Investigation of methods of achieving continuity in composite concrete bridge decks

by B P Pritchard and A J Smith
(W S Atkins Consultants Ltd)

The work reported herein was carried out under a contract placed on W S Atkins Consultants Ltd by the Transport and Road Research Laboratory. The research customer for this work is Bridges Engineering Division, DTp.

This report, like others in the series, is reproduced with the authors' own text and illustrations. No attempt has been made to prepare a standardised format or style of presentation.

Copyright Controller of HMSO 1991. The views expressed in this Report are not necessarily those of the Department of Transport. Extracts from the text may be reproduced, except for commercial purposes, provided the source is acknowledged.

Bridges Division
Structures Group
Transport and Road Research Laboratory
Old Wokingham Road
Crowthorne, Berkshire RG11 6AU

1991

ISSN 0256-7045
Ownership of the Transport Research Laboratory was transferred from the Department of Transport to a subsidiary of the Transport Research Foundation on 1st April 1996.

This report has been reproduced by permission of the Controller of HMSO. Extracts from the text may be reproduced, except for commercial purposes, provided the source is acknowledged.
EXECUTIVE SUMMARY

INVESTIGATION OF METHODS OF ACHIEVING CONTINUITY IN COMPOSITE CONCRETE BRIDGE DECKS

Many UK multi-span bridges are constructed as a series of simply-supported pretensioned precast concrete beams composite with an in-situ reinforced concrete deck slab. The majority feature deck expansion joints over each pier. Deck and substructure corrosion damage caused by winter de-icing salt has been identified as a serious and potentially expensive problem for UK bridges (Reference 20). Other investigations have indicated that many bridge deck expansion joints are not only maintenance-prone, but also contribute to this problem.

DTp is preparing an advice note which will encourage the use of continuous multi-span bridge decks in order to minimise the numbers of expansion joints in new designs.

TRRL therefore commissioned a study by WS Atkins Consultants Limited to investigate and recommend methods of building-in deck continuity for this type of bridge in order to reduce the number of deck joints. This report covers that investigation and it is expected that some of its recommendations will be included in the DTp advice note.

The investigation commenced with data collection on the various continuity methods proposed or in use. This involved a literature search and correspondence with UK, and a limited number of international, bridge authorities, ranging from UK County Councils and American States to UK precast pretensioned beam manufacturers.

Five distinct methods of building-in continuity to composite precast pretensioned bridge decks in the UK and America were identified and are described below. The methods have been in use for up to thirty years and a significant number of structures have been built, most apparently demonstrating the enhanced durability to be expected with the elimination of intermediate deck joints. Some recent American examples also eliminated deck-end joints by building-in to the abutments, creating what are now termed 'integral bridges'.

The methods have been classified as the following types:-

- **Type 1 Wide in-Situ Integral Crosshead**

  Precast beams shorter than the spans and temporarily supported by trestles built off the pier foundations, are made continuous by embedment in wide in-situ integral crossheads permanently supported on single rows of bearings over the piers.

- **Type 2 Narrow In-situ Integral Crossheads**

  Precast beams are supported on twin rows of permanent bearings seated on the piers, and embedded on narrower integral crossheads. In some cases the twin rows of bearings may be temporary, with support eventually transferred to a single row of permanent bearings.

- **Type 3 Integral Crosshead Cast in Two Stages**

  A variant of types 1 and 2, where the integral crosshead is cast in two stages.
Type 4 Continuous Separated Deck Slab

Only the deck slab is made continuous over the piers, its flexure being locally isolated from the precast beams by a layer of compressible material.

Type 5 Tied Deck Slab

The deck slab is hinged over the piers using dowelling which is partly debonded to accommodate flexure.

All five methods used reinforcement located in the deck slabs over the piers to eliminate deck expansion joints. The degrees of moment continuity thereby established ranged from full, excluding beam self weight, with beams embedded in integral crossheads, to nil for the hinged tied deck slabs and continuous separated deck slabs.

A further proposed method using prestressing located in the deck slab over the piers was also identified and designated Type 6.

The report provides a commentary comparing all six methods under twelve headings, ranging from Design Complexity to Comparative Costs. It takes into account both UK and American practice and adds valuable information from the recently published American NCHRP Report 322 "Design of Precast Prestressed Bridge Girders Made Continuous" (Reference 19). This Report, undertaken over three years, and devoted mainly to the Type 2 connection used almost exclusively in America, is important enough to merit a contents summary as an Appendix.

The various comparisons were made against similar properties measured for simply supported multi-span composite bridge decks.

Design and construction complexity was generally greatest with Type 6, followed in descending order through Types 1 to Type 5, which offered little or no complexity. Using the recommendations of the American NCHRP Report 322, the design and construction complexity of the Type 2 method could be significantly reduced by eliminating complicated bottom continuity reinforcement over the piers.

As regards deck geometry, skews up to 50° had been successfully built using Types 1, 2 and 4. Plan curvature was also readily accommodated by Type 1.

Type 1 also offered easily detailed deck drainage for long multi-span viaducts and considerable savings in numbers of bearings and sizing of piers and foundations.

The removal of the deck joints appeared to improve durability and indications from the very small sample survey were that Types 1 to 5 were performing well, with Type 5 marginally the least durable.

Aesthetics were also considered. The removal of water and salt staining from leaking deck joints would improve appearance and the reduction in deck depths and pier sizes possible with Types 1 and 2 could also give aesthetic benefits.
All Types of continuity are likely to give construction cost savings over similar simply supported decks, with Types 1 - 3 offering most saving, possibly between 5 and 10%.

The report concludes with a recommendation that all 5 types of connection previously built (Types 1 - 5) should be recommended in the DTp advice note, setting out the advantages and disadvantages of each for user choice.

Other recommendations included further research and adoption and simplification of some of the NCHRP 322 Report proposals.
CONTENTS

1 INTRODUCTION
1.1 The Problem
1.2 Scope of the Study

2 DATA COLLECTION ON CONTINUITY METHODS
2.1 Literature Search
2.2 Correspondence with UK and International Authorities
2.3 Collected Data

3 METHODS OF ESTABLISHING CONTINUITY
3.1 General
3.2 Type 1 - Wide In-situ Integral Crosshead
3.3 Type 2 - Narrow In-situ Integral Crosshead
3.4 Type 3 - Integral Crosshead Cast in Two Stages
3.5 Type 4 - Continuous Separated Deck Slab
3.6 Type 5 - Tied Deck Slab
3.7 Other Methods

4 COMPARISON OF METHODS
4.1 General
4.2 Design Complexity
4.3 Construction Complexity
4.4 Moment Transfer
4.5 Skew
4.6 Plan Curvature
4.7 Deck Drainage
4.8 Bearings and Piers
4.9 Compliance with BS 5400
4.10 Relevant Research and Load Testing
4.11 Durability
4.12 Aesthetics
4.13 Comparative Costs

5 RECOMMENDATIONS
5.1 Further Investigations
5.2 Most Suitable Continuity Details to be Promoted for UK

6 ACKNOWLEDGEMENTS

7 REFERENCES

FIGURES 1 - 14

APPENDIX A - Summary of Structures Identified by Survey

APPENDIX B - Commentary on NCHRP Report 322
1 INTRODUCTION

1.1 The Problem

Corrosion caused by winter de-icing salts is currently the most damaging threat to the integrity of UK bridges. It requires ever-increasing expenditure on inspection, assessment, maintenance and repair.

Recent Department of Transport (DTp) investigations (Reference 20) have confirmed that most bridge deck expansion joints leak and contribute more than any other bridge element to deck and substructure corrosion damage. The joints themselves also have a poor maintenance record.

DTp is currently preparing an advice note which will encourage the use of continuous multi-span bridge decks in order to minimise the number of expansion joints. To this end the Transport and Road Research Laboratory (TRRL) commissioned WS Atkins Consultants Limited (WSACL) to undertake a desk study on the ways in which continuity can be provided for composite bridge decks where the primary beams are of pretensioned precast concrete.

The initial 3 month study commenced in July 1989. The Contract was subsequently extended in January 1990 to include a review of the American NCHRP Report 322 (Reference 19).

1.2 Scope of the Study

The study consisted of four elements, as follows:

a) obtaining details of the various methods of providing continuity which have been used or proposed worldwide by means of a literature search and by contact with Highway Authorities and bridge designers. Details of each type listed to illustrate the method used;

b) provision of a commentary on each of the details provided on matters such as complexity of design, limitations of moment transfer, compliance with BS 5400, associated research and testing, ease of construction, durability, anticipated or reported performance, and comparative cost;

c) suggestions for achieving continuity which have not yet been applied or published and any future research;

d) recommendations towards the most suitable method(s) to be promoted for the UK.

2 DATA COLLECTION ON CONTINUITY METHODS

2.1 Literature Search

A literature search was carried out using the IRS/Dialtech Service (File 4 - Compendex and BRIX) whilst TRRL supplied details of a similar search carried out through the IRRD service. The input key words were 'bridge deck continuity' and 'precast beams'. The results were disappointing and indicated little more of interest than the references that were
already included in BCA’s recent publication "Composite Concrete Bridge Super-structures" (Reference 2).

TRRL supplied a copy of a PhD Thesis submitted to the University of Leeds in January 1989 entitled "Continuity Development Between Precast Beams Using Prestressed Slabs" (Reference 12) by A D C Jayanandana. This also contained a useful set of references.

All references are given in Section 7.

2.2 Correspondence with UK and International Authorities

WSACL used their personal contacts with UK Local Authorities, Consultants and Precast Pretensioned Beam Manufacturers in seeking relevant information. There was also considerable information in-house and from WSACL regional offices.

A good response resulted in sufficient information and drawings of bridges built and continuing to be built using precast beam continuity. Although the survey cannot be considered to be comprehensive it is felt that it is representative enough for the purposes of this report.

WSACL’s knowledge of international use of bridge deck precast beam continuity is limited to the USA and Holland. Letters concerning USA practice were sent to AASHTO, FHA and Professor Klaiber at Iowa State University. Replies were received from AASHTO and FHA, resulting in a long list of States using this method of bridge deck construction. A selected number were written to, and two replies were received, from Florida and Illinois. Professor Klaiber provided details of research in the United States (Reference 19).

