VOLUME 3 INSPECTION AND MAINTENANCE

SECTION 2 MAINTENANCE

PART 5

BD 54/15

MANAGEMENT OF POST-TENSIONED CONCRETE BRIDGES

SUMMARY

This standard provides the requirements and guidance for the management of post-tensioned concrete bridges. It implements a risk based approach to management and includes advice on Special Inspections, monitoring, repairs and strengthening.

INSTRUCTIONS FOR USE

1. Remove existing contents pages for Volume 3 and insert new Contents pges for Volume 3 dated February 2015

2. Remove BD 54/93 from Volume 3, Section 1, which is superseded by BD 54/15 and archive as appropriate.

3. Insert BD 54/15 into Volume 3, Section 2 after Part 4.

4. Archive this sheet as appropriate.

Note: A quarterly index with a full set of Volume Contents Pages is available separately from The Stationery Office Ltd.

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BD 54/15
Volume 3, Section 2,
Part 5

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1. INTRODUCTION

Background

1.1 Generally post-tensioned concrete bridges give good service and few have significant problems. However, they are particularly vulnerable to corrosion and severe deterioration where internal grouting of tendon ducts is incomplete and moist air, water or de-icing salts can enter the ducting system. The ingress of water and salts into tendon ducts is most likely at joints in segmental construction, other construction joints, at anchorages and over intermediate supports when the deck is continuous.

1.2 A programme of Post-tensioned Concrete Bridge Special Inspections (PTSI) undertaken in the UK in the 1990s showed that the condition of the post-tensioning system in most cases was satisfactory although many contained voids in the ducts and light corrosion of tendons. A small number were less satisfactory and needed repair work. Others are likely to need attention in the future (Woodward R, 2001).

1.3 Since the 1990s PTSI programme Overseeing Organisations have tended to rely on the principal and general inspection programme to identify signs of deterioration. Experience has shown that it is the construction detail and practices that are of the greatest significance in affecting deterioration of post-tensioned concrete bridges. However, these inspections do not identify the internal conditions of the post-tensioning system so as time elapses beyond the original PTSI, the level of uncertainty about the internal condition of tendons increases. Whereas for most forms of construction, defects tend to be relatively easy to interpret and result in ductile modes of failure, safety critical defects in post-tensioned concrete are typically hidden, very difficult to detect and may result in a brittle mode of failure. The primary problem with PT bridges which sets them apart from other types is the difficulty of establishing the internal condition of the tendons because outward signs of distress are not generally expected to occur. For this reason the most vulnerable bridges need monitoring and reinspection after an appropriate interval. Visual inspection methods alone cannot give warning of imminent collapse and internal inspections can be expensive and potentially damaging for the structure and should only be carried out if there is a clear need. It is essential that a combination of techniques and procedures be adopted as part of a system of risk assessment and risk management, to assist bridge managers in decisions as to when to undertake further intrusive investigations and when to use other risk management measures.

1.4 This document provides a process of risk review, risk assessment and risk management for post-tensioned concrete bridges with advice on the activities that may be needed for the successful risk management of post-tensioned concrete bridges.

1.5 This standard supersedes BD 54/93, BA 50/93 and BA 43/94, which are hereby withdrawn.

Scope

1.6 This standard is applicable to the management of post-tensioned concrete bridges that are the responsibility of the Overseeing Organisations. The scope includes bridges, any parts of which have been constructed using post-tensioning techniques in which tendons (bars, single strands or multi-strands) are tensioned so as to apply a compressive force to pre-cast concrete or hardened in situ concrete elements. The tendons could be in ungrouted, grouted or greased internal or external ducts, or otherwise protected (eg concrete or mortar surround), or unprotected. The tendons could be in longitudinal, transverse, secondary, vertical or tie down members.

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Purpose

1.7 This purpose of this standard is to set out the requirements and advice for risk review, risk assessment and risk management, special inspections, repair, strengthening and monitoring of post-tensioned concrete bridges that are the responsibility of the Overseeing Organisations.

Mandatory Sections

1.8 Sections of this document containing mandatory requirements are identified by being contained in boxes. These requirements must be complied with or a prior agreement to a Departure from Standard must be obtained from the Overseeing Organisation. The text outside boxes contains advice and explanation, which is commended to users for consideration.

Mutual Recognition

1.9 The requirements and guidance in this document are given on the basis that construction and/or maintenance of post-tensioned concrete bridges will be carried out using the Specification for Highway Works (MCHW Vol.1). However, products conforming to equivalent standards and specifications of other member states of the European Union and tests undertaken in other member states may be acceptable in accordance with the terms of the 104 and 105 Series of Clauses of that Specification.

Devolved Administration Issues

1.10 Not Applicable.

Implementation

1.11 This Standard must be used forthwith on all projects for the assessment, design, construction, operation and maintenance of motorway and all-purpose trunk roads (and all roads in Northern Ireland) except where procurement of works has reached a stage at which, in the opinion of the Overseeing Organisation, its use would result in significant additional expense or delay progress (in which case the decision must be recorded in accordance with the procedure required by the Overseeing Organisation).

Personnel

1.12 The person responsible for overseeing Risk Review, Risk Assessment and PTSI must have specialist experience of post-tensioned bridge design and construction methods, and have proven and demonstrable wide experience of post-tensioned bridge management, inspection, testing and monitoring procedures.

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Definitions and Abbreviations

1.13 For the purpose of this standard, the following definitions apply:

Maintaining Organisation – The organisation appointed by the Overseeing Organisation to manage highway assets on its behalf.

PTSI Site Inspection – A visual inspection of a bridge in accordance with Chapter 6 of this standard.

PTSI Site Investigation – A detailed investigation of a bridge involving, external examination, materials testing, NDT and internal examination, in accordance with Chapter 7 of this standard.

Project Manager – The person appointed by the Maintaining Organisation to manage a PTSI in accordance with paras 1.12 and 5.4.

Risk Review – an examination of a previous risk assessment and other information with the objective of identifying the need for further risk assessment, in accordance with Chapter 2 of this standard.

Risk Review Report – A report on the Risk Review as defined in paras 2.18 to 2.22.

Risk Assessment – the identification of hazards, risk events, likelihood, consequences, risk level and Risk Rating in accordance with Chapter 3 of this standard.

Risk Assessment Report – A report on the Risk Assessment as defined in para 3.36.

Risk Management – The use of various measures to mitigate or remove identified risks in accordance with Chapter 4 of this standard.

Risk Management Plan – The application of a set of Risk Management measures to an individual structure.

1.14 The following abbreviations are used in this standard:

AADT – Annual Average Daily Traffic

ALL – Assessment Live Load

ASR – Alkali Silica Reaction

NDT – Non-Destructive Testing

PTSI – Post-Tensioned concrete bridge Special Inspection

SA – Sulfate Attack

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2. RISK REVIEW

General

2.1 An ongoing system of Risk Review, Risk Assessment and Risk Management will be used to provide assurance on the safety of the stock of post-tensioned concrete bridges. The overall process is illustrated by the flowchart in figure 2.1. The process starts with a Risk Review.

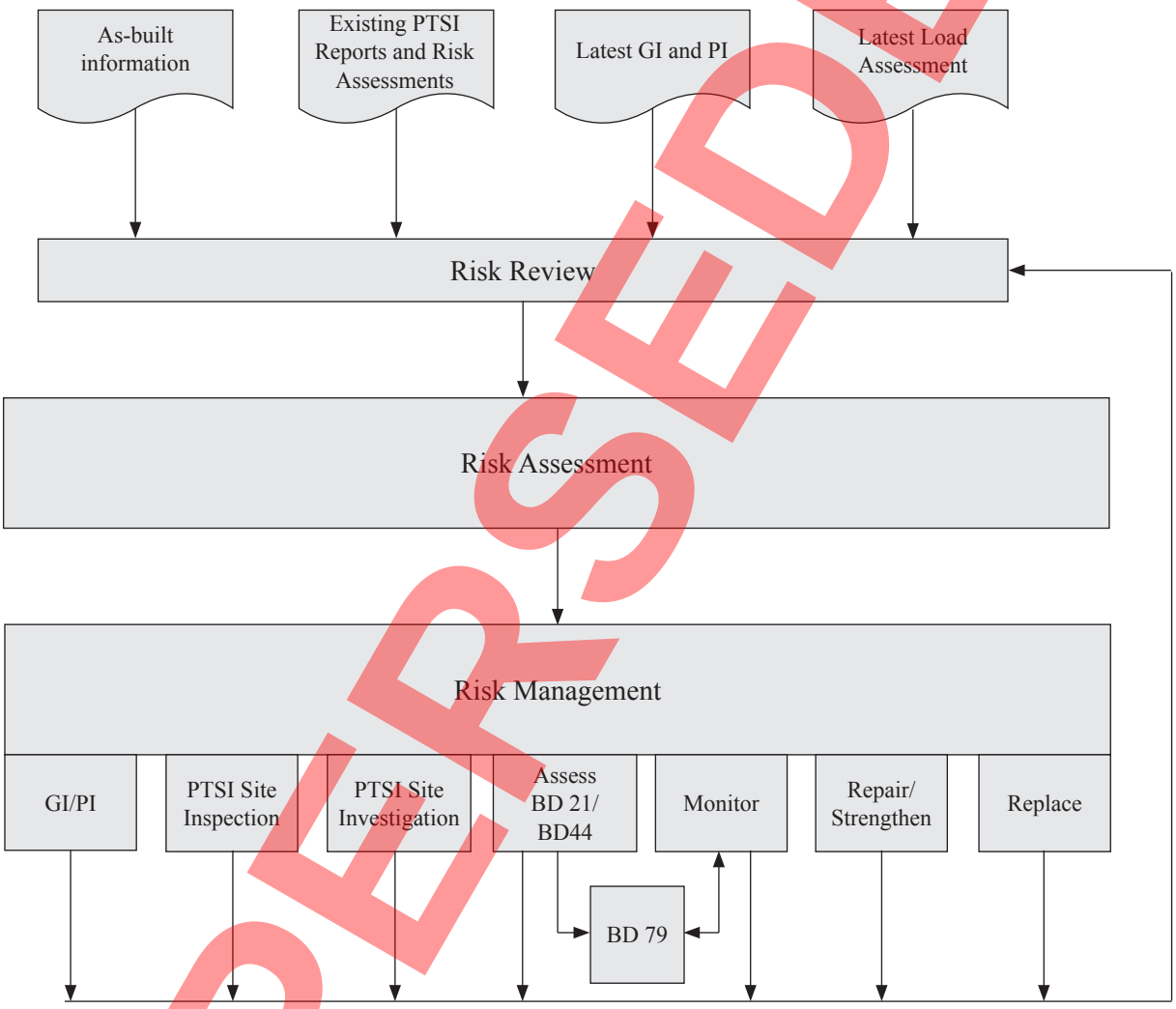

Figure 2.1 – Overview of the Risk Review, Risk Assessment & Risk Management System

2.2 A Risk Review must be carried out concurrently with each Principal Inspection. Further Risk Reviews must be carried out when further significant information is generated from ongoing Risk Management activities. The first Risk Review may be carried out before the next scheduled Principal Inspection with the agreement of the Overseeing Organisation.

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Risk Review

2.3 The primary purpose of the Risk Review is to consider the adequacy of the most recent Risk Assessment. Where there is no Risk Assessment available, or the most recent Risk Assessment is found to be inadequate, obsolete, or otherwise in need of updating, the purpose of the Risk Review is also to assemble the relevant information necessary to undertake a new Risk Assessment or update an existing one.

2.4 The following information should be examined in the risk review:

- The most recent Risk Assessment
- As-built drawings, construction records and other as-built information
- Historical PTSI reports
- Recently completed PTSI reports
- Recommendations from previous PTSI checked against maintenance records for actions taken
- Principal, General and Special Inspection reports to BD 63
- Latest load assessment to BD 21, BD 44 and other relevant assesment standards
- BD 79 reports and records
- Monitoring reports.

2.5 Where a Risk Review (or a PTSI Preliminary Desk Study to BA 50/93) has been carried out previously, this should be cross-referenced to avoid unnecessary duplication of work.

