THE ASSESSMENT OF FILLER BEAM BRIDGE DECKS WITHOUT TRANSVERSE REINFORCEMENT

by A McC Low and N J Ricketts

Prepared for: Bridges Engineering Division, DOT
Lancashire County Council
and the County Surveyors Society

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Bridges and Ground Engineering Resource Centre
Transport Research Laboratory
Crowthorne, Berkshire, RG11 6AU
1993

ISBN 0 9521860 0 4
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SUMMARY

This report considers the strength of filler beam bridge decks without transverse reinforcement. It describes the testing and analysis of three such bridge decks and refers to previous work on a half-scale model. It discusses how the additional, uncodified strengths found in this type of structure can be invoked in the assessment of existing bridges.

The testing involved one load test to collapse on a bridge in Herefordshire (Wormbridge) and non-destructive vehicle load tests on two bridges in Lancashire (Pepper Hills and Lancifliff Cross). In each case the structures proved to be far stronger than could be predicted using the conventional assessment rules. Assessments were obtained from the bridge owners and, in the case of the collapse test, independent conventional and non-linear assessments by a consultant. A back analysis of the test results was done by TRL and a model for the behaviour of these decks at the ultimate limit state has been developed.

The conclusions and recommendations are given in section 8. They contain a principal conclusion which, on its own, will be able to justify many bridges which would otherwise fail their assessment. To use this principal conclusion it is only necessary to read section 8. To use other beneficial actions mentioned it will be necessary for an assessment engineer to read the sections of the report referenced so that he or she can judge their relevance in a particular context. It is emphasised that caution is required when extrapolating from a test result and the engineer should read section 2.

Sample assessment calculations based on this report have been prepared for Lancashire County Council by Parkman Consultants and are available from The County Surveyor and Bridgemaster, PO Box 9, Guild House, Cross Street, Preston PR1 8RD.

1. INTRODUCTION

Filler beam bridge decks are concrete slab decks which are reinforced principally with steel I beams enclosed within the depth of the slab. Sometimes the flange of the beams is exposed on the soffit where it must be painted. More generally there is a depth of cover concrete under the bottom flange so the resulting slab is initially indistinguishable from a slab reinforced with bars. For this reason it is difficult to know how many such bridges exist. This form of construction was common in the second quarter of this century. On closer examination the line of the steel beams can often be discerned on the soffit because the concrete under them has fine cracks which might cause differential weathering. It is sometimes debonded from the steel and this causes a dull response when the soffit under a beam is struck with a hammer.

In England and Wales the assessment of such decks is covered by the DOT's bridge assessment standard BD 21/84. For specific topics this standard refers to the clauses on filler beam decks in the current design code which are in section 8 of BS 5400: part 5:1979 (implemented by DOT standard BD 16/82). These clauses require transverse reinforcement sufficient to resist the transverse moments predicted by an elastic analysis. Many of the existing bridge decks have no transverse reinforcement or an insufficient amount and yet they appear to have performed satisfactorily in service. This report describes investigations into the adequacy of such bridges, considers their implications and makes recommendations for assessments in accordance with current standards.

2. ASSESSMENT PRINCIPLES

2.1 THE REQUIREMENTS OF BD 21/84

The DOT's assessment loading, philosophy and many detailed requirements are set out in the Departmental Standard BD 21/84 and its accompanying Advice Note BA 16/84. The principal requirement is that bridges must be shown to perform satisfactorily at the ultimate limit state (ULS). In some cases it may be possible to demonstrate this directly with a load test. However, assessment methods need to be focused on the behaviour of bridges whose adequacy is marginal. It is unlikely that such a bridge would be undamaged by an application of its ULS load. Hence its adequacy must be demonstrated by calculation. In section 5.7 of the standard which covers load testing it states: "The object of load testing shall be to check structural behaviour under load and/or verify the method of analysis being used...".

This philosophy limits the role of load testing because the demonstration of the bridge carrying its nominal load cannot be taken as proof of its load capacity. However, in another direction it extends the role of load testing because the lessons learnt from one bridge can be applied to other similar bridges.

The adequacy of every bridge must be demonstrated by relating it to a similar bridge or test structure which has been loaded to failure. Usually this is demonstrated indirectly through the use of code methods which are based on simplified idealisations of structural behaviour and are necessarily conservative. A code method must be valid for a wide range of design parameters so it is necessary to calibrate it against tests on many shapes and sizes of structure. Assembling a code is a lengthy process.
In a real bridge there are often beneficial structural actions which are not covered by the codified methods. No bridge should be condemned or downgraded without considering these. This is stated clearly in BD 34/90 which says "...where such methods indicate that a structure may be sub-standard, more refined methods of assessment...shall be applied." The beneficial actions may be quantified either by test or by theoretical justification based on first principles. Judgement is needed to identify where the latter is appropriate.

### 2.2 THE USE OF TEST RESULTS

When considering the nationwide assessment of all bridges there is a wide variety of beneficial actions which need to be quantified. It is impractical to provide the resources which would be needed to develop codified methods for every action and the time delay associated with such an operation would probably be unacceptable. Instead it will be necessary for assessors to be aware of any relevant tests to failure so they can make informed judgements about individual bridges. A "case law" of reported tests will evolve. The tests described in this report should be seen as part of this case law and it should be recognised that the generalised conclusions drawn are based on the circumstances of the specific tests. The codifying activity has not been carried out so it is up to individual assessors to decide on the applicability of any conclusions to their specific bridge.

A test to failure provides a strength value which is specific to one bridge and one load arrangement. The loading may be widespread across the deck like the lane loading in the code or it may be chosen to demonstrate the ability of the bridge to distribute a heavy wheel load or concentrated group of wheel loads. If the result is to be useable for different bridges and load positions it is necessary to make an analytical model of the failure.

The first step is to make a back-analysis. This is a subjective exercise in which a structural model is developed which best fits the test data at failure. The data recorded on the way to failure are also a guide to the form of failure which develops. The "prediction" of this analysis is predetermined to be the test failure load so it is usually necessary to adjust the strength parameters to achieve this result. Often the failure load is much higher than would be predicted using usual parameters so their values are increased. When it is not clear how the parameters should be increased relative to each other their values should be chosen on the subjective basis of equal credibility. If it is necessary to postulate incredible values then a different model must be sought which invokes more of the reserves of strength which are inherent in the real structure.

Once the back-analysis model is established a judgement must be made whether the actions invoked are sufficiently robust to be relied upon in assessments. Usually this requirement is demonstrated by evidence of ductility in the test but all links in the chain must be considered. If the model is judged to be insufficiently robust an alternative model must be found which will predict a failure load lower than that reached in the test.

Next the strength parameters should be reset to the usual values used for assessment with partial factors applied or, for prediction, without partial factors. These models, respectively called the assessment and prediction models, can then be applied directly to other bridges of the same type with a similar load pattern. The models can also be used with a different load pattern but in this case a further subjective judgement is required. If the revised configuration requires a disproportionate reliance on an action which was less significant in the back analysis of the test then the limitations of the model must be recognised. It may be necessary to apply an additional partial factor to cover actions not adequately demonstrated in the test.

A test under working load on a bridge which has been instrumented will show how that bridge is behaving and the results can be compared with the results at a similar load level in a previous test to failure. This will help determine the applicability of the failure results to the bridge being tested.

Although structural capacity cannot be demonstrated at working load the lack of it may be. If the deflections or strains are higher than expected this indicates that in some way some part of the structure is missing or fractured.

### 3. TESTS ON FILLER BEAM DECKS

The results of tests on four structures are available and they are listed in Table 1 with some of their basic dimensions. The three full sized bridges had no transverse

<table>
<thead>
<tr>
<th>Bridge</th>
<th>Clear span m</th>
<th>Skew</th>
<th>Width m</th>
<th>Slab depth mm</th>
<th>Beam depth mm</th>
<th>Beam spacing mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thomas and Short</td>
<td>2.74</td>
<td>0°</td>
<td>4.72</td>
<td>228</td>
<td>152</td>
<td>305</td>
</tr>
<tr>
<td>Wormbridge</td>
<td>5.79</td>
<td>17°</td>
<td>9.37</td>
<td>457</td>
<td>305</td>
<td>457</td>
</tr>
<tr>
<td>Pepper Hills</td>
<td>4.80</td>
<td>39°</td>
<td>5.63</td>
<td>300</td>
<td>203</td>
<td>510-670</td>
</tr>
<tr>
<td>Langcliffe Cross</td>
<td>4.25</td>
<td>7°</td>
<td>6.41</td>
<td>330</td>
<td>203</td>
<td>590-665</td>
</tr>
</tbody>
</table>
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The work described in this report was funded by the Department of Transport, the County Surveyor's Society and Lancashire County Council.

A copy of the full Research Report Special 383 (price at publication £35, price code J, report prices are subject to change) may be obtained on written request from TRL Post Box 303, Wokingham, Berkshire, RG11 6YX. Cheque made payable to TRL.

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reinforcement. Two of the structures were tested to destruction. The first of these was reported by Thomas and Short (1952). They were conducted on a half scale model with a span of 2.74m representing a span of 5.48m with 300mm deep beams at 600mm centres in a 450mm deep slab. The supports were carried on bearings which were free to rotate and slide at one end. The results of the tests are not repeated here although some of the values are used in Section 4.6.3 to corroborate a back-analysis model.

The tests on the full sized bridges have not been published and so shortened and modified versions of the reports prepared for the commissioning agents are given in Sections 4 to 6 below. These reports include descriptions of back-analysis models and comparisons of the predictions of these models with the test results.

4. TESTS ON WORMBRIDGE

This section describes the assessment and two tests to failure of a 5.79m clear skew span highway bridge at Wormbridge near Hereford. This bridge was built during 1938/39 and carried the A465 Hereford-Abergavenny road across Worm Brook. The deck was of filler beam construction, comprising 305mm x 127mm rolled steel joists at 450mm centres, cast within a 380mm thick concrete slab and was typical of many small highway bridges still in service throughout the country.

The bridge maintenance responsibility was that of the County Engineer of Hereford and Worcester acting as agent for the Department of Transport. The bridge was assessed to The Department of Transport’s Assessment Standard BD 21/84 (Department of Transport, 1984) by the County Engineer’s Department and found to be substandard in bending both longitudinally and transversely. However a note was made that the deck would be capable of sustaining full assessment loading if the transverse bending capacity could be enhanced.

The decision to rebuild the bridge involved a number of factors not least was that the abutment walls were below current design standards (unreinforced and only 457mm thick.)

The scheme to replace the bridge was designed to cause minimum interference to traffic and resulted initially in the demolishing of approximately one third of the deck width. The remaining two thirds of the structure was offered to TRL for testing at an appropriate point in the contract just prior to demolition.

It was decided that the bridge was suitable for testing because little research had been done on filler beam decks. Also a great many decks of this type exist throughout the country and while they often show no signs of distress in service, they are commonly found to be under strength when assessed.

The width of the structure was such that, with care, two tests could be attempted. It was decided to load the deck directly at slab level, and to follow this with a test load at the level of the road surface. In both cases the deck slab would be instrumented to measure strain and displacement.