Further international uses of precast beam deck continuity also came to light from several of the UK consultants.

A list of organisations that contributed data is included in Section 6.

2.3 Collected Data

The collected data has been assembled into tabular form and is attached as Appendix A. One set links the type of connection, the designer/authority, the structure name and the date of construction. The other set shows the connection details with leading dimensions.

3 METHODS OF ESTABLISHING CONTINUITY

3.1 General

Five distinct methods of building-in continuity to composite precast pretensioned bridge decks in the UK were established from the data collection. Some of the methods are also used internationally.

The classification of the methods indicated that two make only the deck slab continuous or hinged over the piers whilst the other three provide full monolithic continuity connection of the beams and slab over the piers.
All methods require transverse beams at or near the piers. These beams can be arranged as twin, one either side of the pier, or singly, located over the pier. Transverse beam depth can be less than, equal to, or greater than the longitudinal beam depth.

For clarity in this report, single transverse beams located over the pier and pier bearings are designated as integral crossheads. All other transverse beams are designated as diaphragms.

3.2 Type 1 - Wide In-situ Integral Crosshead

This type of continuity detail, shown in Figure 1, uses precast pretensioned beams significantly shorter than the spans between support piers. The beams are usually supported on temporary trestles, built off the pier foundations. The wide in-situ integral crosshead over the pier is then cast between and around the beams to provide about 1 metre embedment. Longitudinal continuity is accomplished by reinforcement within the continuous composite deck slab generally supplemented by reinforcement and sometimes, pretensioning strand ends extending from the top and bottom of the embedded beams. Transverse strength is assured by either prestressing tendons or reinforcement, some of which may pass through holes in the ends of the precast beams. The crosshead is supported on a single row of bearings set centrally on the pier.

For UK organisations this detail was the most common discovered, with 20 examples. Spans range from 12 to 35 m and skew angles from 0° to 48°. Beams are generally U or M type, spaced or contiguous. There are examples of considerable plan curvature.

3.3 Type 2 - Narrow In-situ Integral Crosshead

Figure 2 shows this continuity detail. The precast pretensioned beams are long enough to be erected on two parallel rows of temporary or permanent bearings, or one row of permanent bearings on the pier tops. As with Type 1, the in-situ integral crosshead over the pier is then cast between and around the beams to provide about 1 metre embedment. The crosshead is, however, narrower than Type 1 because of the necessarily small gap between the embedded beams. The same narrow gap makes adequate reinforcement connection difficult between beam ends. Longitudinal hogging bending continuity is again readily established by top reinforcement within and extending well into the continuous composite deck slab. Transverse strength of the crosshead is generally provided for by reinforcement, some of which passes through holes in the ends of the precast beams.

Where twin rows of temporary bearings are used, a central row of permanent bearings located under the crosshead is brought into use by removing the temporary bearings after the crosshead concrete has gained sufficient strength.

For this detail, 7 examples are listed from UK organisations. Spans range from 10 to 20 m with skews up to 30° and no plan curvature. Beams are generally U or M type, spaced or contiguous.

The Florida authorities provided drawings of a chronology of evolution of Type 2 precast beam continuity which commenced in 1979. Typical beams are American bulb tees 6 ft (1.83 m) deep spanning 143 ft (43.6 m), the connection using bottom overlapping dowel or hooked bar reinforcement in the narrow gaps between the beams, which sit on twin rows of bearings.
The Illinois Department of Transportation indicated that they have built a large number of continuous span bridges using precast prestressed concrete I beams since the late fifties. The Type 2 connection detail uses protruding overlapping hooked beam reinforcement in the narrow 6" gap between beams, which again are seated on twin rows of bearings. They state further that:

"...Numerous modifications were made in 1983 to alleviate the cracking problems we were experiencing in the diaphragms. To our knowledge the current details are performing to expectations."

Examination of the current continuity detail shows one interesting change to the original Type 2 arrangement. This requires the bonding of roofing felt to the sides of the beam embeddings within the diaphragms, presumably as bond-breaking devices.

If United States practice is included then this method is at least as common as Type 1. United States practice also extends the span range to in excess of 40 m.

3.4 Type 3 - Integral Crosshead Cast in Two Stages

This type of continuity detail, Figure 3, is a variant of Types 1 or 2 where the integral crosshead is cast in two stages. The crosshead is of greater depth than the main deck precast beams and the bottom section is cast first to support these beams on thin mortar beddings. The second stage completes the integral crosshead in the manner described in Type 1.

Examples of this detail were found in 4 locations using U or PCDG precast beams. Spans range from 13 to 31 m with skews up to 18° and with slight plan curvature in one case.

3.5 Type 4 - Continuous Separated Deck Slab

This continuity detail, shown in Figure 4, confines itself to the deck slab only, which flexes to accommodate the rotations of the simply supported deck beams erected in the conventional multispan manner. To permit this flexure the deck slab is separated from the support beams for a length of about 1.5 m by a layer of compressible material. In-situ reinforced concrete diaphragms are located at the ends of the separated slab. The method is apparently the subject of a patent taken out by Dr A Kumar of Kumar Associates, UK.

Some four examples of this method were discovered with only limited dimensional information. The two detailed examples show spans up to 21 m with one skew of 31° and no significant curvature.

Florida also provided details of shorter span beam decks, typically using 60 ft (18.3 m) span I beams with a modified Type 4 connection over the piers. This provides slab reinforcement but without any slab separation over the beams.

3.6 Type 5 - Tied Deck Slab

Figure 5 shows the tied deck slab detail which was developed for multispan precast beam decks during the UK Standard Bridge exercise of the seventies. The bridge decks are designed and constructed in the conventional multispan simply-supported manner with slab
trimmer diaphragms at the beam ends. These ends are, as with Type 4, carried on two parallel rows of bearings on the piers.

Long connecting reinforcement dowels are incorporated at the mid-depth of the slab to tie the slabs together over the pier, eliminating expansion movement at deck level and permitting the use of a buried deck expansion joint. To permit deck rotation, the dowels are debonded and sleeved from the surrounding slab concrete over short lengths either side of the joint. Also, the slab and trimmer beam downstands are 'necked' using compressible joint filler below and above the dowel connection.

Details of one structure only of this type have been obtained, with 21 m spans and 25° skew, although it is certain that many more have been constructed.

3.7 Other Methods

One other proposed method of providing deck continuity came to light. This, shown in Figure 6, was put forward by Dr Jayanandana in his Leeds PhD Thesis (Reference 12). It is based on the Type 2 method with beams erected on two rows of permanent bearings in the same way. The proposed variation is that the deck slab is constructed in two stages, the first stage being poured over the integral crosshead approximately 1/3 span either side. Tendons located in the slab are prestressed to make the deck continuous over the pier prior to completing the centre reinforced concrete sections of the deck slab. This method is designated as Type 6 for the comparison of methods given in Section 4, which follows.

4 COMPARISON OF METHODS

4.1 General

Types 1 to 5, established methods of providing continuity in composite bridge decks, where the primary beams are of pretensioned precast concrete, plus the proposed method Type 6 are now compared in general terms.

The comparisons are made under headings ranging from Design Complexity to Comparative Costs. Where more detailed comparison appears necessary, recommendations are made for further investigation.

4.2 Design Complexity

The design complexity of each of the various continuity methods can be measured in part as the extra design effort required to convert the multi-span simply supported composite beam decks into continuous decks. A further measure is the complexity of this extra design effort.

Undoubtedly, the least complex design method is vested in Type 5. Little extra design effort is required as the simply supported decks remain simply supported. The tied deck slab is accomplished by merely adding the dowel/hinge detail to the end slab trimmer diaphragms. The extra design effort is required to cater for the forces directly transmitted through the tie connection such as deck traction and braking, wind, concrete shrinkage and bearing friction. This can, however, introduce some economy in bearing design since braking and traction loads can be distributed over more bearings and substructures.
Type 4 method requires more design effort. The continuous separated deck slab must be
designed for the same directly transmitted forces as with Type 5. However, the slab must
also be designed to accommodate the flexure of the connected composite beam decks. Such
flexure requires investigation of the following:-

a) creep and shrinkage of the precast beams;
b) creep and shrinkage of the deck slab;
c) live loading;
d) thermal effects;
e) deck settlement - including differential displacements of the two rows of bearings at a
support.

Types 1, 2 and 3 methods require even more design effort with greater complexity. The
flexure items listed above must be translated into moments applied to the integral crosshead
and the embedded precast beams. Additionally, continuity moments arise due to the self
weight of deck slabs added after deck continuity has been established.

As a result of the partial continuity, large negative, or hogging moments are created over the
support regions. These are typical of any continuous structure and are mainly due to live
load. Continuity also reduces the midspan sagging moments due to live loads and dead loads
due to surfacing and the deck slab. This is illustrated by several Type 1 M-beam examples,
where midspan moment reductions permit spans which are between 15% and 30% greater,
depending on the number of spans, than the maximum achievable with simply-supported
spans. However, large positive, or sagging moments are also created in the support regions
due to the restrained creep rotations of the precast beams. These can be critical when
combined with sagging bending moment due to settlement and thermal effects on the deck
and live load on remote spans.

The analysis and design of these support moments is quite complex, as indicated in Kumar's
booklet (Reference 1). The sample simply-supported 18.1 m span composite beam deck
requires 40 pages of calculations. Providing Type 2 continuity for a pair of 18.1 m spans
requires 77 pages.

The most difficult aspect of this design is the magnitude of the support sagging moments.
An analysis for support sagging moments due to creep in a series of precast composite decks,
using the proposed new Y-beams as M-beam replacements, indicated moments higher than
the sagging moments at midspan due to live load. These support sagging moments are
extremely difficult to cater for in the precast beams with the large amount of projecting
reinforcement vying for space with pretensioning tendons in the beam bottom flange. In the
modest 18.1 m span Kumar example there are 8 No 25 mm embedded reinforcing bars and
15 No 15.2 mm pretensioning strands to be crowded into a 970 mm wide x 120 mm (av)
flange. For spans nearer 30 m, the accommodation of a greatly increased number of bars
and strands within the same flange area would prove impossible.