2.6 The Risk Review should be carried out as a desk study to determine the design and construction details and to review the previous inspection and maintenance records for the bridge that are needed for the Risk Assessment. This will also enable construction details to be verified and previously recorded deterioration or repairs to be checked should the Risk Assessment deem a new PTSI necessary.

Review of Risk Assessments

2.7 In examining the most recent Risk Assessment, factors to consider include:

- time since last Risk Assessment
- whether hazards previously identified are still existing
- whether the recommended Risk Management measures are in place
- whether any known hazards are missing from the Risk Assessment.

As-built details and construction records

2.8 The details required include the type of deck, mode of articulation, degree of redundancy in the deck and the structural dimensions of the primary and secondary post-tensioned members. Information on the type of post-tensioning system, location of individual tendons and end anchorage positions is also needed, since observed surface defects may be related to the internal pre-stressing details.

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2.9 The construction records should be examined to determine the method and sequence of construction, stressing and grouting and the type of deck waterproofing. These details may indicate the possible locations and reasons for defects in the bridge.

2.10 The original specification for the grout and concrete mixes should provide information on the cement content, water/cement ratio and compressive strength. Where possible, the construction records should be consulted to ascertain the type of cement, sand and aggregate used in the grout and concrete mixes, the curing times and the age of the sections at the time of stressing. Information on any additives, air entraining agents or cement replacements will be particularly relevant. These details may indicate the likely permeability, durability and relative performance of the various sections in a bridge.

Previous PTSI Reports

2.11 Previous PTSI reports should be examined. These are likely to include the required information on as-built details, construction records and critical sections. Gaps and inadequacies in previous PTSI reports should be identified. Recommendations from previous PTSI reports should be checked against maintenance records to identify whether they have been implemented.

Critical Sections

2.12 If not already done in a previous Risk Review or PTSI, critical sections should be identified as part of the risk review. A critical section is one at high risk from water ingress or corrosion, including regions where voids may form preferentially in ducts, or where yield points may form in a collapse mechanism, or both. Typically critical sections will include end support and anchorage regions, half joints, midspan areas, regions over intermediate supports (and other duct high points) and any form of construction joint transverse to a post-tensioned cable and duct.

2.13 Critical points on tendons located at critical sections should be identified, classified for type (eg mid-span, construction joint), and the number present for each type stated.

2.14 The Risk Review should identify what investigation has already been carried out to each critical section and whether sufficient site investigations have been made to identify the condition of the post-tensioning system at these sections.

Inspection, Assessment and Maintenance records

2.15 Previous inspection reports should be reviewed to identify known defects and differentiate them from problems identified for the first time during any new PTSI deemed necessary by the Risk Assessment. Particular attention should be paid to the reasons for any previous repairs to the bridge. Results of testing carried out during the course of previous inspections, such as chloride content, cement content, sulfate content, alkalinity, carbonation, cover, resistivity and half-cell potential, should be recorded to supplement data obtained during any new PTSI, so that an assessment can be made of changes in condition over time.

2.16 Reports on previous load assessments should be reviewed for consistency with the current condition, assessment standards and operational loading. The adequacy of previous load assessments with respect to identifying sensitivity to loss of prestress and possible failure modes should be considered. Methods of analysis used should be identified. Where the structure is or has previously been managed under BD 79, the BD 79 reports and records should be reviewed.

2.17 Details of any previous monitoring, reference points or datum readings may be valuable during any new PTSI. Similarly, information on the materials and techniques employed in carrying out previous repairs will be very helpful when judging their performance and relative value in future repairs of the same nature.

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Risk Review Report

2.18 The report should advise on the adequacy of any previous risk assessment and make a recommendation on the need for further risk assessment. The report should identify whether there is sufficient information available to undertake or update a risk assessment and the adequacy of any previous investigations at critical sections. Where the available information is not considered sufficient to carry out a viable risk assessment, for example missing information on material properties, section geometry, the prestressing system and reinforcement details at critical sections, recommendations on the action needed to collect the necessary information should be provided in the Risk Review Report. This is likely to involve use of the risk management tools identified in figure 2.1, typically a PTSI Site Inspection followed by a PTSI Site Investigation.

2.19 Where the Risk Review identifies the need for a PTSI Site Inspection, the objectives for the PTSI Site Inspection should be clearly identified in the Risk Review Report. These objectives should be based upon the available design and construction information and the previous maintenance history of the bridge, and any missing information that needs to be collected should be identified.

2.20 It is not necessary to duplicate reporting that has been completed satisfactorily under previous Risk Reviews (or PTSI Preliminary Desk Study to BA 50/93). However, where this has not previously been completed, the report should include a summary of the essential design, construction and maintenance details, presented in a systematic format in an Appendix to the main report.

2.21 The design details and construction records should include the following principal groups of information:

a. List of available record drawings.

b. Form of construction.

c. Type of concrete.

d. Prestressing and reinforcement.

e. Protection systems – ducts and grout

e. Construction information.

2.22 The differences between design and actual construction should be identified. The maintenance history of the structure should also be summarised. The information is required under three main headings:

a. List of previous inspection reports.

b. Summary of previous defects and repairs undertaken.

c. Actions taken in response to recommendations in previous PTSI reports.

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3. RISK ASSESSMENT

Risk Assessment

3.1 The primary purpose of the Risk Assessment is to ensure that bridge owners and bridge managers understand the risks associated with a particular structure, enabling selection of the appropriate risk management tools over the life of the structure. Risk assessments are also used to derive risk ratings that can be used for the prioritisation of inspections and repair works. Risk Assessments and updates to Risk Assessments must not be carried out unless recommended by the Risk Review and the Overseeing Organisation agrees with the recommendation.

3.2 The Risk Assessment should draw on information identified in the Risk Review including previous PTSI and Risk Assessment reports. It should not unnecessarily duplicate previous work. The Risk Assessment is intended to be a live document that is updated throughout the life of the structure as new information is fed in. The Risk Assessment should identify the hazards that could affect a structure and the associated risk event. The likelihood and consequences of the event occurring should be derived.

3.3 The Risk Assessment Comprises two steps that must both be applied in all cases:

1. Primary Risk Assessment comprising a detailed qualitative risk assessment, (see paras 3.4 – 3.5), which is carried out first, followed by;

2. Risk Rating, for prioritisation and ranking purposes (see paras 3.6 – 3.12).

Primary Risk Assessment

3.4 All available evidence must be examined and used to identify and list each hazard. Hazards must be identified under the headings age, form, vulnerable details, condition, history and assessment with reference to paras 3.13 – 3.35 and tables 3.5 – 3.10.

3.5 For each hazard identified, the assessor must state:

- the risk event (what could happen if the hazard is not dealt with in the short term, ie within 3-4 years)
- the likelihood of the risk event occurring (low, medium, high or certain, with explanation – see Table 3.1 for guidance)
- the consequences of the risk event occurring (low, medium or high, with explanation – see Table 3.2 for guidance)
- a hazard risk level – see Table 3.3 for guidance
- a proposal for risk management measures.

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Table 3.1 Guidance on Likelihood Levels

Likelihood Level	Description
Certain	Certainty – evidence exists that the event has already happened, is happening or about to happen
High	Highly likely – experience shows such risk events do typically occur if such hazards are not dealt with
Medium	Possible and likely – theoretically possible and experience shows that such events sometimes occur if such hazards are not dealt with
Low	Possible but unlikely – theoretically possible but experience shows such events are rare

Table 3.2 Guidance on Consequence Levels

Consequence Rating	Description
High	Global structural collapse/High number of fatalities/closure of a strategic route
Medium	Local structural element failure/Small number of fatalities/high number of injuries/closure of a regional route/restriction of a strategic route
Low	Serviceability failure/Small number of injuries/fatalities unlikely/restriction of a regional route/closure or restriction of a local route

Table 3.3 Guidance on Hazard Risk Levels

		Consequences		
		High	**Medium**	**Low**
Likelihood	**Certain**	Very High	High	Medium
	High	Very High	High	Medium
	Medium	High	Medium	Low
	Low	Medium	Low	Low

Risk Rating

3.6 This rating is based on the age, form, vulnerable details and materials, present condition and consequences of collapse. The hazards identified in the Primary Risk Assessment are used to calculate the rating.

3.7 The Age Factor (F_A) is determined from Table 3.5.

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3.8 The Bridge Form Factor (F_F) is determined from Table 3.6.

3.9 To determine the vulnerable details and materials factor (F_V), score 1 for each vulnerable detail present from Table 3.7 up to a maximum score of 10.

3.10 To determine the Condition factor (F_C), score 1 for each condition hazard present from Table 3.8 that is present on the structure, up to a maximum score of 10.

3.11 Consequence Factor (F_Q) is intended to represent the consequences of a bridge collapse. F_Q is selected from Table 3.4. Where more than one situation applies, the highest score is to be selected. There are some circumstances where the traffic flow alone does not fully reflect the importance of a bridge. Examples include:

- bridges with no suitable diversion route or the diversion route is very long
- bridges on rural roads to ports serving island communities where there is no diversion route
- bridges that provide a link within a community where loss of the bridge would result in unacceptable community severance.

In such cases, the Consequence Factor may be increased by one (maximum score is 5).

Table 3.4 Consequence Factor (F_Q)

Feature	Consequence Factor (F_Q)
Traffic on or below the bridge (Two way AADT) Over 80000 Over 60000 to 80000 Over 40000 to 60000 Over 20000 to 40000 Up to 20000	 5 4 3 2 1
Railway below	5
Other areas below occupied by people, valuable installations, environmentally sensitive areas (such as conservation areas), storage facilities for hazardous materials, navigable river. Score according to the assessor's judgement.	1 to 3

3.12 The Risk Rating (R) can now be determined by combining the age, bridge form, vulnerable details and materials, condition and consequence factors. The Risk Rating is expressed as a percentage using the following expression:

$$R = 100\ [\ ((4F_A + F_F + F_V + F_C)\ F_Q) - 6\ ]/254$$

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Hazards

3.13 The hazards that are known to affect post-tensioned concrete bridges can be grouped together under six main headings:

- age
- form
- vulnerable details and materials
- condition
- history
- load assessment.

Age

3.14 Hazards related to age are a reflection of the fact that knowledge, experience, design standards, and specifications for methods and materials have improved over time. For example, the requirement for bridge deck waterproofing was first introduced around 1963 when mastic asphalt was the usual waterproofing material. Development of sheet membranes followed and by 1999 mastic asphalt was considered inadequate. The low age factor (Table 3.5) for bridges built from 1997 onwards recognises the first publication in 1996 of TR47 Durable Post-Tensioned Concrete Bridges (Concrete Society, 1996) which recommended new design and construction standards and practices (a second edition was published in 2002, which was in turn replaced by TR72 in 2010).

Table 3.5 Age Factor (F_A)

Date Constructed	Age Factor (F_A)
Pre 1965	5
1965 to 1975	4
1976 to 1985	3
1986 to 1996	2
1997 onwards	1

Bridge Form

Segmental Bridge Decks

3.15 In comparison with monolithic construction, all types of segmental bridge decks have a higher probability of a sudden mode of collapse. Many forms of segmental construction have been used for both simply supported and continuous bridge decks. The basic distinctions that can be made between them relate to the direction of the joint, the joint material and the width of the joint.

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3.16 Some segmental bridge decks were built without any form of composite action. In the extreme case of simply supported segmental beams, it is necessary to consider monitoring methods to provide a reliable warning of imminent failure. A combination of specialist techniques can be applied, but the technical approach needs careful planning and considerable experience. Longitudinal cracks may indicate that tendons have severed and re-anchored, and these cracks should be investigated and monitored with suitable instrumentation.

3.17 The probability of a sudden mode of collapse is reduced when simply supported segmental beams are transversely connected to form a grillage. There has to be a degree of load sharing and severe corrosion of longitudinal prestressing tendons along a potential fracture line before a failure can take place. Nevertheless, the risk is high if exposure conditions are severe and a general loss of prestress allows water to penetrate more easily into the joints. It should be noted that fracture lines across a grid may not be straight, so any long-term monitoring system has to be planned accordingly to take this into account.

