INTEGRAL BRIDGE ABUTMENTS

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M.Eng. Project Report

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ABSTRACT

This report presents information collated on the earth pressures and settlements that develop behind model and full-scale integral bridge abutments. The objective is to facilitate the design of integral bridges; for which the current UK guidelines are arguably overly conservative. The report concludes that integral bridge design lengths should be incrementally increased. Modifications to BA 42/96 are suggested based on measured earth pressure increases due to cyclic loading. With adequate compaction and drainage, approach slabs are unnecessary.

1.0 INTRODUCTION

An integral bridge may be defined as having no expansion joints or sliding bearings, the deck is continuous across the length of the bridge. Integral bridges are alternatively referred to as *integral abutment bridges*, *jointless bridges, integral bent bridges* and *rigid-frame bridges*. *Semi-integral* or *integral backwall* bridges typically have sliding bearings, but no expansion joints.

Expansion joints and bearings have traditionally been used to accommodate the seasonal thermal expansion and contraction of bridge decks, typically of the order of tens of millimetres. A survey of approximately 200 concrete highway bridges in the UK, carried out for the Department of Transport, however, revealed that expansion joints are a serious source of costly and disruptive maintenance work (Wallbank, 1989 cited in Springman *et al.,* 1996). In response to this, the Highways Agency published Advice Note BA 42 in 1996, promoting the design of integral bridges and stating that all bridges up to 60m in length should be integral with their supports.

Although the integral bridge concept has proven to be economical in initial construction for a wide range of span lengths, as well as technically successful in eliminating expansion joint/bearing problems, it is susceptible to different problems that turn out to be geotechnical in nature. These are potentially due to a complex soil-structure interaction mechanism involving relative movement between the bridge abutments and adjacent retained soil. Because this movement is the result of natural, seasonal thermal variations, it is inherent in all integral bridges.

There are two important consequences of this movement:

1) Seasonal and daily cycles of expansion and contraction of the bridge deck can lead to an **increase in earth pressure behind the abutment**. This build-up of lateral earth pressures is referred to as 'soil ratcheting' (England & Dunstan, 1994 cited in England *et al.,* 2000). This can result in the horizontal resultant earth force on each abutment being significantly greater than that for which an abutment would typically be designed and represents a potentially serious long-term source of integral bridge problems.

2) The second important consequence is **the soil deformation adjacent to each abutment**. It has been postulated that settlement troughs occur as a result of the soil slumping downward and toward the back of each abutment. In many cases this is addressed by the incorporation of an approach slab into the bridge design, whereby the slab is intended to span the void created underneath it. However, there is also evidence to suggest that such a slab is unnecessary and that regular maintenance of the bridge surface can be sufficient to largely overcome this problem.

1.1 Purpose and Scope of Project

Guidance on the design of integral bridges in the UK can be found in BA 42/96; aspects of this document, however, are regarded as being overly conservative (England *et al*. 2000). These issues are currently being addressed by the Highways Agency and the Advice Note is in the process of being updated. New variations of integral bridge designs are continuously emerging, however, and it is important that design guidelines are not inappropriately used in such cases. The process of modifying this document is therefore inherently time-consuming.

In the interim, some bridge designers are reaching agreements on departures from the code with the Highways Agency, based on more recent research findings. The aim of this report is to draw information from a wide variety of sources in order to increase the confidence of designers in the performance of integral bridges and subsequently facilitate this design process.

Numerical modelling will undoubtedly become an increasingly powerful tool for the design and analysis of integral bridges, but time constraints on a project of this type have resulted in the focus being placed solely on experimental testing and field testing. It is hoped that these results may also be used to help improve numerical modelling techniques. This report is also limited to quantifying the earth pressures and settlement behind an abutment, rather than postulating a mechanism for this behaviour.

Integral bridges are not a new concept. The first section of the M1, constructed in 1958-59, required 127 bridges, of which 88 are of a continuous portal type, that act integrally with the surrounding soil and range in span up 41m. Various integral bridge construction techniques are widely and successfully used in the USA, Sweden, Canada and Australia, which include spans up to 100m (Burke, 1989; Hambly 1991 - cited in Springman *et al.,* 1996). The boundaries of design are being pushed further still with the use of innovative backfill materials, leading to a bridge in the USA totalling 300m in length, which is also performing well. (Frosch, 2002).

1.2 Mode of Bridge Movement

With the elimination of expansion joints, the thermal expansion and contraction of the bridge deck must be alternatively accommodated. Card & Carder (1993) postulated that for portal frame bridges, such as those found on the M1, this could be achieved by vertical deflection of the bridge deck, rather than by longitudinal thermal movements of the bridge deck being transmitted to the abutments. Subsequent experimental research on two such bridges by Darley and Alderman (1995), however, concluded that vertical movements were generally very small, effectively disproving this theory. These findings are supported by results from field studies on a shallow abutment bridge supported by piles (Lawver, 2000) and a shallow spread-base abutment bridge (Darley *et al.*, 1998) which showed that the primary abutment movement was horizontal translation. Embedded abutments that are pinned at their base, however, rotate about the toe of the abutment wall (Barker & Carder, 2001).

1.3 Magnitude of Deck Expansion

The magnitude of the longitudinal deck expansion is dependent primarily on the bridge temperature. Extensive research carried out in the UK (Emerson, 1973, 1976, 1977, cited in England *et al.,* 2000) has resulted in temperature parameter, the *effective bridge temperature* δT_{EB} (or EBT), which relates well to the shade temperature. The research resulted in published EBT values for concrete, composite (steel-concrete) and steel box section bridges in different geographical locations in the UK (Emerson, 1976). Composite and steel decks may be assumed to have the same coefficient of thermal expansion as concrete, but they experience higher changes in EBT, so that the seasonal movements of composite decks and steel decks are about 121% and 145% of that of a concrete deck, respectively (England *et al*., 2000).
This movement will be combined with deck strains (since the deck axial load must be in equilibrium with the lateral resistance of the soil behind the abutment) and post construction effects such as shrinkage.

2.0 LITERATURE REVIEW - Model Test Procedures

There are two principal texts published in the UK on the design of integral bridges, both of which were commissioned by the Bridges Engineering Division of the Highways Agency. The first is TRL Report 146: Cyclic loading of sand behind integral bridge abutments, (Springman *et al.*, 1996), which is the basis of the recommendations in Advice Note, BA 42/96. The second is Integral Bridges: A fundamental approach to the time-temperature loading problem, (England *et al.*, 2000), which presents the findings of a second phase of research aiming to examine the issues of settlement and the build-up of lateral earth pressures. What follows is a review of the model test procedures from the two reports. Both studies involved numerical modelling but this report analyses only the experimental modelling techniques, although the conclusions of the reports draw from both sets of results.

2.1 TRL Report 146: Cyclic loading of sand behind integral bridge abutments

TRL Report 146 follows on from a literature review conducted by Card & Carder (1993) and, based on centrifuge model tests and subsequent numerical analyses, makes recommendations on the design of integral bridges. This section aims to analyse some of the assumptions made and clarify the notation used. Salient results are presented in Sections 3 and 5.

2.1.1 Modelling technique

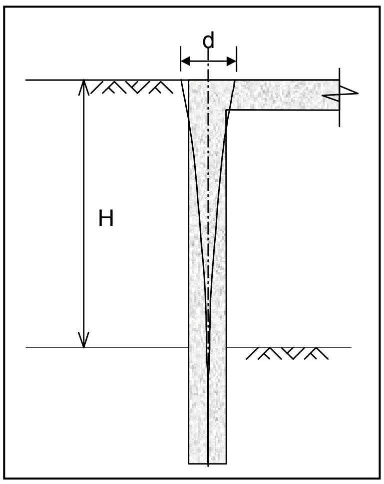

Figure 2.1a Embedded abutment wall

The experimental work was carried out at the Cambridge Geotechnical Centrifuge Centre using the 10m balanced beam. The principle of centrifuge modelling is to recreate the stress conditions that would exist in a full-scale construction (prototype), using a model of greatly reduced scale (Schofield, 1980, cited in TRL 146). During these tests a radial acceleration of 60g was applied to the models, which were correspondingly 1/60 of the prototype size.

Seven centrifuge model tests were carried out on an embedded abutment wall (see Fig 2.1a) and two on spread base abutment walls (see Fig 2.1b). The tops of the walls were cyclically displaced to model the thermal expansion and contraction of the bridge deck. The flexibility and roughness of the model walls was varied, in addition to the density of the fill and the magnitude of the displacements.

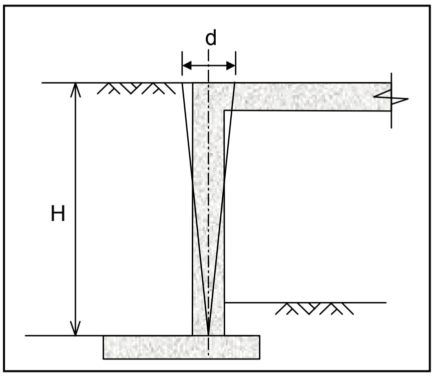

Figure 2.1b Spread base wall

Of the seven embedded wall tests, one test was carried out using a stiff wall and one test using a rough wall. The results showed that increasing *either* the stiffness *or* roughness of the wall generated larger lateral earth pressures. The potentially worst case (rough and stiff) was not modelled.

2.1.2 Variable parameters

2.1.2.1 Direction of initiation of first cyclic movement

The direction of initiation of first cyclic movement was varied to model the effects of constructing the bridge either in the summer or winter. The results showed that the time of year in which the bridge was constructed had no influence on the behaviour of the bridge in the long term.

2.1.2.2 Magnitude of cyclic movement

The aim of the tests was to determine the effect of abutment movement on the magnitude of the induced soil strain and soil behaviour. Integral abutments are subject to a range of movements including deformations due to dead loads, settlement due to consolidation of founding strata and swelling/shrinkage/creep of concrete deck/ abutments (Springman *et al*., 1996). Provided the bridge has been designed to accommodate these, it is primarily the cyclic movements that will have the greatest detrimental effect on the bridge.

The model test system was set up whereby small, cyclic displacements were imposed at the top of the abutment wall using a rotating cam device mounted on an eccentric vertical shaft. The magnitude of these displacements was based on the predicted longitudinal movement of the bridge deck, based on the work by Emerson (1976). If the deck length and thermal expansion properties are known, these can be used to determine the magnitude of the cyclic displacements imposed on the backfill. The maximum horizontal displacement at the beam-abutment junction, d can therefore be calculated using equation (1), where the variables are shown in Figures 2.1a and 2.1b.

$$d = \alpha \delta T_{EB} L \qquad (1)$$

where L = span (m)

α = coefficient of thermal expansion (e.g. $12x10^{-6}$/°C for concrete)

d/2 = amplitude of abutment displacement (m)

A bridge needs to be designed to withstand the following:

- One 1:120 year cycle of ~46°C for concrete decks
- Seasonal cycles between summer and winter temperatures
- Daily cycles between day and night temperatures

The selected input displacement ranges were:

- 100 cycles at ±6 mm (θ_i = 0.12°) for daily cycles
- 100 cycles at ±12 mm (θ_i = 0.23°) for serviceability state,
- 100 cycles at ± 30mm (θ_i = 0.57°) for annual cycles (ultimate state),
- 100 cycles at ± 60mm (θ_i = 1.15°) for 1:120 year cycles(s)

These movements are equivalent to a 200m span bridge, with a coefficient of thermal expansion α = $12x10^{-6}$/°C subjected to δT_{EB} ranges of 5°C, 10°C, 25°C and 50°C respectively.

e.g.
$$d/2 = 12x10^{-6} x5x200/2$$
$$d/2 = 6mm$$

A span of 200m is significantly longer than the current maximum recommended length of 60m and this goes some way to demonstrating the extent to which this limit is conservative.

It is important to note that H (as shown in Figures 2.1a and 2.1b) represents the retained height of the abutment walls, in contrast with H representing the overall height, as defined by Card and Carder, 1993.

2.1.2.3 Density of retained material

The fill used in the centrifuge models was a fine sub-angular silica sand (100/170 grade, Fraction E) with particle sizes ranging between 90 and 150μm (5.4 - 9mm at prototype scale). The maximum and minimum attainable void ratios of the sand were e_{max} = 1.014 and e_{min} = 0.613. The sand had a specific gravity G_s of 2.65 where γ_w = 9.81kN/m^3 and ϕ_{crit} was found to be 32°.

The soil densities ranged from 23 to 97% I_D, where $I_D = [(e_{max} - e) / (e_{max} - e_{min})]$. The first soil placement technique involved pouring the sand from a hopper at 1g with a minimal drop height and small aperture to create a loose deposit with I_D ~20 - 35%. It is extremely unlikely that backfill would ever be placed at such a low density, nor as consistently, but this test serves to illustrate the detrimental effects of placing poorly compacted fill. Higher relative densities (I_D ~80%) were achieved similarly, but with a greater drop height and larger aperture; this again resulted in a very consistent fill. The densest deposit (I_D ~95%) was achieved by the successive vibration of 20mm layers of soil, more closely representing the compaction methods used in construction. The current requirement of the Specification for Highway Works (SHW) is for an "end product density equivalent to 95% relative compaction" (Steel & Snowdon, 1996). Relative compaction is not the same as relative density; links between densities obtained experimentally and those achieved in the field can be found in Jewell, 1992.

These densities were measured prior to the centrifuge being briefly accelerated to 100g to artificially increase the earth pressure. The effect this had on the density of the fill immediately prior to testing has not been recorded. The report also notes that the "voids ratio and hence the relative density of the sand changed during the model preparation stages as the sample was handled and loaded onto the centrifuge arm; more so for the loose samples". Reliance should not therefore be placed on the absolute quoted values of the relative densities; the overall trends they exhibit however are sufficient from which to draw conclusions.

2.1.2.4 Stiffness of embedded wall

The flexural rigidity of the embedded wall was varied by a factor of 10 ($1.5x10^5$ - $1.5x10^6$ kNm^2/m) to model a flexible piled wall and a concrete diaphragm wall. The inverted T wall was designed to model a typical 1m spread-based reinforced concrete abutment wall and base with 1.5% reinforcement and a flexural stiffness of 1.21×10^6 kNm^2/m at prototype scale.

