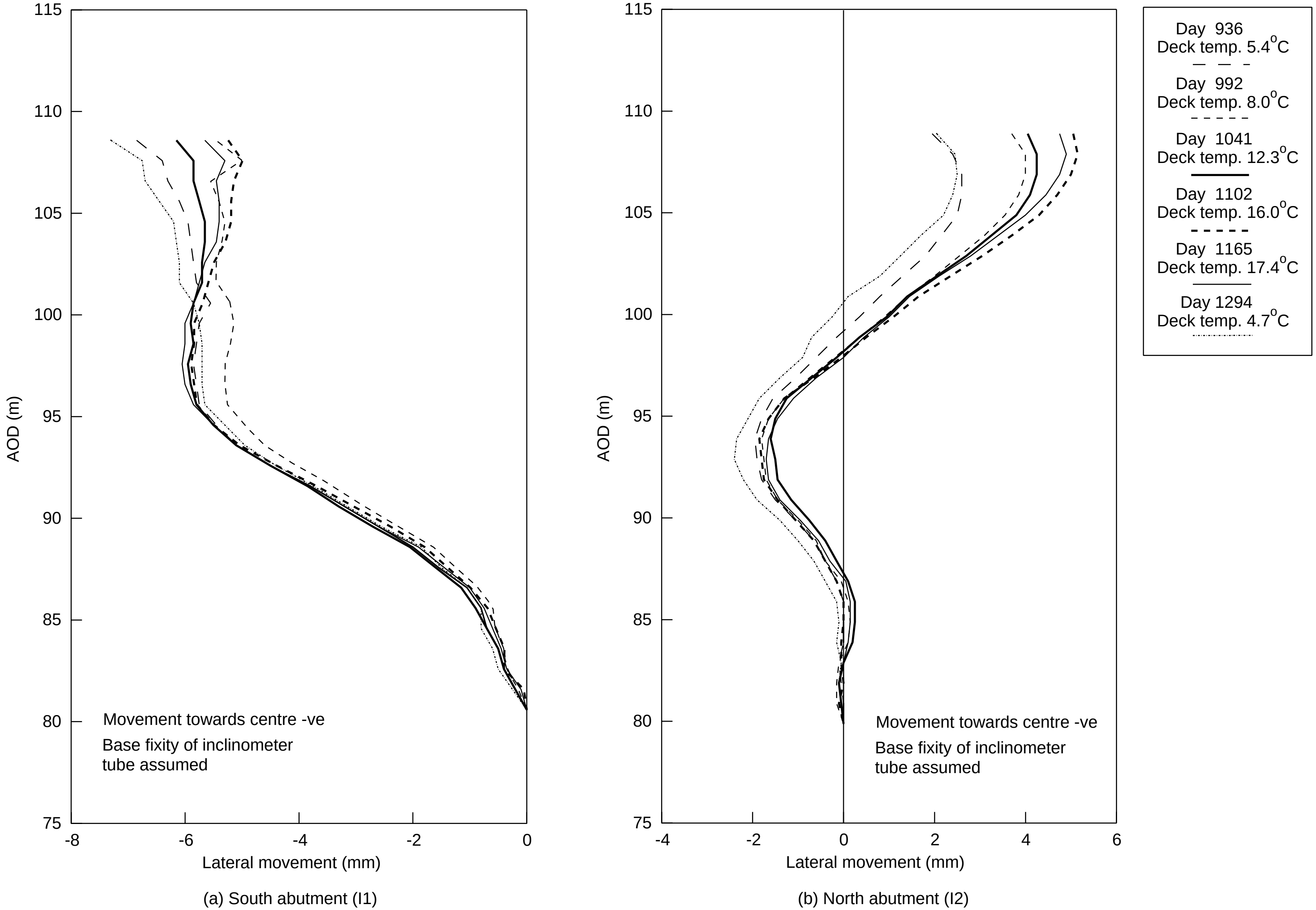


Figure A1 Lateral movements measured after bridge opening