A contract was let to a consultant to assess the bridge independently. The specification required firstly a conventional BD 21/84 (1984) assessment and secondly an assessment using extended methods to produce a more realistic strength prediction. In addition a back analysis was made at TRL and the method used is proposed as the basis of an assessment technique for similar bridges.

4.1 BRIDGE STRUCTURE AND MATERIAL PROPERTIES

4.1.1 Deck construction

The deck at Wormbridge (see figures 1 to 3) was constructed using 305mm x 127mm x 48kg (12"x5" x 32lb) RSJ sections as filler beams. Spanning 5.79m, they were spaced at 457mm centres and cast within a 380mm thick insitu concrete slab. The beams were held in position by the mass concrete only. No transverse tie bars were used and there was no transverse reinforcement in the slab other than a layer of “Expamet” type mesh below the beams. The bridge was built to a skew of 17 degrees and the beams were placed at this angle within the deck. No distinct bearings were provided for the span which was cast directly onto the abutments. There was no evidence of any attempt to form movement joints or bearings. An original drawing for the structure was available but there was no evidence that it was an "as built" record.

4.1.2 Abutment construction

The abutments at Wormbridge were simple unreinforced mass concrete walls 457mm thick constructed on a 1m wide strip footing (figure 2). The proportions were what might be expected in a small reinforced portal bridge, but as a mass concrete wall with no effective shear connection to the slab they were obviously below current design standards. As was common with mass concrete of this age the walls contained a small proportion of bricks and other large aggregate, though not enough to merit describing the material as masonry. Despite this the abutment walls showed no signs of distress or defects.

4.1.3 Properties of structural steel

After the tests a steel beam section was recovered. When these beams were manufactured universal beams and rolled steel joists were produced in the same 12"x5" serial size and therefore the section was measured. This confirmed it was a joist as stated on the drawings. Tensile tests to BS 18: 1987 (British Standards Institution, 1987) were carried out on specimens from the flange and the web, the results of which are given in Table 2.
TABLE 2
Tensile test results for Rolled Steel Joist

<table>
<thead>
<tr>
<th>Dimensions</th>
<th>Yield stress</th>
<th>Max stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diam mm</td>
<td>Area mm</td>
<td>GL mm</td>
</tr>
<tr>
<td>Flange section</td>
<td>7.98</td>
<td>50</td>
</tr>
<tr>
<td>Web section</td>
<td>5.65</td>
<td>25</td>
</tr>
</tbody>
</table>

GL = Gauge length
EL% = Percentage elongation
RA% = Percentage reduction in area

4.1.4 Properties of concrete

Four 100 mm dia concrete cores (yielding six tests) were recovered from the deck. The results are tabulated below:

TABLE 3
Concrete compressive test results

<table>
<thead>
<tr>
<th>Core No</th>
<th>Density kg/m</th>
<th>Compressive Strength N/mm</th>
<th>Insitu Cube Strength N/mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>1A</td>
<td>2360</td>
<td>48</td>
<td>49.5</td>
</tr>
<tr>
<td>1B</td>
<td>2350</td>
<td>46</td>
<td>47.5</td>
</tr>
<tr>
<td>2</td>
<td>2350</td>
<td>40.5</td>
<td>42</td>
</tr>
<tr>
<td>3A</td>
<td>2350</td>
<td>44</td>
<td>45.5</td>
</tr>
<tr>
<td>3B</td>
<td>2370</td>
<td>42.5</td>
<td>44</td>
</tr>
<tr>
<td>4</td>
<td>2350</td>
<td>43.5</td>
<td>45</td>
</tr>
</tbody>
</table>

12.9mm super strands with sufficient length to project three metres above the bridge deck.

4.3. TEST 1

4.3.1 The loading system

As the bridge had no transverse reinforcement and the ability of the deck to distribute load laterally had been questioned, the loading system was arranged to investigate this aspect. The most onerous loading effect in service was considered to occur when two 40 tonne vehicles passed on the bridge (Based on that specified for longitudinal trough decking in BD 21/84 (Department of Transport, 1984)). In this case the offside wheels of the vehicles could produce point loads at a spacing of 700mm laterally and therefore both test rigs were designed to produce load at this spacing. The loading effects of the near side wheels were not considered. The object of Test 1 was to determine whether the bridge deck would behave differently if the loads were applied directly onto the concrete deck slab. A small test rig was positioned within the verge close to the kerb line. The load was applied on two 500 x 300 pads 700mm apart as shown in figure 5

4.3.2 Design of test rig

The test rig for Test 1 was composed of a bearer formed from two stiffened 305 Universal Column sections. These were welded together as shown in figure 5 with a bearing plate on the top surface on which to stand a single stressing jack. The ground anchor tendons passed through a hole in this plate, between the bearers and through a hole cored in the deck of the bridge. The ends of the main bearer assembly were mounted on two beam load cells which were in turn attached to two Universal Column section loading pads.

Load was applied using a 3000 kN jack mounted on a loose collet block. This allowed the rock anchor to be locked off and a further stroke of the jack taken as required.

4.2 PREPARING FOR LOAD TESTING

4.2.1 Site survey

Before any decisions were taken regarding position of loads and design of loading rigs a full survey of the structure was undertaken. The survey was able to confirm the skew angle, dimension accurately the cut line forming the limit of the demolition and determine with reasonable accuracy the position of the steel beams.

4.2.2 Installation of ground anchors

Five ground anchors were installed through holes cored in the bridge deck at the positions shown in figure 4. Each anchor was required to be able to sustain a temporary working load of 1500 kN and was composed of ten...
4.3.3 Instrumentation

Loads were measured during the test using two 2000 kN beam load cells positioned immediately above each foot of the loading rig (see figure 5). For each load increment the readings from each load cell were added to give total load on the bridge. The data were recorded using a Solartron Orion data logger linked to a PC.

Strains were measured at the soffit of the deck using an array of fourteen TRL type surface mounted vibrating wire gauges. These were mounted below the steel beams at the positions shown in figure 4 and were read remotely using a portable miniature strain meter.

Displacements were measured using nine displacement transducers mounted on a scaffold grillage built beneath the bridge and attached to hooks glued to the soffit. The displacement data were also recorded using an Orion data logger.

4.3.4 Test procedure

Before the test commenced final zero readings were recorded from the load cells and instrumentation. A small load increment of 30 kN was applied to bed down the stressing jack and tension the anchor tendons. From this point load on the bridge was increased in approximately 100 kN increments. At each load increment the instrument readings were recorded. The loading was increased to a maximum of 1864 kN, the theoretical capacity of the ground anchor tendons. At this point the cracking beneath the bridge was spreading towards the area of the second test and it was decided not to proceed further. The jack was retracted which resulted in the loose collets beneath the jack locking off at a load of 1080 kN. The jack was removed from the test rig which was then left stressed overnight to see if any creep of the bridge deck occurred. Twelve hours later the load cells still recorded 1080 kN and the rig was then cut off the tendons. A crack at the south face of the bridge between the deck and the parapet wall which had formed during the test closed after the load was removed.

4.3.5 Visible damage during loading

The initial loading increments produced no signs of visible damage and it was not until a load of 980 kN that the first crack appeared. It was approximately 500mm long and ran transversely from the anchor hole towards the downstream face of the bridge. During the next increment (1080 kN) the crack extended the width of two beams either side of the anchor hole. Cracking became more extensive over the next few load increments and at 1510 kN, three main cracks formed a radial pattern over a width of four beams either side of the test rig position. The test was continued to a load of 1860 kN, by which time the crack pattern was more extensive but still exhibited the same basic radial characteristics as shown in figure 6. Very little longitudinal cracking had occurred and the crack pattern was such that an uninformed observer might wrongly assume the deck to be an isotropically reinforced slab.

4.3.6 Interpretation of data

The load/displacement graphs (figure 7) form an approximately bi-linear shape up to a load of 750 kN with a change of slope at a load of 400 kN. From 750 kN to the point at which the test was terminated at 1860 kN the graphs were non-linear with the displacement progressively increasing with load. The slope of the non-linear portions of the curves at the maximum test load provided some evidence that the failure load for the structure would have been much higher.

The transverse strain gauges 1 to 6 when plotted against load appear to show cracking starting under the load position at a load of about 500 to 700 kN. This can be determined by noting large increases in strain, often the result of cracks passing through gauge lengths or marked reduction in strain, caused by cracks occurring close to gauges. The first crack however was not visibly apparent until a load of 1000 kN was reached. All the strain gauges except gauge No 3 which was directly under a loading pad showed a linear response up to a load of about 700 kN which compared well with the displacement gauge results. Interpretation beyond this point was difficult as it depended on the local effects of cracking, either close to, or through the gauge length.

4.4 TEST 2

4.4.1 The loading system

The loading system for Test 2 was arranged to produce four point loads 700mm apart laterally and 1800mm longitudinally at the surface of the carriageway. Each point load was again applied over an area of 500mm x 300mm. The arrangement was designed to model the offside rear wheels of two passing 20 tonne bogies. As with Test 1 the loading effects of the near side wheels were not modelled.

4.4.2 Design of test rig

The test rig for Test 2 shown in figure 8, consisted of two 2000 kN crossheads linked longitudinally with a length of universal column section. Four 2000 kN beam load cells were mounted above the loading pads and the loads were applied using four 3000 kN capacity stressing jacks.

4.4.3 Instrumentation

Loads for Test 2 were measured using four 2000 kN capacity beam load cells. These were positioned between the crossheads and the loading pads in a similar manner to the arrangement for Test 1. The data from the load cells were recorded by the Orion data logger as for the first test.

Strains were measured using fifteen vibrating wire gauges attached to the soffit of the bridge below the beams as shown in figure 4. Seven of the gauges were placed transversely at the centre of the span with the remaining eight placed longitudinally beneath the two loaded beams.
Displacements were measured using eleven displacement transducers attached to a scaffold frame beneath the bridge. Seven of these were positioned transversely beneath the beams at the centre of the span with the remaining four placed at the abutments as shown in figure 4.

4.4.4 Test procedure

The preliminary tensioning of the stressing jacks resulted in an initial load of 100 kN. From this point, load on the bridge was applied in approximately 100 kN increments. At each load increment the instrumentation readings were recorded. The loading was increased to a recorded maximum of 2900 kN. The test was then continued for a further five increments to confirm this load could not be exceeded. On completion of the test the jacks were retracted and removed from the test rig.

4.4.5 Visible damage during loading

The visible damage during Test 2 was extensive and is described in list form for clarity. This should be read in conjunction with figure 9, the crack map for Test 2.

Increment 2 (435 kN)
An existing crack along beam 15 was seen to widen to 0.4mm.

Increment 4 (740 kN)
The existing crack, beam 15, continued to open to 0.6mm.

Increment 5 (840 kN)
First signs of new damage: a longitudinal crack between the upstream anchor points (0.25mm) and a transverse crack on the centre line (0.15mm).

Increment 6 (950 kN)
Pre-existing crack on beam 15 opened to 0.8mm

Increment 7 (1040 kN)
A transverse crack formed on the centre line of the bridge starting in the slab on the upstream side of the ground anchor points.

Increment 8 (1196 kN)
A longitudinal crack formed on the line of beam 20 and began leaking water.

Increment 9 (1373 kN)
Longitudinal curving cracks formed in close proximity to vibrating wire (VW) strain gauges 9 and 12.