3.18 Simply supported box girder bridge decks are similar to a beam grillage in terms of failure mode and the probability of a rapid failure mechanism is still high. Therefore, the monitoring procedures considered for this form of deck should be similar in principle to a beam grillage deck.

3.19 Composite decks generally represent a much safer type of bridge structure, since the presence of an in situ slab connecting precast segmental units provides a better degree of redundancy if isolated tendons should fail. Moreover, the slab helps to protect the beams from the ingress of salts including chlorides or sulfates.

3.20 Where a simply supported composite deck is also formed with an in situ bottom slab, the formation of a central hinge is less likely. The presence of untensioned reinforcement should be sufficient to spread the risk of failure between adjacent segments. In addition, the bonded untensioned steel will also provide a degree of gradual yielding, so that a regular monitoring procedure should be adequate to detect the onset of any failure mechanism.

3.21 In general, continuous forms of segmental post-tensioned decks carry a lower probability of a sudden mode of failure. Due to continuity over the intermediate supports, it is necessary for a mechanism to form before a collapse can occur. Therefore, at least two complete deck sections must yield in an end span of a continuous deck. Internal spans require three hinges to form a mechanism so a collapse condition is more unlikely to develop, but it may still occur without visible warning.

3.22 Where continuous bridge decks are formed from a series of beam segments with a composite reinforced concrete top slab, there is normally a significant amount of bonded untensioned reinforcement crossing the transverse joints. Hence, there should be an additional reserve of strength in the vicinity of the piers and a sudden type of collapse is less likely.

3.23 The broad categories of segmental decks in Table 3.6 are intended to illustrate the degree of risk of a brittle mode of failure associated with various types of structures. Where the risk is high, risk management measures to include PTSI and monitoring may be necessary. Sudden failure is more likely where there is no secondary reinforcement across the joints (however secondary reinforcement across joints may also mask loss of prestress that might otherwise be detected by the joints opening).

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Monolithic Bridge Decks

Table 3.6 Bridge form factor (F_F)

Bridge Form	Risk of Brittle Failure Mode	Factor (F_F)
Segmental		
Beams or box girders, simply supported, non-composite, transverse joints, longitudinal prestress	Very High	12
Beam grillage, simply supported, non-composite, longitudinal and transverse joints and prestress	High	10
Beams or box girders, simply supported with composite slab, transverse joints and longitudinal prestress	Medium	8
Continuous beams, box girders or portals with composite slabs, transverse joints and longitudinal prestress	Low	4
Continuous beams or box girders, non- composite, transverse joints and longitudinal prestress	Medium	8
Monolithic		
Beams, simply supported, non-composite, longitudinal prestress	Medium	6
Beam or box girder, simply supported, composite, longitudinal or longitudinal and transverse prestress	Low	4
Beams, simply supported/continuous, non-composite, transverse prestress	Very Low	2
Solid or voided slab, simply supported/continuous, longitudinal and/or transverse prestress	Very Low	2
Tie Downs		
Continuous bridges, cantilevers and suspended spans on half joints with anchor spans tied down for:		
• Live load only	High	10
• Both dead load and live load	Very High	12
Tied Supports		
Bridges with buried ties between supports	Very High	12

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3.24 Most forms of in situ post-tensioned monolithic construction carry little risk of sudden structural collapse. Solid slabs and voided slab decks represent the safest form of construction. Monolithic beams with or without composite slabs and monolithic forms of box construction are all unlikely to collapse without prior warning. Providing there are no built-in planes of weakness arising from construction joints, there is a low probability of all the prestressing tendons across a deck failing at specific transverse sections. However, the effects of tendon failure will also depend on the quality of the grouting. Where a duct is substantially ungrouted the whole tendon may become ineffective as a result of serious local corrosion. In an extreme case, where many ducts are ungrouted, it is conceivable that a hinge could form when several tendons corrode even when the loss of steel section does not occur on a single transverse section. Table 3.6 illustrates the degree of risk of a brittle mode of failure associated with various types of structures with monolithic decks.

3.25 The presence of continuity in a deck reduces the potential risk whereas half joints which rely primarily on the integrity of the tendons may present a higher risk. The significance of detailing should not be overlooked: for instance, the presence of reinforcement may reduce the risk of sudden collapse or increase the probability of detecting distress by allowing cracks to develop before failure.

Tie-down

3.26 A variety of concrete bridge decks, both reinforced and prestressed have been constructed on the tied-down principle. A common form of this type of construction is the cantilever and suspended span decks, using half joints to support the suspended span. The cantilever sections are stabilised by an anchor side span, which can be tied down by vertical post-tensioning if the side span has insufficient self-weight to counter-balance the suspended span. Similar tie-down side spans have also been formed in continuous bridge decks, where the end spans could go into uplift under certain loading conditions. This category of structure, regardless of the form of deck construction, carries a high risk of a sudden mode of collapse, if the vertical post-tensioning should fail.

Tied Supports

3.27 A small number of post-tensioned overbridges are known to have been built with tied supports, whereby a concrete encased steel tie is laid beneath the carriageway of the road under the bridge, connecting the bridge abutments. These members are extremely difficult to inspect and maintain and are considered highly vulnerable.

Vulnerable Details and Materials

3.28 The Risk Assessment should identify existing vulnerable details of design and construction that are more likely to allow deterioration (corrosion) in the post-tensioning system. For example critical sections where little spare capacity is available or areas of extensive cracking or poorly compacted concrete that may reduce the protection of the tendons against corrosion.

3.29 Table 3.7 provides a list of vulnerable details that has been compiled from past experience. For the purposes of the Risk Rating, this table is used for scoring current details. In cases where vulnerable details and materials have been removed but resulting deterioration remains, that deterioration is scored using Table 3.8.

3.30 Deck waterproofing started around 1963. Decks that are still not waterproofed are highly vulnerable to penetration of chloride laden water. Decks with older mastic asphalt systems are also vulnerable. Decks with modern roll, board, spray or brush applied systems are not considered vulnerable provided they are in sound condition.

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Table 3.7 Vulnerable Details and Materials

Segmental Joints	In descending order of vulnerability • Narrow in situ mortar • Wide in situ mortar or concrete • Match cast glued
Other Joints	• Construction joints intersecting anchorages or tendons/ducts • Half joints • Hinge
Prestressing System	• Lack of redundancy, eg small number of large tendons where a severe local defect might have a serious effect on strength • Tendons grouped together in one or two ducts rather than each tendon protected in its own duct • Tendons located close to the upper surface of the deck where failure of deck waterproofing may lead to corrosion • Use of metal spacers to separate post-tensioning wires (vulnerability to crevice corrosion – see Annex C) • Unlined ducts • Tendons protected only by mortar/concrete surround • Unprotected anchorages on external faces of beams • Anchorages concealed within joints or on upper surfaces of decks • Grout tubes exposed in top of deck slab
Water Management System	• Absence of an adequate drainage system • Absent or old deck waterproofing system • Absence of deck joint seals
Other Materials & Durability	• Low cover to reinforcement • Low concrete grade • Admixtures containing chlorides used in concrete or grout • Inadequate longitudinal reinforcement eg at joints or inadequate shear reinforcement (as determined by structural assessment and observation of cracks)

Condition

3.31 This part of the Risk Assessment considers the present condition of the structure. Table 3.8 provides a list of condition hazards that will typically be identified by routine inspections and PTSI.

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Table 3.8 Condition hazards

Cracking	In prestressed concrete sections – Various locations, crack directions and causes (see Table 6.1)
Water Management System	• Cracks and potholes on carriageway surface • Surface ponding on deck • Blocked drainage systems • Water trapped in boxes and other structure voids • Damaged or missing deck joint seals • Water leaks and staining on soffit and at joints and cracks
Deflection	• Excessive deflection • Differential vertical deflection
Spalling	• Corrosion • Freeze/thaw • Stress concentrations
Reinforcement Corrosion	• Visual evidence • Adverse half-cell and chloride test results
Joints	Cracks, leaks and stains
Bearings	• Deterioration and damage • Unexpected movement/rotation or failure to move/rotate as expected
Internal	
Ducts	• Incorrect location (due to flotation or other displacement) • Corrosion • Perforation • Presence of water
Grout	• Cracked or shattered • Soft • Moist • High chloride content • Voids • Ungrouted
Tendons	• Incorrect type or size • Missing tendons • Corrosion • Fracture of wires or tendons • Loose tendons (unexpected lateral movement)
Anchorages	• Voids • Moisture/water • Chlorides in grout/concrete • Corrosion

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3.32 Only corrosion rated severity 3 or above in accordance with table G10 of the Inspection Manual for Highway Structures can generate a score for the Risk Rating. Corrosion rated severity 1 or 2 does not score.

History Hazards

3.33 History hazards cover the areas of construction, operation, environment, inspection and investigation.

Table 3.9 History hazards

Construction	• Grouting problems • Tensioning problems • Honeycombing
Maintenance	• Failure to maintain water management systems
Operation	• Changes in highway cross-section (effect on loading) • Changes in loading (permanent or imposed)
Environment	• Use of de-icing salts (harshness of climate) • Proximity to chemical works • Proximity to the sea
Inspection and investigation	• Inadequate Principal/General/Special Inspections • No previous PTSI • Previous PTSI was incomplete or inadequate • Recommendations from previous PTSI have not been implemented

Assessment Hazards

3.34 Assessment hazards cover hazards identified by assessment findings or hazards due to lack of, or inadequate assessment.

Table 3.10 Assessment hazards

Assessment
• Structure has not been assessed for current condition, assessment standards or operational loading • Structure has been assessed as substandard • Structure has not been assessed for sensitivity to loss of prestress (eg % strand loss that can be tolerated) • Assessment has not identified and considered all possible failure modes

3.35 Tables 3.6 – 3.10 are intended to be comprehensive but not exclusive. Assessors should include in their assessment any hazards they identify that are not in the tables.

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Risk Assessment Report

3.36 The results of the Risk Assessment should be reported in a Risk Assessment Report. An example report is shown at Annex A. The report should include:

- A brief summary of the findings from the most recent Risk Review
- Details of the Primary Risk Assessment describing all identified hazards with explanation of the corresponding risk event, the likelihood of the event occurring, the consequences and risk level.
- A Risk Management Plan
- The Risk Rating calculation.
- Signature by the person defined in para 1.12.

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4. RISK MANAGEMENT

General

4.1 Each post-tensioned concrete bridge should be allocated an individual Risk Management Plan, which will comprise a set of bridge specific risk management measures.

4.2 The risk management measures will typically be derived from the Risk Assessment and should be included in the Risk Assessment Report. Figure 2.1 shows some typical risk management measures. Routine Principal and General Inspections will apply to all bridges and these alone are likely to be sufficient for low risk bridges. Additional measures could include Special Inspections, PTSI, load assessment, monitoring, repair, strengthening and in extreme cases replacement of elements, decks or whole structures. Guidance on Principal and General Inspections can be found in BD 63 Inspection of Highway Structures (DMRB 3.1.4). Guidance on PTSI can be found in chapters 5, 6 and 7 of this standard. Refer to BD 21 and BD 44 for the requirements and guidance for load assessment. Guidance on monitoring is in chapter 8, repair in chapter 9 and strengthening in chapter 10 of this standard.

4.3 Decisions on selection of risk management options should take into consideration the costs and benefits of those options drawing on individual structure asset management plans and whole life cost analysis as appropriate. In some cases monitoring schemes have been used successfully to delay repair or replacement until such time as it can be implemented in a cost effective manner.

4.4 The outputs from risk management measures should be fed back to the Risk Review and if necessary the Risk Assessment which should be routinely updated as knowledge of the structure increases. Decisions can then be taken on the need for further risk management measures.

Record Keeping

4.5 Record keeping is an essential element of risk management. The following must be stored on the Overseeing Organisation's designated record management system.

- Risk Review reports
- Risk Assessment reports
- Risk Management Plan
- Any reports or other outputs arising from Risk Management measures
- A list of recommendations arising from Risk Assessments and Risk Management Plans with a record of actions taken and when.