It is however, argued that a large amount of sympathetic creep must arise in the reinforced
concrete connection to counteract the slow creep of the restrained precast beams, generally
resulting in a considerable reduction in the reinforced concrete sagging moments. Further
research is recommended.
The NCHRP Report 322 (Appendix B) offers considerable improvement by proposing to allow bottom cracking and using either no positive reinforcement or only a small amount of crack minimising steel, considerably less than the UK requirements illustrated above. Of course, the effect is generally to reduce the effective surfacing and live load continuity and increase midspan sagging moments.

Beam embedment and the resulting continuity restraints can be located further into the span with Type 1 method. This means that the continuity moments arising are less than with the precast beams in Type 2 and some Type 3 continuity details, which are located closer to the supports. The large sagging moments occurring at the ends of the precast beams in Type 2 and 3 connections are particularly difficult to design for as there is little room between beams to provide laps or mechanical fixings to effect projecting reinforcement continuity over the pier. This adds to the difficulties of embedding the bars in the beam ends referred to above. It is possible that reinforcement located closely around the beam periphery rather than embedded in it would provide a better connection detail. Further research and testing is recommended.

Type 2 and some Type 3 details have larger hogging moments applied to the embedded beam ends because they are closer to the supports than the Type 1. This adds complexity to the design of the precast beams, which are not well-conditioned to cater for hogging moments. Indeed, resort must be made to deflected pretensioning tendons accompanied by extensive debonding of horizontal tendons. It has been proposed that hybrid pre and post tensioned beams might provide a better solution (References 4 and 5). It is encouraging to note that the new Y beams have thicker webs and top flanges than M beams to provide room for post-tensioned tendons.

Probably the most complex continuity method is represented by the proposed prestressed deck slab Type 6. All the analytical problems associated with Types 2 and 3 are compounded by the additional effects of slab prestressing and staged slab construction.

Thus, it can be said that Type 6 exhibits the most design complexity, closely followed by Types 1, 2 and 3, with Type 5 offering the least connection design complexity. Type 4 occupies a middle position between 1-3 and 5.

4.3 Construction Complexity

Construction complexity is again relative to the construction of a series of simply supported composite precast beam decks. Once more Type 5 tied deck slab offers the least complexity, requiring only the addition of the hinged dowel connection.

Type 4, the continuous separated deck slab, also requires only minor extra construction activity. However, there is the complication of a patent, although the early use of the detail in Chile (Appendix A) and its current use in Turkey do not seem to have suffered any patent problems.

Types 2 and 3 methods are next in complexity, requiring only the pouring of the integral crosshead in one or two stages. The use of temporary bearings for some Type 2 methods can add further complexity.
Type 6 method adds prestressed two stage deck slabs, requiring more construction than Types 2 and 3.

Type 1 method probably involves the most construction complexity, with the requirement for foundation based trestling to temporarily support the precast beams. However this can be offset by the considerable advantage of using precast beams within the readily transportable 27.5 m maximum length for deck spans up to 5 m greater. End projecting reinforcement can be removed from embedded screw couplers during transport. The method can also offer considerable construction advantages for plan curved viaducts (see 4.6).

Integral crosshead reinforcement can become quite congested, making concreting difficult, particularly where longitudinal continuity and transverse reinforcement cross at top level. Type 1 and, occasionally, Type 2 methods of establishing continuity can provide some relief of this congestion by substituting prestressing tendons for lateral reinforcement. The tendons can be located nearer the mid level of the crosshead unlike transverse reinforcement, which must be located just under the crossing top longitudinal reinforcement.

4.4 Moment Transfer

There is no moment transfer for Types 4 and 5. For Types 1 and 6 all loading but the beam self weight can be considered to act on a fully continuous structure. This also applies to Types 2 and 3 provided the large sagging moments can be accommodated with effective reinforcement and limited cracking. This can be particularly advantageous in reducing the precast beam size required and the overall deck depth, quantities of deck materials and weight to be carried by the piers and abutments. Continuous spans up to 30% greater than simply supported spans can be accommodated. If the recommendations of NCHRP Report 322 (See Appendix B) are adopted then reduced values of moment transfer would result.

4.5 Skew

Although there is no backup from testing, which was limited to 0° skew models, high skews of nearly 50° have been successfully used for Type 1 and Type 2 methods, Figures 7 and 8. 30° skew has been accommodated for Type 4.

4.6 Plan Curvature

Only Types 1 and 3 can readily accept significant plan curvature of the deck. With radially located piers, the curvature requires significantly longer spanning precast beams on the outside than on the inside of the curve. Beams can be made of identical length and the curvature accommodated by varying the width of the integral crosshead to form a plan trapezium shape, Figure 9.

4.7 Deck Drainage

Drainage of surface water on multi-span viaducts ideally requires collector downpipes spaced no further apart than the pier support positions. This means lateral connecting piping between kerb gulley drains at the deck edges and downpipes within or attached to the sides of the piers. Normal simply supported composite beam span decks are so congested with precast beam ends over the piers that it is usually too difficult to pass the drainage pipes...
through. Resort has therefore to be made to ugly external piping passing very prominently down the deck sides or to more complex longitudinal deck drainage systems.

The Type 1 continuity method uses wide in-situ integral crossheads with the precast beam ends embedded some distance from the pier. This readily allows enough room for the passage of a concealed internal lateral connecting drain using generous diameters and bends, Figure 10.

4.8 Bearings and Piers

Multi-span simply supported composite beam decks generally require two parallel rows of bearings seated on piers or pier crossbeams. In turn this means that the pier or crossbeam must extend to sufficient width to pick up the outer deck beams and be thick enough to seat the two parallel rows of bearings.

Such requirements apply equally to Types 4, 5 and 6 continuity methods and to some cases of Types 2 and 3. However, Type 1 connections and the remaining cases of Types 2 and 3 can offer considerable savings in bearings and piers by using a single central row of bearings. This immediately halves the numbers of bearings required, although individual bearing size will marginally increase.

It also means that piers are thinner, not only because a single line of central bearings takes up less room at the pier top, but because the dead and live load moments applied to the piers by off centre pairs of bearings are removed. Significant savings in pier foundations also result.

Full width piers or crossheads are not required either. The integral crosshead in the deck can be top reinforced or prestressed laterally to allow considerable deck cantilevering outside the pier, Figure 11. Resulting inboard piers can offer considerable savings in ground intrusion. This in turn can allow ground level slip roads to be located under the deck edges and can reduce flyover spans at skew crossings, both particularly important in cramped urban locations. There is also a useful saving in numbers of bearings.

4.9 Compliance with BS 5400

It is unfortunate that very few of the listed structures were designed to BS 5400, most using the older BS 153. However, although the new code sets more onerous requirements concerning shear and reinforcement lapping, it is believed that attempts to 'equalise' BS 5400 with the original BS 153 during its preparation have been successful enough to create few new design problems. Little information was given in earlier codes on the positive support moments generated by creep and it is possible that these effects may not have been fully taken into account in some earlier designs.

4.10 Relevant Research and Load Testing

Major research and testing of precast beam continuity was undertaken by the Portland Cement Association in Chicago in the early sixties (Reference 7). The research mainly involved Type 2 connection method and covered all aspects, ranging from creep and shrinkage studies to horizontal shear connections. It also covered extensive testing of a half-scale continuous two span bridge, some 20 m long x 5 m wide.
Testing of the Type 1 connection method was undertaken by the then C&CA at Wexham Springs in 1974 (Reference 8). The tests were full-scale but limited to the connection detail only. This was based on a transversely prestressed integral crosshead connection designed by the then WS Atkins & Partners for the M11 Woodford Interchange (Reference 3).

Both testing programmes confirmed the structural integrity of Types 1 and 2 connection methods. However, one parameter not included was skew.

4.11 Durability

With the removal of the deck expansion joints and the associated ready intrusion of chloride contaminated deck drainage water, it could be expected that the durability of decks using Types 1-6 continuity connections would be considerably improved. Deck cracking over the supports can still occur and long term deck reinforcement corrosion is possible if the deck waterproofing fails. A sensible 'belt and braces' approach might lead to the future use of epoxy coating for the important deck reinforcement used in Types 1 to 4 connections.

A survey of the current state of the listed structures of Appendix A could reveal much of the relative durabilities of the various connection Types 1-5 constructed. This was not required at this stage, but it was decided to visually inspect several structures conveniently located nearby at the M25/M40 interchange.

Three Type 1 structures were chosen because their high skews, up to 48°, appeared to be taking the method well beyond the limits of the associated non-skew C&CA load testing. Particular attention was paid to an underside examination of the prestressed concrete integral crosshead. A binocular examination revealed an extremely good condition after some 4 years of service, with no signs of cracking, Figures 7 and 8.

A Type 5 structure of similar age was also examined, this probably being the least durable because the tied deck joint retains a rotational capacity, with a sealed notch required in the deck surfacing. Again, a binocular survey revealed excellent condition, with no apparent cracking, leakage or surface damage, Figures 12, 13 and 14.

The NCHRP report indicates that the widely circulated questionnaire replies generally pointed to the good durability, despite bottom cracking in several cases, of the Type 2 method.

It was also encouraging that several of the bridge authorities contacted indicated that they had had no durability problems with their continuous composite precast beam deck bridges.

It therefore appears from a preliminary sample that durability of all methods could be excellent. However, it is recommended that a full visual survey should be undertaken to confirm this tentative conclusion.

4.12 Aesthetics

Undoubtedly any of the six types of bridge deck continuity over the support piers will contribute to the appearance by eliminating the unsightly joint leakage which develops and becomes so evident on the decks and supports of multi-span simply supported decks.
Types 4 and 5 methods require deck width solid piers or crossbeams on columns. Pier or crossbeam widths are large in order to accommodate the two parallel rows of bearings, as occur with some examples of Types 2, 3 and 6 methods. Such wide fat piers do not generally contribute to the bridge or viaduct appearance, besides being over-intrusive in urban locations. Types 4 and 5 methods also add maximum deck depth to their heavy appearance.