Increment 10 (1373 kN)
This increment produced no increase in load but cracks formed transversely through to the upstream free edge of the slab and longitudinal along beam 16 around VW gauge 11.

Increment 11 (1570 kN)
The cracks along beam 16 joined to become one continuous crack.

Increment 12 (1608 kN)
A crack started to open along beam 14 and a vertical abutment crack formed on the north abutment opposite beam 16.

Increment 13 (1706 kN)
The crack on beam 16 had opened to a width of 1mm.

Increment 17 (2010 kN)
A crack was formed through VW gauge 6 and the abutment crack had widened to 1.5mm.

Increment 19 (2050 kN)
A crack formed along beam 12. The maximum displacement at this point was 14.6mm

Increment 20 (2040 kN)
The transverse crack at the centre line of beam 15 had opened to a width in excess of 2mm. For safety reasons, accurate crack measurement was not possible at this level of loading.

Increment 24 (2315 kN)
Extensive abutment cracking, north abutment opposite beams 16 and 13.

Increment 27 (2450 kN)
From this point in the test the bridge structure was noticeably different in the way that it carried load. The deck started to act as three components articulated along the line of beams 15 and 17 and the upstream free edge of the slab started to crack and lift clear of the abutments. As a portion of the bridge had been demolished on the upstream side the mass and built in effects from a parapet wall were not available to prevent the slab lifting at high loads.

Increment 30 (2500 kN)
From increment 27 to increment 30 the load increased by only 50 kN but the maximum displacement increased from 24.3mm to 31mm.

Increment 31 (2700 kN)
The load again started increasing and a piece of concrete, 300mm x 50mm, was seen to fall from beam 17.

Increment 34 (2903 kN Maximum load)
Cracks continued to open considerably and the maximum displacement increased to 64mm.
Increment 36 (2864 kN)
The crack from beam 14 increased back to the abutment.

Increment 41 (2227 kN)
The test was discontinued at this point. The displacement was 116mm. Over the previous five increments load had been slowly decreasing as the structure was being jacked down, with the deck elements rotating about the major fractures shown in figure 9. The centre section of the slab appeared to have fractured into several approximately rectangular blocks spanning between the joists. The slab continued to lift from the abutments at the upstream corners. At the upstream face the tarmacadam had become separated from the sub-base but there was no indication of any separation between the sub-base material and the slab. Had the test been continued beyond this point it was obvious that a point would have been reached where the centre section of the fractured slab would have separated from the rest. All the visible evidence indicated that this would have been a mechanism failure due to large geometry changes rather than any shearing action.

4.4.6 Interpretation of data
The load/displacement graphs (see figure 10) are all of similar shape and exhibit the same bi-linear features seen in those from Test 1. From the curves it can be seen that the graphs were linear to a load of approximately 700 kN and after a slight change in slope they remained linear until the load increased to 1370 kN. From this point the structure behaved in a non-linear manner with the curves gradually decreasing in slope to become horizontal at a load of 2450 kN. At this load the slab fractured into three discernable components. Following this the load increased to the maximum of 2900 kN before reducing steadily with rapidly increasing displacement.

The transverse displacement/load distributions, shown in figure 11, were plotted to examine how the load distributed transversely and highlight the points at which the structural behaviour changed during the test. The curves show marked increases in rate of displacement at loads of 1370 kN, 1960 kN and 2500 kN. These points coincide with those identified from the load/displacement curves with the exception of that at the 1960 kN load. From further examination of the load/displacement graphs in figure 10, this point could be identified and possibly linked to the increased opening of transverse and longitudinal cracks or the abutment fracture. However at this high loading it was not safe to approach the structure and measure crack widths.

The displacement/load distributions give a good indication of how the load distributed laterally during the test and also when this distribution started to break down. For displacement gauges 1 to 5 the curves remain parallel up to a load of 2430 kN at which point gauges 2 to 4 start to show an increasing rate of displacement. These gauges coincide with the centre section of the fractured slab. Data from the strain gauges proved difficult to interpret because of sensitivity to local effects such as cracking. As described in Test 1, this sensitivity had proved useful during the test in providing an early indication of the position and load at which initial cracking occurred.

4.5 ANALYSIS AND ASSESSMENT
The original assessment of this structure produced by the Hereford and Worcester County Engineers Department was made available to assist the research work. In addition to this a contract was let to produce an independent conventional assessment and an assessment using extended methods.

4.5.1 Original assessment
The assessment analyzed the structure using three different methods. Initially the bridge was analyzed as a series of simply supported beams with full lateral restraint. Transverse distribution within the slab was neglected as there was no transverse reinforcement. Secondly the deck was treated as an isotropic slab using design charts (Rowe, 1966). Finally a yield line analysis treated the deck as an orthotropic slab.

The results of the simple beam analysis found that the limiting factor was the bending resistance of the beams. The critical loading was found to be the accidental vehicle load and a permissible load of FE1 (Group 1 fire engine) was obtained. The shear capacity of the beams was found to be adequate to resist full assessment loading to BD 21/84 (1984).

The isotropic slab analysis using design charts found that the critical loading case was HA. The longitudinal moment caused by full HA assessment loading at ULS was calculated to be 304 kNm/m, compared with a calculated ultimate moment of resistance of 313 kNm/m. The transverse moment produced was 146 kNm/m which compared with an ultimate moment of resistance of 6 kNm/m calculated assuming an allowable concrete tensile stress of 0.25 N/mm² and no transverse reinforcement.

For the yield line analysis a method was used that transformed the deck to an equivalent isotropic (affine) slab to take into account the differences between the longitudinal and transverse bending resistance (Jones 1989). The transverse moment calculation again assumed a concrete tensile stress of 0.25 N/mm². There was no appreciable increase in carrying capacity when compared to the simple beam analysis.

The conclusion of the assessment was that the bridge deck was deficient both in longitudinal and transverse bending resistance, but that if the transverse bending capacity of the deck could be enhanced to fully distribute the applied loading, it would be capable of carrying the full assessment load.
4.5.2 Independent assessment

The independent assessments were made by Gifford & Partners Consulting Engineers. The assessment calculations were designed to be specific to the testing so that the resulting predictions of failure load could be directly related to the test data. Therefore the loading positions, concrete core and steel test data detailed in sections 3 and 4 were supplied with the specification.

An assessment to BD 21/84 (1984) was produced using the mean measured material properties and partial safety factors of unity. A load spread of 1 to 2 through the fill and 1 to 1 through the concrete was used for assessing the effect of concentrated loads. The resulting calculated failure loads were 386 kN for the two point loading used for Test 1 and 651 kN for the four point load used for Test 2.

For the extended assessment the “Non Linear Grillage” computer program, developed by Jackson was used. This non-linear program is similar to a grillage but considers all six degrees of freedom at each node. It uses simple line elements and the “smashed crack / distributed steel / layered” approach. The failure loads predicted using this program were 1400 kN for the two point loading used for Test 1 and 2100 kN for the four point loaded used for Test 2. In obtaining these loads some transverse restraint was assumed due to friction at the abutments. The coefficient of friction used was 0.4, which is the concrete-on-concrete value used in the design of falsework. This figure was intended to be conservative as the deck at Wormbridge was actually cast onto the abutments. The analysis was repeated using full lateral restraint which resulted in a 40% increase in failure load for the two point test (Test 1). Further analysis however using an increase in restraint of 50% found only marginal increases in strength.

The main conclusions of this assessment were that the higher failure load obtained from the non-linear analysis was mainly due to the enhanced distribution properties it predicted. This was largely due to compressive membrane action and is dependent on lateral restraint at the abutments. Predicted strain hardening was also significant with a maximum stress in the steel beams of 270 MPa compared to a yield stress for the flanges of 239 MPa. It was also noted that in all cases the failure mode predicted was flexural and ductile.

4.6 BACK ANALYSIS OF RESULTS

A back analysis was made at TRL (see Section 2.2). The models used differed from those used for the extended assessments only because the studies were performed by different people using different analysis tools.

The studies were based on Test 2 as it provided more data. The model derived was later applied to Test 1 (see 8.3 below). The beams are at 457mm centres and it was natural to consider the loads being carried by each deck strip of this width centred on each beam. An initial analysis showed that each strip might be able to carry about 250 kN. This was based on a flexural failure assuming full composite action, with the concrete in tension ignored and the load applied at the same positions as in the test. A more refined analysis described in 8.2 below gave a strip capacity of 284.7 kN with a flat yield plateau. As the deck supported 2904 kN before failing this suggested that it had a good distribution mechanism which was able to spread the load to more than ten beams. The deflected form of the bridge under maximum load (figure 11) appears to indicate that the load resistance is more concentrated. However, many of the deck strips are on the yield plateau so they carry the same load even though their deflections are very different.

4.6.1 Distribution of the concentrated load

Figure 12 indicates that four beams fall within a direct 1:2 spread from the applied loads although only part of the outer beams is within the spread lines. It was assumed that these four beams were directly loaded and were working at their full capacity at failure.

There was no transverse reinforcement and the concrete was clearly cracked along the lines of the beams. Hence it was assumed that transverse flexure was not contributing to this distribution. The only mechanisms available to distribute the load appeared to be raking compressions in the concrete and torsion. Each strip of concrete between adjacent beams is well gripped by the beams. It was assumed that racking compressive forces, transverse to the line of the beams, would act from the top of the web of a loaded beam to the bottom of the web of its neighbour. A significant horizontal force is required to resist this raking thrust. As the load is distributed further across the deck additional horizontal reactions are mobilised at each beam. The only force available to resist these horizontal reactions is the friction generated by the beam reactions at the abutments. Initial studies showed that the friction force would be much too small.

Unreinforced concrete sections are able to sustain some torsion although they fail in a brittle manner. Hsu (1984) gives the nominal limiting torsional shear stress as 0.415 f'c MPa where f'c is the cylinder strength in MPa. Assuming f'c = 0.8 * fcu then this becomes 0.371 * fcu where fcu is the cube strength in MPa. He also states that members with only longitudinal reinforcement are slightly stronger in torsion than unreinforced members and they are much more ductile. His data indicate an increase of about 10% and he says it is seldom more than 15%. It has been assumed that the steel beams give a benefit equivalent to that provided by longitudinal reinforcement and the strength is increased by 10%. If the mean value for use in predictions and back analyses is taken as 1.23 times the nominal value then the mean value of the limiting shear stress is 0.500 * fcu. Initial studies showed that the torsion capacity was insufficient to carry the distributed load across the first bay.

The “raking strut” and “torsion” models were then combined into the model indicated in figure 12. With both the torsion and dead load friction fully mobilised and
some of the strength parameters increased slightly it was possible to show that the full 2904 kN test failure load could be carried. The torsional shear stress was increased a further 13% to 0.565f_{cu}. The coefficient of friction was given the high value of 1.2. This is discussed in 8.3.

In the combined model torsion is the primary means by which the load is distributed from one beam to its neighbour. If the torsion capacity of the concrete bay is exceeded then sufficient lateral horizontal reaction is mobilised so that the torsion is reduced to its limit. The horizontal reactions reduce the torsion because they act as a couple with the depth of the steel beam as the lever arm. In figure 12 the horizontal and vertical forces on the bay are shown combined as a raking force.