Risk management to keep substandard structures in service

4.6 The investigation and load assessment of post-tensioned structures supports risk assessment decisions on their adequacy for supporting the required traffic load. The risk assessment process may conclude that the structure is adequate to stay in service with only normal levels of inspection or it may be concluded that the structure is definitely unsafe and requires immediate closure and replacement or strengthening. However, between these extremes are a wide range of possibilities where it might reasonably be concluded that the structure is "sub-standard" but where it may be appropriate nevertheless to keep it in service, either in the interim whilst a more permanent solution is developed or sometimes indefinitely.

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4.7 Even if a post-tensioned concrete bridge has failed a structural assessment and investigations have shown defects in the post-tensioning system, case studies have shown that it may still be possible to keep the bridge in service. A combination of tools have been used to achieve this including structural analysis and load assessment, load mitigation, propping, acoustic monitoring, in situ stress testing, strain, displacement and temperature monitoring, load testing and visual inspection. In such cases structures are managed in accordance with BD 79 (DMRB 3.4.18).

4.8 Monitoring measures are typically central to management plans for keeping substandard structures in service (see Chapter 8). Specific issues with various other measures are considered below.

Structural Analysis and Load Assessment

4.9 Structural analysis and load assessment is an important risk management tool that should be used in conjunction with other risk management measures. It can be used before PTSI to identify failure modes and critical points to guide the site investigation. It can also be used after PTSI to analyse the effect of deterioration on the capacity of the structure and to identify sensitivity to loss of prestress. It informs the design of monitoring schemes to enable monitoring equipment to be placed at the most appropriate locations to detect the onset of failure.

4.10 BD 79 enables monitoring and other interim measures to be used either for provisionally sub-standard structures pending completion of a more refined assessment or for sub-standard structures pending strengthening or repair. The expectation is that the assessment will ultimately reach a conclusion which will be sufficiently definite to determine that the strength of the bridge is, or is not, adequate. In the assessment of existing post-tensioned structures, the biggest unknown will normally be the extent and detail of deterioration of the prestress. It is rarely practical to undertake sufficient intrusive investigation to attach a high degree of certainty to the assumptions about this which are used in the assessment. In assessing the structure and determining the risk of the structure collapsing, it will therefore be necessary to make conservative assumptions for this. This has two important implications:

1. A structure assessed as inadequate may, in fact have adequate strength.

2. To reach safe conservative assumptions about the deterioration and behaviour of tendons, it is necessary to understand which assumptions are safe. For assessing the current strength of the bridge, it will almost always be conservative to assume the maximum loss of tendon force, area and bond. However for other purposes, (e.g. detecting further deterioration, considering the integrity of a bridge with temporary props and also for designing some types of strengthening systems) other cases may be more critical. It will therefore often be necessary to consider a range of possible values when management of structures is considered.

4.11 Particular examples of the second point above include the effect of loss of bond. This will normally reduce the strength but will also make loss of prestress easier to detect as it will apply to all sections and not just the section where breaks occur. Another issue is loss of prestress. Many of the monitoring and other approaches considered in this standard assess prestress force. However, the ultimate strength of the structure is normally more sensitive to loss of area. Hence, it is very unlikely that measurement of force can be used to infer a tendon area due to various inaccuracies including initial loss of prestress and estimating prestress loss.

Reliability-based Methods of Assessment

4.12 BD 79 contains advice on reliability-based methods of assessment. The text in this section should be read in conjunction with the advice in BD 79.

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4.13 Past experience of satisfactory service can be used to justify the integrity of a structure, such as under a reduced load, even when assessment calculations do not. Where the assessment uses probabilistic reliability approaches, this can be done quantitatively by using Bayesian updating. A portion of the theoretical probability distribution can be eliminated because it would imply that the structure would have already collapsed, or at least shown serious signs of distress. Lack of collapse under a particular load or configuration cannot on its own prove safety is adequate in that situation as the factor of safety may be very small. However, where analysis shows failure would be preceded by detectable signs, it may be possible to infer from the lack of such signs that there is an adequate safety margin. If the signs would only be detectable whilst the load is applied, e.g. if they take the form of reversible strains or cracks that close on unloading, load testing may be appropriate (see BA 54 (DMRB3.4.8)). This approach could be used for revising a lower-bound to remaining prestress.

4.14 The approach can also be useful in combination with load mitigation or propping or other strengthening methods. By determining that a structure has a margin (although not necessarily an adequate margin) in one situation, a greater certainty can be attached to the assessment under a less onerous loading or in a less onerous configuration. Most commonly, in the case of post-tensioned structures, this will be done by updating upwards the estimate of the lowest possible prestress level used in the assessment.

4.15 The approach is only valid if the condition of the structure is and continues to be similar to that in the previous period considered. It is therefore valid for resolving issues of uncertainty in the as-built assessment. In this application, for many larger post-tensioned bridges, the satisfactory completion of construction is sufficient to eliminate a substantial portion of the theoretical distribution of failed structures that are implicit in a normal reliability assessment.

4.16 The approach is more limited for considering effects of deterioration. It can, however, be useful in combination with techniques that monitor continuing deterioration, such as acoustic emission. If this combination of approaches is used after emergency propping or load mitigation measures it is important to bring the monitoring into effect as quickly as possible. This minimises the risk of the safety margin being compromised by deterioration between the imposition of the mitigation methods and the mobilisation of the monitoring.

Load Mitigation Measures

4.17 This section should be read in conjunction with the requirements and guidance on Load Mitigation Interim Measures in BD 79.

4.18 It may be possible to keep structures in service by reducing the live load. This may be done by restricting width or height of the carriageway to reduce the number of lanes or to prevent heavy good vehicles from using the bridge. Weight limits can also be used. However, although effective where there are problems of fatigue or possibly on long span bridges where several vehicles are needed to apply full design loading, they cannot normally be enforced sufficiently reliably to be a safe approach for avoiding collapse due to static overloading. Where shortfalls are only under accidental wheel loading, installation of partially effective barriers as considered in BD 79 may be appropriate.

4.19 A further possibility on short span bridges is to restrict speed to such an extent that the dynamic factor included in BD 21 (DMRB 3.3.3) can be reduced. Restricting width to make it difficult for large vehicles to travel quickly is a more effective way of doing this than imposing speed limits and also has the advantage of reducing unnecessary restrictions on lighter vehicles which, being narrower, will be less affected by width restrictions. However, reference should be made to road geometry standards and road safety implications.

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Propping

4.20 Propping has frequently been used as a temporary measure for structures which are considered unsafe. The major difficulty is the practical one of providing props in the area of a span where it was previously considered undesirable to have supports. Where there is risk of the props being hit by errant vehicles this should be assessed. Because of the very high reliability expected of structures, it is quite possible for a propping system that appears desirable from a structural point of view, to actually increase overall risk due to increased risk of collisions.

4.21 Propping prestressed structures away from the areas where they were intended to be supported imposes particular issues. Props can induce unacceptable stresses due to inducing hogging moments in regions designed only for sagging moments. This can sometimes be controlled either by using flexible props or by ensuring the props have only minimal load under permanent conditions and only take live load. In assessing these effects, it will be necessary to consider an upper bound to prestress as well as the lower bound used in the assessment without the propping. Alternatively, safeguarding props can be considered. These carry no permanent or live load under normal operating conditions and only accept load at the point of deck failure, thus minimising the consequences.

4.22 Propping at mid span could cause excessive cracking in the top of the section or compressive stress in the bottom. Where the structure is to remain in this condition permanently or later be strengthened, these conditions should be checked for SLS in the normal way. However, the prime reason for restricting compressive stress to $0.4f_{cu}$ (rather than $0.5f_{cu}$ for reinforced concrete) is to avoid high creep. In this particular circumstance, this would only reduce the prestress towards the level assumed in assessment. Hence, compressive stresses of up to $0.5f_{cu}$ induced by assumed prestress greater than the minimum value considered in the assessment may be disregarded. Where the propping is temporary pending imminent demolition, an ultimate strength only assessment will be adequate. In such cases, it is possible for the shear assessment to give a higher strength with minimum prestress so maximum as well as minimum prestress should be considered.

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5. PTSI – ORGANISATION

General

5.1 The general requirements and procedures for inspections are described in Departmental Standard BD 63 Inspection of Highway Structures (DMRB 3.1.4). Further advice is available in the Inspection Manual for Highway Structures (Highways Agency, 2007) and in BA 35 (DMRB 3.3).

5.2 The aims of a PTSI are:

- To identify bridges within which the post tensioning system has deteriorated significantly and recommend remedial measures
- To identify bridges where deterioration of the post tensioning system has started or the conditions for deterioration are present and establish the basis for future management decisions.

5.3 A PTSI is a risk management measure that may follow a Risk Review and Risk Assessment (see chapters 2 – 4). The PTSI comprises a Site Inspection and if necessary a Site Investigation. The PTSI is intended to be a flexible process that can respond to circumstances as they arise. Details of the PTSI Site Inspection can be found in Chapter 6 and the PTSI Site Investigation in Chapter 7.

Inspection Management

5.4 The Project Manager for a PTSI must be as specified in para 1.12. They must prepare the programme for a PTSI, obtain all necessary approvals from the Overseeing Organisation, plan, manage and direct the site activities, and prepare the final report. The appointment of the Project Manager is subject to the approval of the Overseeing Organisation.

Inspection Team

5.5 The Project Manager is responsible for the delegation of tasks within the Inspection Team. Team members should have a sound basic knowledge of the design of post-tensioned concrete structures, stressing procedures, grouting techniques and the operation of standard sampling and test equipment. Specific experience available within the Inspection Team should include the supervision of specialist methods of inspection for detection of reinforcement corrosion, voids in tendon ducts and corrosion of tendons, methods for concrete removal and instrumentation techniques.

Planning

5.6 Planning of the PTSI Site Investigation will be aided by knowledge of the sensitivity of the structure to the loss of prestress or tendon area. If this information is not available from previous assessments or the design calculations, an assessment of the structure in as built condition is recommended before the Site Investigation. This is particularly important if the Site Inspection indicates a potential problem.

Special Techniques

5.7 A variety of highly specialised techniques are available (see Chapter 7) for the detection of voids in ducts, the corrosion of tendons and the determination of the stress conditions in the concrete and steel. However there is a need to recognize their limitations and they should not be employed unless it is clear that the output will be useable for analysis to demonstrate the capacity of the structure. It is essential

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that the proposed use of all specialised techniques should be clearly identified in the programme for the site investigation and approval should be obtained from the Overseeing Organisation in advance of preparing any contract documents. Techniques in this special category should be based upon fundamental research and calibration tests, but due to their complexity, the interpretation of the data requires extensive experience of the method. Therefore, all site staff operating these techniques will be subject to the approval of the Overseeing Organisation and full evidence of their relevant experience will be called for.

Safety

5.8 Attention is drawn to the need to comply with current health and safety legislation. Refer to BD 63 Inspection of Highway Sturctures (DMRB 3.1.4) and the Inspection Manual for Highway Structures (Highways Agency, 2007) for further requirements and guidance.

Risk of collapse

5.9 If at any time during the PTSI defects are discovered that raise an immediate concern for public safety, the Project Manager must follow the procedures in BD 79 for a Provisionally Sub-standard Structure. The Overseeing Organisation must be notified and approval sought for subsequent actions.

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6. PTSI – SITE INSPECTION

General

6.1 The objectives of a PTSI Site Inspection are:

- verify the type of bridge construction, form of articulation, geometry of the sections and locations of all construction joints
- identify any areas showing signs of distress and
- if necessary, plan an appropriate investigation programme to determine the causes and consequences of the deterioration (Technical Plan).

6.2 The basic steps relevant to the PTSI Site Inspection are illustrated in Figure 6.1. The Site Inspection is completed by a report, which reviews the findings and makes recommendations for consideration in a further Risk Assessment. If necessary the report should define the work to be carried out and the technical plan for the PTSI Site Investigation.