Type 1 and the single central row of bearings in cases of Types 2 and 3 offer much thinner piers together with the possibility of a much reduced width by transversely cantilevering the deck. These slender inboard piers (Figure 11) can add considerably to the appearance. In addition, with the slimmed down deck arising from the beneficial continuity, the bridge or viaduct is lighter and less obtrusive in appearance.

NCHRP Report 322 appears to cover only Type 2 connections using twin rows of bearings on wide, thick, unattractive and expensive piers. Whether the findings would cover a connection using a single row of bearings after using temporary twin bearings during construction is not clear. This should obviously be aimed for in view of the possible savings in appearance and cost.

The crosshead downstands associated with Type 3 are generally obtrusive to the aesthetically desirable uniform and uninterrupted deck fascia. The beam ends can be notched to reduce the effect, but only at significant extra expense.

4.13 Comparative Costs

Detailed cost exercises to compare the relative economies of all the various continuity methods have not been undertaken in this investigation, although this would be a useful addition to the further work recommended. Nevertheless, with WSACL's recent experience of a Type 1 structure it was possible to make cost comparisons of this type with an equivalent simply supported structure, described later.

Several factors, however, indicate the probable cost order of the continuity methods Types 1-6. For this purpose, all are compared with the cost of a multi-span simply supported bridge using precast pretensioned beam composite decks.

The tied deck Type 5 method probably gives a marginally cheaper bridge than the comparative simply supported structure. The extra cost of the dowels and slab hinge formation is more than offset by the savings in deck jointing, bearings and substructure (4.2).

The continuous separated deck slab Type 4 method may prove marginally more expensive than Type 5 and there may be the added complication of a patent. However, offering similar savings in deck jointing, bearings and substructure will again ensure that a Type 4 continuity structure will be marginally cheaper than the cost of the comparative simply supported structure.

Undoubtedly Types 1, 2, 3 and 6 methods will prove significantly more expensive to design. However, the possible reductions in beam sizes, supports and foundations can provide cost savings to more than offset the extra construction complexity.
Probably the most significant factor is the possible beam size reduction, which not only reduces quantities of deck material and weight to be carried by the substructure, but also can reduce the overall structure length. A typical example is the Woodford Interchange Viaducts (Reference 3) where the Type 1 continuity permitted the use of M8 precast beams instead of M10 beams required for simply supported spans. The 160 mm deck depth reduction allowed some of the interchange flyovers to be reduced in length by 8 m, calculated by dividing the depth saving by the 4% deck slopes at the ends, ie. 160 mm x 25 x 2. For the deck width of 12.7 m, this represents a deck saving of some 100 m², or £60,000 - £80,000 at today’s prices.

The Type 1 structure comparison referred to earlier was made for the 361 m long Woodford Viaduct, completed in North London in late 1987. The 12 span viaduct is made up of twin continuous decks, each using 8 contiguous M9 precast pretensioned beams. There are 10 spans of 30.9 m and 2 end spans of 26.0 m using embedded beam lengths of 27.5 m and 24.60 m respectively. The 1987 cost was about £3.5 million.

The structure was very similar to the earlier 6 viaducts of the Woodford Interchange (Reference 3) although larger beams and slightly shorter spans proved necessary because of an additional design requirement for a 400 tonne vehicle.

The spans of the equivalent simply supported viaduct of the same total length using the same M9 contiguous decks were determined from the span tables produced by PCA for M9 beams, marginally adjusted down to allow for the extra 400 tonne vehicle. This resulted in a requirement for 14 equal simply supported spans of 25.8 m, total length 361.2 m. The cost comparison could then be made by substituting the following items:-

i) 14 x 2 x 8 M9 beams 25.8 m long for the existing beams.

ii) 2 x 13 full width fatter piers for the original 2 x 11 unboard slimmer piers.

iii) 14 x 2 x 8 x 2 simple support bearings for the lesser number of the original continuous support bearings.

iv) 2 x 15 expansion joints for the original 2 x 2.

v) Twin end diaphragms for each integral crosshead.

The comparison indicated no significant cost difference. However, it was apparent that the short crosshead post tensioned tendons of the original Type 1 viaduct represented an expensive additional item. This would probably have been justified if the minimum ground intrusion of the piers had been exploited by locating ground level slip roads under the viaducts, thereby saving expensive urban land. However, this did not happen.

The crosshead prestressing was incorporated at Woodford because the original C & CA tests (Reference 8) had been based on this type of construction. Later constructions have successfully substituted normal reinforcement for prestressing. It was therefore felt that such a substitution should be made for this cost comparison.

The substitution of reinforcement for the crosshead prestressing showed considerable cost savings, making the Type 1 structure some 5% cheaper than the simply supported equivalent.
To this, of course, can be added the significant savings arising from the elimination of intermediate joint maintenance and the associated possible pier and bearing damage due to road salt penetration.

Thus, it can be stated that Type 1 method of providing continuity shows significant first cost and maintenance cost savings over a simply supported equivalent structure.

It can be reasonably assessed that Types 2, 3 and 6 methods of providing continuity will also show these savings. Due to the expense of short prestressing tendons it is probable that Type 6 will prove marginally more expensive than Type 1, 2 and 3.

Eliminating or minimising the bottom reinforcement for the Type 2 method in accordance with NCHR.P 322 would provide some economy, but this would probably be more than countered by the extra costs of providing for reduced continuity. In addition, the expensive wide and fat piers option would be required.

5 RECOMMENDATIONS

5.1 Further Investigations

It is recommended that the following further investigations should be undertaken to extend the findings of this report:-

i) a desk study of the effect of sympathetic creep on the support sagging moments of Types 1, 2, 3 and 6 methods;

ii) testing of support sagging moment reinforcement located around the beams rather than embedded in them;

iii) a durability survey of the structures listed in Appendix A, particularly Type 5, more examples of which should be identified.

iv) Detailed Cost Studies.

5.2 Most Suitable Continuity Details to be Promoted for UK

It is recommended that all the methods covered by Types 1-5 should be promoted for UK use, leaving the Client or Designer free choice. The most important issue is to promote reductions in deck joints for all new bridge designs as soon as possible.

The advantages and disadvantages identified in this report should be clearly set out in the DTp Advice Note. Some reservations should be expressed towards Type 2 method until the recommended investigations, i) and ii) above, are completed, hopefully to simplify the design and construction details. Any modifications or amplifications arising from the recommended investigations, iii) above, could be incorporated at the same time in a later revision to the Advice Note.

It is recommended that NCHRP 322 proposals should be adopted for Type 2 UK twin bearing applications, with the following reservations:-
i) a small amount of crack minimising positive reinforcement should always be provided over piers, possibly as a standard percentage of the precast beam area;

ii) a range of effective restraint moment multipliers should be tabulated for typical construction time scales to permit simpler design for what is the simplest construction method of providing beam continuity;

iii) the NCHRP 322 proposals should be modified to cater for the more desirable single bearing Type 2 applications;

iv) the NCHRP 322 proposals for ductility and fatigue considerations should be adopted for UK Type 2 practice, possibly extended to similar aspects of Types 1, 3, 4 and 5.

6 ACKNOWLEDGEMENTS

The authors of this report wish to take this opportunity to extend their thanks to all those organisations who have given their time to provide the basic information. Without their assistance this report could not have been completed in as meaningful a way.

The following lists organisations who provided information used in this report:

° UK Authorities

- Cheshire County Council
- Northampton County Council
- Avon County Council
- North Yorkshire County Council
- DoE Northern Ireland
- Clwyd County Council
- Kent County Council

° UK Consultants

- Sir Owen Williams & Partners
- Brian Colquhoun & Partners
- Sir Alexander Gibb & Partners
- Scott Wilson Kirkpatrick & Partners
- Sir William Halcrow & Partners
- Ove Arup & Partners
- Gifford & Partners
- Dobbie & Partners
- Kumar Associates
Overseas Organisations

- American Association of State Highway and Transportation Officials
- Iowa State University
- Illinois Department of Transportation - USA
- Florida Department of Transportation - USA
- Federal Highway Administration - USA

REFERENCES


Fig. 1

SECTIONAL ELEVATION A-A

SECTION D-D

These components may be omitted.

Typical Features.

Beams erected on temporary supports, generally on pier foundations.

CONNECTION DETAIL TYPE 1-
WIDE IN-SITU INTEGRAL CROSSHEAD.

Fig. 1
ALTERNATIVE ELEVATION A-A WITH SINGLE BEARING.

Typical Features:
- Beams supported on permanent bearings
- May be single or twin line of bearings
- Single diaphragm for beams in both spans
- These components may be omitted

SECTION C-C

END SPAN

BEAM ENDBEAM

INTERNAL SPAN

B

TRANVERSE REINFORCEMENT THROUGH BEAM WEBS

BEARING SPACING CROSSHEAD LENGTH

INSITU FACIA

PART SECTIONAL PLAN B-B

SECTIONAL ELEVATION A-A

CONTINUOUS BEAM AND INSITU FACIA

SPACED BEAM AND INSITU FACIA

INSITU FACIA

Fig. 2
SECTION C-C

**Typical Features:**
Beams supported on first stage crosshead construction, reinforcement as Type 1 or Type 2 connection.

- These components may be omitted.

**Fig. 3**

- L - Length of beam
- D - Depth of beam
- E - Elevation

**Fig. 3 Text:**

- Contiguous Beam
- Insitu facia
- Spaced Beam
- No insitu facia

**Diagram Notes:**
- Mortar bed or pad bearing
- Crosshead monolith with temporary flexibility during construction
- Extent of stage 1 construction
- Reinforcement omitted for clarity
SECTIONAL ELEVATION A-A

Compressible filler between beam and slab

Slab length continuous across joint

Transverse reinforcement through beam webs

Diaphragm set back from beam ends

SECTION C-C

PART SECTIONAL PLAN B-B

Typical Features:
Beams on individual bearings.
Separate diaphragms for each span.
No bottom continuity reinforcement.
No live load continuity.

CONNECTION DETAIL TYPE 4-
CONTINUOUS SEPARATED DECK SLAB.
Fig. 5

SECTIONAL ELEVATION A-A

Transverse reinforcement through beam webs

SECTION C-C

DETAIL X

Typical Features:
- Beams on individual bearings.
- Separate diaphragms for each span.
- No bottom continuity reinforcement.
- No live load moment continuity.