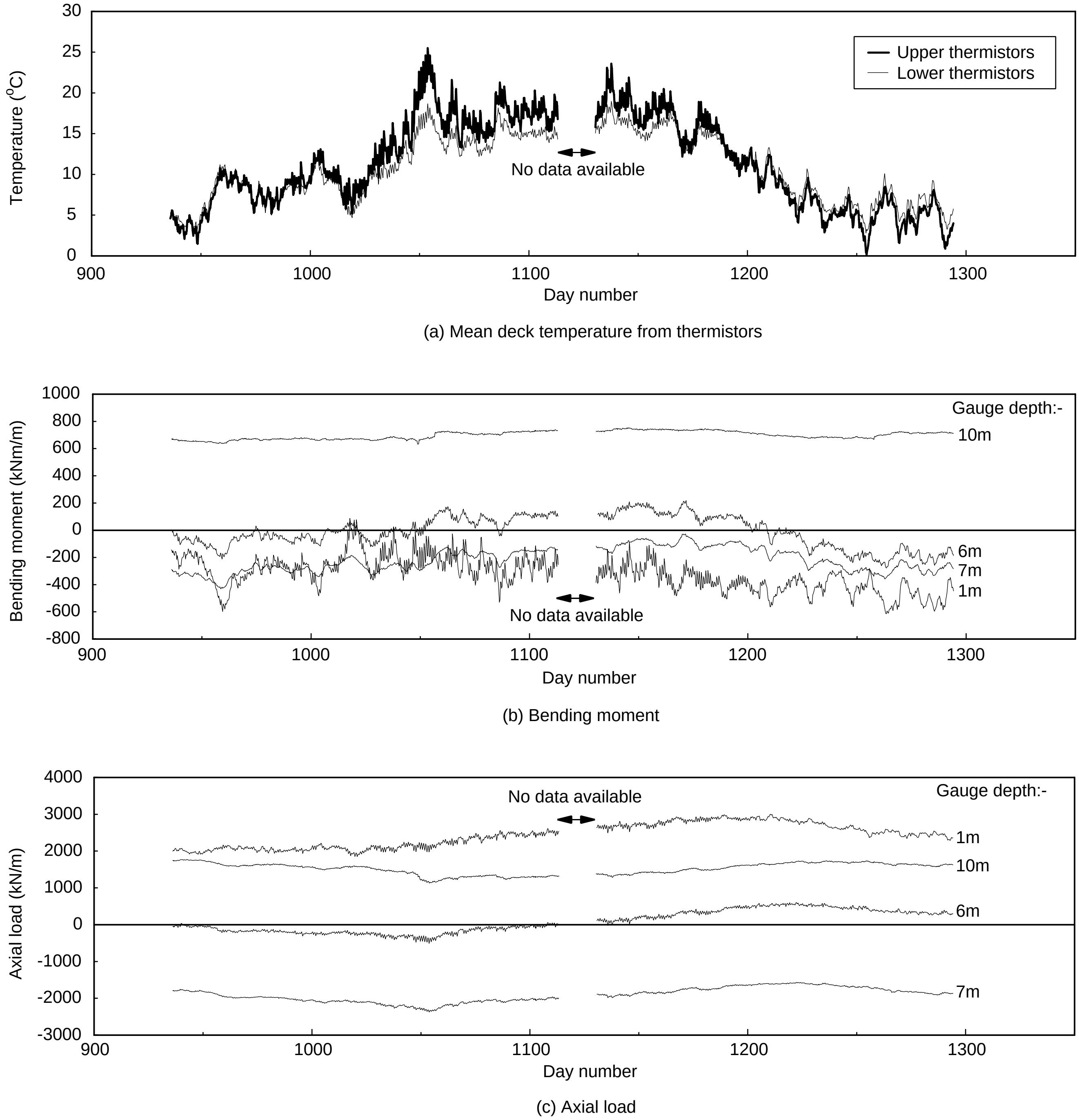
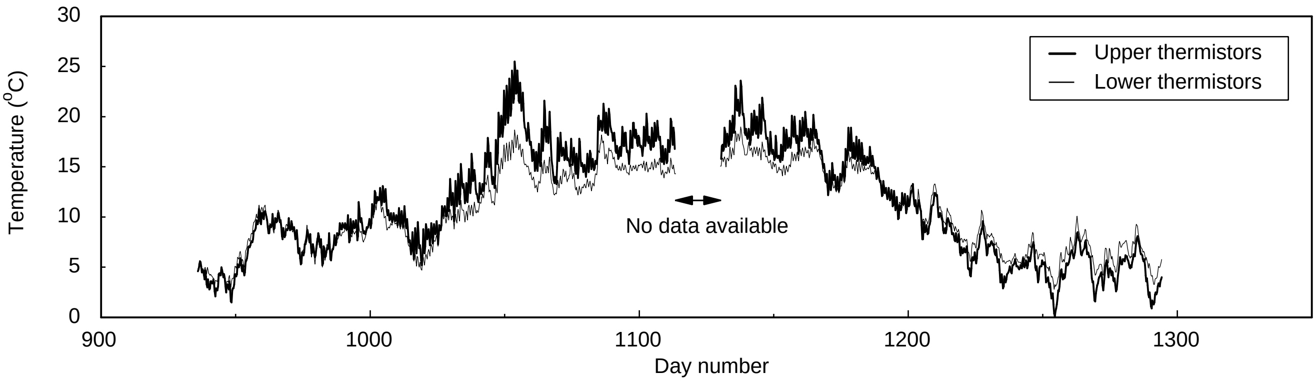
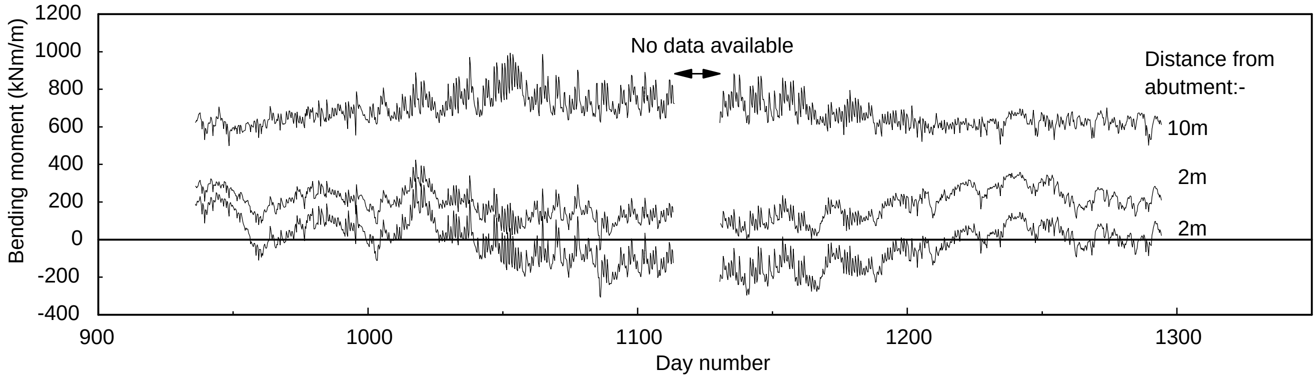