2.1.2.5 Roughness for both wall types

In integral abutments a higher value of wall friction leads to a more conservative design since a larger passive earth pressure will be generated, which in turn creates a greater bending moment in the wall. The first five embedded wall tests were carried out on models described as being 'smooth', these had a ratio of $\delta/\phi \approx 2/3$, where δ is the angle of friction of the soil/structure interface and ϕ is the internal angle of friction of the soil. These were followed by a single test on a 'rough' flexible embedded wall onto which a layer of retained sand had been glued in order to obtain a ratio of $\delta/\phi=1$. The spread base abutment walls were both 'rough' and stiff. The results for the 'smooth' wall may be more relevant if a geosynthetic liner is placed behind the wall.

2.1.3 Measurements

The model was set up with an array of displacement transducers, bending moment transducers, an axial load cell and pressure cells and was photographed during the centrifuge flight. The pressure cells were used to measure the total lateral earth pressures acting on the wall during cyclic displacements in flight. Soil markers were placed in the fill to form a grid that was photographed in flight and used for spot chasing to give contours of shear strain. The interpretation of the data is discussed in Section 4.

2.2 Integral Bridges: A fundamental approach to the time-temperature loading problem (England et al.*, 2000)*

This investigation comprised three parts: theoretical analyses, experimental tests and analytical studies; again this report focuses only on the experimental aspect of the work. The experiment was limited to a single type of abutment, but the results are well presented and reproducible.

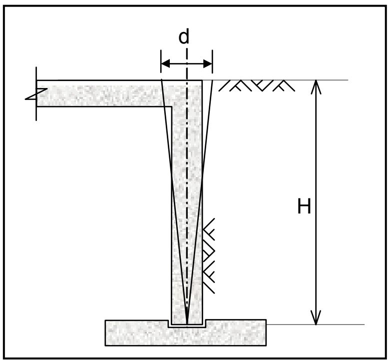

Figure 2.2a A stiff abutment wall with a pinned base

2.2.1 Modelling technique

The main difference between the physical modelling carried out at UCL and the work carried out by Springman *et al*. (1996) was that the wall was pinned at its base (see Figure 2.2a) and subjected to single- and double-cycles at 1g. England's stress level is therefore a factor of 60 too small. Following three preliminary tests (to verify the suitability of the test apparatus and identify the importance of shakedown behaviour) three single-cycle tests of different amplitudes of rotation were carried out to model seasonal effects. These were followed by one double-cycle test to investigate the combined influence of daily and seasonal temperature cycles on soil behaviour, stress escalation and settlement.

2.2.2 Variable parameters

2.2.2.1 Magnitude of cyclic movement

Once again the displacement was imposed at the top of the wall to represent the expansion and contraction of the bridge deck. The different amplitudes of rotation were based on different deck lengths, however, rather than different temperature ranges. The first three tests had imposed rotational amplitudes of d/2H = ±0.13%, ±0.25% and ±0.35% corresponding to a seasonal temperature range of 50°C on 60, 120 and 160m span bridges respectively. The coefficient of thermal expansion was assumed to be $12x10^{-6}$/°C for concrete.

2.2.2.2 Density of retained fill

The retained fill initially had a relative density, I_D, of 94.1 ±0.2% for all of the tests.

2.2.2.3 Stiffness and roughness of abutment wall

The abutment wall is described simply as being 'stiff'. It was constructed as a concrete wall pinned to a strip footing. The soil-abutment interface was assumed to be smooth and to develop no friction.

2.2.3 Measurements

Pressure transducers were installed in the face of the retaining wall to measure changes in the lateral earth pressure during the cyclic displacement. Spot chasing photographic methods were also used to observe the development of the shear slip band and resulting settlement.

3.0 EARTH PRESSURES - Experimental Results

Drawing information from the work of Broms & Ingleson, 1972, CIRIA, 1976 and England, 1994, England *et al*. (2000) report that an escalation of soil-wall stresses occurs during successive temperature cycles. In order to produce a design guideline, it is necessary to quantify this increase and establish the factors upon which it is dependent. What follows is a comparison of the Design Manual for Roads and Bridges (DMRB) Guideline with model test data from *TRL 146* and *Integral Bridges*.

3.1 BA 42/96 - The Design of Integral Bridges

The design recommendations for the magnitude of the lateral earth pressures in BA 42/96 are largely based on the findings of centrifuge and analytical studies reported by Springman *et al*. (1996). The report recognises the potential for stress escalation with time and proposes earth pressure distributions for the following different structural forms:

(a) Shallow height bank pad and end screen abutments

(b) Full height frame abutments

(c) Full height embedded wall abutments

The shallow height bank pad is assumed to mobilise full passive pressures. A distribution of the earth pressures is proposed for the full height abutments (see Figures 3.1a and 3.1b).

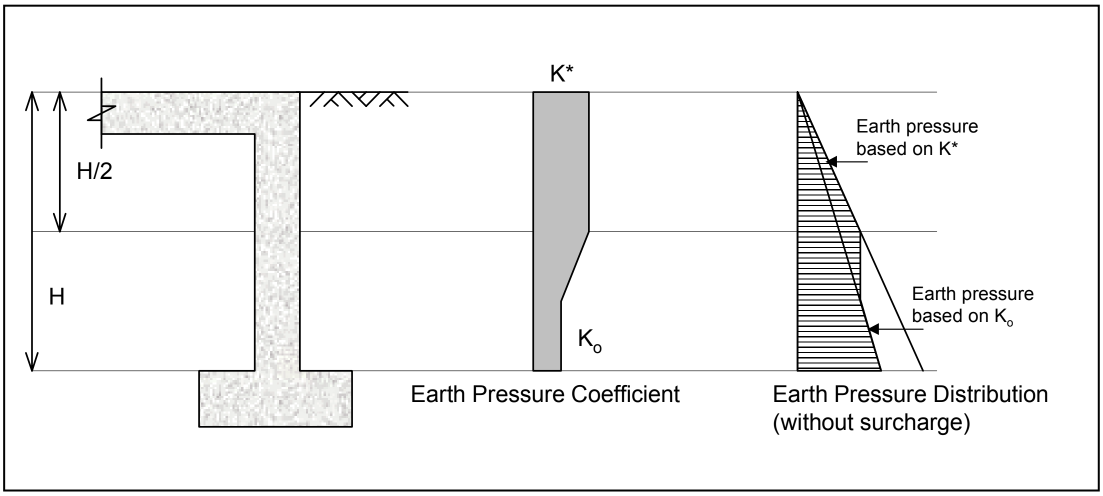

Figure 3.1a Earth Pressure Distribution for Frame Abutment

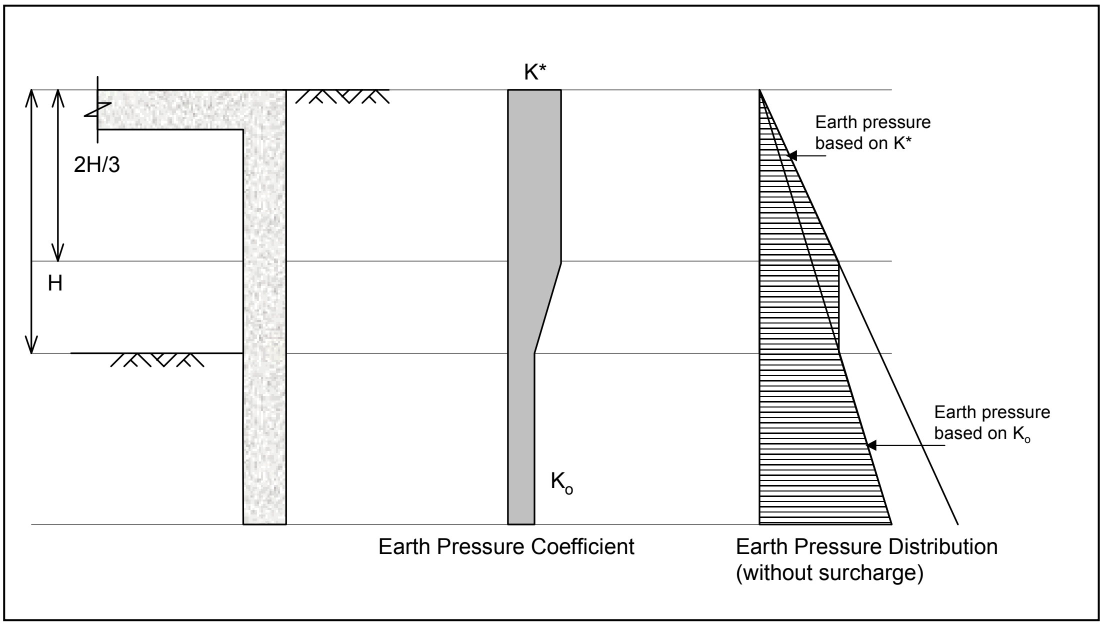

Figure 3.1b Earth Pressure Distribution for Full Height Embedded Wall Abutments

These pressure distributions are expressed in terms of K_o and K*, where K* is defined as follows in terms of the retained height (H) and thermal displacement of the top of the abutment (d), based on wall friction δ of ϕ' /2.

$$K^* = (d/0.05H)^{0.4} K_p$$

The Guidance Note also stipulates that K* should be greater than the 'at rest' earth pressure K_o and $K_p/3$, where:

$K_o = (1-\sin\phi')$ with ϕ', the effective angle of shearing resistance and K_p, the passive lateral earth pressure coefficient, also defined in BA 42/96.

It is this latter requirement that is thought to be particularly over-conservative (England *et al*., 2000).

3.2 TRL Report 146: Cyclic loading of sand behind integral bridge abutments (Springman et al. 1996)

The work carried out by Springman *et al*. draws information from a literature review of the geotechnical aspects of the design of integral bridge abutments (Card & Carder, 1993). This review concluded that the primary controlling factor of the lateral earth pressure was the magnitude of the shear strain, defined as γ_i = d/H where d and H are defined in Figures 2.1a and 2.1b. “In the absence of any current published information or research findings it is considered that linear interpolation in earth pressures from K_o to K_p should be adopted over the shear strain interval $1x10^{-5}$ to $1x10^{-3}$" (Card & Carder, 1993). Springman *et al*. (1996) recommend, however, that this approach of relating input shear strain γ_i at the wall with the magnitude of earth pressure coefficient, should not be used for design purposes without being able to link γ_i with the residual shear strain in the backfill.

One of the objectives of the centrifuge tests was to measure the earth pressures (and subsequently deduce the earth pressure coefficients), on the back of both an embedded wall and a spread-base wall, resulting from cyclic displacement of the top of the wall.

The report concludes that the overall lateral earth pressures increased with the number of cyclic displacements. The modelling also concluded that lateral earth pressures immediately decreased to active values in all cases when the wall rotated away from the fill. This is not important in terms of the structural design of the wall and deck, which is dominated by passive wall pressures, but highlights the possibility of a gap forming behind the abutments during the winter months, into which debris may fall.

Five tests were carried out on smooth flexible embedded walls, one on a rough flexible embedded wall and one on a smooth stiff wall. The lateral earth pressures increased both with increasing roughness and increasing stiffness, but no tests have been carried out in which the two parameters were combined. The input wall rotation is defined as $\theta_i = d/2H$.

- At serviceability state ($\theta_i < 0.23^o$), K<1 for flexible walls and K<2 for stiff walls.
- At ultimate state ($\theta_i > 0.5^o$), 1<K<2 for flexible walls and K~4 for stiff walls.

Results are also given for the earth pressures developing after 100 1:120 year cycles; this can have no real bearing on bridge design, however, since only one of these cycles is anticipated during the lifetime of the structure.

- After a single 1:120 year cycle 1.5<K<2.5 for a flexible wall and K~5 for a stiff wall.

The flexible embedded abutment therefore never reaches the theoretical limiting passive earth pressure $K_p = 6.5$ (based on $\phi'_{crit} = \delta = 32^o$, Caquot & Kerisel (1948)) at ultimate state or after a single 1:120 year cycle.

Even if wall friction is not considered, it remains less than the theoretical value K_p = 3.3 (based on ϕ_{crit} = 32º, δ = 0º, Caquot & Kerisel (1948)). Only the stiff embedded wall pressure exceeds K_p = 3.3 after 100 cycles at the ultimate state or a single 1:120 year cycle, but it still remains less than K_p = 6.5.

Two tests were carried out on rough stiff spread-base walls, one with dense fill (83% I_D) and one with loose fill (23% I_D).

- At serviceability state (θ_i < 0.23º), K<2, which is substantially lower than K_p = 6.5 and without wall friction it is still less than K_p = 3.3.
- At ultimate state (θ_i >0.5º), K<3.3 following 100 cycles at ultimate state and also after a single cycle at the ultimate state. The authors recommend, however, that K_p = 6.5 is a more appropriate limit for a rough wall

3.3 *Integral Bridges: A fundamental approach to the time-temperature loading problem (England* et al.*, 2000)*

George England, Neil Tsang and David Bush (2000) re-examined the problem for abutments backfilled with granular material. As described in Section 2.2 the physical modelling involved single- and double-cycle tests at 1g of a stiff wall, pinned at its base. The tests were carried out to simulate the movement of 60, 120 and 160m span bridges over a 50ºC temperature range. The authors assessed the model test results in conjunction with the numerical simulations to propose recommendations directly related to the current standard BA 42/96. The conclusions drawn were:

- K quickly changes during the early temperature cycles to span the hydrostatic wall reaction ratio, K=1. K subsequently increases at a progressively decreasing rate until a steady state value K_{ss} is reached, see Figure 3.3a.
- The extent of the stress escalation and the time required to reach the steady state are determined primarily by the bridge dimensions and the *effective bridge temperature* (EBT) fluctuations, both seasonally and daily.
- The combined response of daily and seasonal fluctuations is very similar to that of seasonal fluctuations only. Daily EBT variations help to keep the value of K closer to the hydrostatic state, for the same soil density, but also cause the additional densification of the backfill and hence increase K, resulting in nearly no net change.
- Numerical results suggest that the long-term value of K is not significantly influenced by either the initial density of the backfill or the completion date of

construction. The numerical models also suggest that K_{ss} is higher for backfill material with a higher internal angle of friction.