*These components may be omitted.

CONNECTION DETAIL TYPE 5-TIED DECK SLAB
(a) Stage 1: Precast Prestressed Girders are Placed on Supports

(b) Stage 2: Casting of In-Situ Concrete Top Slab and Diaphragm

(c) Stage 3: Post-Tensioning of Top Slab

(d) Stage 4: Casting of the Remainder of the Slab

Sequence of Construction for the Proposed Method

CONNECTION DETAIL TYPE 6 —

As proposed by A D C Jayanandana.  

Fig. 6
45/20 In situ concrete crosshead post tensioned transversely by 24 No 10/0.76 tendons.

M.O.T./C.C.A. type H.B. pretensioned inverted 'T' beams with 28 No 15.2 mm. dia. 'Dyform' strands. 14 No strands to be deflected and 6 No debonded at both ends.

32 000 centres of Piers.

PART LONGITUDINAL SECTION.
SCALE 1:100

PART SECTION THROUGH DECK.
PART SECTION THROUGH CROSSHEAD.
SCALE 1:50

FIGURE 10

Type 1 Connection - Typical Section with Inboard Columns
Showing Drainage - M11 South Woodford
Figure 11  Type 1 - Connection - View Showing Plan Curvature and Inboard Columns - M11 South Woodford

Figure 12  Type 5 - Connection - Typical Elevation - M25, M4 to Maple Cross

Figure 13  Type 5 - Connection - Joint in Over-Road Surfacing M25, M4 to Maple Cross

Figure 14  Type 5 - Connection - Soffit View of Joint M25, M4 to Maple Cross
APPENDIX A

Summary of Structures Identified by Survey

This appendix provides, in tabular form, the details of continuous bridge decks using precast beams that have been identified by the survey. The first table provides data with respect to the design organisation, name of structure, type of continuity connection and date of design or construction. The second table provides key structural data and is organised by type of connection.

The information has, in general, been extracted from drawings supplied, some of which did not include all the details required.

NOTES ON TABLES

A GENERAL

Reference No: This is used to identify particular structures, allowing cross-reference between the two tables.

Structure Name: The name of the structure as given on the drawings.

B LIST OF STRUCTURES

Connection Type: The type of continuity connection as described in the text.

Designer/Authority: The name given is of the organisation providing the details or whose name appears on the drawings.

Date: The date of construction (C) if known or the date of design (D), the latter generally being taken from the drawings.
C  STRUCTURAL DATA

These tables provide key structural data including beam type and size, span and beam lengths, connection dimensions, skew and the connection design designated as Types 1-5. These tables should be read in conjunction with Figures 1 to 5.

The data is intended to be indicative of the range of structures and rounding has been applied to dimensions and skews in some cases.

An asterisk (*) has been included in the tables where data was not available.

D  COMPLETENESS

The tables are based on limited enquiries to Consultants and Local Authorities and are therefore not fully comprehensive of all continuous structures in the UK and certainly not abroad.
<table>
<thead>
<tr>
<th>Ref No</th>
<th>Joint Type</th>
<th>Designer/Authority</th>
<th>Structure Name</th>
<th>Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3</td>
<td>Sir Owen Williams &amp; Partners</td>
<td>R. Neath Eastern Approach Viaduct</td>
<td>D 1987</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>Cheshire County Council</td>
<td>M56 Motorway The Cedars Group 1 Bridges</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>2</td>
<td>Cheshire County Council</td>
<td>M56 Motorway - Accommodation bridges</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>1</td>
<td>Northamptonshire CC</td>
<td>Barnes Meadow Viaduct</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>4</td>
<td>Avon County Council</td>
<td>River Frome Bridge, Avon Ring Road</td>
<td>C 1988</td>
</tr>
<tr>
<td>6</td>
<td>1</td>
<td>North Yorkshire CC</td>
<td>Rawcliffe Bridge, York Outer Ring Road</td>
<td>D 1985</td>
</tr>
<tr>
<td>7</td>
<td>1</td>
<td>North Yorkshire CC</td>
<td>Copmanthorpe Bridge, York Outer Ring Road</td>
<td>D 1984</td>
</tr>
<tr>
<td>8</td>
<td>1</td>
<td>North Yorkshire CC</td>
<td>A1 Catterick North Junction</td>
<td>D 1981</td>
</tr>
<tr>
<td>9</td>
<td>1</td>
<td>North Yorkshire CC</td>
<td>Millfield Railway Bridge, York Outer Ring Road</td>
<td>D 1981</td>
</tr>
<tr>
<td>10</td>
<td>X</td>
<td>Cheshire County Council</td>
<td>A41/A55 Interchange</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>2</td>
<td>DoE Northern Ireland</td>
<td>Trinamadan Bringe, Gortin Co. Tyrone</td>
<td>D 1985</td>
</tr>
<tr>
<td>12</td>
<td>2</td>
<td>DoE Northern Ireland</td>
<td>Union Bridge widening, Lisburn</td>
<td>D 1985</td>
</tr>
<tr>
<td>13</td>
<td>1</td>
<td>DoE Northern Ireland</td>
<td>Bessbrook River, Newry</td>
<td>D 1986</td>
</tr>
<tr>
<td>14</td>
<td>2</td>
<td>DoE Northern Ireland</td>
<td>Kings Road Bridge, Whitehead</td>
<td>D 1985</td>
</tr>
<tr>
<td>15</td>
<td>2</td>
<td>DoE Northern Ireland</td>
<td>Shimna Bridges, Newcastle</td>
<td>D 1988</td>
</tr>
<tr>
<td>16</td>
<td>1</td>
<td>DoE Northern Ireland</td>
<td>Dock Street Bridge, Belfast</td>
<td>D 1980</td>
</tr>
</tbody>
</table>

WP Ref No A911CPB
<table>
<thead>
<tr>
<th>Ref No</th>
<th>Joint Type</th>
<th>Designer/Authority</th>
<th>Structure Name</th>
<th>Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>17</td>
<td>1</td>
<td>WS Atkins &amp; Partners</td>
<td>M11 Motorway S. Woodford Interchange</td>
<td>D 1972 / C 1976</td>
</tr>
<tr>
<td>18</td>
<td>1</td>
<td>WS Atkins &amp; Partners</td>
<td>Woodford Via, Woodford to Barking Relief Rd</td>
<td>D 1984 / C 1987</td>
</tr>
<tr>
<td>19</td>
<td>3</td>
<td>Brian Colquhoun &amp; Partners</td>
<td>Kuwait Outer Bypass Junctions 3A &amp; 7A</td>
<td>D 1982 / C 1987</td>
</tr>
<tr>
<td>20</td>
<td>1</td>
<td>Clwyd County Council</td>
<td>Typical details</td>
<td></td>
</tr>
<tr>
<td>21</td>
<td>1</td>
<td>Kent County Council</td>
<td>Stour Viaduct</td>
<td>C 1979</td>
</tr>
<tr>
<td>22</td>
<td>1</td>
<td>Kent County Council</td>
<td>Queenborough Viaduct</td>
<td>C 1982</td>
</tr>
<tr>
<td>23</td>
<td>1</td>
<td>Kent County Council</td>
<td>Barham Bridge</td>
<td>C 1987</td>
</tr>
<tr>
<td>24</td>
<td>2</td>
<td>Sir Alexander Gibb &amp; Partners</td>
<td>M4 River Wyrardsisbury Bridge</td>
<td>D 1962</td>
</tr>
<tr>
<td>25</td>
<td>1</td>
<td>Scott Wilson Kirkpatrick &amp; Partners</td>
<td>A835 Dingwall Ullapool Black Bridge</td>
<td>D 1980</td>
</tr>
<tr>
<td>26</td>
<td>Box Portal</td>
<td>Scott Wilson Kirkpatrick &amp; Partners</td>
<td>Renfrew Motorway Bellahouston Bridge</td>
<td>D 1971</td>
</tr>
<tr>
<td>27</td>
<td>Box Portal</td>
<td>Scott Wilson Kirkpatrick &amp; Partners</td>
<td>Renfrew Motorway Gower Street Bridge</td>
<td>D 1972</td>
</tr>
<tr>
<td>28</td>
<td>1</td>
<td>Sir William Halcrow &amp; Partners</td>
<td>M25 M4 to Maple Cross M40 Overbridge</td>
<td>D 1982</td>
</tr>
<tr>
<td>29</td>
<td>1</td>
<td>Sir William Halcrow &amp; Partners</td>
<td>M25 M4 to Maple Cross South Links Underbridge</td>
<td>D 1981</td>
</tr>
<tr>
<td>30</td>
<td>1</td>
<td>Sir William Halcrow &amp; Partners</td>
<td>M25 M4 to Maple Cross West Links Overbridge</td>
<td>D 1981</td>
</tr>
<tr>
<td>31</td>
<td>5</td>
<td>Sir William Halcrow &amp; Partners</td>
<td>M25 M4 to Maple Cross A40 Tatling End O’bridge</td>
<td>D 1981</td>
</tr>
<tr>
<td>32</td>
<td>1</td>
<td>Sir Alexander Gibb &amp; Partners</td>
<td>Pitaldonich Bridge, A9 PerthInverness Trunk Rd</td>
<td>D 1981</td>
</tr>
<tr>
<td>33</td>
<td>1</td>
<td>Sir Alexander Gibb &amp; Partners</td>
<td>Killiecrankie Viaduct, A9 PerthInverness Trunk Rd</td>
<td>D 1987</td>
</tr>
</tbody>
</table>