The friction forces depend on the vertical reactions at the ends of the beams. The vertical reactions from each beam are modified by the torsion reactions from each adjacent strip. The torsions shift some of the beam reaction back to the previous beam, the one nearer the applied load. These interactions are difficult to visualise and to make them manageable they were modelled in a small BASIC program called FILJO which was run on a pocket calculator. The program is simple because it performs a limit analysis which only considers the situation when the limit is reached. It distributes a given load across the deck in one direction and calculates the transverse forces mobilised at the furthest loaded beam and the dead load friction force available there to resist it. This friction force depends on the distance to the free edge of the deck which is different on the two sides of the load. Hence the program is run for each side and the capacities are added. In this model friction at the abutment is mobilised in two directions without considering the interaction between the two friction forces. The interaction would have introduced complexity and the slight loss of conservatism was considered to be acceptable.

For Test 2 FILJO gives a capacity of 1343.5 kN for the side nearer the free edge and 1560.5 kN for the other side which give a total load of 2904 kN as in the test. The analysis shows the load carried in each beam and indicates that the load is distributed over a deck width of about 6m.

This model can be used to predict the load capacity with the load moved progressively towards the free edge at beam 21. Figure 13 shows how the capacity reduces and it is encouraging to note that most of the capacity is sustained until quite close to the edge. However it must be emphasised that this is speculation. Failure test data are not available for a load which is closer to the edge. If the edge beam is not sufficiently embedded in the concrete it may suffer premature failure due to lateral buckling.

4.6.2 Load capacity of a deck strip

For this back analysis the vertical load capacity of a beam strip has been taken as 284.7 kN. The derivation of this value must be explained. With the effective span taken as 5850mm and the steel yield stress as 229 MPa then the capacity is 208 kN based on the fully plastic flexural capacity of the composite section. This is much too small to use in a viable back analysis. Other effects which contribute to the capacity must be included. For example:

- Strain hardening of the steel.
- Frictional restraint at the abutments acting at the level of the underside of the deck in the longitudinal direction.
- Concrete in tension.
- The use of a realistic concrete compressive stress block in place of the notional stress used in the plastic section analysis.
- Composite action with the surfacing.

All these effects can be modelled in a simple non-linear beam computer model. "Simple" is not a word which can usually be applied to a non-linear computer analysis. However, new programs were written which were kept as simple as possible. Intentionally much of the development was done on a pocket calculator.

Two programs were written. The first, REF, was written on a pocket calculator. It evaluates the resultant force in a general member section under a specified pattern of linear strain. The section is composed of any number of subsections each of a different material with the stress/strain characteristics of each material specified as any number of points with linear interpolation. For a specified resultant force or position the program allows some automatic iteration to find the strain in the top or bottom fibre.

The second program incorporates REF and is run on a PC for speed. It models a Symmetrical End Restrained Beam and is hence called SERB. Given the applied load on the beam and the end friction coefficient the trajectory of the thrust line can be found. In a double iteration the top and bottom strain are varied to match the resultant to the thrust line in position and magnitude. Hence the curvature can be found at a number of sections along the beam and this is numerically integrated twice to give the deflected shape. It also estimates the maximum load capacity. By trial and error the capacity can be confirmed quite quickly.

Early runs with SERB predicted that the maximum capacity would be reached when the midspan deflection was about 400mm. This is very different from the observed behaviour in the test which indicated that the maximum load was reached with a deflection of about 50mm. Because of the axial load due to friction it was realised that geometric non-linearity (GNL) might be significant. It was very easy to model GNL in the program. It was only necessary to adjust the thrust line eccentricity by the calculated deflection and include an iteration loop. With this modification the model reached its maximum load at a deflection of about 40mm which was close to the observed behaviour.
There was approximately 220mm of surfacing in the deck but the under layer of this appeared to be a dense but unbound sub-base material with rounded aggregate. It was assumed that there was no composite action between the deck and the surfacing. Hence the principle benefit came from friction at the supports with the remainder from the realistic concrete stress block.

For the SERB analysis the concrete stress-strain curve had a peak compressive strength of 50 MPa and a peak tensile strength of 2.7 MPa. The friction coefficient at the support was taken as 1.2. It was assumed that the single yield test result for the flange of 229 MPa had been a low result and a value of 237 MPa was used instead. With these parameters the capacity of the beam strip was found to be 284.7 kN, as assumed above.

Under this load the local bond stress between the beam and the compressive zone of the concrete calculated as 8.5.1 of BD 16/82 is 1.47 MPa. This can be compared with the SLS limit given of 0.7 MPa.

### 4.6.3 Corroboration and summary

Although Test 1 did not reach failure it can be inferred from the load/displacement graphs in Figure 7 that failure would have occurred at about 2200 kN. Re-running the back analysis with the load configuration for Test 1 and all other parameters unchanged gave a failure load of 2233 kN.

The only other available data for a filler beam bridge deck loaded to destruction comes from Thomas and Short (1952). They tested a half scale model with a single point load at midspan. Some details are given in section 3. They recorded failures from an unspecified number of tests in the range 470 to 530 kN. Using the little material strength data given in the paper the failure load predicted by FILJO, using the same parameters as the back analysis, is 480 kN. This is an encouraging corroboration.

The hypothesis that the loads are distributed principally by torsion in the concrete can also be tested with results from the linear elastic stage of the test. From the load/deflection graphs the elastic limit load was taken as 1600 kN. Figure 15 shows how the recorded midspan deflections vary across the width of the bridge at this load. Also shown are deflections calculated using the equations of Cusens and Pama (1975) for shear-key decks. These assume that there is no transverse bending moment and concentrated loads are distributed by torsion. Composite section properties were used with the concrete assumed to be uncracked with a Young's Modulus of 32500 MPa. This is the stiffness appropriate to the measured concrete strength. The comparison shows good agreement between theory and test. Note that the analysis assumes that longitudinal cracks exist along the line of the beams.

In the elastic analysis the most heavily loaded strip was carrying 9.76% of the applied load which compares with 8.43% found in the back analysis of the failure. This indicates favourable redistribution characteristics for decks which are designed or assessed elastically. Alternatively it shows that the capacity assessed using an elastic distribution analysis will be 14% less than that from an ultimate distribution analysis assuming the same strip capacity is used in both assessments. However this conservatism when using an elastic distribution will be much less under the distributed loads used for assessment.

Summarising, the back analysis has been achieved with a failure mechanism which in most respects is believable. Credulity may be stretched over the use of a coefficient of friction of 1.2. In using this value it was realised that some of the friction force was derived from bond or cohesion within the concrete at the junction between the deck and the abutment. The Thomas and Short (1952) results were derived in laboratory conditions with notionally no friction. Hence in fitting the parameters to both results the friction parameter has been, to a degree, determined by the data.

Alternatively the discrepancy with Thomas and Short (1952) could be explained by composite action with the surfacing because their models have none. The greater the strength assumed in the surfacing the lower the coefficient of friction need be.

For an assessment a lower friction coefficient would be used, say 0.6. Also the torsion stress would be limited to 0.50 fcu and the yield stress would be taken as its test value of 229 MPa. Together these give a predicted failure load of 2177 kN which is 75% of the test value.

The bond stress deduced of 1.47 MPa is about twice the yield test result for the flange of 229 MPa. Together these give a predicted failure load of 2177 kN which is 75% of the test value.

4.7 DISCUSSION

4.7.1 Test 1

Test 1 demonstrated the very effective distribution behaviour of the deck and was curtailed before damage spread into the Test 2 area. From the load/displacement curves (figure 7) it can be deduced that failure under the two point loading was likely to occur at approximately 2200 kN which was greater than the safe capacity of the ground anchor tendons. Further evidence to support this figure can be gained from examining the cracking pattern (figure 6) which shows radial cracking over four and six beams respectively on each side of the load. There were few signs of longitudinal cracking and the slab was clearly well below its ultimate load.

An ultimate capacity of 2200 kN compares with a load of 386 kN predicted by the conventional assessment, 1390 kN predicted by the non-linear assessment, 2233 kN from the back analysis of Test 2 reworked for the Test 1 configuration and 1870 kN using the back analysis method with assessment parameters. The assumed failure load is 5.7 times greater than the conventional
assessed, 1.6 times greater than the non-linear assessment and 1.2 times the assessment based on the back analysis. A summary of these results is given in Table 4.

### TABLE 4

Summary of assessment values

<table>
<thead>
<tr>
<th></th>
<th>Test 1</th>
<th>Test 2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>kN</td>
<td>%</td>
</tr>
<tr>
<td>Conventional assessment</td>
<td>386</td>
<td>18</td>
</tr>
<tr>
<td>Non-linear assessment</td>
<td>1390</td>
<td>72</td>
</tr>
<tr>
<td>Test failure load</td>
<td>2200</td>
<td>100</td>
</tr>
<tr>
<td>Back analysis</td>
<td>2233</td>
<td>102</td>
</tr>
<tr>
<td>Back analysis assessment</td>
<td>1870</td>
<td>85</td>
</tr>
</tbody>
</table>

The features which are different between the non-linear assessment and the back analysis assessment are the use of torsion by the latter and different friction coefficients (0.4 and 0.6 respectively). There could also have been some composite action between the deck slab and the compacted sub-base layer as this was undisturbed over most of the area. Neither method considered this. It should be noted that the non-linear assessment analysis modelled the downstream parapet as an upstand beam. During the test however the deck slab deflected opening a wide crack at the deck/parapet interface and leaving the parapet spanning between the abutments.

The back analysis assessment can be reworked with the same coefficient of friction, 0.4, as the non-linear assessment. This gives a value of 1756 kN which indicates that most of the difference between the two methods is due to the use of torsion.

From the test it was evident that the deck distributed the load laterally far more than was expected. This was despite having beams spaces between flange tips considerably greater than the 2/3rds section depth specified in cl 8.3.1 BS 5400: Pt 5 (1984). The radial crack pattern gave no visual evidence that the slab was of filler beam construction. The recorded midspan displacements were only about 4mm at 1000 kN increasing to 17mm at the maximum load of 1860 kN. Unfortunately in the first test it was not possible to damage the structure sufficiently to determine how the lateral load distribution was taking place.

#### 4.7.2 Test 2

For Test 2 it was possible to load the structure to failure, and the ultimate load capacity can be compared directly with the assessments. The ultimate load of 2904 kN was 1.4 and 1.3 times the predictions from the non-linear assessment and the back analysis assessment respectively and it was 4.5 times the conventional assessment. A strength of 2450 kN, the load at which the deck fractured into three discernable components articulating along the lines of the fractures, would give factors of approximately 1.2, 1.1 and 3.8 times the various assessments respectively and is probably the more realistic figure to use for comparison with similar structures.

The nature of the failure produced by Test 2 led to conclusions about how the structure supported and distributed the load. As described in Section 4.6.1, at the ultimate load the centre section of the slab had fractured into several rectangular concrete blocks supported on the bottom flanges of the joists. There was little longitudinal cracking other than along the lines of the joists. Further experimental work on models could determine the joist spacing limits to this failure mode and discover if at wide spacing the mode changes to a shear or snap-through buckling failure.