The experimental result for the K from the 1 in 12 scale model retaining wall will be higher than the corresponding values from the 7m high prototype because granular soils exhibit stiffer (in terms of stress ration to strain) behaviour at lower confining pressure. The absolute experimental results therefore represent a very conservative upper bound, which the authors have improved by introducing a scaling factor.

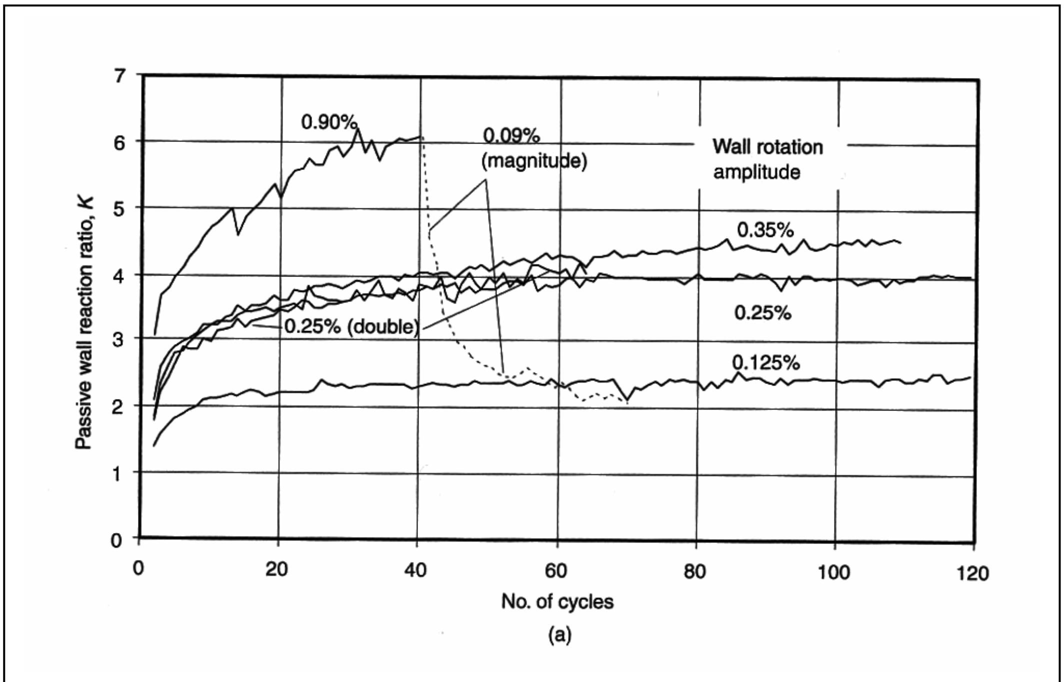

Figure 3.3a Influence of wall rotation amplitude (±d/2H) on wall reaction ratio

The following equation has been recommended with the caveat that it is based on limited experimental data obtained from model tests on a retaining wall hinged at its base:

$$K^* = K_o + (d/0.03H)^{0.6} K_p \qquad (2)$$

where K_o = initial at rest stress ratio

K_p = passive lateral earth pressure coefficient

d/H = ratio of double amplitude displacement to retained height as defined in Figure 2.2a.

The authors argue that although the lateral earth pressure distribution assumed in BA 42 is acceptable, the evaluation of K* with an origin at K=0 is fundamentally incorrect. The additional requirement of K* not less than $K_p/3$ is also too conservative; instead the initial at rest stress ratio K_o is a more suitable limit at zero wall rotation.

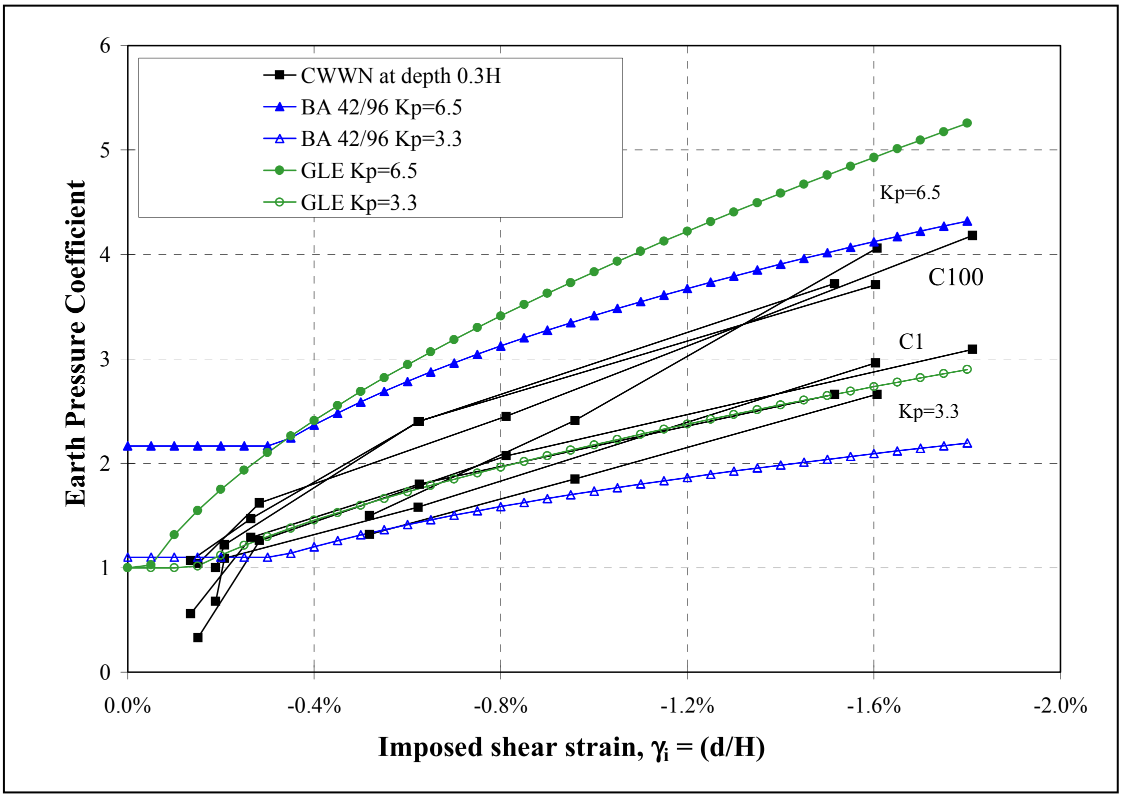

Figure 3.3b Comparison of experimental results with design guidelines

The above equation is compared graphically (GLE in Figure 3.3b) with the current design guideline and experimental values from centrifuge modelling on a spread-base wall by Ng (1996) [cited in Cheng (1999)]; this demonstrates the advantage of modifying the lower limit.

The degree of pressure increase is greater at shallower depths so only the experimental results of the uppermost pressure cell (at a depth of 0.3H) are shown for the spread-base abutment. The design lines are shown for K_p values of 6.5 and 3.3 corresponding to the theoretical passive earth pressure limits of the soil used in the centrifuge models, with and without wall friction (as discussed in Section 3.2).

It is important to recognise that the recommendations proposed by England *et al.* (2000) are based on limited experimental data obtained from model tests on a retaining wall hinged at its base. Therefore, they should not be extracted into practical design without due care. A separate research project (Arsoy *et al.,* 2002) concluded, however, with the recommendation that integral abutments with hinges should be used for longer integral bridges. This is discussed in Section 3.4.

3.4 Experimental and Analytical Investigations of Piles and Abutments of Integral Bridges (Arsoy et al.*, 2002)*

A project under contract for the Virginia Transportation Research Council, involved large-scale cyclic load tests of three pile types (H-pile, pipe pile and prestressed reinforced concrete pile) and three abutment types, reached the conclusion that integral abutments with hinges should be used for longer integral bridges. The hinge in this case was not at the toe of a spread-base abutment however, but was between the abutment and pile cap as illustrated in Figure 3.4a.

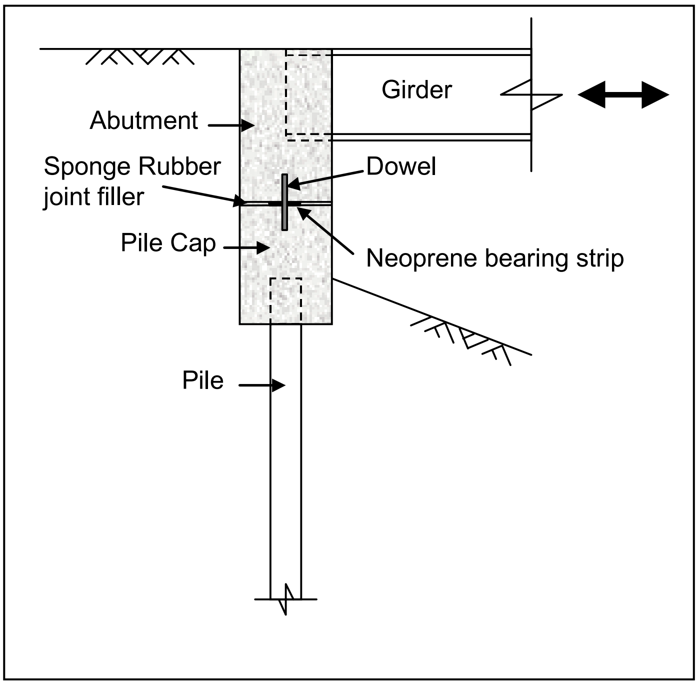

Figure 3.4a Diagram of hinged abutment

The aim of this project was to investigate the ability of an integral abutment with hinge, and three different piles types, to withstand cyclic lateral displacements induced by temperature variations. It gives no insight into the earth pressures that develop as a result of such displacements, but can be used to provide guidance on aspects of detailed design.

The data from the experimental program indicated that weak-axis steel H-piles are the best pile type for support of integral abutment bridges. The use of stiff concrete and pipe piles for the support of integral bridges is discouraged.

4.0 EARTH PRESSURES - Field Measurements

The maximum allowable lengths of integral bridges currently tend to be based on empirical data. The US Federal Highway Administration recommends the maximum span lengths indicated in Table 4.1 (FHWA, 1980 cited in Nielsen, 2001), all of which are greater than the recommended maximum length in the UK (60m) (BA 42/96).

Material	Max recommended span (m)
Steel	91.4
Reinforced concrete	152
Prestressed concrete	183

Table 4.1 FHWA Recommended Maximum Spans

A survey carried out in the United States in 1992 (Soltani & Kukreti, 1992) revealed that much of the progress in the use of integral bridges resulted from successive extension of limitations based on acceptable performance of prototype installations. One of the aims of this project was to gather information on the performance of existing bridges, primarily through data obtained from bridges instrumented and monitored over several years, to aid the advancement of integral bridge design.

The Highways Agency in the UK has commissioned a number of research projects on integral bridge performance to enable further refinement of BA 42/96, the results of several of these are presented in Sections 4.3, 4.5 and 4.6. The 1992 survey in the USA showed that 29 of the 38 States that responded were either using or had used integral-type abutments (Soltani & Kukreti, 1992). In order to update these data, 46 of the United States Departments of Transportation (DOT) were contacted individually during the course of this project, requesting relevant data on the design and performance of integral bridges. Responses to this request indicated that integral bridges were being used by 12 further states, which had either not responded to the previous survey or had not been using integral bridges at the time. Research has been carried out in several states (the results of which are presented in Sections 4.2, 4.4, 4.7 and 4.8) and four further investigations are in progress or scheduled for 2002/2003.

4.1 Field tests

This section covers the key findings of eight investigations on existing integral or semi-integral bridges. The bridge designs show considerable variation, which is part of the difficulty in producing generalised design guidance.

In the past, bridge engineers in the UK have tended to avoid using piled abutments to avoid the bridge becoming a hard spot when embankments settle (Hambly & Burland, 1979 cited in Hambly, 1992). In the USA however, shallow concrete bridge abutments are routinely supported by piles (Soltani & Kukreti, 1992), which are often orientated to bend about their weak axis. Table 4.2 below summarises the salient features of each bridge.

	Ref	Location	Bridge	Length (m)	Skew (deg)	air temp (^oC)
1	Elgaaly, 1992	Maine	Steel frame	50.3	20	$-33<T<23$
2	Darley, 1995	UK	Concrete	56.7	0	$-1<T<19$
3	Darley, 1995	UK	Concrete	47.5	0	$-1<T<19$
4	Hoppe, 1996	Virginia	Semi-integral	98	5	$-22<T<34$
5	Darley, 1998	UK	Shallow abut.	60	0	$4<T<23$*
6	Barker, 2000	UK	Full height abut.	50.2	0	$0<T<28$*
7	Lawver, 2000	Minnesota	Concrete	66	0	$-33<T<27$
8	Frosch, 2002	Indiana	Steel/Concrete	45.7	25	$-22<T<38$

Table 4.2 Summary of bridge features

* denotes range of deck temperatures

Each investigation had different specific objectives, primarily as a result of previous research. The aims and main conclusions of each research project are presented in the following.

4.2 Testing an Integral Steel Frame Bridge: Elgaaly et al., 1992; Skew Effects on Backfill Pressures at Integral Bridge Abutments: Sandford & Elgaaly, 1993.

The Forks Bridge in the state of Maine has integral abutments and approach slabs that are connected to the abutments. It consists of five rigid steel frames resting on shallow foundations, the steel legs are encased in concrete to form the abutments. Instruments were installed into the bridge during construction, which was completed in October 1989, the bridge was subsequently monitored for three years. The Maine DOT used a Rankine passive pressure in the design of the upper third of the wall, transitioning to an at-rest case at the base of the wall. Thus the effect of expansion of the deck, as found by Broms & Ingleson (1971), was incorporated into the design, but no effect of the skew was considered. The measured dry unit weights of the backfill averaged 94.9% of maximum, and the sub-base averaged 96.4%.