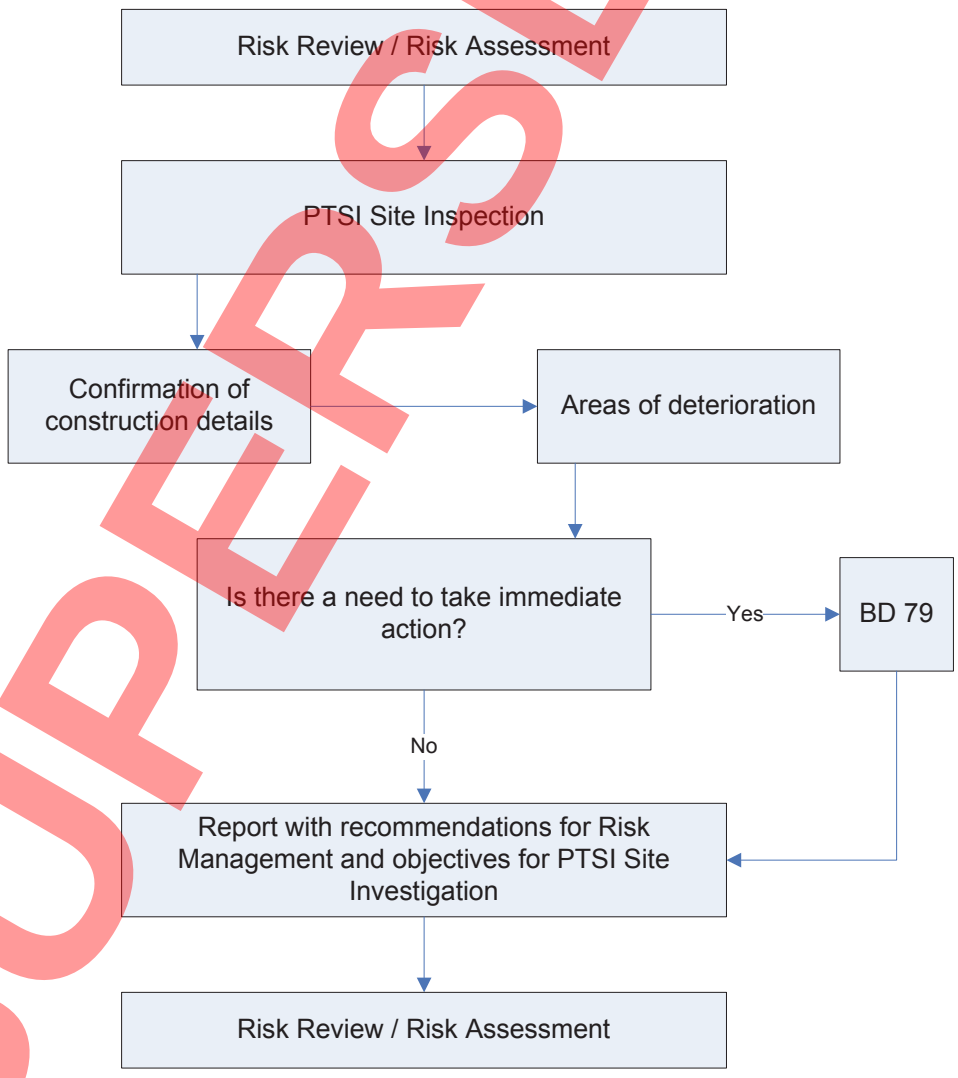

Figure 6.1 – Basic steps for a PTSI Site Inspection

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6.3 The faults detected at this stage are likely to be cracking, deflections, water leakage and staining, breakdown of bearings and expansion joints, steel corrosion and losses of concrete section.

6.4 All signs of deterioration should be examined to assess whether they warrant inclusion in the subsequent site investigation. Particular attention should be given to areas adjacent to critical sections, as defined in Para 2.12 and an initial appraisal made of the risk of a sudden mode of collapse. If the PTSI Site Inspection confirms the presence of deterioration at critical sections the Project Manager should consider the need to take action to maintain an appropriate level of public safety (see paragraph 5.9).

6.5 It is important to select an appropriate level of inspection so that its objectives can be achieved without unnecessary duplication of the close range inspection that may follow in the Site Investigation, particularly where the cost of access and traffic management to achieve it is high.

Visual Inspection

6.6 The inspection should systematically record the defects and interpret their significance. The inspection should be carried out in such a way as to identify actual and potential areas of distress. Particular attention should be paid to determining the presence of, and reason for, any cracks and the location of water leakage through the deck. The conditions surrounding all end anchorage zones, half joints and construction joints should also receive examination.

6.7 Signs of general corrosion on the surface of the concrete may be indicative of conditions within the concrete which are conducive to corrosion of the tendons. Detailed advice on the possible cause and interpretation of visual defects is given below.

Cracking

6.8 The location and direction of cracks are a valuable indication of the present condition of a structure. Prestressed bridges are normally designed to avoid cracks in the concrete. As such, the development of cracks can have serious durability implications and may indicate a loss of prestress. Useful advice on diagnosing non-structural cracking may be found in the Concrete Society Technical Report 22 “Non-structural Cracks in Concrete” (Concrete Society, 2010). Typical cracks found in prestressed concrete structures are summarised in Table 6.1, together with the possible causes to which they may be attributed. A particular crack can be the result of a combination of several causes. Table 6.1 is not exhaustive and there may be other possible causes.

6.9 It is important to identify the cause of cracking, since it will lead to a more precise determination of the actual structural damage. Cracks which appear to be critical to the structural integrity of the bridge should be monitored at regular intervals to check the development of further deterioration. Such information will be necessary for the assessment of the residual strength of a section.

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Table 6.1 Typical Cracks in Prestressed Concrete Sections

Structural Element	Location	Crack Direction	Possible Cause
Soffit (beam or slab)	End of span	Longitudinal	Bursting stresses Lack of end block reinforcement ASR/SA in concrete
	Midspan	Longitudinal	ASR/SA in concrete Broken tendons
		Transverse	Loss of prestress Excess live load
Web	End of span	Diagonal	Shear stresses Loss of prestress
		Longitudinal	ASR/SA in concrete Duct flotation Broken tendon Frozen water in ducts
Web (cantilever/continuous beam)	Over support	Vertical	Loss of prestress
Top flange (T-beam/box beam)	Midspan	Transverse	Differential shrinkage
	Over support	Longitudinal	ASR/SA in concrete Broken tendons
		Transverse	Differential shrinkage Loss of prestress Excess live load

6.10 Transverse flexural cracks may be an indication of a significant loss of prestress or tendon failure in the midspan or intermediate support region of a deck. However, the force in a fractured tendon is mainly transferred to adjacent tendons and it is likely that the only significant damage will be in the reanchorage zones on each side of the fracture. The damage to the concrete section will depend primarily upon the grouting in the duct and the amounts of surrounding shear reinforcement.

6.11 Cracks along the line of tendon ducts may be indicative of corroded and broken wires or tendons. Such cracks may be formed by the bursting forces that are generated as a broken wire slips and then re-anchors.

6.12 Where a tendon fails in the region of an end anchorage, structural damage may take several forms. Local splitting along the line of a fully grouted duct is likely to be well contained, since the areas of shear reinforcement and end anchorage bursting steel should be adequate. However, tendons at the ends of a deck are often inclined and there may be a significant reduction in the contribution to shear capacity, leading to inclined shear cracks in the webs of beams.

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6.13 Inclined shear cracks in webs may also occur in bending moment cross-over regions in continuous bridge decks, where prestressing tendons for sagging or hogging moment regions may be anchored. A quantitative assessment of damage in this type of region may require both concrete stress measurements using coring techniques and direct measurements of steel stress in the local shear reinforcement.

6.14 As UK codes do not limit maximum principal tensile stress in webs, it is possible to have safe and compliant structures that have shear cracks (indeed without them the links are not working).

6.15 Severe structural cracking can also occur in prestressed bridge decks as a direct consequence of vehicle impacts. Apart from the local crushing at the point of impact, it is possible for secondary shock waves to generate large scale longitudinal cracking in the bottom flanges of beam and slab decks. Diagonal shear cracks may also develop in the webs of beams which are supported on the lateral restraint bearings.

Water Management

6.16 The bridge deck drainage system should be the subject of a thorough visual examination. All gullies, downpipes and manholes should be checked to determine whether the system is working effectively. Ideally, checks should be carried out during intervals of heavy rain and subsequent dry periods. The influence of the carriageway surface condition, including cracks or pot-holes, should be noted. The location of any surface ponding on the deck should be recorded and related to midspan and intermediate support regions.

6.17 Surface water on the deck may enter the footways and any central reserve areas along the kerblines. Water and de-icing salts may remain trapped in these areas or penetrate into service bays, service ducts or voids within the deck construction. Water may also flow towards the ends of the deck and remain trapped against upstands formed at the deck expansion joints.

6.18 The introduction of drainage holes into the soffit of a deck may permit trapped water to escape. Safety precautions should be exercised in drilling any exploratory drainage holes, since the trapped water may be alkaline and, in extreme cases, may cause severe burning of the skin or damage to eyes. Specific advice on draining voids within bridge decks is given in BA 35 (DMRB 3.3).

6.19 During the detailed inspection, all signs of water leaks through cracks in the deck slab, construction joints, expansion joints and half joints should be recorded, with comments relating to the cause and source of the water. Similarly, all surface leaching should be recorded together with any signs of discolouration which may indicate the presence of internal rusting or other contaminants.

Deflections

6.20 The fracture of tendons in grouted or partially grouted ducts is unlikely to produce any visible or measurable deflections. Loss of stiffness along the length of the member will be small since the tendons will re-anchor on either side of the break. At the fracture position the local loss of effective section will not generally be significant and the adjacent tendons may take up part of the force released, producing only a local change in steel and concrete strain. Where the tendon ducts in a beam are ungrouted, broken tendons will release their force along the entire length and the resulting loss in prestress may produce a change in deflections and end rotation.

6.21 The presence of hogging deflections in the midspan region of a post-tensioned bridge deck is normally indicative of a satisfactory level of residual prestress. Where sagging deflections are observed, it suggests there may be excessive losses in prestress due to creep, shrinkage or temperature effects.

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Concrete Spalling

6.22 Concrete spalling may occur for a variety of reasons and note should be taken of the location and orientation of all surface spalling. The most common cause is likely to be corrosion of ordinary reinforcement producing surface delaminations. Corrosion of prestressing tendons may or may not be expansive, depending upon the supply of moisture and oxygen and the type of iron oxide formed. If corrosion of the tendons has caused splitting of the concrete surface layers, the cracking is likely to be more deep rooted compared to that caused by corrosion of ordinary surface reinforcement.

6.23 Large-scale concrete spalling may be observed where water filled voids in tendon ducts have frozen. Local spalling of the concrete surfaces may also occur due to stress concentrations arising from bursting stresses, misfit between segments or misalignment of tendon ducts across joints between precast units.

Steel Corrosion

6.24 Normal methods for detecting the potential for corrosion and the presence of excessive amounts of chloride should be sufficient. These tests and procedures are fully described in BA 35 (DMRB 3.3). No general non-destructive procedures currently exist for detecting corrosion in prestressing tendons and serious corrosion of the tendons may exist without any visual signs of distress.

Construction Joints

6.25 The type of construction joints within a bridge structure should be identified and recorded in separate categories. The condition of joints between precast units and the materials used to complete the joint should be noted. In situ joints between precast units may be formed with dry packed mortar, concrete or epoxy resins. Plain construction joints between deck pours may be simple dry joints and the joint direction could be vertical, horizontal, stepped or inclined. The lengths and widths of all joints should be recorded.

6.26 All construction joints in post-tensioned structures represent a potential plane of weakness. Concrete sections may act in a partially-cracked manner when compressive stresses across the joint fall between 1-2 N/mm^2. All joints should be examined throughout their length to search for signs of microcracking or water-staining. The start and finish of any cracking or water penetration may be significant and should be recorded in detail. Any areas causing concern should be noted and marked for potential monitoring in the future.

Bearings

6.27 The inspection of the bearings should be carried out with the objective of confirming the movements of a bridge structure are occurring as intended without damage to the deck, the fixings or the sub-structure. The parapet and carriageway surfacing should be checked for signs of movement or rotation at both ends of a bridge and at all intermediate supports. Similarly, all bearings should be examined for signs of movement and rotation, irrespective of their intended design function. The condition of the materials in the bearings and all forms of deterioration should be noted.