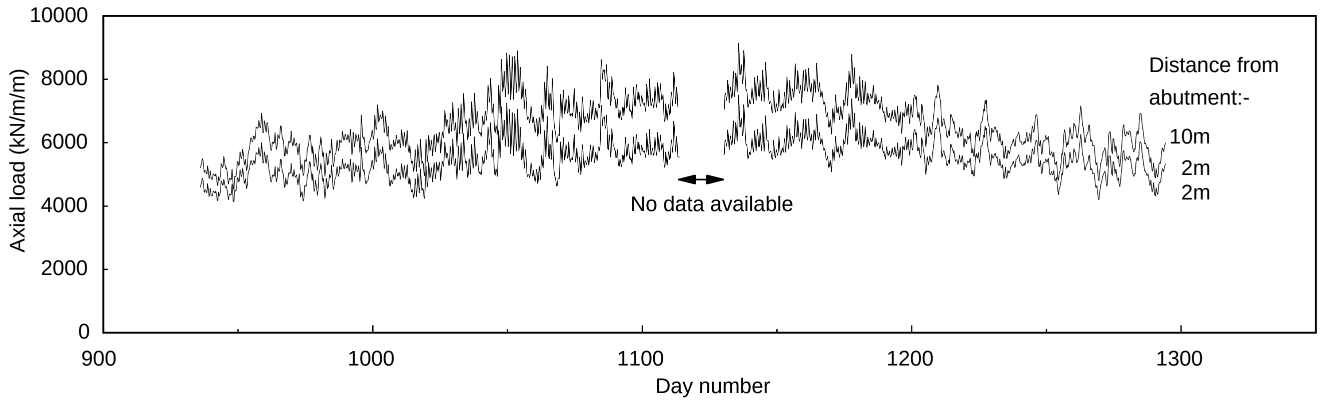
Figure A2 Variations in abutment loads and moments with temperature