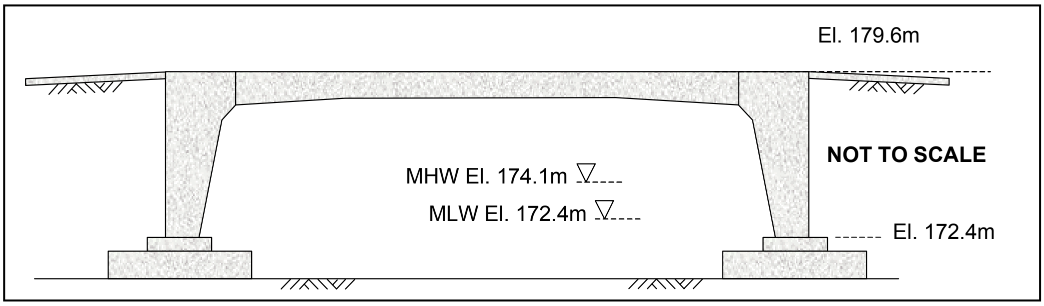

Figure 4.2a Sketch of Forks Bridge

4.2.1 Aim

The aim of this study was to determine if the skew of an integral bridge affects the backfill pressures and how these relate to deck movements arising from thermal changes.

4.2.2 Conclusions

- The pressure cells gave no indication of stiffening of the soil.
- The pressure envelope (similar to that of Broms & Ingleson (1971)), shown in Figure 4.2b., appears to be conservative.
- The effect of skew on the earth pressures is shown in Figure 4.2c. During the warmest months there are marked differences between the pressure on the obtuse side and the acute side. The effect appears to be dimishing with time, thus there is no indication that the effects of cyclic stiffening are increasing skew differences. For skewed abutments, a horizontal soil envelope that has the Rankine passive pressure at the obtuse end of the wall and the Rankine active pressure at the acute end should be considered. See Appendix A for clarification of passive pressure definition.

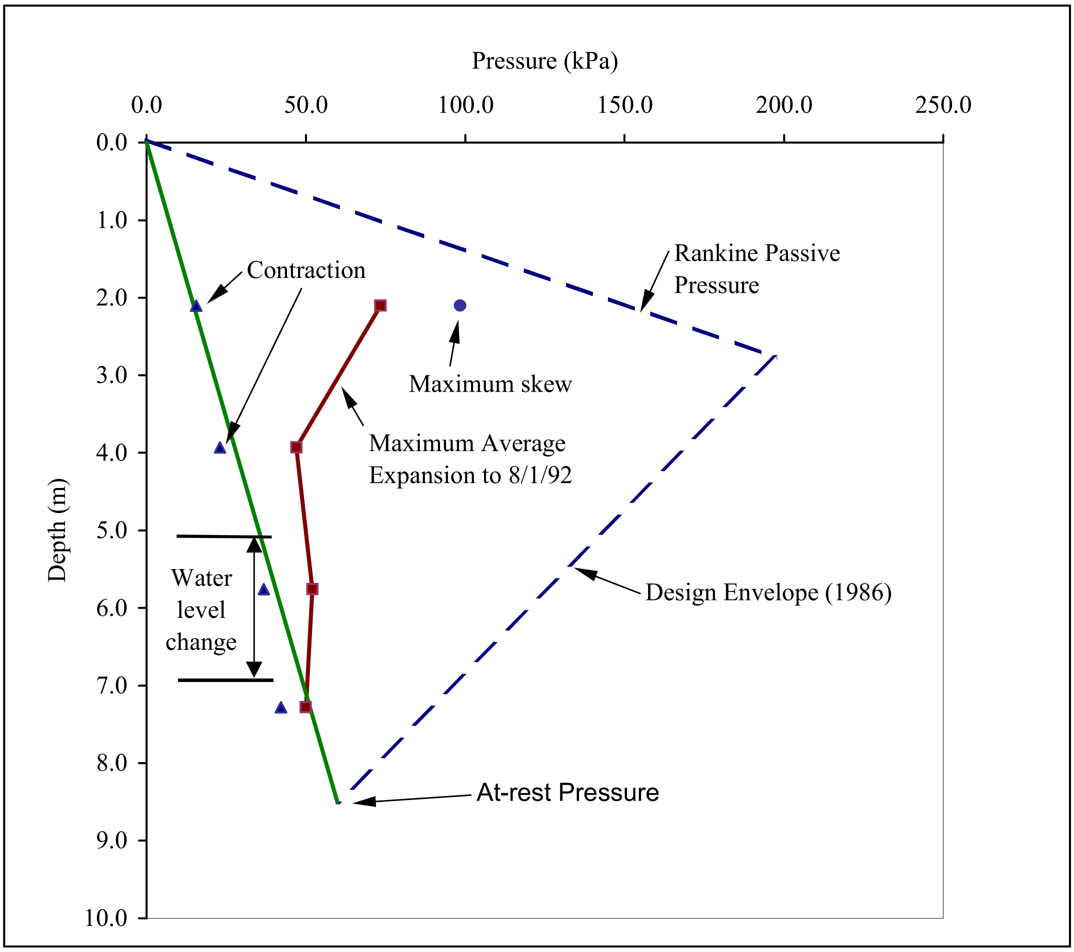

Figure 4.2b Measured earth pressures vs depth with design envelope (redrawn from Sandford &Elgaaly, 1993)

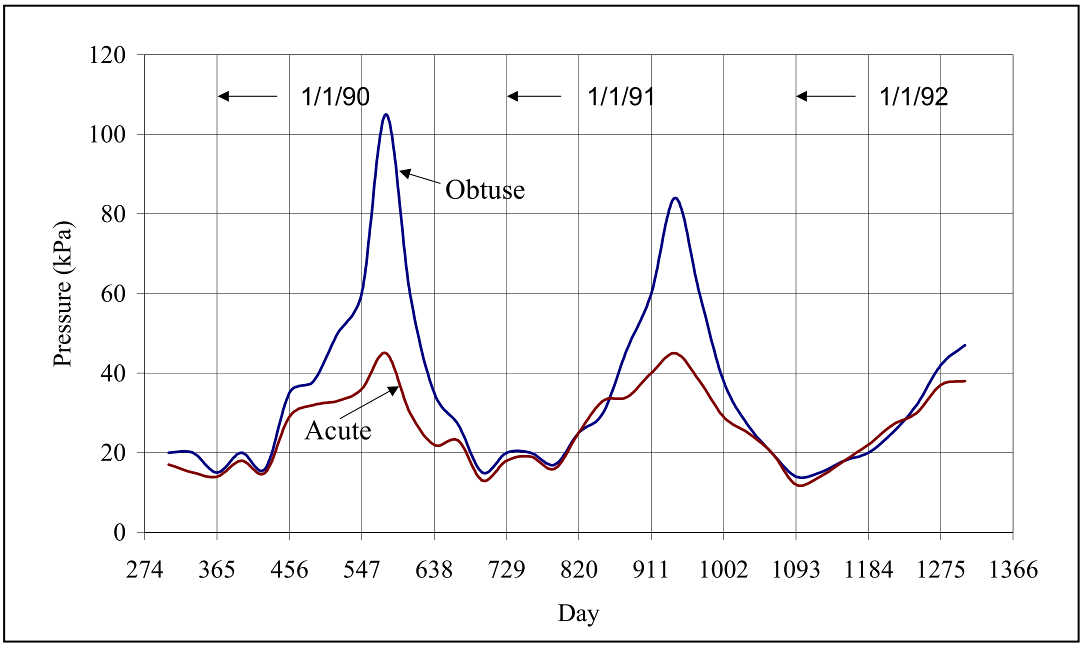

Figure 4.2c Skew effects on pressure, elevation 177.7m (based on Figure 9 from Sandford & Elgaaly, 1993)

4.3 Measurement of thermal cyclic movements on two portal frame bridges on the M1: Darley & Alderman, 1995

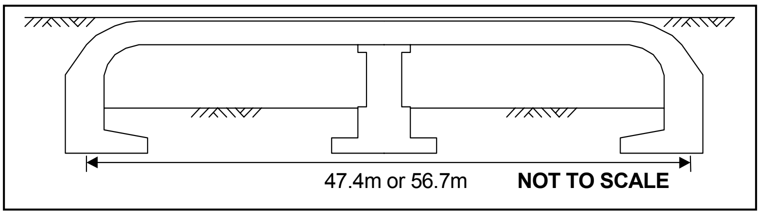

Figure 4.3a Sketch of portal structure

Both bridges were designed by Sir Owen Williams and Partners and built over the M1 in 1959. They are portal frame structures comprising of reinforced concrete abutments and solid reinforced concrete decks. Bridge 1 was instrumented in October 1993 and Bridge 2 during February 1994. Both bridges were monitored over a period of 14 months.

4.3.1 Aim

It was postulated that the thermal expansion of a bridge deck could be accommodated, to a certain extent, by cambering of the deck. The purpose of this report was to determine whether or not longitudinal thermal movements of the deck were actually being transmitted to the abutments, rather than causing the deck to hog.

4.3.2 Conclusions

- The report confirmed that vertical movements were generally small ($-3.5mm<\delta<2.5mm$) therefore hogging is not the primary response of the bridge deck to a temperature increase.

4.4 Field Study of an Integral Backwall Bridge: Hoppe & Gomez, 1996

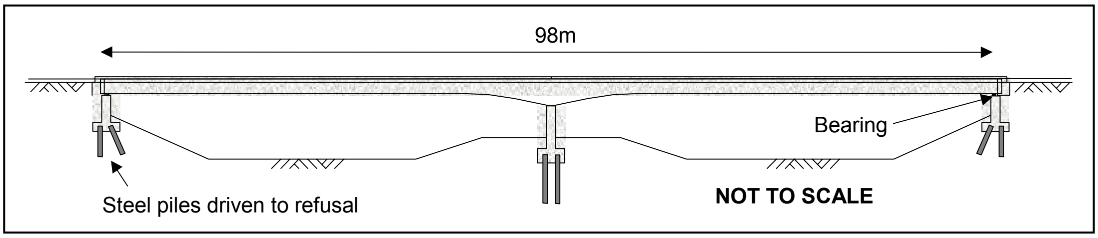

Figure 4.4a Sketch of integral backwall bridge

This bridge differs from the others presented in this report in that it is an integral backwall (or semi-integral) bridge; it has a bearing between the pile-cap and the abutment as illustrated in Figure 4.4b. In fully integral bridges, research with abutments supported by steel piles has shown that flexural piling stresses can sometimes exceed the yield strength (Burke, 1993 cited in Hoppe, 1996).

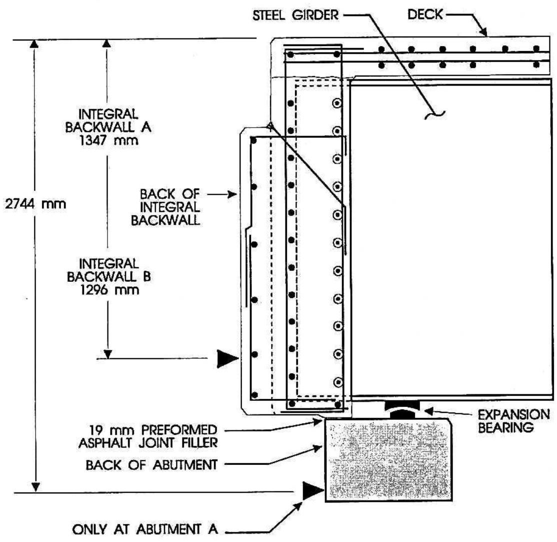

Figure 4.4b Semi-integral detail

This condition does not occur in semi-integral bridges since there is no moment transfer to the abutment (Hoppe, 1996). Field monitoring began during bridge construction in the summer of 1993 and continued until January 1996. A significant design exception of this bridge is that it was built without approach slabs, which is unusual in the USA (Hoppe, 1999).

4.4.1 Aim

The aim of the study was to investigate the long-term performance of an integral backwall bridge to determine the effects of secondary forces acting on the superstructure. This included the evaluation of soil pressures behind the integral backwall and the abutment and approach settlement.

4.4.2 Conclusions

- The bridge performed satisfactorily over 2.5 years, with no signs of structural distress.
- Fully passive earth pressures ($K_p = 6.2$) were recorded behind the integral backwall. See Appendix A for clarification of K_p. The earth pressure increased following the end of construction as a result of continuous backwall movement. This may be indicative of 'soil ratcheting'.
- Soil pressure behind the integral backwall varied considerably on a daily basis. A trend of the maximum weekly pressures at integral backwall B from January 1994 to January 1996 is shown in Figure 4.4c.

- No significant change in mean soil pressure acting behind abutment A was observed during the monitoring period.

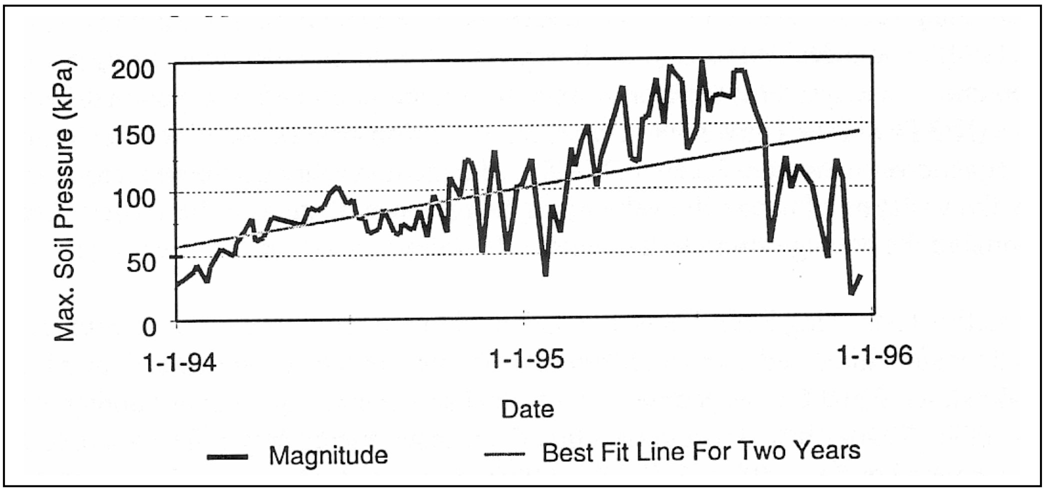

Figure 4.4c Maximum weekly soil pressure at integral backwall B

4.5 Seasonal thermal effects over three years on the shallow abutment of an integral bridge in Glasgow: Darley et al., 1998.