The back analysis assessment gives a better fit with the results than the non-linear assessment. This suggests that the distribution of the load laterally was by means of combined torsion and strut action as shown in figure 12 and described in Section 4.6.1. If the distribution is entirely by strut action then lateral restraining forces are required. This is probably one of the reasons that the simple rules for filler beam decks in BS 5400: Pt 5 (1984) require a minimum area of transverse reinforcement. However adequate distribution can be achieved entirely by torsion as was demonstrated by the back analysis of the Thomas and Short (1952) tests in which it was assumed there was no friction.

Composite action with the fill and surfacing, neglected by the assessments, was probably present and some evidence for this was seen at the upstream free edge of the slab. The surfacing separated from the sub-base over a considerable width of the span. However, the sub-base, though of poor material, remained in contact with the slab and showed no signs of disturbance despite significant deflections. It was not possible to quantify the degree of composite action as the top of the slab could not be gauged to obtain strain profiles.

### 5. TESTS ON PEPPER HILLS BRIDGE

Pepper Hills Bridge is in the village of Bolton-by-Bowland, Lancashire (map reference SD 787496). It was instrumented and load tested by TRL during November 1991. The bridge is of filler joist, or filler beam, construction with ten RSJs which are reported to be 203mm x 152mm (8" x 6"), in a 300mm thick concrete slab. There appears to be no transverse reinforcement. It has a clear skew span of 4.80m with a skew of 39° and is 5.635m wide. The deck has a nominal surfacing thickness of 135mm. The bridge had been assessed to BD 21/84 and found to be capable of carrying group 2 FE assessment loading. A general arrangement of the bridge is shown in figure 16.

#### 5.1 THE TESTS

The purpose of the tests was to learn as much as possible about the behaviour the deck under working...
loads, and higher loads if possible. In particular it was hoped to observe the strains due to the local actions of wheel loads and also the articulated panel global behaviour which was postulated in the Wormbridge back-analysis. Neither effect was measured directly at Wormbridge. Pepper Hills bridge was chosen for testing partly because some of its beams are widely spaced so the local effects are more pronounced.

Preliminary versions of the SERB and FILJO programs were available before the tests and they were used to estimate a failure load of 1280 kN under the test bogie wheel configuration. The method assumes a right span and the skew would increase the failure load. This provided the confidence to proceed with the test using a 400 kN bogie. After the tests the programs were developed further and for the next bridge at Langcliffe Cross the prediction increased by 25%.

The deck was instrumented with 22 vibrating wire (VW) strain gauges and 14 displacement gauges as shown on figure 17. The standard strain gauges were 140mm long. They were placed principally to record the pattern of longitudinal strains due to maximum moment in the beams. It was possible to remove sufficient lengths of the cover concrete to fix the strain gauges directly to the undersides of the steel beams. 8 of the 10 beams were instrumented in this way with the interest concentrated on the upstream beams where the spacing is greater. Under a two axle load the maximum moment position is theoretically offset from the midspan position by one quarter of the axle spacing. Nine of the gauges were positioned on the beams 300mm south of the midspan line. Another four were positioned on the underside of the beams adjacent to the abutments to record any abutment restraint. The remainder of the gauges were attached to the concrete soffit. Some special 200mm long VW gauges had been bought so they could span across the beams and record the relative movement between adjacent panels.

Most of the displacement gauge wires were attached to the underside of the beams on the midspan line with their recording transducers on steel stools or spikes in the stream bed directly underneath. Four of the gauges were used to check for possible displacement at the abutments. There was also a dummy displacement gauge recording a fixed length to check for system errors and a temperature gauge.

All the gauges were connected to data-loggers in a mobile laboratory parked on the verge to the north-east of the bridge. From the laboratory the readings could be recorded remotely and a scan of all data for one load position could be completed in about 20 seconds.

The bridge was loaded with the bogie of an articulated lorry. The footprint of the bogie is shown in figure 18 with its dimensions. The trailer had been chosen for bridge testing because it had unusually close axles, only 1.36m apart. With this spacing the legal weight limit is 90 kN per axle. The trailer had been modified with special tyres and strengthening blocks in its suspension so that it could safely carry 200 kN per axle.

The trailer was loaded with concrete blocks held in place with steel angles fixed to the deck. The axle loads were weighed at TRL. First the permanent blocks were placed to make up the maximum legal load for travelling. Then additional blocks were added to give the maximum load for testing. The positions of the additional blocks were recorded and they were then transferred onto two other lorries for the journey. The recorded axle loads are:

<table>
<thead>
<tr>
<th></th>
<th>Travelling</th>
<th>Full load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>front axle</td>
<td>rear axle</td>
</tr>
<tr>
<td>Travelling</td>
<td>87 kN</td>
<td>91 kN</td>
</tr>
<tr>
<td>Full load</td>
<td>187 kN</td>
<td>202 kN</td>
</tr>
</tbody>
</table>

It was shown by calculation that the full load produced moments in the deck strips similar to those produced by full HA loading to BD 37/88 which is 9% greater than the unrestricted assessment loading in accordance with BD 21/84 uprated by its amendment 1, 1989. The term “deck strip” signifies a steel beam together with its associated concrete, extending halfway to each of the adjacent beams. For this analysis the global service model was used with the equations of Cusens and Pama (1975) as described in section 7.

The load was applied to the deck in 17 passes listed below along the 8 loading lines A to H. Figure 18 shows the positioning arrangements. The lorry driver was guided by the marker board which was laid on the roadway. Its arrowhead was set on the midspan line. The centre of the offside rear wheel was used as a reference and it was stopped at the position numbers which were marked on the board. The driver kept the wheels about 50mm - 100mm from the board. For each position this distance was recorded so that for each pass it was possible to estimate the mean actual offset of the reference point from the bridge centre line.

For each pass 14 reading scans were taken. One immediately before the load was on the bridge, then positions 1 to 12 with the X dimension in metres (from the midspan line to the rear wheel) having the respective values of -2.9, -2.3, -1.7, -1.3, -0.9, -0.5, -0.1, 0.3, 0.7, 1.1, 1.7, and 2.3 and finally one immediately after the load had left the bridge.

A full cycle of readings for each line or pass took 15 to 20 minutes. This included the time required for positioning the vehicle. Tests were made with the travelling load first. The offset in metres for each pass completed was:

<table>
<thead>
<tr>
<th>Pass</th>
<th>Travelling</th>
<th>Full load</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1.12</td>
<td>1.19</td>
</tr>
<tr>
<td>B</td>
<td>1.37</td>
<td>1.40</td>
</tr>
<tr>
<td>C</td>
<td>1.54</td>
<td>1.59</td>
</tr>
<tr>
<td>D</td>
<td>1.76</td>
<td>1.75</td>
</tr>
<tr>
<td>E</td>
<td>1.97</td>
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<td>F</td>
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<td>0.75</td>
</tr>
<tr>
<td>H</td>
<td>0.26</td>
<td>0.25</td>
</tr>
</tbody>
</table>

This pass was repeated immediately in the reversed direction.
5.2 THE ANALYSIS

A linear elastic computer grillage analysis was made to provide theoretical results for comparison with the test results. The analysis was made using the GSA program from Oasys Ltd. The model used is shown in figure 19. The lines of pin supports indicate the abutments. The small circles on the transverse members indicate that they are all articulated at one end. This ensures that there are no moments transverse to the line of the beams and imposes the behaviour of a shear-key deck.

The beams are modelled at their measured spacings which are uneven and the stiffness properties are specified separately for each beam, appropriate to their spacing. The torsional stiffness of each concrete strip between adjacent beams is modelled on the beam on the left hand side of it. Following this pattern Beam 10 has no torsional stiffness.

The stiffness properties were calculated assuming full composite action, with the concrete uncracked and using an elastic modulus of 30,000 MPa for the concrete.

The analysis was run under dead load and 37 live load cases, one for each vehicle position with the vehicle on lines A, C and F under full load and one for the assessment loading to BD 12/84.

Figure 20 shows that the wheel loads were applied as point loads on the adjacent beams so there was no attempt to model local moments in the slab between the beams. The axle loads were taken from the weighbridge values with the rear axle taking 8% more than the front axle. Figure 21 shows how the model deflects.

5.3 RESULTS AND INITIAL COMMENTS

A selection of the results are presented here in figures 22 to 52. All except some of those in figure 47 relate to the bridge under full load, referred to as a nominal 40 tonnes. All are displayed in a graphical form. Compressive strains are positive and a downward displacement of the deck is plotted downwards. Calculated predictions from the grillage model are shown where they are available as plain lines with no symbols on them.

5.3.1 Deflections

Figures 22 to 24 show the recorded deflections for the load on lines A, C and F together with the calculated values. In general the peak recorded values are similar to the predictions with some of the recorded values being greater by about 30%. This is consistent with the deck being generally uncracked but with some regions where cracking is developed.

For line C two sets of results are given because an additional set of readings were taken as the vehicle returned over the bridge along the same line. The lateral shift between the two sets of readings can be explained by creep. Figure 23(e) is a good example which shows the results for the first pass with a significant phase shift relative to the calculated response and with a residual deflection of about 20% of the peak value. The phase shift has a magnitude of about 1.0m which is about two load positions or about 3 minutes in time. Note that in figure 23 the datum was taken as the deflection after the load had returned across the bridge.

5.3.2 Longitudinal strains

For each pass under the full load some of the longitudinal midspan strains are shown in figures 25 to 32. Where calculated values are available it can be seen that in general the recorded strains exceed the predictions by about 20%.

The longitudinal influence lines for the gauges 5 and 6 on beams 5 and 6 are shown in each figure in graphs (a) and (b). The results for line C indicate that the phase shift due to creep is much less marked than it is for deflection. This can be explained because the strains recorded are steel strains and the steel does not creep under these conditions. At the gauge position the local concrete has been broken away to expose the beam so, even if the deck were carrying the moments with uncracked behaviour generally, at the gauge it would behave more like a cracked section in which the steel strains are insensitive to the concrete strains.

Wheels pass directly over the gauge positions at beam 6 for line F and beam 5 for line C and the prediction shows a double peak for the two axles which is not exhibited in the readings. In both cases there is a slight indication of a vestigial peak. The discrepancy suggests that the load distribution between the axles was not as it was on the weighbridge. This is quite possible because the suspension of the trailer was short circuited through steel blocks to take the extra load for the test. The blocked-out suspension is very stiff and will be sensitive to minor unevenness in the road surface. The discrepancy could be explained by a redistribution of about 10 kN to the front axle from the rear axle.

In graph (c) of each figure the distribution of midspan strain across the deck is shown at different stages during the pass. For those passes where the results of the grillage analysis are available the comparison is made between the envelopes of maximum values. These comparisons indicate a relationship between the predictions and the results which is more erratic than that for the deflections. This indicates some differences between beams in the internal stress patterns at the gauged sections which is probably due to different degrees of cracking. However, for any particular beam there is no consistent relationship common to the three comparisons. The torsion in the concrete may complicate the response of the cracks to flexure.

Figures 33 to 35 show the influence lines for the bottom flange strains in beam 7 adjacent to both abutments and at midspan. Note that the compressive strains adjacent to the supports are predicted by the grillage model even though it invokes no end fixity or horizontal end restraint. However, the readings are significantly more compressive than the predictions which might indicate a slight degree of end restraint.
5.3.3 Transverse strains

Transverse strains were measured mid-panel between the beams and also using special, longer 200mm gauges which spanned across the beam positions between adjacent panels.