(a) Mean deck temperature from thermistors

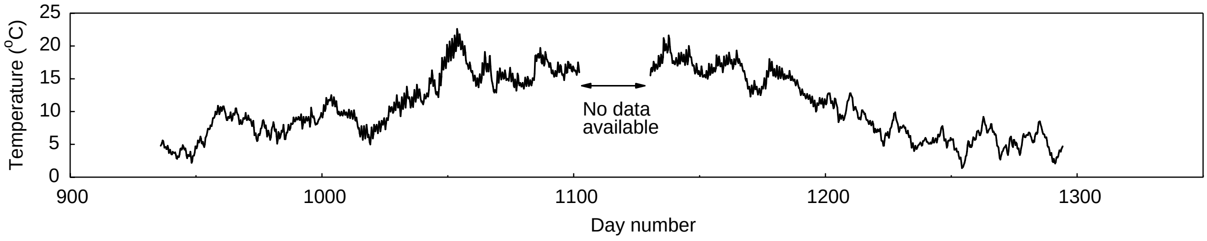


(b) Bending moment

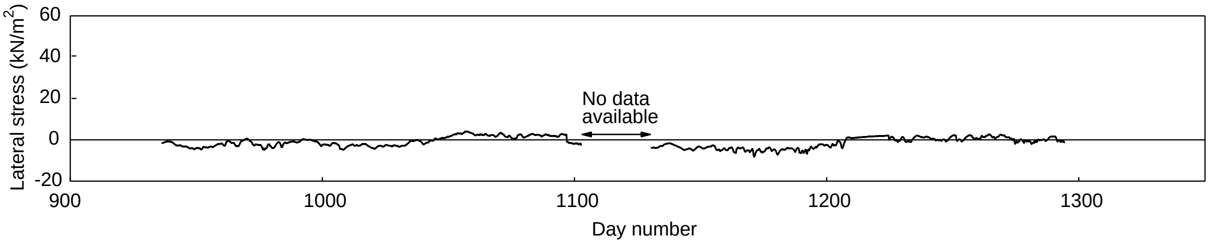


(c) Axial load

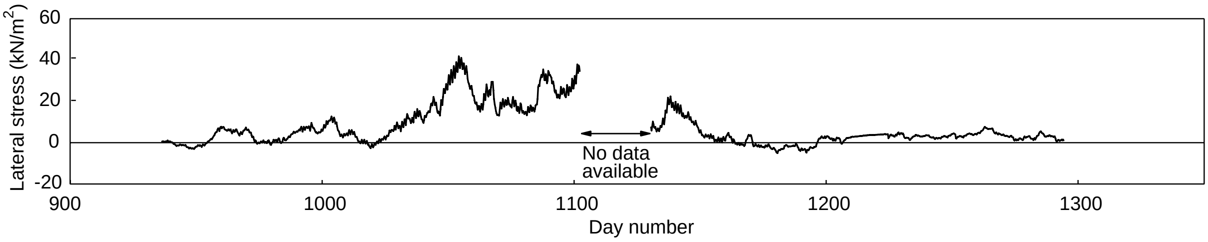
Figure A3 Variations in deck loads and moments with temperature



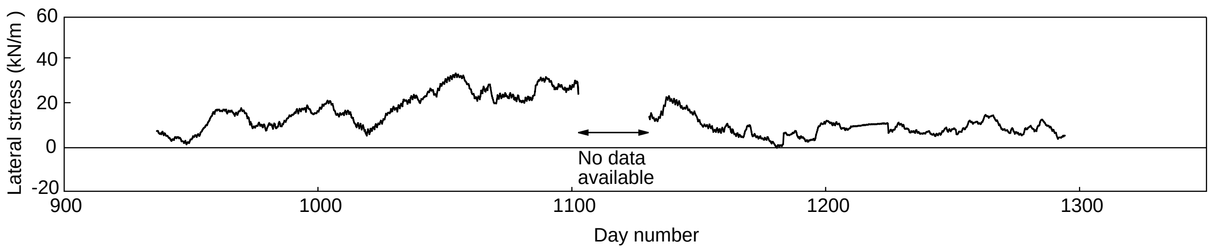
(a) Mean deck temperature (^{o}C)



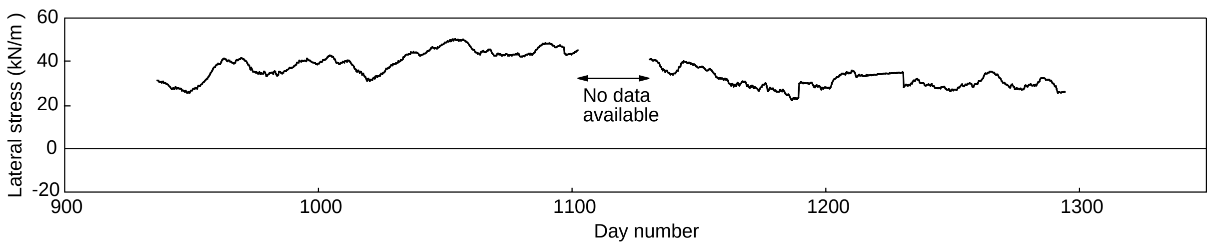
(b) Average of cells 6 to 9 (1.62m below deck)