Monitoring of this bridge commenced in 1993, the results during construction and up to 1995 have been reported in TRL R178. This report describes the results of the monitoring from February 1995 to January 1998. The bridge was constructed with shallow integral abutments and three intermediate supports.

Well-graded granular fill with ϕ' of 41° was used immediately behind the abutment.

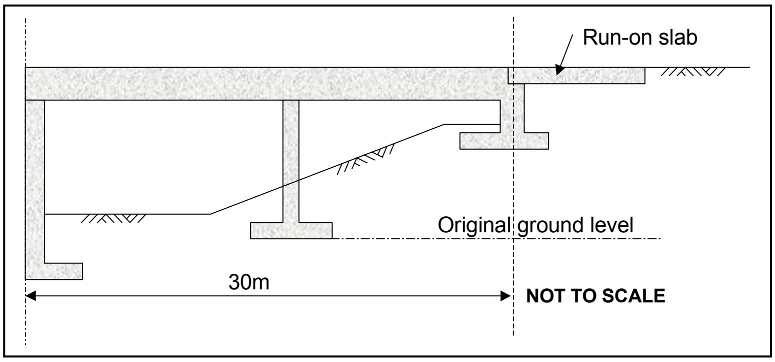

Figure 4.5a Sketch of shallow abutment bridge

4.5.1 Aim

The aim of this research was to provide more design advice on the earth pressures acting behind integral bridge abutments, following on from the work of Springman et al. (1996), Card & Carder (1993) and England & Dunstan (1994).

4.5.2 Conclusions

- The concrete abutment moved by a combination of tilting and translation.
- Bridge temperatures of up to 23°C gave rise to significantly increased lateral stresses, all of which were higher than a K value of 1 and, in the case of the top

and bottom cells, exceeded a K value of 2. A maximum effective bridge temperature of 33°C can be expected in Glasgow (BD37, DMRB 1.3), which would result in even higher K values. The K_p values corresponding to this backfill are 4.8 and 10 assuming an unfactored ϕ' value and wall friction angle, δ, of zero and $\phi'/2$ respectively. See Appendix A for clarification of passive earth pressure coefficients. This might suggest that the recommendation of designing for fully passive pressure is too conservative. The authors recommend that further backfill stress measurements are required for a bridge which has been in-service for more than a decade, or better quantification of wall friction, is needed to enable further refinement of BA 42/96.

- Lateral stress measurements on two dates when cell and deck temperatures were effectively the same indicated an increase on lateral stress between June 1996 and May 1997. This provides some indication that the density and stiffness of the backfill may have increased over the yearly cycle of expansion and contraction.

4.6 Performance of an integral Bridge over the M1-A1 Link Road at Bramham Crossroads: Barker & Carder, 2001

This bridge, which was constructed following the advice of BA42/96, is approximately 50m in length and comprises of prestressed concrete beams composite with a 200mm thick reinforced concrete slab. The deck beams are structurally connected to full height reinforced concrete abutment, which are founded in Magnesian Limestone. This report describes the findings from measurements during construction and over the first three years in service.

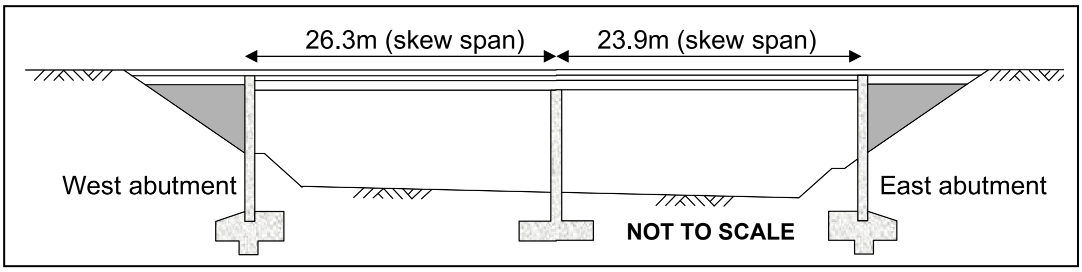

Figure 4.6a Sketch of Bramham Crossroads North Bridge

4.6.1 Aim

The aim of this project was to assess the bridge's seasonal performance at full scale. The measurements taken included the movement of the abutments and the lateral earth pressures acting on the abutments.

4.6.2 Conclusions

- Measurements indicated a significant shortening of the deck associated with creep and shrinkage of the prestressed beams, although the overall deformation rate was marginally less, at 87% of that estimated from BS5400.
- During the first three years creep and some shrinkage induced abutment movement away from the backfill, which limited the development of lateral earth pressures.
- 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.

4.7 Field Performance of Integral Abutment Bridge: Lawver et al., 2000

This investigation started in 1996 during construction, in which over 180 instruments were installed in and around the bridge. The monitoring is ongoing, but the first set of results has been published in the Journal of the Transportation Research Board.

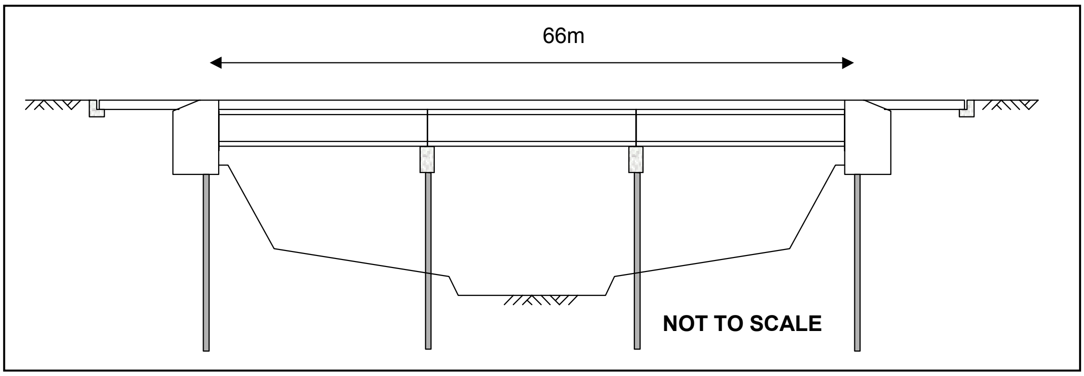

Figure 4.7a Sketch of bridge near Rochester, Minnesota

4.7.1 Aim

The objective of this research carried out near Rochester, Minnesota is to better understand the behaviour of integral abutment bridges. This bridge is fully integral, with shallow abutments (1.5m) supported by a single row of six P12 x 53 piles, approximately 24m in length, oriented in weak-axis bending.

4.7.2 Conclusions

- The primary movement of the abutment was horizontal translation; there was very little rotation ($<0.06^o$) of the abutment.

- A net inward movement of the abutments over time was observed. The overall shortening during the first year was expected due to shrinkage, but this is unlikely to account for the continued overall shortening. A possible cause proposed by Lawver *et al*. (2000) was soil collection and compaction behind the abutments during the winter.
- Changes in pressure during the summer were found to be less than 172 kPa. Average changes in pressure were 90 and 94 kPa for the north and south abutments, respectively.

4.8 Integral Bridge in West Lafayette, Indiana. Frosch, 2002

This 150ft (45.7m) long, 25° skew, steel girder composite deck bridge in West Lafayette, Indiana, was instrumented in the summer of 2000. Data acquisition began in September 2000. The project is ongoing and no results have yet been published. Preliminary results, courtesy of Dr Robert Frosch and David Fedroff of Purdue University, Indiana are presented below.

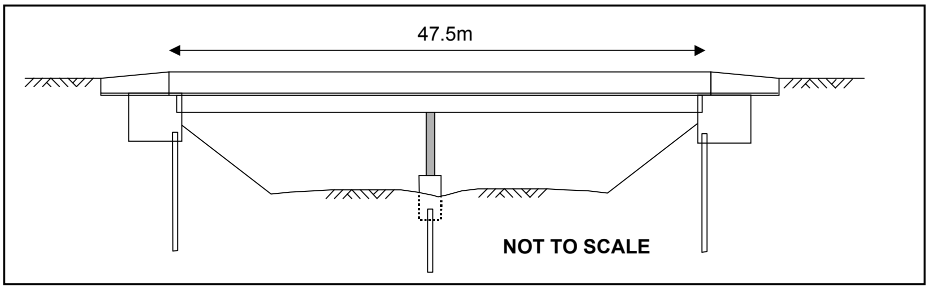

Figure 4.8a Sketch of bridge in West Lafayette, Indiana

4.8.1 Aim

The objective of the investigation is to determine if the current length (300ft for pre-stressed concrete, 250 feet for steel) and skew (30°) limitations imposed by the Indiana Department of Transportation are necessary and relevant. [Fedroff, 2002].

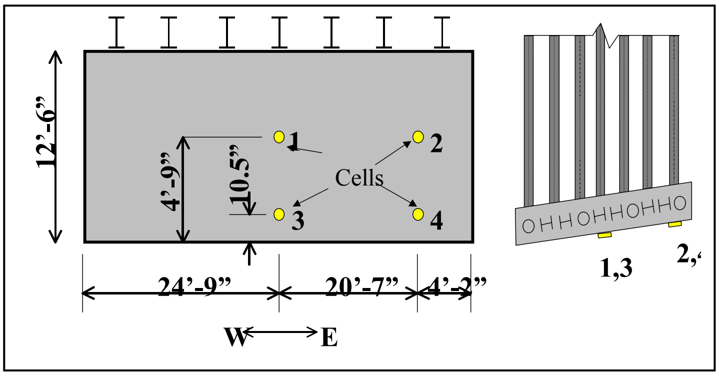

Figure 4.8b Earth pressure cell locations

Four pressure cells were installed behind the south abutment as shown in Figure 4.8b to determine the effect of thermal expansion and contraction on the earth pressures.

4.8.2 Conclusions

- N.B. Results of this research have not yet been published, the following are the author's conclusions drawn from preliminary results.
- The overall earth pressures behind Cell 1 appear to be increasing (see Figure 4.8c), which may be an indication of stress escalation with time due to the densification of the retained soil as observed by England & Dunstan (1994). This effect is not apparent in Cells 3 and 4.

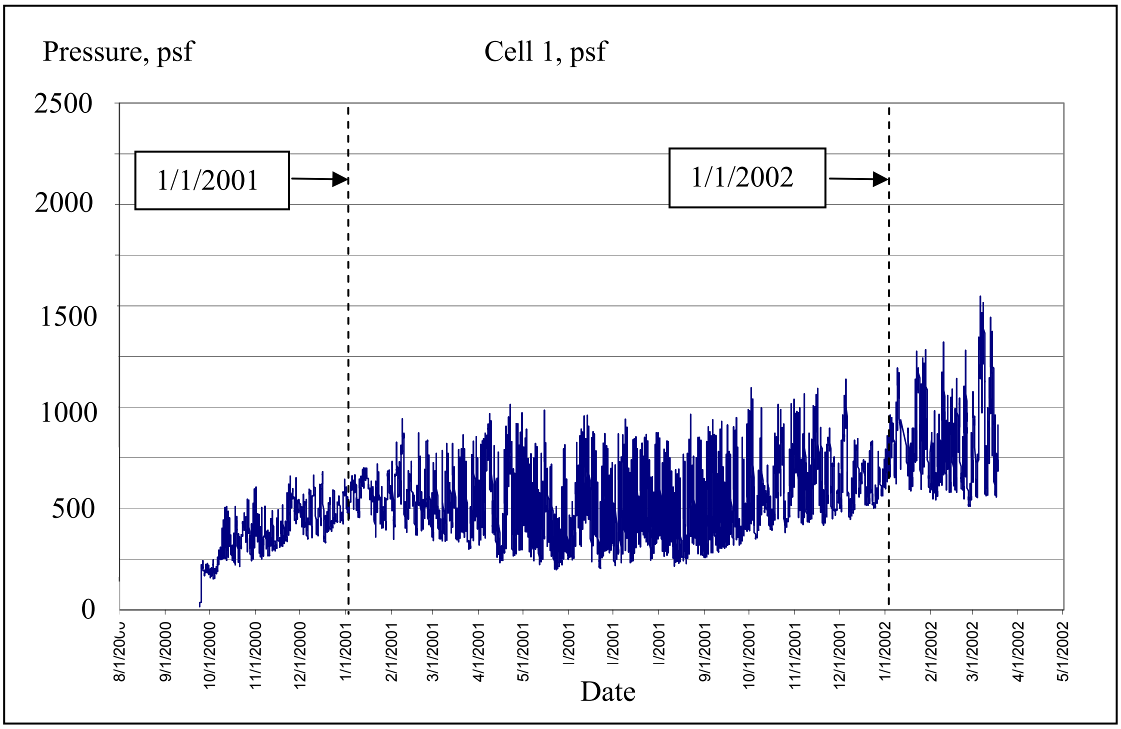

Figure 4.8c Variation of earth pressure with time, Cell 1.

- The maximum pressures recorded in Cells 1, 2, 3 and 4 were approximately 72, 108, 62 and 36kPa respectively. Taking Cell 2, at a depth of 2.36m and estimating the uncompacted fill as having a bulk unit density of 18kN/m^3 and ϕ' of 35°, the maximum earth pressure coefficient is approximately 2.5. It would appear, therefore, that fully passive earth pressures have not yet been mobilised.