WP Ref No A911CPB - A4 -
<table>
<thead>
<tr>
<th>Ref No</th>
<th>Joint Type</th>
<th>Designer/Authority</th>
<th>Structure Name</th>
<th>Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>34</td>
<td>4</td>
<td>Ove Arup &amp; Partners</td>
<td>River Bridge Turkey</td>
<td>D 1989</td>
</tr>
<tr>
<td>35</td>
<td>3</td>
<td>Gifford &amp; Partners</td>
<td>Central Station Bridge Bournemouth</td>
<td></td>
</tr>
<tr>
<td>36</td>
<td>2</td>
<td>Gifford &amp; Partners</td>
<td>Typical Details</td>
<td></td>
</tr>
<tr>
<td>37</td>
<td>4</td>
<td>Gifford &amp; Partners</td>
<td>Bio Bio Bridge Chile 1960's</td>
<td>C</td>
</tr>
<tr>
<td>38</td>
<td>3</td>
<td>WS Atkins &amp; Partners</td>
<td>Approach Viaducts M1 Fiveways Interchange</td>
<td>C 1970</td>
</tr>
<tr>
<td>39</td>
<td>4</td>
<td>Dobbie &amp; Partners</td>
<td>Railway Bridge</td>
<td>D 1989</td>
</tr>
<tr>
<td>40</td>
<td>2</td>
<td>Florida D. of Transport</td>
<td>Various Structures</td>
<td>D 1984</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>-1988</td>
</tr>
<tr>
<td>41</td>
<td>2</td>
<td>Illinois D. of Transport</td>
<td>Typical Details</td>
<td></td>
</tr>
<tr>
<td>42</td>
<td>2</td>
<td>Florida D. of Transport</td>
<td>St Mary River</td>
<td>D 1989</td>
</tr>
</tbody>
</table>

WP Ref No A911CPB - A5 -
<table>
<thead>
<tr>
<th>Ref No</th>
<th>Structure Name</th>
<th>Beam Type</th>
<th>Span</th>
<th>Key Dimensions (mm)</th>
<th>Plan Geometry/</th>
<th>Beam Section</th>
<th>Combined Transverse Detail</th>
<th>Combined</th>
<th>Shotcrete</th>
<th>Be</th>
<th>Bi</th>
<th>Li</th>
<th>Le</th>
<th>D</th>
<th>C</th>
<th>E</th>
<th>Details</th>
<th>Diameter</th>
<th>Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>M56 Mokoway, The Centre Group 1 Bridges</td>
<td>M</td>
<td>-</td>
<td>22.517</td>
<td>Bar</td>
<td>RC</td>
<td>PS</td>
<td>36</td>
<td>Bar</td>
<td>914</td>
<td>2.134</td>
<td>610</td>
<td>2900</td>
<td>2900</td>
<td>2900</td>
<td>2900</td>
<td>2900</td>
<td>2900</td>
<td>2900</td>
</tr>
<tr>
<td>4</td>
<td>Bannock Bridge, York Outer Ring Road</td>
<td>M</td>
<td>6</td>
<td>27.000</td>
<td>27.738</td>
<td>3.000</td>
<td>1.000</td>
<td>5000</td>
<td>27.000</td>
<td>27.738</td>
<td>3.000</td>
<td>1.000</td>
<td>5000</td>
<td>27.000</td>
<td>27.738</td>
<td>3.000</td>
<td>1.000</td>
<td>5000</td>
<td>27.000</td>
</tr>
<tr>
<td>6</td>
<td>Rawlins Bridge, York Outer Ring Road</td>
<td>M</td>
<td>6</td>
<td>27.000</td>
<td>27.738</td>
<td>3.000</td>
<td>1.000</td>
<td>5000</td>
<td>27.000</td>
<td>27.738</td>
<td>3.000</td>
<td>1.000</td>
<td>5000</td>
<td>27.000</td>
<td>27.738</td>
<td>3.000</td>
<td>1.000</td>
<td>5000</td>
<td>27.000</td>
</tr>
<tr>
<td>7</td>
<td>Cumnor Bridge, Oxford</td>
<td>M</td>
<td>4</td>
<td>12.000</td>
<td>17.000</td>
<td>4.000</td>
<td>1.000</td>
<td>6000</td>
<td>94.000</td>
<td>17.000</td>
<td>4.000</td>
<td>1.000</td>
<td>6000</td>
<td>94.000</td>
<td>17.000</td>
<td>4.000</td>
<td>1.000</td>
<td>6000</td>
<td>94.000</td>
</tr>
<tr>
<td>8</td>
<td>Al Catterick North Junction</td>
<td>M</td>
<td>4</td>
<td>12.000</td>
<td>17.000</td>
<td>4.000</td>
<td>1.000</td>
<td>6000</td>
<td>94.000</td>
<td>17.000</td>
<td>4.000</td>
<td>1.000</td>
<td>6000</td>
<td>94.000</td>
<td>17.000</td>
<td>4.000</td>
<td>1.000</td>
<td>6000</td>
<td>94.000</td>
</tr>
<tr>
<td>9</td>
<td>Millfield Railway Bridge, York</td>
<td>M</td>
<td>4</td>
<td>12.000</td>
<td>17.000</td>
<td>4.000</td>
<td>1.000</td>
<td>6000</td>
<td>94.000</td>
<td>17.000</td>
<td>4.000</td>
<td>1.000</td>
<td>6000</td>
<td>94.000</td>
<td>17.000</td>
<td>4.000</td>
<td>1.000</td>
<td>6000</td>
<td>94.000</td>
</tr>
</tbody>
</table>

**Type 1: Wide In-Situ Integral Crosshead**
<table>
<thead>
<tr>
<th>Ref No</th>
<th>Structure Name</th>
<th>No Span</th>
<th>Beam Type</th>
<th>Key Dimensions (mm)</th>
<th>Btm Slab</th>
<th>Beam Hog Design</th>
<th>Crosshead Transverse Detail</th>
<th>Bottom Continuity Steel</th>
<th>Plan Geometry/Skew</th>
</tr>
</thead>
<tbody>
<tr>
<td>22</td>
<td>Queensborough Viaduct</td>
<td>8</td>
<td>M7</td>
<td>Le Li Be Bi C D E</td>
<td>yes</td>
<td>PS</td>
<td>PS</td>
<td>Bar/Strand</td>
<td>Curved</td>
</tr>
<tr>
<td>23</td>
<td>Barham Bridge</td>
<td>4</td>
<td>M7</td>
<td>12 990 19 139 11 240 17 639</td>
<td>1 500 4 500 1 500</td>
<td>* *</td>
<td>1 000</td>
<td>RC</td>
<td>Bar/Strand</td>
</tr>
<tr>
<td>24</td>
<td>River Wyaradiabury</td>
<td>3</td>
<td>Own</td>
<td>7 163 14 630</td>
<td>* *</td>
<td>610 Infill Slab</td>
<td>--</td>
<td>Bar</td>
<td>*</td>
</tr>
<tr>
<td>28</td>
<td>M40 Overbridge</td>
<td>4</td>
<td>U12 U8</td>
<td>23 550 29 621 20 225 29 530</td>
<td>3 756 5 576 (skew)</td>
<td>1 000</td>
<td>* *</td>
<td>PS</td>
<td>*</td>
</tr>
<tr>
<td>29</td>
<td>South Links Underbridge</td>
<td>4</td>
<td>U8 U5</td>
<td>18 550 25 538 24 403</td>
<td>* *</td>
<td>5 484 7 484 (skew)</td>
<td>1 000</td>
<td>PS</td>
<td>*</td>
</tr>
<tr>
<td>30</td>
<td>West Links Underbridge</td>
<td>4</td>
<td>U8 U5</td>
<td>18 550 24 970 23 810</td>
<td>* *</td>
<td>5 246 7 246 (skew)</td>
<td>1 000</td>
<td>PS</td>
<td>*</td>
</tr>
<tr>
<td>32</td>
<td>Pitaldonich Bridge</td>
<td>3</td>
<td>U10</td>
<td>25 300 28 600 25 200 27 000</td>
<td>850 3 750 1 450 (skew)</td>
<td>--</td>
<td>RC</td>
<td>Coupled Bar</td>
<td>15</td>
</tr>
<tr>
<td>33</td>
<td>Killiecrankie Viaduct</td>
<td>8 + 11 + 4</td>
<td>SBB</td>
<td>* * * *</td>
<td>1 800 Infill Slab</td>
<td>--</td>
<td>--</td>
<td>Coupled Bar</td>
<td>Square</td>
</tr>
</tbody>
</table>
## TYPE 2 NARROW IN-SITU INTEGRAL CROSSHEAD

<table>
<thead>
<tr>
<th>Ref No</th>
<th>Structure Name</th>
<th>No. Spans</th>
<th>Beam Type</th>
<th>Key Dimensions (mm)</th>
<th>Btm Slab</th>
<th>Beam Hog Design</th>
<th>Crosshead Transverse Detail</th>
<th>Bottom Continuity Steel</th>
<th>Plan Geometry/skew</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>M56 Motorway Accommodations bridges</td>
<td>2</td>
<td>M</td>
<td>Le 17 812 Li 152 F 1 372 C 610 D 0 E 0</td>
<td>no</td>
<td>*</td>
<td>*</td>
<td></td>
<td>20</td>
</tr>
<tr>
<td>11</td>
<td>Trinamadan Bridge, Gortin, Co Tyrone</td>
<td>2</td>
<td>M7</td>
<td>Le 16 500 Li 750 F 1 372 C 610 D 0 E 0</td>
<td>no</td>
<td>*</td>
<td>*</td>
<td>RC</td>
<td>Bar</td>
</tr>
<tr>
<td>12</td>
<td>Union Bridge widening, Lisburn</td>
<td>3</td>
<td>M1</td>
<td>Le 11 275 Li 11 320 F 1 372 C 610 D 0 E 0</td>
<td>no</td>
<td>*</td>
<td>*</td>
<td>RC</td>
<td>Bar</td>
</tr>
<tr>
<td>14</td>
<td>Kings Road Bridge, Whitehead</td>
<td>2</td>
<td>M2</td>
<td>Le 12 142 Li 19 600 F 1 372 C 610 D 0 E 0</td>
<td>yes</td>
<td>*</td>
<td>*</td>
<td>RC</td>
<td>Bar</td>
</tr>
<tr>
<td>15</td>
<td>Shinna Bridges, Newcastle</td>
<td>2</td>
<td>M2</td>
<td>Le 10 210 Li 680 F 1 372 C 610 D 0 E 0</td>
<td>no</td>
<td>*</td>
<td>*</td>
<td>RC</td>
<td>Bar</td>
</tr>
<tr>
<td>25</td>
<td>A835 Black Bridge</td>
<td>2</td>
<td>M6</td>
<td>Le 19 600 Li 19 600 F 1 372 C 610 D 0 E 0</td>
<td>yes</td>
<td>*</td>
<td>*</td>
<td>RC</td>
<td>None</td>
</tr>
<tr>
<td>36</td>
<td>Typical Details (Gifford)</td>
<td>–</td>
<td>M</td>
<td>Le – Li – F – C – D – E –</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>40</td>
<td>Various Structures (Florida)</td>
<td>5x4</td>
<td>T</td>
<td>Le 44 196 Li 44 196 F 1 372 C 610 D 0 E 0</td>
<td>no</td>
<td>PS</td>
<td>RC Bar Square</td>
<td>PSA Bar Square</td>
<td>Square</td>
</tr>
<tr>
<td></td>
<td></td>
<td>24x4</td>
<td>T</td>
<td>Le 43 586 Li 43 586 F 1 372 C 610 D 0 E 0</td>
<td>no</td>
<td>PS</td>
<td>RC Bar Square</td>
<td>PSA Bar Square</td>
<td>Square</td>
</tr>
<tr>
<td></td>
<td></td>
<td>11x4</td>
<td>T</td>
<td>Le 43 282 Li 43 282 F 1 372 C 610 D 0 E 0</td>
<td>no</td>
<td>PS</td>
<td>RC Bar Square</td>
<td>PSA Bar Square</td>
<td>Square</td>
</tr>
<tr>
<td>41</td>
<td>St Mary's River</td>
<td>5</td>
<td>I</td>
<td>Le 15 850 Li 15 850 F 1 372 C 610 D 0 E 0</td>
<td>no</td>
<td>*</td>
<td>RC Bar 35°</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>42</td>
<td>Typical Details (Illinois)</td>
<td>–</td>
<td>–</td>
<td>Le – Li – F – C – D – E –</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
</tbody>
</table>
## TYPE 3 INTEGRAL CROSSHEAD CAST IN TWO STAGES