6.28 The integrity of any fixings to the bearings should be noted and any local failures recorded. Holding down bolts may be required to carry occasional uplift forces in some forms of bearing and the performance of these bearings should be observed under live load. General loss of prestress in continuous or semicontinuous post-tensioned structures may lead to significant increases in the axial forces carried by uplift bearings or tied-down post-tensioned anchors.

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Site Inspection Report

6.29 This report should contain the findings from the Site Inspection. Any variations in the construction details or new areas of serious deterioration should be identified and recorded. The main text of the report should present information relevant to the Risk Assessment, particularly the "Condition" section of the Risk Assessment. The report should contain a summary of critical points in the structure. The conclusions should state whether a site investigation is needed and should highlight issues to be fed back for consideration in a revised Risk Assessment.

6.30 Where a PTSI Site Investigation is considered necessary, the technical plan and particular objectives for the Site Investigation should be defined in the Site Inspection report. This should include a planned schedule of tests where appropriate. It should also include provisions for the installation of monitoring ports or internal instrumentation when it is proposed to undertake these operations on selected ducts within the Site Investigation period.

6.31 The inspection data should be grouped into two main headings:

a) Amendments to construction information.

b) Principal areas and defects requiring site investigation.

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7. PTSI – SITE INVESTIGATION

General

7.1 The general procedure for a Site Investigation is illustrated in Figure 7.1. In many cases, it may be sufficient to carry out only the initial testing tasks. Where problems are encountered and additional testing is considered necessary, then further investigation should be undertaken.

7.2 The main objective of the Site Investigation is to determine the cause and extent of deterioration for the purpose of assessing structural integrity. It should determine the existing conditions at critical sections, so that a realistic appraisal of the residual strength of the bridge can be made. Sufficient numerical information will be required in order that a present condition load assessment of the structure can be undertaken. The results of this inspection may also form the basis for remedial measures and strengthening works and monitoring in the future.

7.3 The technical plan and particular objectives for the PTSI Site Investigation are typically defined in the PTSI Site Inspection report but should be confirmed by the Risk Assessment before commencement of the PTSI Site Investigation.

7.4 The Site Investigation includes an initial testing stage and may include a further testing stage. At the initial testing stage an examination should be carried out with sufficient scope to give an adequate assessment of the present condition of the prestressing system and the presence of factors that might lead to future deterioration. In general the aims should be:

- to obtain sufficient data in terms of types of test, elements inspected and sample rates such that no further testing should be necessary if the structure is found to be free from problems
- to allow for a degree of flexibility in the plan to respond to the circumstances as found
- to provide for a limited amount of additional testing in the event that problems are found or the testing is otherwise not conclusive.

7.5 The areas selected for detailed investigation should take into account the form of deck, construction details and the type of deterioration already detected. This information will influence selection of the most appropriate techniques for identifying the presence of voids in ducts and the conditions of the grout, tendons and anchorages. Detailed guidance on the relative merits and application of various methods is given in this chapter.

7.6 Methods of investigation for post-tensioned concrete bridges range from a visual inspection to complex non-destructive and semi-destructive methods. The Site Investigation should commence with a visual examination (where it has not been possible to complete this during Site Inspection) and routine surface and material tests. Progression to the more complex methods of the Site Investigation may be justified if there is evidence of tendon corrosion and a risk of sudden failure in a structure. However, serious corrosion of the tendons can occur without any external visual evidence.

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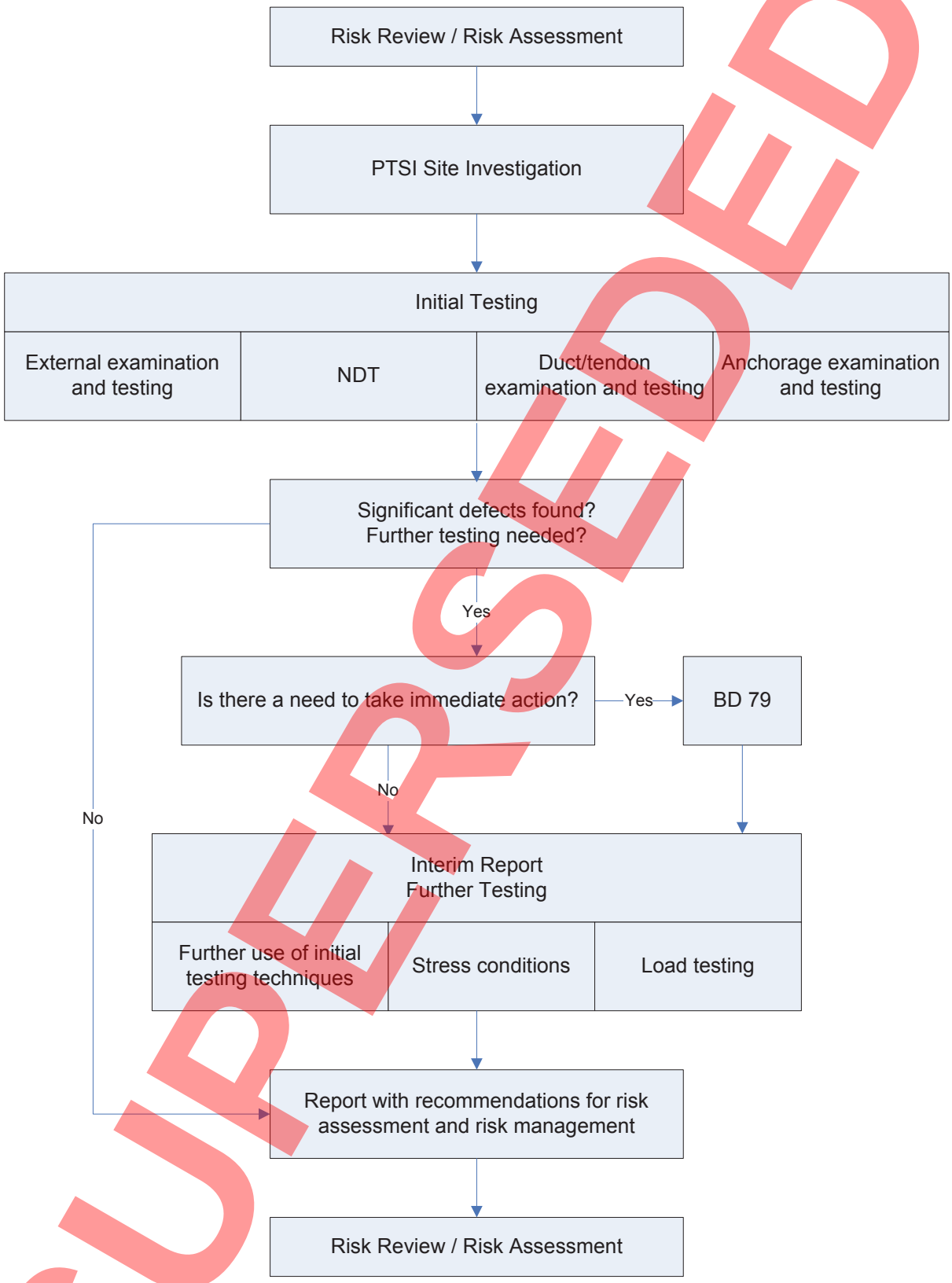

Figure 7.1 – Basic steps for a PTSI Site Investigation

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7.7 An indication of general corrosion of the reinforcement in the concrete, may be taken as indicative of the potential for corrosion occurring in the prestressing steel. Therefore, the methods of determining corrosion risk, as outlined in BA 35 (DMRB 3.3) and the Inspection Manual for Highway Structures (Highways Agency, 2007), provide a valuable precursor to the use of other inspection techniques. In particular, high concentrations of chloride ions increase the probability of tendon corrosion. Therefore, chloride ion content of the concrete should be determined.

7.8 Non-destructive testing can be used in the initial testing stage to assist in the detection of voids in the tendon ducts (for more information see BA 86 (DMRB 3.1.7)). If no voids are found this does not preclude the possibility of corrosion occurring. However, in fully grouted ducts with good quality grout, any corroded and broken wires will quickly re-anchor and the risk of full loss of prestress should be reduced.

7.9 If voids are found and the conditions within the concrete are conducive to corrosion of the steel then internal examination of the duct to inspect the tendons should be undertaken. The method for gaining access to the tendon duct should be chosen considering the position of the duct and the degree of damage that will be caused.

7.10 In all cases, drilling holes and other methods of exposure of tendons and anchorages must not be carried out without the agreement of the Project Manager and utmost care must be taken to ensure that the tendon is not damaged. The Project Manager must ensure there is close supervision of the exposure operation by a suitably experienced member of the Inspection Team.

Critical Sections

7.11 At the initial testing stage an examination should be carried out at a sufficient number of tendons at the critical sections (see 2.12) to give an adequate assessment of their condition at these points. Critical points on tendons should be classified for type and the number present for each type stated.

Number of points to be inspected

7.12 In larger bridges it has generally been found impractical to examine a significant number of tendons at all critical sections. Instead, it is acceptable for critical sections of similar type to be grouped together so that a representative sample of inspection points may be taken from this larger population. If serious defects are found the investigation should be broadened, and all critical sections might then be examined. In small bridges it may be more practical to test all critical sections.

7.13 Statistical methods may be used for recommending the number of critical points that should be inspected. Statistical analysis tends to show that the number of points that have to be inspected is only weakly dependent on the number of critical points in the group: ie in practical terms, the proportion of critical points that has to be examined in an inspection reduces as the number of critical points increases. This trend has been observed to occur naturally in previous inspections, without the use of statistical analysis. Based on the overall number of critical points, the proportion inspected has typically been as follows:

30% for approximately 50 critical points
20% for approximately 100 critical points
12 % for approximately 200 critical points
7% for approximately 400 critical points
5% for approximately 800 critical points
3 % for approximately 2000 critical points.

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7.14 However, for a given number of critical points, a large variation has been observed in the percentage inspected, the selection being based on perceived need, practicality and accessibility. Further observations can be made as follows:

- The number of critical points that have to be inspected in a group depends on the number of faults that can be tolerated. More inspections are required when tolerance to faults is low: eg when there are very few tendons (eg critical points below 50) it may be necessary to inspect a high proportion of them if the structure is sensitive to a loss in their effectiveness.
- Not all points are equally critical, and some tendons may be more important to structural integrity or more liable to corrosion than others. The number of points selected from each type should reflect this.
- The critical points on any particularly vulnerable section should generally be treated as an individual group.
- The extent of an inspection has to be increased significantly when serious problems are found. This intuitive conclusion is supported by statistical analysis.
- Where there is evidence that faults may have occurred in a systematic rather than random manner, this should be taken into account when planning the inspection.

7.15 Experience from the 1990s PTSI programme found the most frequent faults to be dry voids which are not in themselves threatening to the structure. The presence of these faults alone would not generally be sufficient reason to widen the investigation substantially. However, when an assessment shows that for adequate strength the tendons must be bonded to the concrete, the sample inspected should be large enough to check the structure for the widespread presence of ungrouted ducts.

7.16 The discovery of serious corrosion and partially fractured tendons would normally require additional testing to establish the safety of the structure. Where corrosion has started and the conditions are present for future deterioration, the level of inspection adopted should be sufficient for recommendations to be made on the future management of the structure. In both instances, account should be taken of the sensitivity of the structure to loss of prestress or tendon area.

7.17 The typical figures for percentage of critical points inspected, quoted above, are representative of experience from the 1990s PTSI programme and are given as a guide for new investigations. The reasoning behind the actual number and selection of inspection points should be recorded in the Site Investigation report.

Initial testing

7.18 The Initial Testing starts with external examination and testing. This is followed by the detection of voids using NDT or by drilling pilot holes with insertion of an endoscope, typically followed by an internal examination of ducts and tendons with tests on materials in tendon ducts (grout and/or water). Anchorage examination and testing should also be included. Exposures of anchorages located in the top surface of bridge decks should generally be included in the initial testing. End face anchorages are typically more difficult to access and it may be prudent to use the results of initial testing to decide whether to expose these during further testing, particularly where dismantling of structural elements would be necessary.