4.9 Coefficients of Thermal Expansion

The magnitude of the lateral earth pressures behind an abutment will depend partly on the degree of longitudinal expansion of the deck, which is highly dependent on its thermal coefficient of expansion. In BA42/96 a value of 12 x 10^{-6}/°C has been assumed for concrete. It is noted, however, that lightweight aggregate concrete, and other materials, can have markedly lower coefficients of thermal expansion and these values may be used in such instances.

The measured deck coefficients of thermal expansion for 4 of the 8 bridges are presented below with data from two earlier experiments.

Ref	Location	Deck	α /°C
Girton, 1989	Iowa (Boone)	Concrete- limestone aggregate	8.1×10^{-6}
Girton, 1989	Iowa (Maple)	Concrete - gravel aggregate	9.0×10^{-6}
Darley, 1995	UK	Concrete on rock	11.3×10^{-6}
Darley, 1995	UK	Concrete on stiff clay	13.7×10^{-6}
Darley, 1998	UK	Shallow abut.	9.0×10^{-6}
Barker, 2000	UK	Full height abut. limestone agg.	7.0×10^{-6}

Table 4.9 Summary of coefficients of thermal expansion

4.10 Influence of deck compression

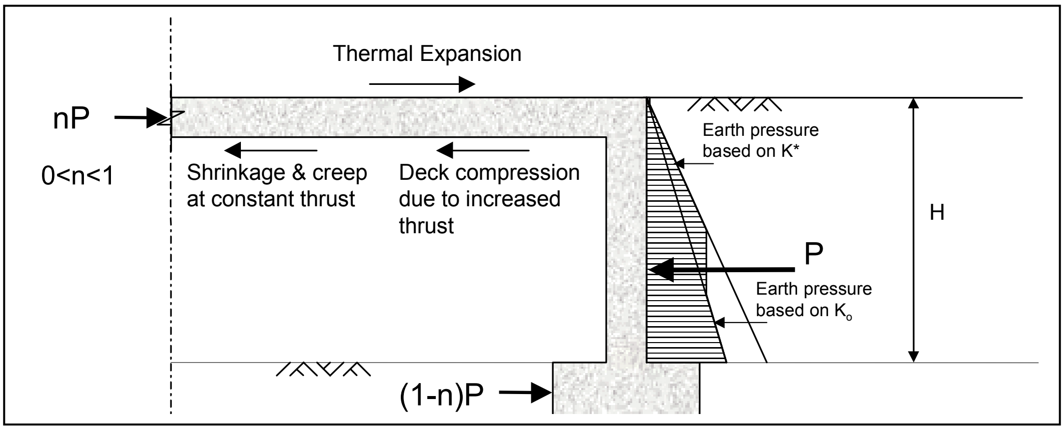

Figure 4.10a Influence of deck compression

4.10.1 Shrinkage and creep

Creep and shrinkage deformation of prestressed concrete beams has a significant effect over the first few years in service (Barker & Carder, 2001) and will build in an additional safety factor against the development of passive lateral earth pressures. It may, however, contribute to increased settlement and an unacceptable moment in the wall (Springman *et al.* 1996).

4.10.2 Deck Compression

A degree of deck compression will occur due to the axial deck load, which must be in equilibrium with the lateral earth pressures behind the abutment. A finite element analysis was carried out by Low (1994) to model the interaction between the deck forces, abutment movements and stresses and strains in the soil, with and without a road pavement behind the abutment.

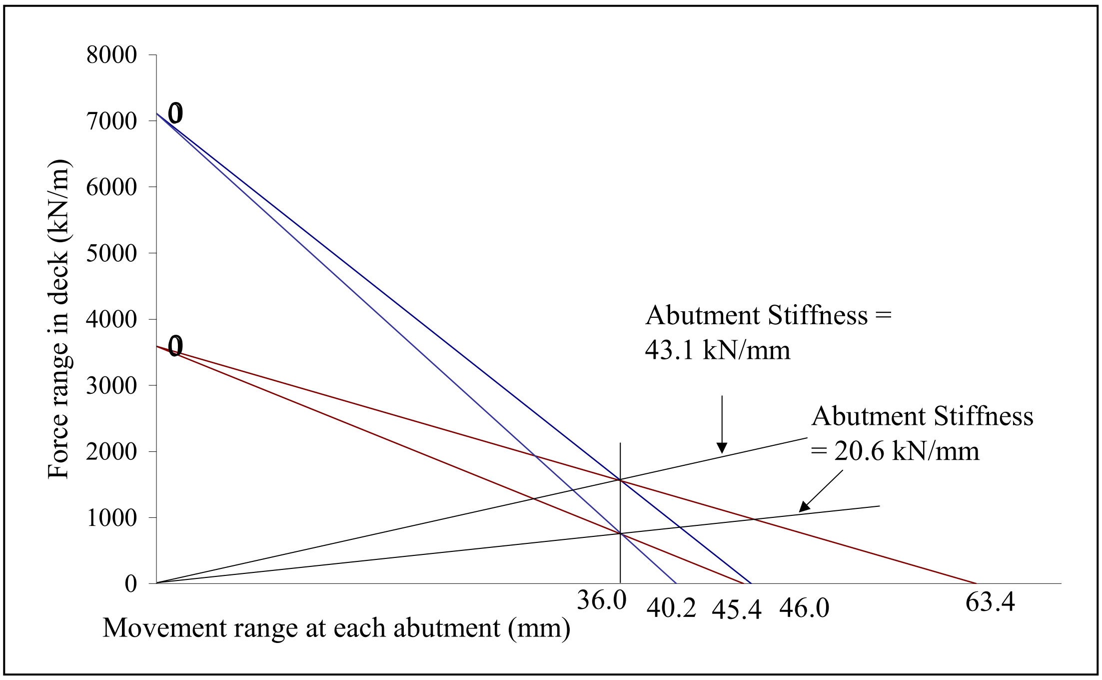

Figure 4.10b Force/displacement interaction diagram from FE analysis reproduced from Low (1994)

The additional force due to the highway pavement doubles the stiffness (20.6 kN/mm to 43.1 kN/mm, see Figure 4.10b) of the abutment and thus increases the proportion of temperature movement that can be absorbed in deck strains (Low, 1994). A concrete bridge deck might have a mean concrete cross-sectional area of about 0.45 m^2/m and be subjected to a temperature range of 47°C. Hence, with the pavement, the maximum deck length, which could operate within the 36mm movement range, would be 163m. Without the pavement it would be 142m (Low, 1994).

5.0 SETTLEMENT - Experimental Results

Settlement at bridge approaches can seldom be traced to a singe cause and a finite amount of differential settlement is inevitable at virtually all bridge approaches (Hoppe, 1999). This section compares the experimental results of *TRL 146* and *Integral Bridges;* field measurements are discussed in Section 6.

5.1 BA 42/96 - The Design of Integral Bridges

BA 42/96 provides no information on the determination of soil deformation or any recommendations on the use of approach slabs.

5.2 TRL Report 146: Cyclic loading of sand behind integral bridge abutments (Springman et al. *1996)*

Soil movement was observed by photographing markers embedded in the backfill material and analysing their relative movement using spot chasing methods. Photographs were taken before, during and after each set of perturbations, then again before the model was removed from the centrifuge arm. These were used to determine the vector displacements and principal strains. The settlement profiles shown in Figure 5.2a have been derived from the displacement vectors for the spread base wall.

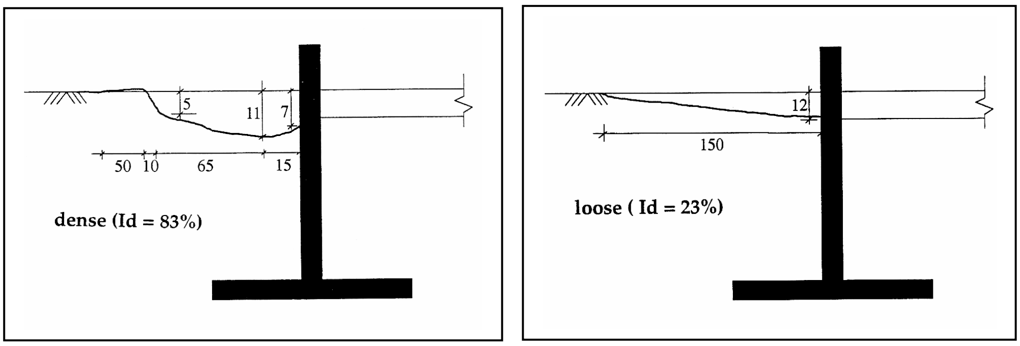

Figure 5.2a Settlement profiles of spread-base walls (mm)

It is not clear from the diagrams, but the dimensions shown correspond to the actual settlement of the model and therefore need to be scaled up by a factor of 60. The maximum settlement of the loose fill, at prototype scale, was ~0.7m (as confirmed in the text) after 100 ± 60mm rotations and the settlement trough extended 9m away from the wall. Similarly the maximum settlement observed in the dense fill was ~0.66m, about 0.9m away from the wall with the settlement trough extending 5.4m away from the wall. These values are misleading however, principally because they are the very final settlement after the successive load cases, including 100 cycles of the 1:120 year case. It is of no use to the engineer to be able to estimate the settlement profile of their backfill after 12000 years. A more reasonable interpretation would have been to take the displacement vectors of the top row of markers after 100 cycles at ultimate state and make an estimate of the soil profile from these. The spot chasing data for the spread-base abutment are only given for the final profile, so this is not possible; intermediate data are given, however, for the embedded wall. These diagrams, shown in Figure 5.2b, demonstrate that apart from the loose fill (35%I_D), considerably smaller settlements are observed.

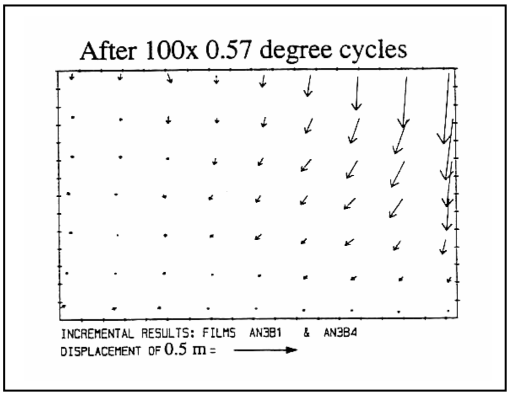

Figure 5.2bi) Smooth, flexible, 35% I_D

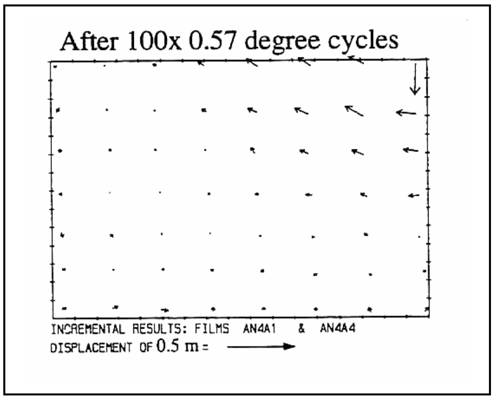

Figure 5.2bii) Smooth, flexible, 95% I_D

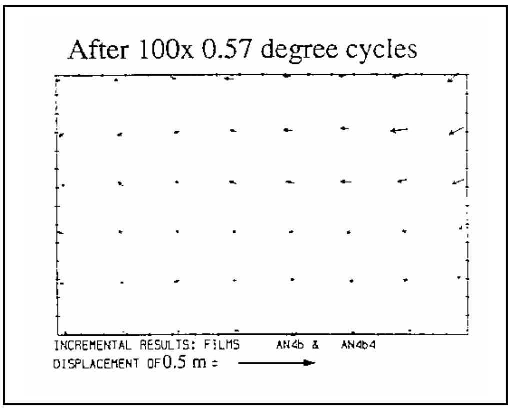

Figure 5.2biii) Rough, flex., 80% I_D

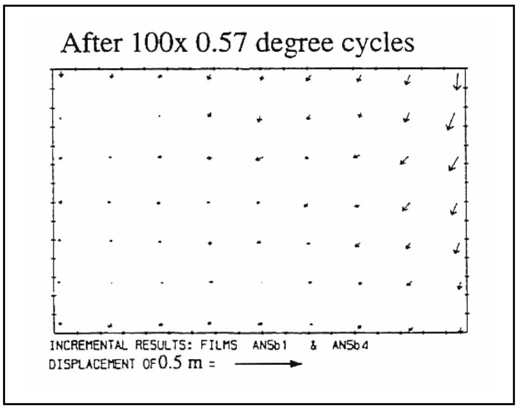

Figure 5.2biv) Smooth, stiff, 80% I_D

It is notable that the densest (95% I_D) deposit exhibits heave.

The settlements of the embedded walls are presented in the TRL report in terms of the length of the settlement zone behind the embedded wall (Figure 5.2c) and the area of ground loss behind the wall (Figure 5.2d). Figure 5.2c shows that the amount to which the settlement zone extends tends not to increase after 100 cycles at the serviceability state. This would suggest that settlement problems are likely to appear in the earlier stages of the life of the bridge, which is consistent with field observations. This behaviour is not, however, demonstrated by the loose fill and would support a requirement that backfill be compacted to a relative density of at least I_D 80%.