<table>
<thead>
<tr>
<th>Ref No</th>
<th>Structure Name</th>
<th>No Spans</th>
<th>Beam Type</th>
<th>Key Dimensions (mm)</th>
<th>Btm Slab</th>
<th>Beam Hog Design</th>
<th>Crosshead Transverse Detail</th>
<th>Bottom Continuity Steel</th>
<th>Plan Geometry/Skew</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>River Neath, Eastern Approach Viaduct</td>
<td>6x6</td>
<td>U11</td>
<td>Le 28 000 Li 31 000 Be 24 800 Bi 27 400 C 3 600 D 6 000 E 1 200</td>
<td>no</td>
<td>RC</td>
<td>RC</td>
<td>*</td>
<td>Curved</td>
</tr>
<tr>
<td>19</td>
<td>Kuwait Outer Bypass</td>
<td>3x3</td>
<td>I &amp; box</td>
<td>Le 28 250 Li 29 150 Be 27 950 Bi 27 950 C 1 200 D 2 200 E 500</td>
<td>no</td>
<td>PS?</td>
<td>RC</td>
<td>Strand 18</td>
<td></td>
</tr>
<tr>
<td>35</td>
<td>Central Station Bridge</td>
<td>10x2</td>
<td>I</td>
<td>Le 26 700 Li -     Be - Bi - C - D - E -</td>
<td>Infill Slab</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>38</td>
<td>Fiveways Approach Viaduct</td>
<td>10</td>
<td>I</td>
<td>Le 13 200 Li 12 800 Be - Bi - C - D - E -</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>PS Bar Square</td>
<td></td>
</tr>
</tbody>
</table>

## TYPE 4 CONTINUOUS SEPARATED DECK SLAB

<table>
<thead>
<tr>
<th>Ref No</th>
<th>Structure Name</th>
<th>No Spans</th>
<th>Beam Type</th>
<th>Key Dimensions (mm)</th>
<th>Btm Slab</th>
<th>Beam Hog Design</th>
<th>Crosshead Transverse Detail</th>
<th>Plan Geometry/Skew</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>River Frome Bridge, Avon Ring Road</td>
<td>5</td>
<td>M6</td>
<td>Le 21 105 Li 21 500 Be 450 C 1 200 D 3 000 E -</td>
<td>no</td>
<td>-</td>
<td>RC</td>
<td>Square</td>
</tr>
<tr>
<td>34</td>
<td>River Bridge - Turkey</td>
<td></td>
<td>U</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>37</td>
<td>Bio Bio Bridge - Chile</td>
<td></td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>39</td>
<td>Railway Bridge</td>
<td>3</td>
<td>M6</td>
<td>Le 19 670 Li 19 670 Be 560 (skew)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>31</td>
</tr>
</tbody>
</table>
## TYPE 5 TIED DECK SLAB

<table>
<thead>
<tr>
<th>Ref No</th>
<th>Structure Name</th>
<th>No Span</th>
<th>Beam Type</th>
<th>Key Dimensions (mm)</th>
<th>Btm Slab</th>
<th>Beam Hog Design</th>
<th>Crosshead Transverse Detail</th>
<th>Plan Geometry/Skew</th>
</tr>
</thead>
<tbody>
<tr>
<td>31</td>
<td>A40 Tatling End Bridge (13 Bridges similar)</td>
<td>2</td>
<td>U8</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>RC</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Le</td>
<td>Li</td>
<td>F</td>
<td>D</td>
<td>2</td>
</tr>
</tbody>
</table>
APPENDIX B

Commentary on NCHRP Report 322

This Appendix describes the findings of NCHRP Report 322 (Reference 19) and comments upon its applicability to UK practice.

Contents of the Report

November 1989 saw the publication of Report 322 "Design of Precast Prestressed Bridge Girders Made Continuous", prepared by Construction Technology Laboratories Inc. (CTL) of Skokie, Illinois for the Transportation Research Board of the National Research Council.

Concrete bridges constructed with simply supported precast prestressed girders made continuous through connections to in-situ deck slabs and diaphragms over bridge piers have been built extensively throughout the USA since the early 1960's. Almost without exception, the means of establishing continuity relies on the Type 2 method described in para. 3.3 using twin rows of permanent bearings on each supporting pier. Designs have generally been based upon procedures developed by the American Portland Cement Association following an extensive experimental research programme in 1961 (Reference 7). Continuity moments over the piers required top and bottom reinforcement, the difficult and expensive latter detail catering for positive moments arising from time dependent creep and shrinkage rotation together with live load applications.

Although existing bridges are generally performing well, it is believed that the PCA design method does not accurately predict the true behaviour of these structures. Report 322 records extensive studies undertaken to resolve this uncertainty. The following items were covered:-

a) Current practice in analysis, design, and construction of bridges built of prestressed girders made continuous was investigated.

b) Early age creep and shrinkage tests were undertaken.

c) Computer programs were developed for the parametric studies and for simplified analysis methods.

d) Parametric studies were conducted to examine the effects of variation in time-dependent material behaviour and variation in bridge design parameters on the resultant service moments at continuity connections and girder midspan regions.

e) Parametric studies were conducted to examine the effects of variation in bridge design parameters on inelastic redistribution of moments and development of the maximum strength of the bridge girders.

f) Recommendations were developed for design procedures for service and ultimate strength studies to be included in revision to AASHTO.

Commentary on these six items follows.
Current Mainly American Practice

A literature review produced 76 references, with not unexpected emphasis on American practice. Notable references were to the so-called 'integral bridges' which not only eliminate intermediate deck joints by using continuity but also the end joints by building the deck girders into the abutments.

A detailed questionnaire was sent to transportation officials, designers and fabricators. The response was good and the following items were recorded:

i) The number of continuous spans ranged from 2 to 29, the majority falling between 3 and 7.

ii) Girders were mainly steam cured AASHTO - PCI Sections, Types II, III and IV. Bulb-Tee and Box Section girders were also used.

iii) The majority used the Type 2 continuity method with twin rows of permanent bearings on each pier with overlapping reinforcing bars and/or prestressing extending from the beam ends to provide positive moment continuity.

iv) There was miscellaneous additional information covering design procedures, reinforcement details, construction sequence and problems encountered - mainly during construction.

Early Age Creep and Shrinkage Tests

With a lack of data on the early age time-dependent effects of creep and shrinkage, tests were undertaken using four precast concrete beam manufacturers. All test elements were steam cured in accordance with current practice. Results were compared with ACI-209 (17) "Prediction of Creep, Shrinkage and Temperature Effects in Concrete Structures" which generally somewhat underestimated test results. Nevertheless, it was decided to use the ACI-209 procedures for the two computer programs selected for studying the next two items.

Computer Analysis of the Effect of Time-Dependent and Bridge Design Parameter Variations on Service and Ultimate Load Behaviour

The service behaviour effects were studied using a modified version of computer program PBEAM, which is capable of analysing composite prestressed/reinforced concrete bridge decks of any cross section having one axis of symmetry. The program accounts for the effects of nonlinearity of stress-strain responses of materials and variation with time of strength, stiffness, shrinkage of concrete and creep of concrete and prestressing steel.

PBEAM allows unlimited flexibility in analysing various timed construction sequences and live load applications. It also caters for the effects of cracking over the supports in both the top deck slab and the soffit of the support diaphragm. A "tension-stiffened" effective stress-strain relationship for top and bottom support reinforcement is used to model behaviour of reinforcing steel at cracked sections. This in turn leads to the analysis of changes in support continuity due to deck slab diaphragm cracking and due to diaphragm crack closure.
Comparisons were made between analysis and test results by using PBEAM to predict the results of the PCA tests undertaken by Mattock in the early sixties (Reference 7 and Section 4.10). The good observed correlation indicated that PBEAM is capable of adequately modelling the time-dependent and nonlinear behaviour of simple span girders made continuous.

PBEAM is a sophisticated computer program developed by Suttikan and uses a discrete linear element method developed by Hays and Matlock. Appendix C of the Report describes this method of analysis but it is not published. It is stated that "Qualified researchers may obtain loan copies, or microfiche may be purchased by written request to NCHRP, Washington, DC". Although this information is currently not to hand, it is apparent that PBEAM could probably be modified to suit UK parameters.