The strains for the 200mm gauges (numbers 13 15 17 & 19) are recorded in figures 36 to 43. They are more difficult to predict. The grillage model has transverse rotation discontinuities along the lines of the beams. To predict strains from these rotations it is necessary to postulate a neutral axis position. The rigid body motion of the panels will also mobilise some lateral restraint which will cause some additional transverse strains. For the predictions shown in the figures it has been assumed that the neutral axis is at the mid-depth of the deck and lateral restraint forces have been ignored.

The concrete under the beams is cracked. In some positions the length of the 200mm gauges may not have been long enough to span onto uncracked concrete in the panels. From the results it appears that gauges 13 and 15 are anchored into sound concrete because they match the predictions quite well for lines F and A respectively. It is not clear why the expected compression becomes a tension for many of the cases.

Gauge 17 appears almost dead although it does respond to the load slightly. It is probably attached to a delaminated section of cover concrete which is separated from a panel by a crack.

Gauge 19 shows more life than gauge 17 but its strain for line F is only about 40% of the predicted value. It also may be on an island.

For the mid-panel strains, peak values have been predicted assuming they span simply supported between adjacent beams and are loaded by the uniform pressure within the spread zone under a wheel patch. Assuming a 1:2 spread through the surfacing and 1:1 to the mid-depth of the slab this gives a tensile strain of 18 microstrain.

Figures 44 to 52 show the recorded values of mid-panel strains. For gauges 14 and 16 the maximum tensile strain is 16 microstrain recorded for gauge 16 under line B. This is a good agreement with the prediction. From figures 17 and 18 it will be seen that in this case the wheel track passes directly over the gauge. Similarly the peak compression of 13 microstrain occurs in gauge 14 for line C when the wheel tracks are on beams 5 and 8, throwing diagonal compressions down towards the bottom flanges of beams 6 and 7 which squeeze the gauge between them.

In contrast the peak tensile stress recorded in gauge 19 is 112 microstrain. Because this is much larger than that predicted for the applied load it is deduced that the panel is cracked and a crack passes through the gauge length. Despite the crack in the unreinforced panel the load was carried because it derived adequate lateral restraint from its adjacent panels.

Figure 52 shows how some of the deflections and strains increase from the travelling load to the full load. The values shown are the maximum values during one pass of the load. The non-linearity indicates that some cracks are developing under full load. The effect is more marked for the deflections than the strains. This is consistent with the usual assumptions about reinforced concrete.

5.4 GENERAL COMMENTS

The results of the test are in reasonable agreement with the predictions of the shear-key deck model as postulated in section 4. It appears that under the full test load the concrete in the deck was slightly cracked.

The articulated behaviour observed implies that under live load there must be some very slight movement of the panels on the flanges of the beams. Over a long period at current working loads this might result in a reduction of the composite action in the deck. Without composite action the flexural capacity is reduced by about 25%.

Under the ultimate load conditions which need to be considered for assessment it is anticipated that the bridge would respond in a similar way to Wormbridge which demonstrated full composite action and favourable redistribution all the way to failure.

Taking into account the results from this test and the test at Wormbridge it is possible to make a provisional assessment of the flexural strength of the deck in accordance with BD 21/84. Using the shear-key deck elastic distribution and assuming a steel yield stress of 230 MPa on the same corroded section used in the original assessment, a concrete cube strength of 25 MPa and composite action but no end restraint. The flexural capacity at ULS is about 55% greater than that required for an unrestricted, 40 tonne loading. This margin removes the concern about the loss of composite action.

At ULS the single wheel required for all assessments above the 7.5 tonne weight restriction imposes a factored load of 165 kN which causes a tension stress in the soffit of about 0.95 MPa. This stress is not small enough to be ignored. Furthermore it appears that at least one of the panels is already cracked. The test only showed that a load of 100 kN could be carried and this was shared between two tyres. The 165 kN load is carried on a cracked panel by raking thrusts to the bottom of the webs of the adjacent beams causing horizontal reactions which must be resisted. Because of the parapet the wheel cannot reach the edge panel so there is always a panel and a half which is able to span the horizontal reaction to the abutments. The parapet, a panel and a half of deck and half of the wheel itself provide sufficient vertical reaction to anchor the force with friction, requiring a friction coefficient of only 0.36.
6. TESTS ON LANGCLIFFE CROSS BRIDGE

Langcliffe Cross Bridge is near the village of Slaidburn, Lancashire (map reference SD 729518). It was instrumented and load tested by TRRL during November 1991. The bridge is of filler joist, or filler beam, construction with ten RSJs which are reported to be 203mm x 152mm (8" x 6") with a cover to the soffit of 50mm, cast in a 330mm thick concrete slab. There appears to be no transverse reinforcement. It has a clear span of 4.25m and is 6.41m wide with a skew of 7°. The make-up and surfacing on the deck has a thickness of about 440mm. The bridge had been assessed to BD 21/84 and found to be capable of carrying group 2 FE assessment loading. A general arrangement of the bridge is shown in figure 53.

6.1 THE TESTS

The purpose of the tests was to learn as much as possible about the behaviour the deck under working loads, and higher loads if possible. In particular it was hoped to observe the strains due to local actions and the articulated panel behaviour which was postulated in the Wormbridge back-analysis. Neither effect was measured directly at Wormbridge.

The deck was thicker than that at Pepper Hills and the span was less. Hence the preliminary analysis for Pepper Hills was deemed also to cover the adequacy of this bridge under the test load of a 400 kN bogie.

The deck was instrumented with 22 vibrating wire (VW) strain gauges and 14 displacement gauges as shown on figure 54. The standard strain gauges were 140mm long. They were placed principally to record the pattern of longitudinal strains due to maximum moment in the beams. Because the cover concrete had debonded, and the bridge soffit was due to be patch repaired, it was possible to remove sufficient lengths of the cover concrete to fix the strain gauges directly to the undersides of the steel beams. Under a two axle load the maximum moment position is theoretically offset from the midspan position by one quarter of the axle spacing. Nine of the gauges were positioned on the beams 300mm south of the midspan line. Another four were positioned on the underside of the beams adjacent to the abutments to record any abutment restraint. The remainder of the gauges were attached to the concrete soffit. Some special 200mm long VW gauges had been bought so they could span across the beams and record the relative movement between adjacent panels.

The soffit of the deck was very wet and it had to be dried locally with hot air before a gauge was fixed. At the position of gauge 21 the soffit was more extensively cracked. This gauge became unbonded before the tests started and it was abandoned.

The displacement gauge arrangement, loading bogie dimensions and loadings were as described in section 5.1.

It was shown by calculation that the full load produced moments in the deck strips similar to those produced by full HA loading to BD 37/88 which is 9% greater than the unrestricted assessment loading in accordance with BD 21/84 uprated by its amendment 1, 1989. The term "deck strips" indicates a steel beam together with its associated concrete, extending half way to its adjacent beams.

The load was applied to the deck in 10 passes along the 6 loading lines A to F. Figure 55 shows the positioning arrangements. The lorry driver was guided by the marker board which was laid on the roadway. Its arrowhead was set on the midspan line. The centre of the offside rear wheel was used as a reference and it was stopped at the position numbers which were marked on the board. The driver kept the wheels about 50mm - 100mm from the board. For each position this distance was recorded so that for each pass it was possible to estimate the mean actual offset of the reference point from the bridge centre line.

For each pass twelve reading scans were taken. One immediately before the load was on the bridge, then positions 1 to 10 with the X dimension in metres (from the midspan line to the rear wheel) having the respective values of -1.6, -0.9, -0.2, 0.2, 0.6, 1.0, 1.4, 1.8, 2.5 and 3.2 and finally one immediately after the load had left the bridge.

Test were made with the travelling load first. The roadway has soft grass verges over the bridge. The range of offsets had to be curtailed against both sides of bridge because, on the final pass under the travelling load, the trailer got bogged down in the verge.

The offset in metres for each pass completed was:

<table>
<thead>
<tr>
<th>Pass</th>
<th>Travelling</th>
<th>Full load</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1.37</td>
<td>1.43</td>
</tr>
<tr>
<td>B</td>
<td>1.10</td>
<td>1.10</td>
</tr>
<tr>
<td>C</td>
<td>0.81</td>
<td>0.77</td>
</tr>
<tr>
<td>D</td>
<td>0.55</td>
<td>0.58</td>
</tr>
<tr>
<td>E</td>
<td>0.26</td>
<td>—</td>
</tr>
<tr>
<td>F</td>
<td>—</td>
<td>1.95</td>
</tr>
</tbody>
</table>

6.2 THE RESULTS

All the results presented in figures 56 to 69 relate to the bridge under full load. Compressive strains are positive and a downward displacement of the deck is plotted downwards.

In many cases calculated values have been included on the graphs for comparison. In these cases the analysis was made on a pocket computer using the shear-key deck equations, Case IV as described by Cusens and Pama (1975). These assume the deck is right and simply supported. The span was taken as 4.32m. The section properties were taken for the uncracked and uncorroded condition. No account was taken in the live load analysis of either the depth of make-up and surfacing on the deck or the stone parapets.
Figures 56 to 60 show midspan displacements. The values take the "before" reading as their datum for each pass of the vehicle so the "after" reading indicates any residual displacement after the vehicle has left the bridge. It was found that the residual value reduced due to creep and it was noted that it was much smaller at the start of the next set of readings. During the testing of the Pepper Hills Bridge, which is similar (see section 5) one of the passes was repeated in the reverse direction. The results showed clearly that there were significant creep strains because the plots from the two passes were skewed relative to each other. In general the residual values were of a similar magnitude to those found at Langcliffe Cross and were consistent with the creep strains demonstrated by the load reversal. Hence there is no evidence of any permanent set due to the load at either bridge.

No adjustment has been made for the recorded displacements on the abutment gauges, numbers 3, 4, 9 and 15. Generally they recorded values less than 3% of the maximum displacement. Gauge 15 gave erratic readings and had to be excluded.

Figures 61 to 69 show strains. Again, in all cases, the "before" reading is taken as the datum.

Measurements under the travelling load are shown in figure 70 and can be compared with the corresponding results under full load. Figure 70(b) indicates that the increase in midspan strains in the beams is slightly greater than that predicted by linear extrapolation. The effect can be explained by a slight amount of cracking. However figure 70(a) shows the deflections under the travelling load to be much smaller proportionately than those under the full load which are themselves about 30% less than the predicted deflections assuming the concrete to be uncracked. The effect is similar for other beams under other loading lines. It is difficult to explain this, particularly as the effect is not apparent in the strain readings.

6.3 COMMENTARY

Even though the analysis used uncracked and uncorroded section properties the calculated values for midspan deflections and longitudinal strains generally exceed the measured values by about 50%. The form of the calculated and recorded plots generally match quite well although it is appears that the stiffness of the stone parapets affects the edges of the deck. This differs from Pepper Hills where the measured values were greater (see section 5.3.2).

The difference can be explained by:

a) Partial composite action with the fill and surfacing on the deck.

b) End fixity, meaning rotational fixity at the supports.

c) End restraint in the form of longitudinal friction at the supports.