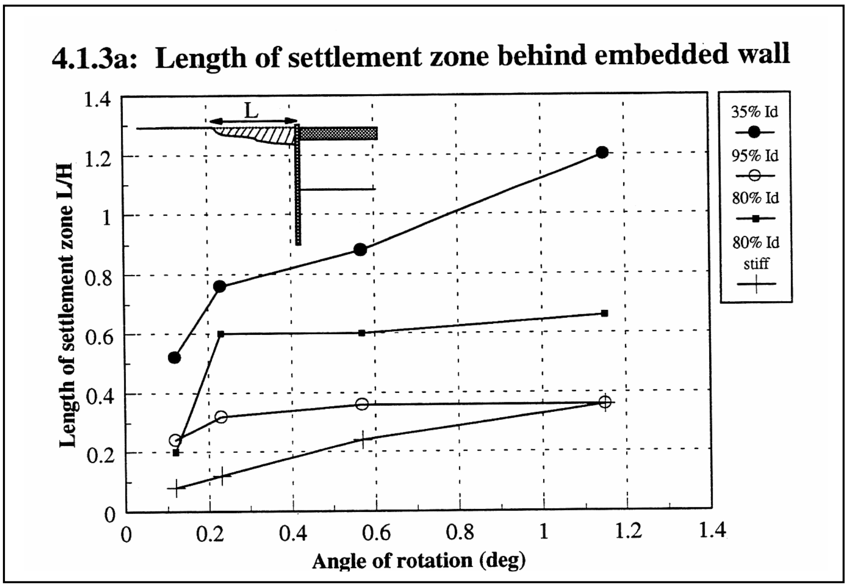

Figure 5.2c   Length of settlement zone behind embedded wall

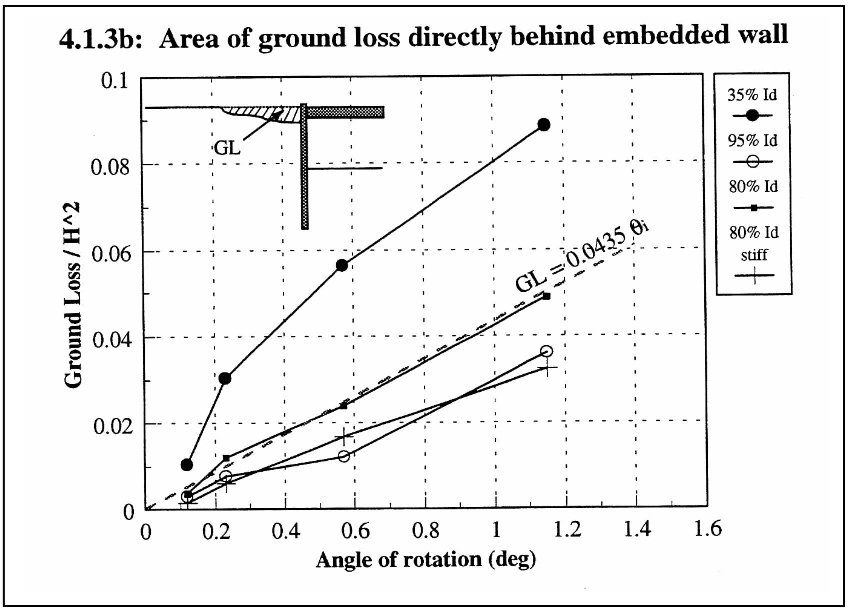

Figure 5.2d   Area of ground loss directly behind embedded wall.

Figure 5.2d gives an estimate of the ground loss. It is unclear how this diagram has been reached since the displacement vectors after 100x1.15° cycles (1:120 year state) clearly shows net *heave* rather than settlement. This is also true, to a lesser extent, for the 100 cycles at ultimate state as shown in Figure 5.2bii). Again, the results for I_D 35% suggest that loose fill should not be used behind an abutment wall.

In the absence of the settlement profile after a single 1:120 year cycle it again seems reasonable to take the profile after 100 cycles at ultimate state as a worst case. This done, the greatest anticipated volume loss appears to be for the rough flexible wall at

80% I_D fill; the vector displacements however, indicate very little settlement. A greater loss would be expected from the stiff wall at 80% I_D, judging by the vectors. The conclusion that loosely placed fill should not be used is an important one, but no reliance should be placed on the given equation for calculating volume loss.

5.3 Integral Bridges: A fundamental approach to the time-temperature loading problem (England et al., 2000)

Whilst the lateral earth pressures measured by England *et al*. (2000) appeared to be approaching a limiting value with an increasing number of cycles, this was not true of the settlement. "Observations confirmed the existence of a flow mechanism, in addition to soil densification." See Figure 5.3a

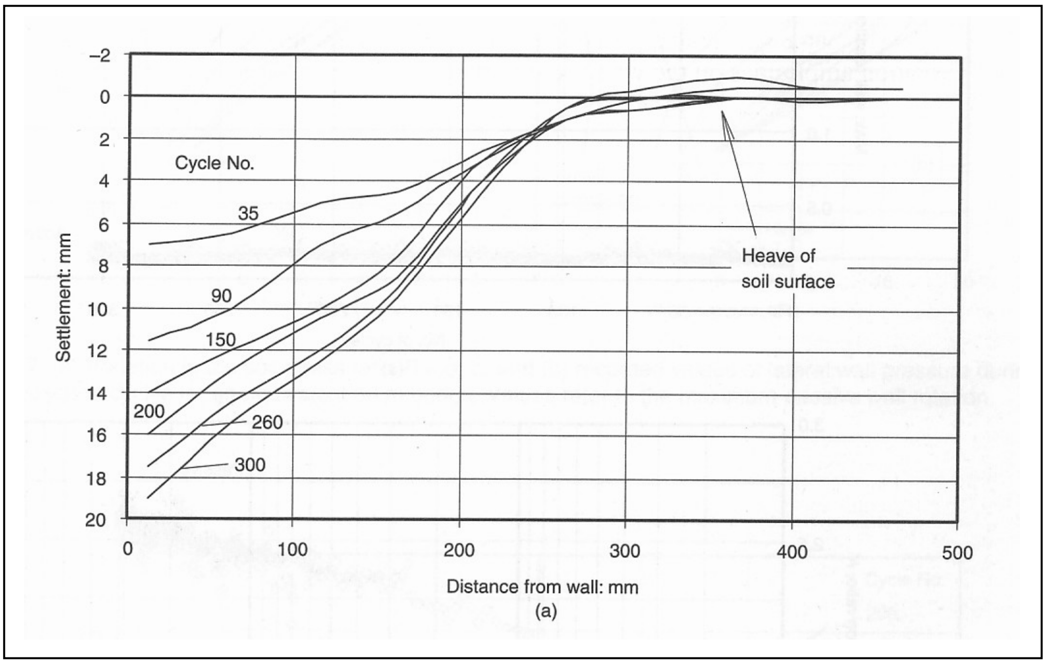

Figure 5.3a Soil settlement for an equivalent 60m bridge

The report also recognises that the soil heaves at a distance away from the wall, a characteristic evident in the centrifuge model results at 95% I_D.

The development of a shear slip band allows rapid downward movement of a soil wedge adjacent to the wall during each active wall movement.

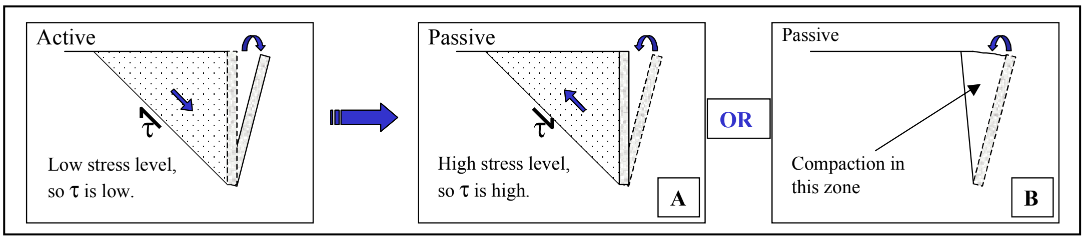

Figure 5.3b Illustration of soil deformation mechanisms

This slip happens as a result of gravity. During the subsequent passive wall movement an upward slip would require work to be done against gravity (Mechanism A in

Figure 5.3b). Alternatively, the soil close to the wall can compact (Mechanism B in Figure 5.3b), resulting in greater soil settlement close to the wall and greater heave away from the wall. This Mechanism requires less energy than Mechanism A. The development of this shear slip band is apparent in Figure 5.3b.
This failure mechanism would only be expected in dense soils, as exclusively used by England *et al*. (2000) and exhibited by the densest samples used by Springman *et al*. (1996). The observed heave is therefore consistent with the postulated flow mechanism; there is less heave for soils of an initially lower density and more for stiffer backfill.

England *et al*. (2000) conclude that settlement is greater for larger effective bridge temperature fluctuations and is increased further by the daily temperature fluctuations, but that it is not particularly sensitive to the initial soil density.

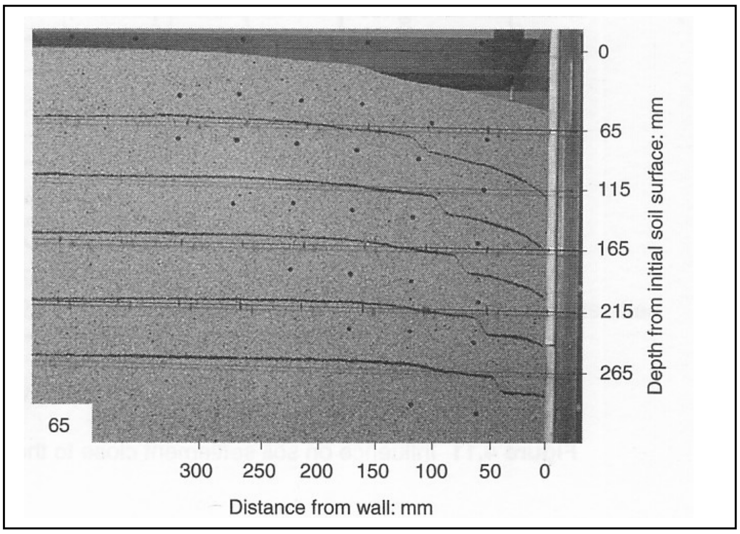

Figure 5.3b Deformation of Leighton Buzzard backfill after 65 seasonal cycles

This contradicts the results of Springman *et al*. (1996), which clearly demonstrate that loose fill results in considerably larger settlements. A refined conclusion would be that settlement is not particularly sensitive to initial density *above a certain limit*. The UCL experiments were carried out at either 90% or 95% I_D, a comparatively small range compared to the 23-97% range used by Springman *et al*. (1996). This smaller range is, however, closer to that used in the field and could support the argument that there is no economic advantage (in terms of mitigating settlement) in employing more costly compaction methods to achieve say, 95% rather than 90% relative density. There may be cases where it is not physically possible to achieve this level of densification with the given materials on site and it would be advantageous to know at the outset that the contractor does not need to be pushed towards achieving this. England *et al.* (2000) conclude that "no recommendation can be made for the evaluation of settlement to the backfill material because there is no evidence to support the existence of a limiting settlement during the normal life-span of an integral bridge, and the interaction between the seasonal and daily cycles is not yet full understood." The authors call for further experimental work to validate the theoretical approach used to assess abutment settlements.

6.0 SETTLEMENT - Field Measurements

Experimental work by Springman *et al*. (1996) suggests that considerable settlement (>500mm for a 6m high spread-base wall) can be anticipated behind integral abutment walls. They therefore recommended a run-on slab to span the depression behind the abutment. England *et al*. (2000) also observed significant settlement (150mm for tests with overall wall rotation d/H of 0.25% for a 60m bridge with 7m of backfill in 120 years), but concluded that further research was necessary to make recommendations on the evaluation of settlement.

Circumstantial evidence, collected from correspondence with practising engineers in the UK, however, suggested that settlements of this magnitude were simply not occurring in the field. In an attempt to solve this paradox, a visit was made to the Highways Agency to view the maintenance records of existing integral bridges in the UK (see Section 6.1). Three of the field studies presented in Section 4 also included monitoring of settlement, both with and without run-on (approach) slabs; the results of these can be found in Section 6.2. The issue of whether or not to use approach slabs is worthy of a research project in itself, but arguments both for and against their use are presented in Section 6.3.

6.1 Highways Agency Maintenance Data

6.1.1 Structures and Technical Approval (SATA) research

In the summer of 2001, some members of the Technical Approval team at the Highways Agency carried out a fact-finding mission on integral bridges, the emphasis of which was placed on the investigation of run-on slabs, but also included sliding bankseats and bridges with piles in sleeves at the abutments. [Brookes, 2002]

- The first structures to be visited were integral bridges (50-60m), without run-on slabs, on the M60 near Manchester, which have been in service for a couple of years. Very minor settlements were observed in the majority of cases, but this was deemed inconclusive with respect to whether or not run-on slabs should have been provided. One bridge approach, which had lightweight backfill on a concrete raft founded on peat, had settled up to 30mm. This again, however, was thought to be insufficient data from which to draw conclusions either on the use of run-on slabs or lightweight fill.

- Two bridges, with top pinned H-piles in concrete sleeves, opened in March 1999 were also visited: Rothwell Haigh (68m) and Pontefract West (81m). The former appeared fine, but some settlement of the surrounding approach embankment (which was on top of 40-50m of uncontrolled compaction landfill) of ~20mm was observed in the latter. Neither of the bridges had approach slabs.
- A bridge at Junction 15 of the M1, opened in Summer 2000 with concrete bored piles in concrete sleeves, a run-on slab and reinforced earth (see Figure 5.1a) showed no evidence of settlement problems.

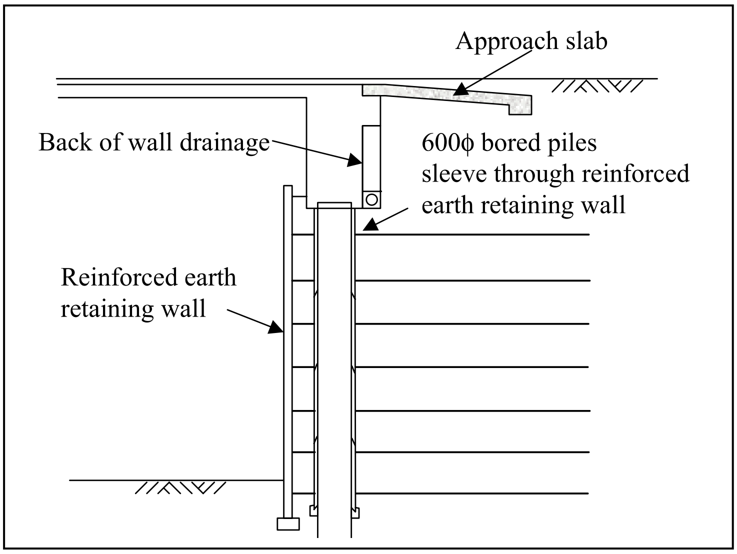

Figure 5.1a Detail of abutment at Junction 15, M1

6.1.2 Sir Owen Williams Bridges

The first section of the M1 motorway, constructed over a 19-month period in 1958-9, included 127 bridges, 88 of which were of a continuous portal type that act integrally with the surrounding soil. The specification for compaction was the Ministry of Transport standard, requiring materials to be compacted at their natural moisture content with air voids not exceeding 10% (Williams, 1960). This was comfortably achieved in the cohesive soils, but proved impracticable in certain sandy and gravelly materials and compaction in these case was based on Proctor density tests. It was later noted by Sir Owen Williams (Williams, 1960) that, "as is frequently the case, differential settlement at the back of bridge abutments has proved a problem, despite particular care being taken with the compaction in these areas".