The effects of ultimate load behaviour were studied using a computer program labelled WALL_HINGE. This was developed to analyse reinforced concrete shear walls and predict failure modes including web and compression zone crushing. A detailed description of the program is given in an unpublished appendix.

To quote:-

"The program is used to determine the strength, inelastic deformation capacity, and failure modes for the hinging region of structures subjected to combined axial load, moment, and shear. The analysis considers both longitudinal and transverse equilibrium through the hinging region and across the transverse plane near the support. The analysis accounts for a nonlinear strain distribution within the hinging region using a compatibility relationship between the summation of tensile strain over the hinge length resulting from the "fanned" cracking pattern with the summation of compressive strains within a relatively short length near the tip of the fan.

This is an important effect to consider in analyses for flexural-compression or shear-compression failure modes that is not accounted for in typical sectional analyses made assuming a linear strain distribution. The important effects of dowel action, aggregate interlock, and interaction of compressive stress and shear stress in the compression zone for analysis of transverse equilibrium across a plane near the support were included. The program uses a complete tri-axial concrete stress-strain relationship developed by Ahmad and Shah, with strength criteria based on a critical octahedral shear stress.

In order to analyse negative moment ductility of composite prestressed girders, the precompression effects of prestressing strand in the bottom flange of the girder were included. The effective prestress in strand was estimated at an assumed critical plane located a distance from the end of the girder equal to the neutral axis depth at ultimate moment. This accounts for experimentally observed behaviour in which concrete crushing occurs beyond the end of the girder enclosed in the cast-in-place diaphragm."

Comparisons were again made between analysis and test results using WALL_HINGE to predict the results of some of the PCA tests undertaken in the same series referred to in 3.4. Again, the good observed correlation, particularly where concrete crushing occurs prior to
steel yield, indicated that WALL_HINGE is capable of adequately modelling the ultimate load behaviour of simple span girders made continuous.

**Service Moment Parametric Studies**

Parametric studies were carried out using PBEAM to examine the effects of varying the following characteristics:-

i) Girder type.

ii) Span length.

iii) Girder spacing.

iv) Positive moment continuity reinforcement.

v) Time-dependent material properties (a) concrete (compressive strength, creep coefficient, shrinkage strain) and (b) prestressing steel (relaxation characteristics).

vi) Girder age when deck and diaphragm are cast.

vii) Girder age at application of live load.

viii) Construction sequence.

Responses to the questionnaire were used to set the limits to the range of values of some of the characteristics.

The parametric studies indicated that, with continuity established at an early girder age the time-dependent positive moment developed at the supports is highly dependent on the amount of positive reinforcement provided. A crack is generally induced in the bottom of the diaphragm concrete. With the application of live load the positive moment crack must close prior to inducing negative moment at the continuity connection.

Increasing the positive reinforcement lessens the cracks, thereby increasing the apparent live load continuity. However, the extra positive moment resulting from this extra positive reinforcement also increases the positive midspan resultant moments. It turns out that the effect of the lack of full continuity caused by not providing positive reinforcement in the diaphragm and allowing the bottom diaphragm crack to open is virtually balanced by the increased positive restraint moment caused by providing this reinforcement. Therefore, the resultant midspan moments, which include moments due to dead load, restraint moments due to creep and shrinkage, and live load moments, are independent of the area of positive reinforcement provided in the diaphragm connections at the supports. It is concluded that providing positive moment reinforcement has no benefit for flexural behaviour of this type of bridge.

The primary advantage of positive moment reinforcement is in maintaining a smaller crack near the bottom of the diaphragm. However, questionnaires returned by four state
departments of transportation (California, Florida, Minnesota and Wisconsin) reported using continuous decks over the supports without any positive moment connection.

There were no serviceability problems associated with the lack of positive moment connections. Because construction of this detail is expensive, time-consuming, and difficult, the provision of positive moment reinforcement at the supports is therefore not recommended in the report.

Results of the parametric study also indicate that the effective continuity for live load can vary from 0 to 100 per cent. Therefore, the time dependent effects must be considered if continuity is to be counted on to reduce positive moments at midspan.

The report states that:

"Continuity performance is highly dependent on the age of the girder when the diaphragm and deck are cast. There is a structural advantage gained for design of the prestressed girders for positive midspan moment by delaying casting of the deck and diaphragms. High negative restraint moments can occur at the support connections when continuity is established at late girder ages. Full continuity for live load can be established depending on the age of girder at time of casting the deck and diaphragm and on the creep coefficient for the girder concrete. However, delaying casting of deck and diaphragm may require a delay in bridge construction. Also, the design moment for negative reinforcement in the deck over the supports is increased and the potential for transverse cracking in the deck is increased by delaying construction."

The report also recommends that the maximum negative continuity moment to be used for reduction of midspan positive moment should not exceed 125 per cent of the negative cracking moment.

The time-dependent restraint moments are calculated by initially assuming full structure continuity and then using a simplified computer program BRIDGERM developed by CTL.

The calculation of additional dead load and live load moments is undertaken using conventional computer programs, but CTL have also developed a further simplified computer program called BRIDGELL for use where no positive reinforcement is provided over supports. In effect, this inserts hinges at supports where positive moments would arise if the positive continuity reinforcement had been provided. Undoubtedly, a similar UK computer program catering for UK live loading could be readily developed.

Ultimate Load Moment Parametric Studies

Time-dependent restraint moments are generally considered to have no effect on the ultimate load behaviour. They are not therefore included in the ultimate load moment calculations, a procedure which applies equally with BS 5400 Limit State Design.

An American requirement is covered by AASHTO (14) Section 8.17.1 whereby:

"Minimum deck reinforcement shall provide strength equal to 120 per cent of the negative cracking moment, calculated using Equation 8-2".
Also to ensure ductile behaviour and to develop the full strength of the composite section, the deck reinforcement ratio \( \left( \frac{A_d}{bd} \right) \) in negative moment regions shall not exceed 0.5\( p_b \), where:

\[
p_b = \left[ 0.85 f'_c (A_f + (x_b - h_f)t_w) \right] / [f_y bd]
\]

In this expression

\[
\begin{align*}
f'_c &= \text{girder concrete compressive strength, psi} \\
A_f &= \text{cross-section area of bottom flange, up to junction with web, sq. in.} \\
x_b &= \text{depth of compression block, in.} \\
h_f &= \text{depth of bottom flange, up to junction with web, in.} \\
t_w &= \text{web thickness, in.} \\
f_y &= \text{steel yield stress, psi} \\
b &= \text{width of bottom flange, in.} \\
d &= \text{depth from extreme compression fibre to centroid of deck reinforcement, in.}
\end{align*}
\]

The parametric study of negative moment ultimate strength indicated that typical American deck sections have adequate rotational ductility. Nevertheless, the 0.5\( p_b \) upper limit should always be checked for negative reinforcement design.

**Proposed Revision to AASHTO Specifications for New Design Procedures for Precast Prestressed Girders Made Continuous**

These proposals are added in Appendix G of the NCHRP report and it is stated that they may be considered for adoption by AASHTO in 1990.

The revisions, based on the reports findings, commence with a new definition:

"**Effective Restraint Moments** - Moments occurring at continuity supports of bridges composed of simple-span precast, prestressed girders made continuous, due to creep and shrinkage of precast girders and cast-in-place deck concrete. The effective restraint moments shall be computed assuming full structural continuity for negative and positive moments at continuity supports."

The most important of the proposed revisions are as follows:

**Service Limit State**

i) Provision of positive moment reinforcement is optional. Effective restraint moments must be computed for either option.

ii) If positive reinforcement is not provided, continuity moments should be calculated assuming lack of positive moment continuity.

iii) If positive reinforcement is provided, calculated service load stresses must not exceed 0.6\( f_y \) or 36,000 lb/in\(^2\) (248 N/mm\(^2\)). This limit
iv) The maximum negative continuity support moment used to determine the positive service moments within the span shall be limited to 125% of the negative cracking support moment.

v) Some existing AASHTO fatigue requirements for the negative reinforcement are quoted.

**Ultimate Limit State**

vi) Effective Restraint Moments shall not be considered for strength (ultimate limit state) design of negative moments in the support regions.

vii) The critical section for concrete compression in the girders occurs away from the support pier and diaphragm. The negative moment strength shall therefore be calculated using the compressive strength of the girder concrete regardless of the diaphragm concrete strength. The effect of initial precompression due to prestress in the girders may be neglected.

viii) Negative reinforcement proportion \( p = (A_v/bd) \) shall not exceed 0.5 of the value of \( p_b \) that would produce balanced strain conditions for the composite section, where:

\[
p_b = [0.85 f'_c A_v] / [f_c bd]
\]

\( A_v \) is the actual concrete compression area on the bottom of the girder at the balanced condition. The effect of compressive strain due to prestress may be neglected in calculation of \( p_b \).
RELEVANCE OF NCHRP REPORT 322 TO THIS REPORT

General

NCHRP Report 322 represents several years of extensive research into precast bridge beam continuity. However, its relevance to this Report is somewhat limited as it is devoted almost entirely to Type 2 applications and says little or nothing about the other five methods Types 1, 3, 4, 5 and 6. Nevertheless, the information on Type 2 applications is considerably enhanced and has been usefully added to Section 4.2, 4.4, 4.11, 4.12 and 4.13.

APPLICABILITY TO UK PRACTICE

Undoubtedly the NCHRP Report 322 proposals, together with an adapted version of BRIDGERM could be used for Type 2 UK applications. Precast beam manufacturers should welcome the positive moment reinforcement recommendations, although most would probably opt for some projecting reinforcement or strand rather than none at all.

However, the design requirements remain complex and it is a pity that the American Report does not propose simpler design methods for what is essentially the simplest of the continuity connections to construct. With all the questionnaire data and the various parametric investigations it is surprising that a range of simple continuity parameters could not be provided to cover the most common time and construction sequences. These could possibly be combined with "deemed to satisfy" specification clauses.

It would also be important to establish whether the NCHRP Report 322 could be applied to the more desirable single permanent bearing Type 2 connection.

One final point concerns the ductility and fatigue considerations recommended. These appear to be desirable additions to UK design requirements.