It was reported that the fill on the deck consisted of a 220mm layer of cobbles over a 194mm layer of clay. Around the time of the test the weather had been wet and considerable quantities of water had percolated through the deck. It unlikely that these materials could transmit the shear for composite action.

End fixity requires that vertical tensile stresses are resisted in the back face of the abutment and the seating of the deck. This is considered unlikely. End restraint only requires friction at the seating. Some degree of friction will be present and the only question is its magnitude. The back analysis for Wormbridge postulated a coefficient of friction of 1.2 although it was recognised that this might include some cohesion. Rerunning the SERB non-linear beam program for Langcliffe Cross with this friction value gives deflections and midspan bottom flange strains which are both factored down by 0.79. This would give a much better fit with the measured results. It also predicts a compressive strain in the bottom flange adjacent to the abutment although its maximum prediction of 1.5 microstrain is much less than the values shown in figure 66 for gauges 3, 4, 7 and 9. This discrepancy can be explained because the gauge positions are very close to the contact point and the force will be more concentrated than the prediction assuming "plane sections remain plane".

Figures 67 and 68 show transverse influence lines for the strains in the 200mm strain gauges which span transversely across the beam positions. Also shown are the influence lines derived by calculation assuming that the deck articulates along the line of the beams with the deck panels between beams remaining straight when viewed in the deck section. The beam deflections are those derived using the shear-key deck assumptions. No allowance was made for the effects of any transverse in-plane forces in the deck. In this calculation it is not clear where the neutral axis should be taken. Arbitrarily it was taken at the mid-depth of the steel beam.

From the results it is clear that only gauge 15 gave results which, to any extent, followed the predictions. It was known that the pattern of cracks in the cover concrete beneath the beams extended over an area beyond the 150mm width of the beam flanges. It had been hoped that 250mm gauges could have been used but none were available. In fixing the 200mm gauges care was taken to choose positions where all visible cracks were spanned by the gauge length. However it appears that for gauges 13, 17 and 19 one of their ends must have been attached to a delaminated section of cover concrete separated by microcracks. At gauge 17 the island may have been pivoting about its middle because the measured results look like a mirror image of the calculated values.

Because of these experimental problems it is particularly encouraging to see that the results for gauge 15 (across beam 7) look reliable. It appears that the strains are consistent with the measured beam deflections rather than the calculated ones. For example under pass D a small compressive strain is expected but a tensile strain of about 200 microstrain is recorded. This is consistent with the top graph in figure 59 which shows a slight concavity predicted in the soffit at beam 7 whereas the
recorded shape shows a marked convexity at this point. This gauge gives evidence supporting the articulated/shear-key model for the deck behaviour under service loads. Furthermore it appears that the assumption that the neutral axis is at mid-depth is approximately correct.

In the articulated model there will be significant torsional stresses in the panels but the only transverse flexural stresses will be due to the local transfer of wheel loads to the adjacent beams. If the wheel loads are dispersed through the 100mm tarmac at 1:2 with no dispersal through the cobbles and clay and at 1:1 through half the depth of the slab (BD 37/88: 6.3.3) then the effective distributed load on the panel is 161 kPa giving a transverse tensile strain mid-panel of 16 microstrain. This ties in quite well with the results in figure 69 which shows mid-panel transverse strains varying up to 20 microstrain in tension which implies a tension stress of about 0.6 MPa. The compressive strains will result from compressive stresses resisting the rigid body articulation of the panels. It is not easy to put figures on the degree of restraint present.

The pattern of the strains agrees qualitatively with the expected pattern considering the relative positions of the wheel tracks. For example gauge 14 is between beams 6 and 7. Figure 55 shows that pass A gave the most direct wheel load on this panel and passes C and D put no load on it. The results show no transverse compressions when the vehicle straddles the panel but quite large compressive strains between the vehicle and the edge of the deck. This is consistent with a transverse arching acting in the deck. Between the wheels the compression is near the top of the slab and is not recorded on the soffit gauges.

The articulated behaviour observed implies that under live load there must be some very slight sliding of the panels on the flanges of the beams. Over a long period this could lead to wear which might impair the composite action in the deck. Without composite action the flexural capacity is reduced by about 25%.

Taking into account the results from this test and the test at Wormbridge it is possible to make a provisional assessment of the flexural strength of the deck in accordance with BD 21/84. Using the shear-key deck elastic distribution and assuming a steel yield stress of 230 MPa, a concrete cube strength of 25 MPa and composite action and ignoring end restraint we find that the flexural capacity at ULS is about 28% greater than that required for an unrestricted, 40 tonne loading. This margin removes the concern about the loss of composite action.

At ULS the single wheel required for all assessments above the 7.5 tonne weight restriction imposes a factored load of 165 kN which causes a tension stress in the soffit of about 0.67 MPa. This is much less than the likely tensile strength of the concrete but code methods require concrete in tension to be ignored. Without tension the wheel is carried by inclined thrusts rising from the bottom flange of the beams where they meet the web. Even if the wheel is at midspan the local outward horizontal reaction can be spanned onto the abutments by in-plane spanning in the slab with the beams acting as tension reinforcement. At the abutment it is anchored by friction from the vertical load of the parapet, a bay and half of the slab and half of the wheel itself. The required friction coefficient is 0.25 which is easily mobilised. Hence the bridge can take the 100 kN wheel.

The local action discussed above is a local flexural failure. The possibility of a local punching shear failure has also been considered. However the punching perimeter always encloses the adjacent beams so there is no possibility of such a failure.

7. ANALYTICAL MODELS

Four models are required to describe separately:
- global behaviour at failure
- global behaviour in service
- local behaviour at failure
- local behaviour in service.

The model of global behaviour at failure is described in Section 4.6. To fit the test result at Wormbridge it was necessary to invoke a significant amount of horizontal end restraint at the abutments. The model is also able to allow for the effect of composite action with the deck topping or surfacing although this facility was not used. When interpreting test results it is difficult to distinguish between these two effects. Neither effect was present in the test devised by Thomas and Short (1952). This analysis procedure would be unwieldy in routine assessment.

The model predicts a reduction in capacity as concentrated loads are moved towards an edge of the deck. The effect is quantified in figure 70. In the Wormbridge tests the load was positioned with at least five beams between it and the edge. For the Thomas and Short tests the distance was greater. When vehicles are positioned closer to the edge it would be prudent to apply an additional partial factor.

The model of global behaviour in service is an elastic model which is consistent with the failure model. As in a shear-key deck, distribution is achieved through torsion with no transverse moments. Its use is described in Section 5.2 where Pepper Hills Bridge is analyzed as a computer grillage and also in Section 6.2 where a much quicker analysis is made of Langcliffe Cross Bridge using the continuum equations of Cusens and Pama (1975). Uncracked section properties are used. The model has been tested against data from Wormbridge, Pepper Hills and Langcliffe Cross and gives reasonable predictions.

The global elastic model may be used by engineers making serviceability checks but this is a relatively infrequent requirement. Because its predicted distributions are slightly less favourable than those given by the
failure model its assessments are inherently safe. Hence this model provides a practical method for making a global analysis for an assessment at the Ultimate Limit State (ULS). The seemingly questionable use of uncracked section properties in this analysis is justified not because the deck is uncracked but rather because the distribution resulting is slightly more conservative than that inferred during the late, cracked stages of the Wormbridge test. When using this model at ULS it should be remembered that there are no test data for the case with the live load close to the edge of the deck and some additional conservatism should be applied in this case. The concern is that the edge beam might be pushed sideways off the abutment shelf. If the form of the abutment precludes such a failure or if transverse ties are used then this concern is removed. In this respect filler beam decks without transverse reinforcement could be strengthened by providing lateral restraint at any point in the span, using for example bonded plates or prestressing.

If the deck is cast directly onto the abutment without any bearings there will be some degree of horizontal end restraint acting on the deck. In the back-analysis of Wormbridge this increased the failure load by 25% so there is the potential to increase an assessment capacity by a similar amount. However it must be remembered that back-analysis is a speculative procedure and the failure model is not sufficiently substantiated in its treatment of this effect to be used in routine assessment. End restraint is present in many traditional forms of bridge deck and will often give benefits much greater than the 25% quoted here. It is hoped that further studies and tests will be made so that this effect can be invoked in assessment.

If the global elastic model were to be used in conjunction with strength enhancement due to horizontal end restraint then it should be recognised that this effect significantly reduces deflections and this should be modelled with extra flexural stiffness in the longitudinal members.

A model of local behaviour at failure has been described in section 6.3. It has not been tested at full load. Currently for assessments it relies on arguments based on the first principles of concrete in diagonal compression transferring the load to the bottom flanges of the adjacent beams, horizontal spanning action in the deck plate resisting the outward thrust and transverse friction at the abutments. This failure becomes significant when the beam spacing-to-depth ratio is high. In one of the panels at Pepper Hills, where the ratio was the high value of 3.15, the concrete was cracked (Section 5.3 and 5.4) so it was assumed the service behaviour matched that of this failure model. The local load intensity in the test was high enough to count as an applied ultimate load for a 3 tonne restriction but not for a higher one.

The model of local behaviour in service is an uncracked slab. It has been tested against data from Pepper Hills and Langcliffe Cross and gives reasonable agreement. However this model has no role in assessment. If concrete in tension is ignored then it behaves as the failure model described above.

8. CONCLUSIONS AND RECOMMENDATIONS

1. Filler beam bridges without transverse reinforcement can carry heavy vehicle loads despite their exclusion from current code methods.

2. In all the collapse tests and the non-linear predictions the global failure mode of filler beam decks without transverse reinforcement was flexural and ductile. Serviceability problems would be apparent long before the ultimate limit state was reached. The term “global failure” is used to mean all failures except the local failure discussed below.

3. A method has been developed which predicts the global failure load of these decks under concentrated load patterns.

4. Despite having no apparent transverse bending capacity the decks were able to distribute both working and ultimate loads well. The good distribution characteristics indicate that it is appropriate to assess this type of deck using the standard assessment loading with full width lanes rather than the concentrated wheel path loading used for trough decks.

5 (The Principal Conclusion). The favourable distribution characteristics (see 4.6.3) indicate that it is safe to design or assess filler beam decks using an elastic load distribution based on a shear key deck model with uncracked section properties. The authors recommend this form of distribution for assessments under standard highway loading.

6. The non-destructive load tests on the bridges at Pepper Hills and Langcliffe Cross have confirmed that the mechanism for the global distribution of concentrated loads continues to be valid with considerably greater beam spacing to depth ratios than those at Wormbridge. However these bridges were not taken to failure and caution should be exercised when assessing bridges with a ratio greater than 2.2.

7. Under the local action of wheel loads between beams the structure should be assessed ignoring the tensile strength of the concrete by considering diagonal compressions and friction at the abutments (see sections 6.3 and 7 (last two paragraphs)). The critical case will be when the wheel load is central in the panel which is closest to a free edge. However if the beam spacing to depth ratio is less than 2 no check for local failure is required.

8. A particular form of local failure is edge failure in which the edge beam is able to separate from the concrete and possibly buckle. If the edge beam is surrounded in sound concrete this is unlikely but a judgement is required for each situation.