There has, however, been no specific monitoring of settlement. Apart from specific subsidence areas where old shallow mine workings had been discovered, movement of the portal structures has not generally caused large settlements. Exceptions to this are approaches to a few farm accommodation overbridges, which suffered considerable settlement, to the order of 300mm. It is possible that evidence of settlement has been masked by resurfacing and maintenance works, but from the records this would appear not to be the case. Maintenance records up to 1983 are minimalist; components of the bridges are simply rated as 'good', 'fair' or 'poor'. These reports may also have been optimistic since a 2-span portal bridge (Charity Farm, M1/70.90), all components of which were rated as 'good' in 1983, was later reported as 'requiring extensive repair work' to the failing approach embankments in 1993. Within a further 5 years the approach slab had failed. A similar record is found for a 4-span continuous farm overbridge (Lodge Farm, M1 69.10).

The Highways Agency holds maintenance data for each bridge along the M1; a record of each type of integral bridge was studied, none of which showed settlement problems. The above examples are isolated cases and the majority of bridges (based on the viewed records and discussions with engineers) are performing well.

6.2 Field Studies

Of the eight field studies discussed in Section 4, three made reference to settlement behind the abutment. In two cases quantitative results are given; the third is simply qualitative.

	Reference	Location	Bridge	Approach slab?	Max settlement (mm)
1	Hoppe, 1996	Virginia	Semi-integral	N	140
2	Darley, 1998	UK	Shallow abut.	Y	7
3	Lawver, 2000	Minnesota	Concrete	Y	"expansive void"

Table 6.1 Summary of field studies on settlement

6.2.1 Field Study of an Integral Backwall Bridge: Hoppe & Gomez, 1996

Damage to the approach pavement due to excessive settlement of the fill was the biggest maintenance problem observed with this bridge (Hoppe, 1996).

A notable feature of this design is the lack of approach slabs (see Section 4.4 for a sketch of the bridge), which is uncommon in the USA where they are used by the majority of state Departments of Transportation (Hoppe, 1999). The approach pavement was completed in October 1993, approximately 2 months after construction and the first major resurfacing was done in May 1994. An example of the Abutment B approach elevations (1994) is shown in Figure 6.2a. In the subsequent 2 years the cost of approach pavement repairs amounted to approximately $10,000. The purpose of omitting the approach slab was that "any remedial action necessary to rectify potential approach slab settlement would be significantly more expensive and inconvenient to the traveling public than re-grading a settling approach pavement" (Hoppe, 1996)

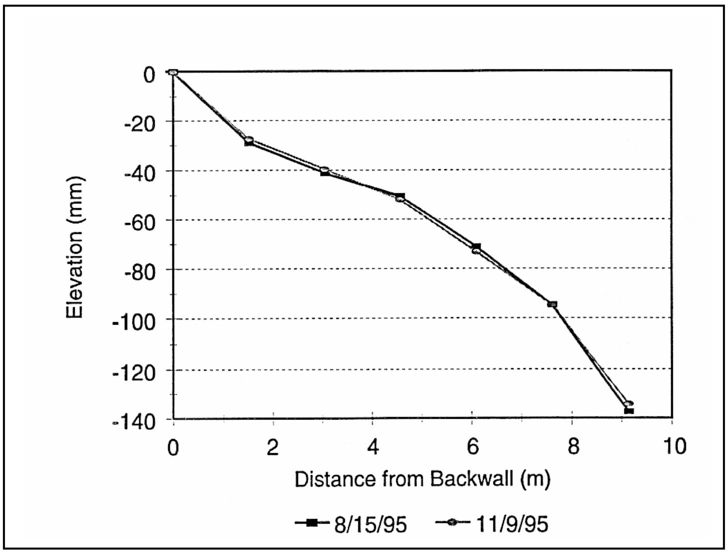

Figure 6.2a Abutment B approach elevations

This considered, the conclusion remained that is was not obvious whether the presence of concrete approach slabs would have minimised, or prevented, asphalt pavement repairs. This is particularly true of Abutment B, where the settlement extended beyond the length of a typical approach slab.

The report concluded that "additional research should be directed at backfill materials and embankment construction techniques associated with integral bridges".

6.2.2 Seasonal thermal effects over three years on the shallow abutment of an integral bridge in Glasgow: Darley *et al.*, 1998

The purpose of this field study was to measure lateral earth pressures and the behaviour of the bridge itself, rather than quantify settlement behind the abutments.

The report concludes, however, that the overall settlement of the fill beneath the run-on slab was 3mm from the start of construction (1993) until February 1995, followed by a further settlement of 4mm up to January 1998. This relatively small settlement may be due to the specification of the abutment fill, which is described as "well graded granular fill" or as a result of the abutment design.

6.2.3 Field Performance of Integral Abutment Bridge: Lawver *et al.*, 2000

Whilst the evaluation of backfill settlement was not one of the objectives of this study, a loss of backfill material was observed within seven months of construction completion. "An expansive void at the base of the abutment behind the riprap was visible". The report concluded with the recommendation that more attention be paid to the backfill plan, including drainage details and the potential use of geotextiles to stabilise the backfill. A workshop was held earlier this year by the Minnesota DOT on compaction testing issues of subgrade and base materials (Siekmeier, 2002). This highlights a difference between design methods in the USA and the UK, where such issues are already being addressed.

6.3 Approach Slabs

The presence of an approach slab has no effect on the magnitude of the differential settlement that will ultimately develop (Hoppe, 1999). The primary function of approach slabs is to provide a gradual transition between the fixed superstructure and the settling embankment. They therefore address the symptom, but not the cause of settlement. Settlement (and sometimes failure) of the approach slabs themselves remains a significant maintenance problem. This section firstly outlines the extent to which approach/transition/run-on slabs are currently used, then presents the arguments for and against their use.

6.3.1 Extent of current use of approach slabs

Design drawings held at the Owen Williams office show that some, but not all, of the Sir Owen Williams bridges along the M1 incorporated approach slabs. Examples of bridges with approach slabs are discussed in Section 6.1, in which the slabs failed as a result of excessive settlement. A number of bridges have also been designed in the UK, which eliminate the use of approach slabs altogether, for example the bridges along the M60 (Brooke, 2002) and all of the newly constructed Channel Tunnel Rail Link (CTRL) bridges (Lamont, 2002).

A survey carried out in the USA in 1999, to which thirty-nine state Departments of Transportation (DOT) responded, showed that 60% of respondents always use approach slabs on integral bridges. The criteria employed by a DOT to determine whether to consider using an approach slab depend primarily on traffic volume, although earlier research by Mahmood (1990) (cited in Hoppe, 1996) indicated that traffic volume has no influence on the magnitude of approach settlement. 71% of respondents using integral structures also reported using mechanical connections at the approach slab/backwall interface, to prevent a gap opening. In the majority of states the approach fill is constructed in 0.2m loose lifts of granular fill, compacted to 95% of the Standard Proctor value. Four states enforce strict 100% Standard Proctor compaction requirements. In Germany, stringent embankment material requirements and compaction control methods (100% Proctor) are also specified and approach slabs are seldom used. [Hoppe, 1999]

6.3.2 Arguments for and against the use of approach slabs

This section draws information from the survey carried out by Hoppe, 1999 and from personal communications with bridge engineers.

The respondents to the survey quoted smooth ride as a primary advantage for the use of approach slabs (81%), followed by reduced impact (41%) on the backwall and enhanced drainage control (16%). The primary disadvantage was initial high construction cost (75%) and, ironically, maintenance problems with settling approach slabs (55%). Two states, Maryland and Kentucky, derive no clearly defined advantages to approach slabs claiming that they only serve to move the bump from the end of the bridge to the end of the approach slab. By moving the expansion joint off the bridge and onto the approach, however, it is argued that even if no improvement is seen in expansion joint performance, tremendous cost savings can be made in terms of structural repair (Beck, 2002). It has also been found that approach slabs can settle considerably. In cases of substantial settlement, the cost of repairing a failing approach slab may be significantly greater than the cost of placing recurrent overlays.

7.0 CONCLUSIONS

7.1 Superstructure

7.1.1 Length

The bridges discussed in this report range up to 98m (Hoppe, 1996) and a survey carried out in the USA (Soltani & Kukreti, 1992) revealed a 280m long precast integral bridge in Tennessee, where integral bridges have been designed for the past 20 years. Integral bridges have also been constructed in the UK on the M1-A1 Yorkshire Link that exceed the 60m recommendation and appear to be performing well (Brookes, 2002). The reasons for better bridge performance than predicted are not fully understood, but may include concrete creep (Tennessee DOT, cited in Soltani & Kukreti, 1992) and additional stiffness due to the highway pavement (Low, 1994). The author would suggest that the length of integral bridges should be incrementally increased and their performance observed. Taking into account the range of *effective bridge temperatures* (Emerson, 1976), concrete bridges of greater length than steel bridges will result in the same abutment movements.

7.1.2 Skew

The interaction between the superstructure and the abutment becomes increasingly complex as the angle of skew increases. The current limit of 30° skew should not be extended at present in light of the effects observed by Elgaaly & Sandford, 1992.

7.1.3 Live load interaction

More research needs to be carried out on the interaction between the live load on the deck and the backfill during the winter, when the earth pressures may have reached active values.

7.2 Abutment design

One of the field tests discussed in this report indicated that there was no stiffening of the soil (Elgaaly *et al*., 1992), whilst the majority demonstrated there was (Hoppe, 1996; Darley *et al.,* 1998; Frosch, 2002) or drew no conclusions.

7.2.1 Shallow abutments

The backfill behind shallow frame abutments appears not to reach fully passive pressures within the first 5 years of service, suggesting that designing for passive pressures is conservative. Further backfill stress measurements are required for a bridge that has been in service for more than a decade, or better quantification of wall friction, is needed to enable further refinement of BA 42/96.

7.2.2 Full height frame abutments and embedded abutments

The earth pressure distribution recommended in BA 42/96 is consistent with the centrifuge model results of a spread-base abutment (Ng, 1996 cited in Cheng, 1999), but the requirement that $K^* \geq K_p/3$ should be removed. A lower limit of K_o, as recommended by England *et al*. (2000) is more reasonable. A more conservative design envelope, may be used where deep penetrating frost might be expected. This is unlikely to be a consideration in the UK.

7.2.3 Piled abutments

Steel H piles can be orientated so that longitudinal deformations of the deck and girders cause bending about the weak or strong axis. The conclusion given by Wasserman and Walker (1996) (cited in Nielsen, 2001) is that "Both methods have proven to be satisfactory to the respective agencies". The author recommends that piles be oriented for weak-axis bending; this reduces bending stresses for a given enforced displacement (Burke, 1993), which makes it easier to achieve fixed-head behaviour in the pile (Wasserman & Walker (1996) cited in Nielsen, 2001) and reduces the possibility of cracking near the pile embedment in the wall. The use of stiff concrete and pipe piles for the support of integral bridges is discouraged (Arsoy *et al.,* 2002).

7.2.4 Semi-integral abutments

Fully passive earth pressures can develop behind integral backwalls; therefore this would appear to offer a reasonable design basis. These pressures will result in plastic deformation of the soil; therefore a better understanding of the subsequent active slip during deck contraction needs to be developed. It may be that traffic loading helps prevent the development of a gap behind the abutment.

7.3 Settlement mitigation

7.3.1 Compaction

Experimental results (Springman *et al.* 1996) suggest that loosely placed backfill should not be used, regardless of whether or not an approach slab is used. The current recommendation that backfill should be compacted, at the optimum moisture content to a dry density of 95% of the maximum dry density, is reasonable.

7.3.2 Approach slabs

The author recommends that with adequate compaction and drainage, approach slabs are unnecessary. Settlement due to thermal movements of the deck will occur irrespective of the presence of approach slabs, therefore their inclusion addresses the symptom but not the cause. Settling approach slabs in the USA cause considerable maintenance problems (Hoppe, 1999), the cost of which can be significantly greater than the cost of placing recurrent overlays. Use of approach slabs in the UK has diminished and the recently built bridges appear to be performing well.

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APPENDIX A Earth pressure coefficient definitions

Frictionless wall:

Passive failure will occur if the horizontal effective stress is increased while the vertical effective stress remains constant or is reduced. This state is described as the passive condition, where K_p is the passive earth pressure coefficient:

$K_p = (1 + \sin \phi')/(1 - \sin \phi')$……………………………………………………….(A)

Effects of soil/wall friction:

The passive earth coefficient derived above for a frictionless wall will lead to uneconomical designs when the wall is rough. δ = soil/wall friction angle

$K_p = \{[1 + \sin \phi' \cos(\Delta + \delta)]/[1 - \sin \phi')]\} \times e^{(\Delta + \delta) \tan \phi'}$…………………………….....(B)

Where $\sin\Delta = \sin\delta/\sin\phi'$

K_p values are often given by or interpolated from tables by Caquot & Kerisel (1948). [Powrie, 1997].

The following values are recommended in BA 42/96 to account for inclined abutment faces:

ϕ'	K_p		
	vertical	20° forwards	20°backwards
30°	5	3	7
35°	6	4	12
40°	9	5	20
45°	15	6	37

Table A K_p for inclined abutments from BA42/96

Rankine Passive Pressure:

The conclusions by Elgaaly & Sandford referred to the Rankine passive pressure coefficient:

$K_p = \tan^2 (45-\phi/2)$…………………………………………………………………(C)

which does not take wall friction into account.