7.19 The Project Manager should prepare a schedule of standard sampling and testing to detect the presence of corrosion activity and assess the condition of the concrete and reinforcement in the vicinity of critical sections under investigation. All material sampling, laboratory and site testing must be carried out by specialist testing firms or laboratories approved for the relevant sampling and testing by the United Kingdom Accreditation Service (UKAS), or by equivalent accreditation bodies of member states within the European Community.

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Further testing

7.20 When defects are found, further testing may be necessary to investigate specific problems identified during the first round of tests and/or to increase the sample rate. In the latter instance the aim is to establish the overall condition of the post-tensioning system with an appropriate level of confidence. A limited amount of further testing should be allowed for in the site investigation where site/traffic conditions are suitable and contractual arrangements can be made to permit.

7.21 If a larger amount of further testing is required, the timing of this may be dictated by constraints such as traffic management requirements, other work nearby, and costs, and it may not be possible or desirable for it to be done immediately following initial testing. An interim report should be provided by the Project Manager in this case. The report should include the results of the tests carried out to date, proposals and reasons for further tests, the extent and type of tests to be carried out. The report should cover any need to widen the search for voids in tendon ducts or the corrosion of tendons. The report should also include an interim risk assessment to give assurance that the further testing can wait until a convenient opportunity arises.

7.22 Depending upon the structural form of the bridge, the results of previous load assessments, or the perceived risk of a sudden collapse, it may be necessary to check local stress conditions in the steel or global stress conditions in the concrete. Where load distribution in the deck depends upon residual levels of transverse prestress, it may also be appropriate to apply incremental load testing techniques. However, it is very important that the maximum applied loading is restricted to serviceability levels to avoid damage to the structure.

7.23 The determination of stress conditions and the use of load testing are considered in paras 7.58 to 7.62. Stress determination may be carried out during the initial testing stage if it is considered critical to an evaluation of the structure or inefficient to obtain later as a consequence of access requirements. Load testing is more likely to be reserved for a further testing stage. The introduction of such specialist testing would require technical approval from the Overseeing Organisation. The organisational requirements for highly specialised techniques are detailed in paragraph 5.7.

Investigation Methods

7.24 Methods of investigation are considered under three headings:

- External examination and testing
- Non-destructive testing and inspection (NDT)
- Internal examination and testing.

External Examination and Testing

7.25 Where it has not been possible to carry out a full visual inspection as described in Chapter 6 as part of the PTSI Site Inspection, this should be completed in the PTSI Site Investigation.

7.26 Methods of determining corrosion risk are outlined in BA 35 (DMRB 3.3.2) and the Inspection Manual for Highway Structures. In particular, high concentrations of chloride ions may increase the probability of "outside-in" corrosion in tendons and anchorages. This part of the investigation may not be required, or its scope may be reduced, if there has been a recent Principal Inspection which included testing in accordance with BA 35 and the Inspection Manual for Highway Structures and covered the areas that are the subject of the PTSI.

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7.27 These methods cannot be used to determine the "inside-out" corrosion risk in tendons without gaining entry to the ducts. No general non-destructive processes are available for this purpose.

Non-Destructive Testing and Inspection

7.28 The detection of voids in post-tensioning ducts is important in identifying areas where corrosion of the tendon may occur. Determining the position of any voids, prior to an internal examination to ascertain the condition of the tendon, should restrict the degree of damage caused to the structure.

7.29 The methods of detection can be non-destructive and guidance on their use is available in BA 86 Advice Notes on the Non-Destructive Testing of Highway Structures (DMRB 3.1.7). Non-destructive techniques include radiography, ground penetrating radar, impact echo and ultrasonic transmission/tomography.

Duct and Tendon Positions

7.30 The first priority in performing a detailed internal examination is to establish the actual location of tendon ducts at each section under investigation.

7.31 Location of ducts and tendons is typically done by reference to as built drawings and the drilling of pilot holes combined with use of a covermeter and covermeter probe for locating and avoiding reinforcing bars and for detecting shallow tendons. Ground penetrating radar has been shown capable of detecting tendons, but is adversely affected by dense reinforcement. Although it has been used in a few inspections, reports of its effectiveness have been mixed. Radiography is capable of duct location but it is inappropriate to use it for this purpose alone, apart from in the most exceptional circumstances.

7.32 During construction, tendon ducts may float upwards between tie-points. Typical vertical movements can be 25-50mm, but displacements exceeding 100mm are not uncommon in some forms of construction. Duct displacements are a potential cause of serious damage during investigations involving any form of hole drilling. Tendons may be hard against the bottom of a duct at any position due to duct flotation. Therefore, drilling holes into the bottom of a tendon duct at midspan to use an endoscope is unlikely to prevent the tendons being damaged. Allowance should also be made for horizontal displacements of tendon ducts.

7.33 Excessive trial and error drilling should be avoided. When a conventional covermeter is insufficient, for instance the tendons are too deep, an improvement may be obtained using a high performance surface-scanning type of covermeter. In trials this has given reliable results.

7.34 Once pilot-hole drilling commences a probe type of directional metal detector can assist tendon/duct location and reduce the number of trial holes required. The detector is mounted on the end of a narrow shaft, which is inserted into the pilot hole. If the pilot hole is directly above the tendon, the readings will indicate the depth to the tendon provided it is within l00mm. If the hole is not directly over the tendon it can indicate the distance and direction, enabling a second hole to be drilled with more confidence. As with most detection systems, the presence of dense steel reinforcement close to tendon ducts reduces its effectiveness.

Endoscope

7.35 Information on endoscopic examination can be found in the Inspection Manual for Highway Structures.

7.36 Instruments (sometimes referred to as flexible videoscopes) are available which comprise a small diameter flexible probe several metres in length with a small camera at the tip to provide a video image. These are helpful where extensive voids are found and can be used to inspect the duct over a distance of several metres from the entry hole.

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Pressure-vacuum Testing

7.37 Information on pressure and vacuum testing can be found in the Inspection Manual for Highway Structures.

Internal Examination

7.38 Once voids and potential corrosion of post-tensioning tendons have been identified, the only certain method of determining the duct location and tendon condition is by an internal examination of the duct. There are a number of ways by which access can be gained to post-tensioning ducts in order that an internal examination can be carried out. The degree of damage caused to the structure will depend upon the method of exposure chosen.

Access to Ducts and Tendons

7.39 The methods of access for internal examination mentioned in para 7.44 may be used individually or in combination, according to the type of problem being investigated. The method employed will be dependent upon site access, safety and the type of structure.

7.40 Access holes for intrusive inspection should be made into the top of a duct where possible. If access is into the bottom of the duct there is a possibility that partial voids, which tend to occur in the top of the duct, will remain undetected. Access holes to tendon ducts should be a minimum of 25mm diameter to allow viewing of the inside of tendon ducts, and adequate grout sampling. An endoscope (see 7.35 – 7.36) can be used for viewing if necessary. Subsequently it may be found to be necessary to enlarge access holes for a better view and/or to take an adequate grout sample, particularly where the tendon lies deep within the concrete.

7.41 Overcoring the original access hole may lead to the ingress of water, particularly when entry is in a downwards direction from above. This is undesirable, and may be avoided by coring to one side of the access hole and stopping short of the duct, completing the hole by hand chisel.

7.42 In order to examine tendons in an undisturbed state and to avoid damage to the prestressing steel, the grout around the tendons should be carefully removed by hand methods. The type and size of the tendons in the duct should be confirmed and the presence of any form of surface corrosion or pitting carefully recorded. The position of the tendon within the duct and the packing of wires or strands in a tendon should be noted, since this may provide useful evidence of duct displacements during construction.

7.43 The repair of all access holes made for the internal examination of tendon ducts requires very careful consideration at the time of planning the inspection. Priming of the exposed steel may be an advisable precaution, depending upon the condition and the local environment. Shrinkage compensated repair grouts and mortars should be adopted for filling holes and consideration given to the addition of a surface repair coating as a further precaution against the ingress of water and chlorides. General guidance on repairs is given in BD 27 (DMRB 3.3.2), Materials for the Repair of Concrete Highway Structures. In some cases “access windows” have been installed to allow future monitoring and silica gel has been placed in duct voids to absorb moisture.

7.44 Advice on methods of access including percussive methods, diamond core drilling, high pressure water jetting and grit blasting can be found in the Inspection Manual for Highway Structures. Use of diamond core drilling requires particular care to avoid tendon damage. Holes can be started with the drill but finished by hand when close to the duct.

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7.45 Although water jetting with a grit additive offers better control for concrete cutting, it is particularly dangerous since it can also cut through the prestressing steel. Hence, its use is not permitted.

Intrusive inspection

7.46 The tendon should be inspected whether there is a void or not. This will entail the removal of grout where present. At each inspection point, measurements should be taken of volume of void and leakage irrespective of whether or not a void is visible. For this, air pressure and vacuum testing techniques can be used. This will determine whether an apparently blind hole is concealing a narrow passage linking it to a nearby void or leak to atmosphere – a potential source of water ingress in the future.

7.47 When a void is encountered its volume should be determined. A record of the void volume at each critical point examined may be sufficient to gauge the overall quality of grouting, but in itself does not provide a complete inspection of the tendons affected. In selected instances, particularly where there is leakage, dampness and corrosion, further information may be obtained by inspecting at additional access holes and checking for the continuity between voids. When an extensive void is encountered, a flexible videoscope will give a better view than a rigid endoscope and allow the extent of the void to be investigated in conjunction with additional access holes where appropriate. It may be helpful to enlarge the opening along the extent of the void to locate the air/water/tendon interface if this can be done without causing excessive damage to the structure. Water contained within a duct should be collected for analysis.

7.48 It should be noted that water can travel along strand tendons within the gaps between the individual wires that make up the strand.

Grout Integrity and Material Testing

7.49 Some grouts have been found to contain inappropriate materials such as chlorides and sulfates. The methods adopted for exposure of the grout should be chosen to allow the condition and properties to be examined in an undisturbed state and samples to be removed in sealed containers. Supervision of the exposure and removal of specimens is of paramount importance.

7.50 Tests to establish the basic constituents of the grout require only a low sampling rate unless there are good reasons to do otherwise. It is important to determine the in situ moisture content, colour and composition of the grout. Where tendon ducts are partially filled, a cement grout is likely to be white rather than grey due to the effect of carbonation. A grout may be completely dry and solid within a duct or it may be a wet paste or shattered and broken in appearance. The composition of the grout, alkalinity, chloride content and the presence of additives should all be determined from dry samples removed from a duct.

7.51 If the grout is found to be wet, contaminated with chlorides or carbonated then there is a risk of local corrosion occurring.

Anchorages

7.52 Anchorage inspection can be difficult, costly and disruptive, and in general is expected to be carried out selectively, targeted to the most vulnerable positions. Factors that indicate the need for anchorages to be exposed include:

- Practicality and ease of accessibility to some anchorages
- Water seepage/staining

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- Poor visual condition of capping mortar
- Large proportion of voided ducts
- Voids in ducts near anchorages
- Vulnerable design details and/or poor construction
- Experience of defects with similar inspections
- Adverse consequences of local loss of prestressing force

7.53 In some cases it is possible to inspect the tendon behind the exposed anchorage by inserting an endoscope down an empty grout pipe or drilling it out if blocked. When it is possible to gain access to the duct sufficiently close to the anchorage, and a void is found, it may be possible to inspect behind the anchor plate using a flexible endoscope or videoscope.

7.54 Exposure of post-tensioned anchorages located at the ends of decks, or similarly restricted areas such as deck half-joints may be carried out using carefully selected water jetting or grit blasting methods. The exposure procedure should be a gradual process, which allows visual examination or the use of endoscope techniques at various intervals.

7.55 The work should be carried out by specialist operators. In particular, all personnel should be made aware of the requirements of paragraph 5.8 and avoid standing behind the anchor plates during water or grit blasting operations.

7.56 The areas selected for exposure at an anchorage may be behind the anchor plates and immediately adjacent to the end block zone. Removal of concrete within the bursting zone of the anchorage should be avoided, since this localised region is likely to be highly stressed. Particular care should be taken not to fracture the end anchorage plate or end wedges of the post-tensioning system.