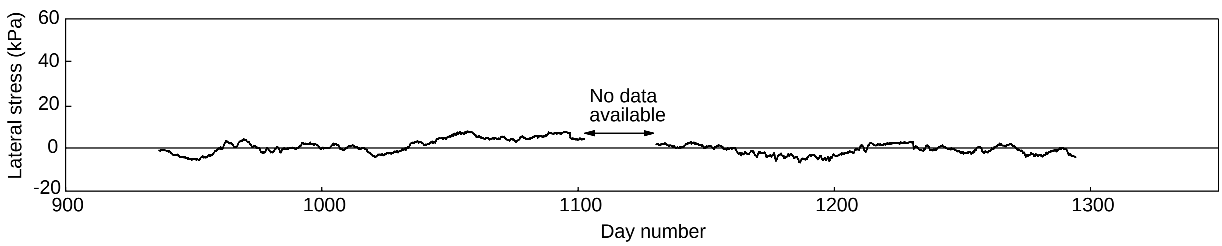
(c) Pressure cell 4 (3.3m below deck)



(d) Pressure cell 3 (4.1m below deck)



(e) Pressure cell 2 (5.2m below deck)



(f) Pressure cell 1 (6.3m below deck)

Figure A4 Variations in lateral stress with temperature

Appendix B: Seasonal monitoring results for Phase 2

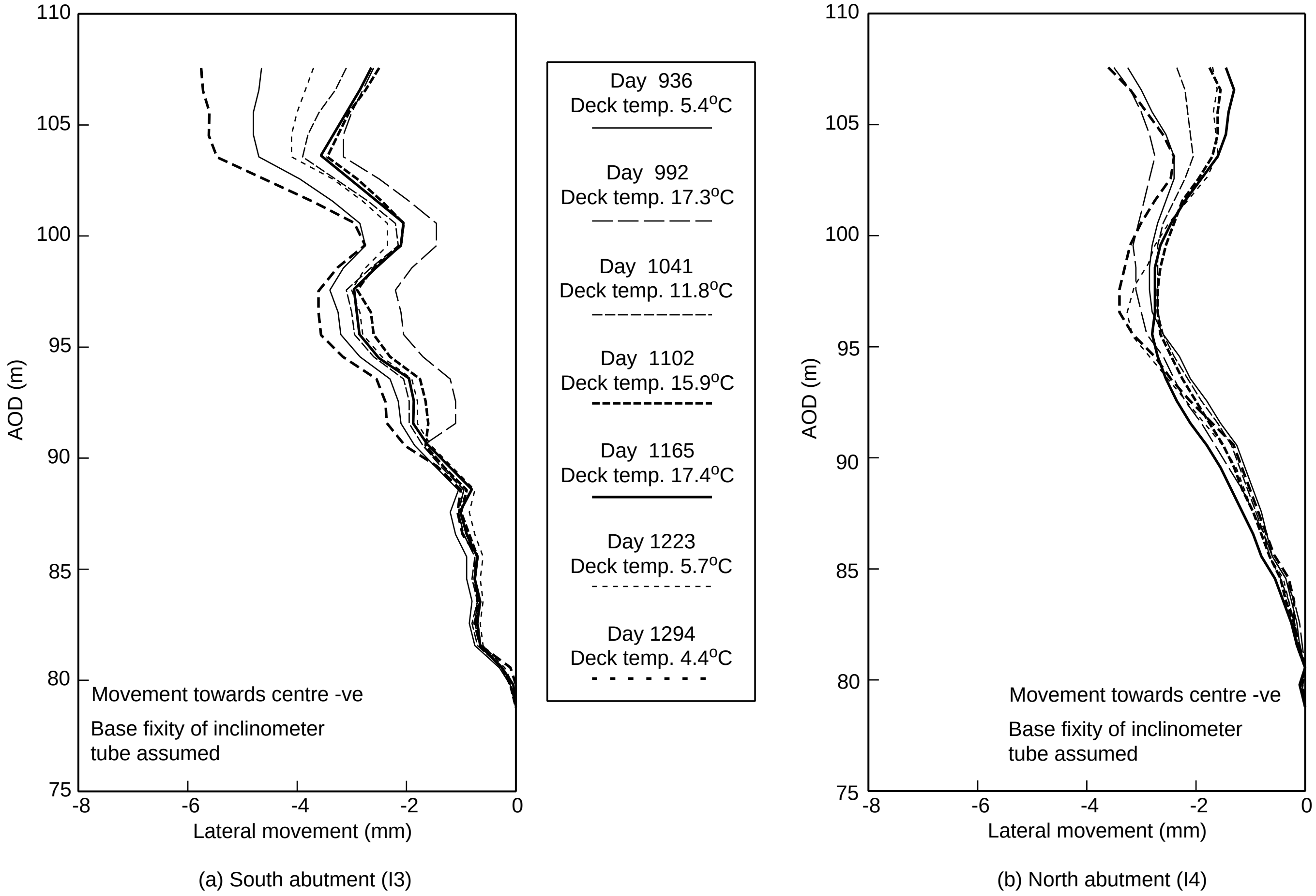


Figure B1 Lateral movements measured after bridge opening

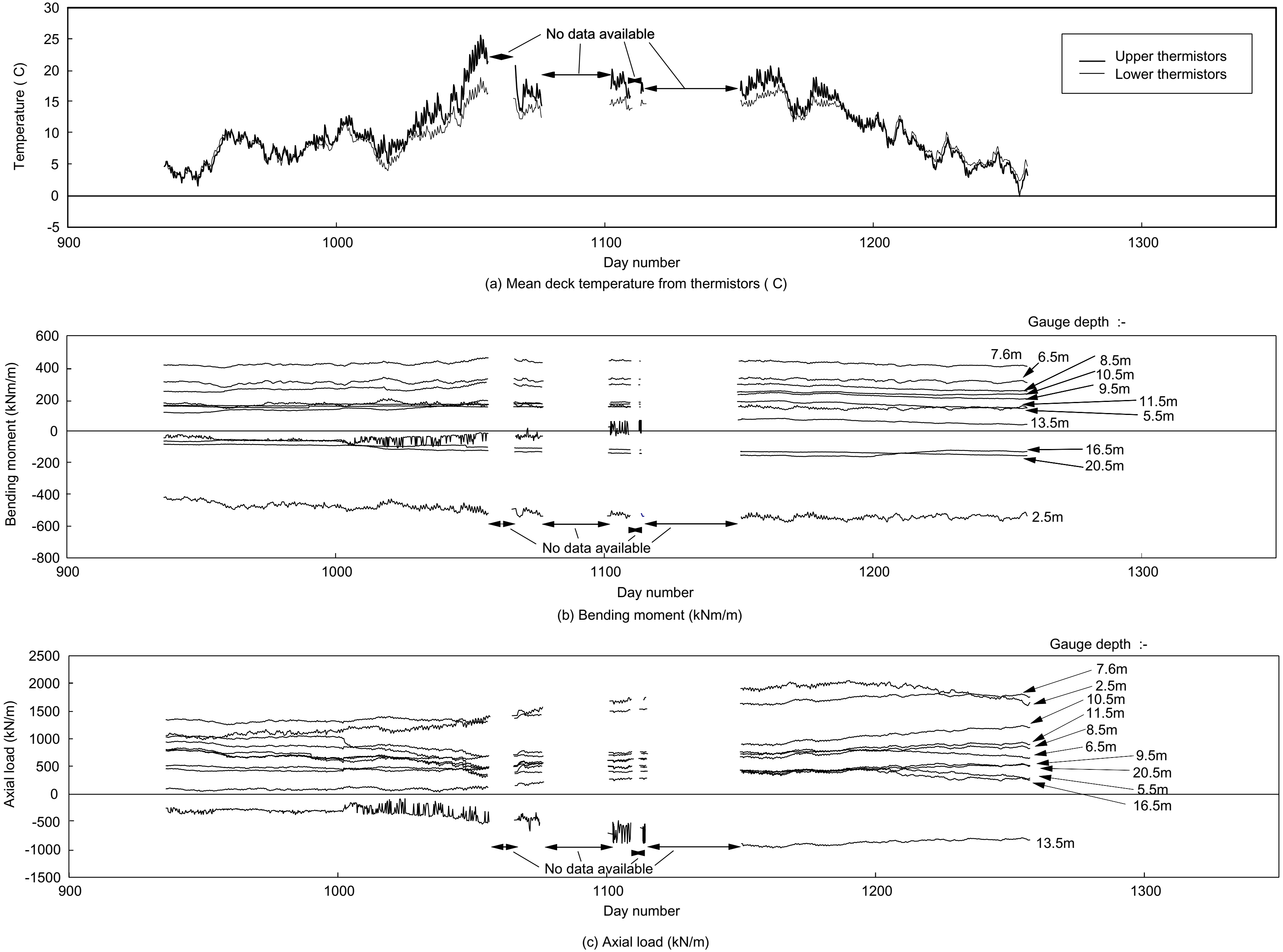


Figure B2 Variations in abutment loads and moments with temperature

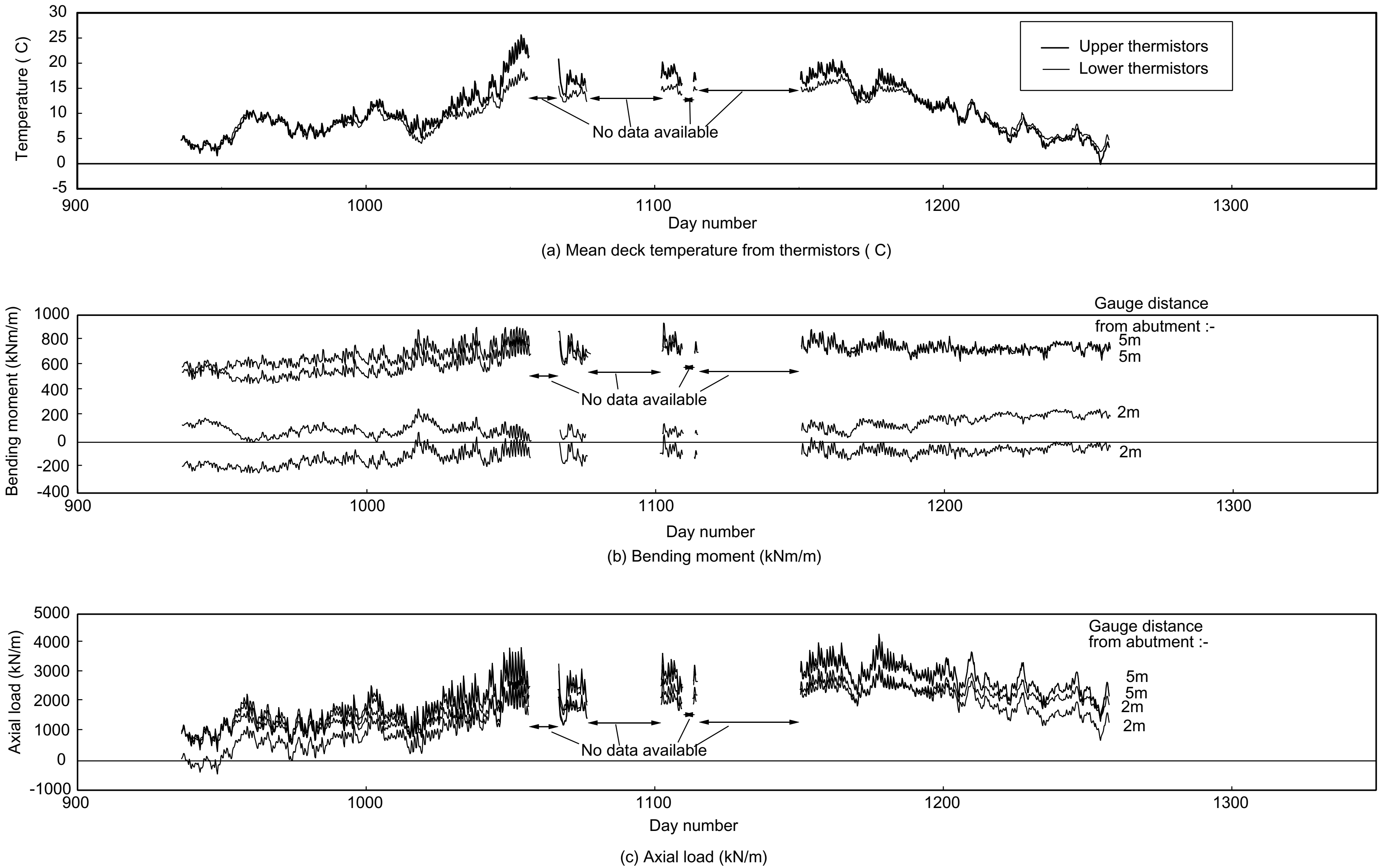