9. Bridges of this type could be strengthened against local and edge failure by providing transverse reinforcement, bonded plates or prestressing at any point in the
span, either below the soffit or within the structural depth. This form of strengthening could also be used to justify global load distributions better than those given by the “principal conclusion” (see sections 4.6 and 7.1).

10. For the back-analysis of Wormbridge composite action was invoked to explain the high load capacity observed. The bond stress required to support this assumption was about twice the SLS limit given in BD 16/82. There are no implications for the code limit.

11. For many bridges (not only filler beam bridges) it is possible that longitudinal friction restraint at the abutments can significantly increase the capacity of each strip of bridge (see section 4.6.2 and 7.1). This is relevant under the distributed loads used for assessment and should be investigated further.

9. ACKNOWLEDGMENTS

The work was carried out in the Bridges and Ground Engineering Resource Centre of the Transport Research Laboratory.

The authors would like to thank the Director of the DOT West Midlands Regional Office and Mr V E Jones the County Engineer, Hereford and Worcester, for making the bridge at Wormbridge available for testing. This test was supported financially by Bridges Engineering Division of the Department of Transport.

The tests on Pepper Hills and Langcliffe Cross bridges were carried out for Lancashire County Council with financial support from the County Surveyors’ Society.

10. REFERENCES


Masonry parapet wall

6-1 mass concrete abutment

12" x 5" x 21' 0" R.S.J.s @ 1' 6" c.c.

Fig. 1 Plan
6-1 mass concrete deck

12" x 5" x 21' 0" R.S.J.s @ 1' 6" c.c.

6.1 mass concrete abutment

1' 5"

3' 0"

3' 0"

Fig. 2 Section A–A

Fig. 3 Downstream elevation
Fig. 4 Plan showing gauge positions
Stressing jack
300 tonne capacity

Dimensions in millimetres

Load cell
Load pads

Load cell
Load pads

Fig. 5 Wormbridge: Test rig No.1
Fig. 6 Test 1: Soffit of slab showing cracking
Fig. 7 Load/displacement graphs: Gauges 1 to 8 : Test 1
Fig. 8 Wormbridge: Test rig No. 2

Stressing jacks: 300 tonne capacity each

Load cells

Load pads
Fig. 9 Test 2: Soffit of slab showing cracking
Fig. 10 Load/displacement graphs: Gauges 1 to 7: Test 2
Selected plots from the above graphs for comparison

Fig. 11 Displacement/load distributions: Test 2 – Mid span displacement gauges 1–7
Fig. 12 Combined model with raking strut and torsion

Fig. 13 Failure prediction as load moves toward edge of deck
$\dot{\sigma}_y = 237 \; \mu = 1.2$

$\dot{\sigma}_y = 229 \; \mu = 0.6$

$\dot{\sigma}_y = 229 \; \mu = 0$

$\dot{\sigma}_y$ = Lower yield stress

$\mu$ = Coefficient of friction

**Fig. 14** Load/deflection graph from ERB

**Fig. 15** Transverse displacement profile
Overall deck length 5750 ave. between abutments 560.

Outer edge of deck 185.

Centres of joists 125.

Edge of deck 4800 clear skew span.

All beams 8" x 6" RSJ's. Ends of joists not proven.

PLAN OF DECK SHOWING LAYOUTS OF JOISTS
SCALE 1:50

Stone parapet 350 wide
Carriageway
Square distance between parapets 4935
Stone parapet 350 wide

Parapet 950 nominal height
Macadam surfacing
100.110
100.130
100.090
Trial hole
99.715

Concrete broken away from beneath bottom flanges of outer joists (exposed flange coated with bitumastic paint).

Fig.16 Pepper Hills Bridge—general arrangement
Displacement gauge
Concrete broken out for 5 1/2" VWSG on flange
5 1/2" VWSG (vibrating wire strain gauge) on concrete
8" VWSG on concrete soffit

SCALE 1:50

Fig.17 Pepper Hills Bridge—gauge positions
Fig. 18 Pepper Hills Bridge—wheel positions
Plan showing articulation
Elements: All

Fig. 19 Plan of computer grillage model
Fig. 20 The test load as modelled on the grillage, line F, position 8

Fig. 21 Grillage deflections due to the load in Fig. 20
Fig. 22 Deflections for line A
Fig. 23 Deflections for line C
Fig. 24 Deflections for line F
Fig.25 Mid-span strains on beams with load on line F

- Pepper Hills Bridge Test
a) Longitudinal influence line for Gauge # 5

Load = 40 tonnes

Dimension X from reference wheel to centre - line (m)

b) Longitudinal influence line for Gauge # 6

Load = 40 tonnes

Dimension X from reference wheel to centre - line (m)

c) Mid-span strain for given load position

Load = 40 tonnes

Dimension Y, transverse position on bridge (m)

Fig. 26 Mid-span strains on beams with load on line E
-Pepper Hills Bridge Test
a) Longitudinal influence line for Gauge # 5

Load = 40 tonnes

b) Longitudinal influence line for Gauge # 6

Load = 40 tonnes

c) Mid-span strain for given load position

Load = 40 tonnes

Fig. 27 Mid-span strains on beams with load on line D

-Pepper Hills Bridge Test
Fig. 28 Mid-span strains on beams with load on line C
– Pepper Hills Bridge Test
a) Longitudinal influence line for Gauge # 5

Load = 40 tonnes

Dimension X from reference wheel to centre - line (m)

b) Longitudinal influence line for Gauge # 6

Load = 40 tonnes

Dimension X from reference wheel to centre - line (m)

c) Mid-span strain for given load position

Load = 40 tonnes

Dimension Y, transverse position on bridge (m)

P5  P6  P7  P8  P9  Calculated

Fig.29 Mid-span strains on beams with load on line B
–Pepper Hills Bridge Test
Fig. 30 Mid-span strains on beams with load on line A
Pepper Hills Bridge Test
a) Longitudinal influence line for Gauge # 5

Load = 40 tonnes

Dimension X from reference wheel to centre - line (m)

b) Longitudinal influence line for Gauge # 6

Load = 40 tonnes

Dimension X from reference wheel to centre - line (m)

c) Mid-span strain for given load position

Load = 40 tonnes

Dimension Y, transverse position on bridge (m)

Fig.31 Mid-span strains on beams with load on line G

-Pepper Hills Bridge Test
a) Longitudinal influence line for Gauge # 5

Load = 40 tonnes

b) Longitudinal influence line for Gauge # 6

Load = 40 tonnes

c) Mid-span strain for given load position

Load = 40 tonnes

Fig.32 Mid-span strains on beams with load on line H
–Pepper Hills Bridge Test
Fig. 33 Strains along beam 7 with load on line F
-Pepper Hills Bridge Test
Fig. 34 Strains along beam 7 with load on line C
–Pepper Hills Bridge Test
Fig. 35 Strains along beam 7 with load on line A
Pepper Hills Bridge Test
Fig. 36 Strains in 8" gauges with load on line F
-Pepper Hills Bridge Test
Fig. 37 Strains in 8" gauges with load on line E
-Pepper Hills Bridge Test
Fig. 38 Strains in 8" gauges with load on line D
–Pepper Hills Bridge Test
Fig. 39 Strains in 8" gauges with load on line C
- Pepper Hills Bridge Test
Fig. 40 Strains in 8" gauges with load on line B
Pepper Hills Bridge Test
Fig. 41 Strains in 8" gauges with load on line A
  - Pepper Hills Bridge Test
Fig. 42 Strains in 8" gauges with load on line G – Pepper Hills Bridge Test
Fig. 43 Strains in 8" gauges with load on line H
- Pepper Hills Bridge Test
Fig. 44 Transverse strains in panels with load on line F
-Pepper Hills Bridge Test
Fig. 45 Transverse strains in panels with load on line E
—Pepper Hills Bridge Test
a) Longitudinal influence line for Gauge # 14
Load = 40 tonnes

b) Longitudinal influence line for Gauge # 16
Load = 40 tonnes

c) Longitudinal influence line for Gauge # 18
Load = 40 tonnes

Fig. 46 Transverse strains in panels with load on line D
—Pepper Hills Bridge Test
a) Longitudinal influence line for Gauge #14

Load = 40 tonnes

Fig. 47 Transverse strains in panels with load on line C
- Pepper Hills Bridge Test
Fig. 48 Transverse strains in panels with load on line B
-Pepper Hills Bridge Test
a) Longitudinal influence line for Gauge # 14
Load = 40 tonnes

b) Longitudinal influence line for Gauge # 16
Load = 40 tonnes

c) Longitudinal influence line for Gauge # 18
Load = 40 tonnes

Fig. 49  Transverse strains in panels with load on line A
-Pepper Hills Bridge Test
Fig. 50 Transverse strains in panels with load on line G
-Pepper Hills Bridge Test

**a) Longitudinal influence line for Gauge # 14**

Load = 40 tonnes

**b) Longitudinal influence line for Gauge # 16**

Load = 40 tonnes

**c) Longitudinal influence line for Gauge # 18**

Load = 40 tonnes
a) Longitudinal influence line for Gauge #14

Load = 40 tonnes

Test

b) Longitudinal influence line for Gauge #16

Load = 40 tonnes

Test

c) Longitudinal influence line for Gauge #18

Load = 40 tonnes

Test

Fig. 51 Transverse strains in panels with load on line H
   —Pepper Hills Bridge Test
Fig. 52. Variations with load
-Pepper Hills Bridge Test
Fig. 53 Langcliffe Cross Bridge—general arrangement
Displacement gauge
- Concrete broken out for 5½" VWSG on steel flange
- 5½" VWSG (vibrating wire strain gauge) on concrete
- 8" VWSG on concrete soffit

Fig. 54 Langcliffe Cross Bridge—gauge positions
Fig. 55 Langcliffe Cross Bridge—wheel positions
Fig. 56 Mid-span deflections, line A 
Langcliffe Cross Bridge Test
Fig. 57 Mid-span deflections, line B
- Langcliffe Cross Bridge Test
Fig. 58 Mid-span deflections, line C
-Langcliffe Cross Bridge Test
Fig. 59 Mid-span deflections, line D  
-Langcliffe Cross Bridge Test
Fig. 60 Mid-span deflections, line F – Langcliffe Cross Bridge Test
Fig. 61 Longitudinal steel strains near mid-span, line A
–Langcliffe Cross Bridge Test
Fig. 62 Longitudinal steel strains near mid-span, line B
-Langcliffe Cross Bridge Test
Fig. 63 Longitudinal steel strains near mid-span, line C
– Langcliffe Cross Bridge Test
Fig. 64 Longitudinal steel strains near mid-span, line D
Langcliffe Cross Bridge Test
Fig. 65 Longitudinal steel strains near mid-span, line F
– Langcliffe Cross Bridge Test
Fig.66 Steel strains along beams 4 and 7, line A
-Langcliffe Cross Bridge Test
Fig. 67 Influence lines for gauges 13 and 15
– Langcliffe Cross Bridge Test
Fig. 68 Influence lines for gauges 17 and 19
– Langcliffe Cross Bridge Test
FIG. 69 Influence lines for gauges 14, 16 and 18
—Langcliffe Cross Bridge Test
Fig. 70 Variations with load
–Langcliffe Cross Bridge Test