Figure B3 Variations in deck loads and moments with temperature

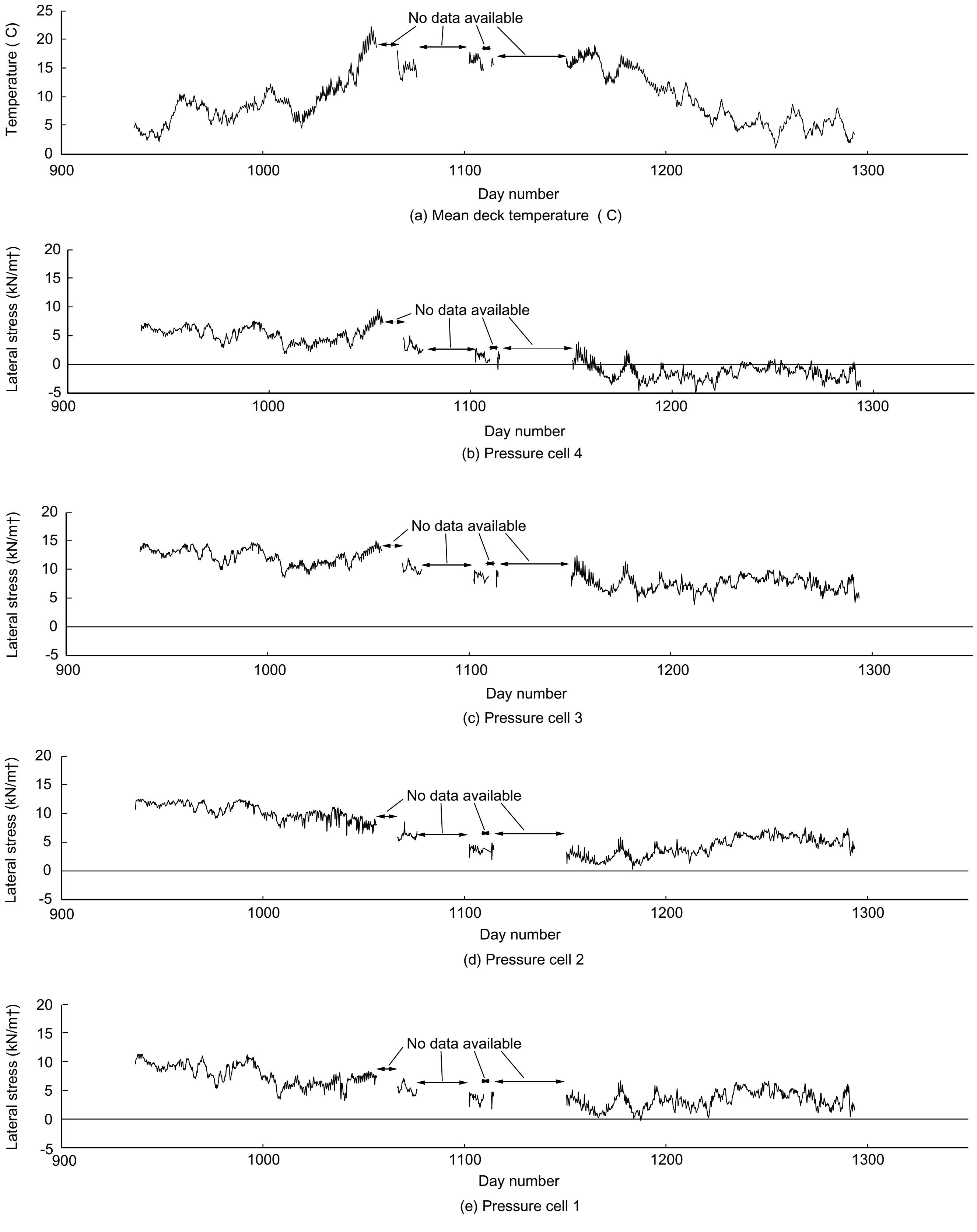


Figure B4 Variations in lateral stress with temperature at 1.53m depth

Abstract

BA42 (DMRB 1.3) gives advice on the design pressures relevant to three main classes of integral abutment, ie. shallow height, full height portal frames and embedded wall abutments. Field measurements on a shallow height abutment have been previously reported, however few data are available on the performance of full height abutments. This study reports on the performance of two full height integral bridges of about 40m span over the Manchester Outer Ring Road. One of the bridges (Phase 2) was constructed with contiguous bored pile abutments founded in glacial till. The other bridge (Phase 1) was a more conventional portal frame structure with the abutments retaining granular backfill. This report describes field measurements during construction and over the first two years in service.

Related publications

TRL344 *Seasonal thermal effects over three years on the shallow abutment of an integral bridge in Glasgow* by P Darley, D R Carder, and K J Barker. 1998 (price £25, code E)

TRL328 *Consultation document on suitability tests for stress absorbing layers behind integral bridge abutments* by P Darley and D R Carder. 1998 (price £20, code A)

TRL290 *Innovative structural backfills to integral bridge abutments* by D R Carder and G B Card. 1997 (price £25, code E)

TRL178 *Seasonal thermal effects on the shallow abutment of an integral bridge in Glasgow* by P Darley, D R Carder and G H Alderman. 1996 (price £25, code E)

TRL146 *Cyclic loading of sand behind integral bridge abutments* by S M Springman, A R M Norrish and C W W Ng. 1996 (price on request, code T)

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