7.57 The condition of the end anchorage zone and anchor plate should be recorded and fragments of the surrounding concrete or mortar protection should be examined and removed for laboratory examination to determine the cement, chloride, and sulfate contents. Repairs to the end anchorage areas should follow the advice outlined in paragraph 7.43 and the recommendations given in BD 27 (DMRB 3.3.2).

Determination of stresses

7.58 Determination of steel stresses and concrete stresses at critical sections gives an indication of the local and general levels of residual prestress, and can be useful for comparison with assessment output. Sufficient measurements should be taken to ensure confidence in the results. The current live load performance and behaviour of the deck in the long-term may be checked using a carefully controlled load test and monitoring of the structure at critical positions.

Steel Stresses

7.59 Advice on methods for the determination of steel stresses can be found in the Inspection Manual for Highway Structures.

7.60 It is essential to be certain there is sufficient spare capacity before cutting any wires on a bridge in service.

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7.61 Unless it is judged necessary to obtain stress from the tendon directly, concrete cutting methods of determining in situ stresses (para 7.62) should be given preference to tendon cutting or drilling because they give overall information on the effective prestress and are less damaging. Specialist advice should be obtained before specifying direct measurement of stress in the tendons.

Concrete Stresses

7.62 Specialist methods have been developed to provide in situ measurements of concrete stresses using instrumented coring and slot-cutting techniques. The results obtained from such stress relief methods represent the total stress conditions in the concrete. The Inspection Manual for Highway Structures contains further advice on the use of these methods.

Load Testing

7.63 The main purpose of load testing any form of bridge deck is to determine the effective levels of transverse load distribution and correlate the structural analysis with the assessed structural action. Very significant reserves of strength can be present in some bridge decks due to secondary effects such as membrane forces, edge stiffening, end restraint and composite action.

7.64 Local damage caused by the failure of a small number of prestressing tendons cannot, in general, be determined by load testing. However, the global behaviour of a bridge deck can be verified by a carefully selected pattern of loading. It is very important that the magnitude and positions of the applied loading is maintained at levels representing serviceability conditions but no higher. The loading should be applied in controlled increments and the deck response monitored by an appropriate pattern of strain gauges in order that the test does not cause any damage to the bridge.

7.65 Deflection monitoring and dynamic stiffness measurements are generally not appropriate for load testing and are unlikely to detect the onset of non-linear behaviour in segmental structures. Load testing should not be used to assess the deterioration in shear strength of a post-tensioned bridge. Load testing should be carried out in accordance with BA 54 (DMRB 3.4.8).

Project Manager's Report on Site Investigation

7.66 The results of a PTSI Site Investigation should be presented and interpreted in the Project Manager's Report. In addition, the structural condition, assessment of risk and monitoring requirements should be summarised to form a basis for future management of the bridge. The quantitative information gathered during the investigation should be recorded in a suitable form for future reference.

7.67 The report should include the results from all previous routine material tests and tests conducted to detect the risk of reinforcement corrosion, additional testing and specialist tests that may be carried out during the Site Investigation. The results from any further standard tests should be incorporated in the report, and should supplement the information previously gathered in the Site Inspection. Analysis of results, conclusions and recommendations should be included in the report.

7.68 The site investigation records should be presented in the report. The detailed information provided should be under three main headings:

a) Tests for corrosion risk.

b) Concrete material tests.

c) Results from internal examination.

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7.69 Conditions at all critical sections should be assessed and presented individually in the Project Manager's Report. Potential collapse mechanisms involving failures at various critical sections should be considered, bearing in mind the present condition and the potential for further deterioration at each section in the future.

7.70 It is important to provide a concise record of the Site Investigation. The report should include a summary of the inspection in tabular format: ie state numbers of each type of critical point in the structure, the number inspected and the result, using the fault classification system (para 7.76). The reasons for any variation from the Technical Plan should be reported.

7.71 Where special methods are introduced to determine in situ stress conditions, the basic information should be summarised. Where stress conditions are investigated, the results should be collected under four separate headings:

a) Temperature conditions.

b) Steel stresses.

c) Principal concrete stresses.

d) Secondary concrete stresses.

7.72 Where possible, estimates should be made of the prestress losses in a structure. The results from any load testing should be fully discussed and interpreted in relation to the condition of the structure and the in situ stress measurements.

7.73 The main conclusions arising from the PTSI should include summary statements on the structural condition, risk assessment and future monitoring requirements. Recommendations should be made on the effects of deterioration on section strength, future management of the structure and the need for remedial measures or the possibility of replacement.

Structural Condition

7.74 The overall condition of the structure should be fully appraised following the Site Investigation. Particular attention should be drawn to the elements of the structure which have suffered from cracking and water leakage.

7.75 The significance of defects in the prestressing system should be carefully assessed in terms of long-term durability and the consequences of structural failure. Where voids are discovered in the grouting, the alkalinity and the presence of any moisture, chlorides and sulfates should be considered in order to assess the potential for corrosion of the tendons. The possible benefits of injecting further grout into voided areas should be reviewed in terms of the potential improvement in durability and ultimate strength. Broken wires or strands from several tendons may not represent a significant loss in strength in some forms of bridge deck. Broken tendons are likely to re-anchor over short distances of 1-2m, but this depends upon the magnitude of the force released, the integrity of the grout and the adjacent shear reinforcement. Therefore, the redistribution of forces in the region of a broken tendon should be carefully reviewed in terms of these basic parameters and the condition of adjacent tendons.

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Fault classification

7.76 For the purpose of summarising the tendon condition at each inspection point tendon corrosion should be classified according to table G10 of the Inspection Manual for Highway Structures. Void size should be classified as follows:

1. No void
2. Small void <25% of section, <50mm linear
3. Medium void <50% of section, < 300mm linear
4. Large void >50% of section, > 300mm linear
5. Ungrouted.

7.77 Classifications of this type should be used in the report. Information on duct leakage, the presence of duct water and its analysis should also be included.

Feedback to the Risk Review/Risk Assessment

7.78 The report is used to feed back results, conclusions and recommendations from the PTSI Site Investigation into the Risk Review and Risk Assessment so that decisions can be taken on the need to update the Risk Assessment and recommendations made for any further Risk Management measures.

7.79 The Risk Assessment should be reviewed to take account of all defects identified in the PTSI Site Investigation, particularly at critical sections. The structural consequences arising from all broken and corroded tendons should be carefully examined. Where grouting of tendon ducts is voided, structural failure may occur at a critical section, even if the tendons are fractured at other locations.

Monitoring Recommendations

7.80 Monitoring recommendations should be developed following the advice in Chapter 8.

Recommendations for Load Assessment

7.81 Details of all major cracks, losses of concrete section and corroded reinforcement and broken tendons should be correlated so that the effect on serviceability and ultimate capacity of sections can be appraised. Cracks in joints between precast segments may relate to a general loss of prestress and subsequent reduction in load distribution behaviour within a deck. Recommendations should be prepared to provide guidance on the likely loss in section properties and reductions in strength of all sections suffering from deterioration. Condition factors for assessment should be proposed.

Recommendations for Bridge Management

7.82 Future management of the structure should be determined according to the extent of the existing defects and the potential for future deterioration. The proposed management of the structure should also be influenced by the results of the Risk Assessment and the monitoring requirements. Factors to be considered include planned maintenance, operational aspects and environmental issues.

7.83 Recommendations should be made on the frequency and type of future inspections. Where serious defects exist or further corrosion of tendons is inevitable, suggestions should be made for the introduction of repairs, strengthening works or forms of replacement.

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8. MONITORING

General

8.1 The chapter should be read in conjunction with the requirements and guidance on Monitoring Interim Measures in BD 79.

8.2 The type and extent of the defects in a structure may require the introduction of frequent inspections and regular monitoring of critical sections. Where any form of monitoring is considered, the frequency of readings and the measurement of temperature conditions should be planned according to the location of the faults and the type of construction.

8.3 Where joints in segmental structures occur at critical locations or there are known defects in the vicinity, proposals should be prepared for regular monitoring. In these circumstances, it is necessary to establish datum values for the in situ concrete stresses adjacent to the joint location and to install a strain gauge monitoring system across the joint. A programme for regular monitoring and interpretation of the results should be prepared in order to provide advance warning of significant changes in structural behaviour.

8.4 Monitoring is an important part of risk management strategies for keeping deteriorating post-tensioned concrete bridges in service. Monitoring systems cannot detect deterioration that has occurred before installation of the system, but they can assist in detecting ongoing changes in structural behaviour and deterioration. For high risk structures, particularly segmental bridges, there may be a case for installing monitoring systems as part of a long term management strategy, even if there is currently no evidence of deterioration.

8.5 When relying on monitoring, it is important to understand the possible failure modes of the structure, and to ensure that the monitoring system is able to detect the onset of these failure modes. Therefore a structural analysis must be carried out first to determine the failure modes, then the monitoring system must be designed to detect the onset of the various failure modes.

8.6 Monitoring systems must be tested and calibrated on installation, and trigger levels set with care. An unrealistically low level may result in high numbers of alarms that could mask a genuinely serious situation. Viable contingency plans must be developed and ready to implement in the event that trigger levels are exceeded.

8.7 Different types of monitoring equipment should be used to enable cross-checks, for example vibrating wire, electrical resistance and fibre optic strain gauges. Equipment needs to be robust in order to function correctly in harsh environmental conditions. Self-checking systems that raise an alarm in the event of malfunction are an advantage.

8.8 Equipment malfunctions are not unusual; therefore systems and their outputs should be regularly reviewed. Experienced engineers with knowledge of both structural behaviour and the issues with monitoring systems are needed to ensure that output data is used to make appropriate decisions about the management of structures, the reliability of the data and the appropriateness of continuing with the monitoring.

8.9 In accordance with BD 79, monitoring should only be considered where the behaviour of the structure is sufficiently understood to give confidence that the monitoring system will give adequate warning of failure.

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8.10 Specific monitoring systems and some particular issues with them will now be considered. With modern data acquisition systems, all the electronic systems can be used for continuous monitoring as considered in BD 79.

Acoustic Emission

8.11 This method detects the sound made by breaking wires or strands. Therefore, to be effective, the monitoring needs to be continuous. The system depends on being able to distinguish the sound of breaking wires from other noise. However, it appears with modern developments the approach is reasonably reliable. The major limitation is that it can only detect change of condition. Unless the pre-existing condition is known exceptionally well, the system cannot be used on its own to identify the point when a structure becomes unsafe. It is very valuable for determining, for example, if measures to reduce deterioration by improving waterproofing have been effective. It can also be used to identify if previous assessment logic (such as using Bayesian updating) has been invalidated by continuing deterioration. It also gives alerts that the structure is deteriorating which draw attention and enable it to be investigated by other means.

8.12 For more information on acoustic emission refer to BA 86 (DMRB 3.1.7).

Deflection

8.13 The major limitation on monitoring deflection of post-tensioned structures is that loss of stiffness due to deterioration can be very localised so that the effect on overall stiffness and hence deflection can be very small right up to the approach of failure. The approach may be more useful in certain specialised cases, for example for detecting if a structure is lifting off bearings.

8.14 In truly unbonded structures, loss of prestress cannot be confined to a small length of structure and deflection is therefore more sensitive to loss of prestress. However, if a monitoring system is to rely on this, it is important to use a conservative high estimate of the degree of bond achieved in the actual structure, rather than the conservative no bond assumption used in assessment.

8.15 If prestressing wires which are fully bonded to the surrounding beam over most of their length begin to fail at one point, the effect upon deflection of the beam will be negligible. If the wires are fully unbonded, the effect upon deflection due to prestress will be proportional to the prestress lost. Hence deflection measurement for monitoring grouted or partially grouted post-tensioned structures is not appropriate.

Natural Frequency

8.16 Natural frequency depends on stiffness and mass distribution. Since the mass of structures does not normally change significantly, monitoring natural frequency is effectively an indirect way of monitoring stiffness and is subject to the same issues considered above.

8.17 Measurement of natural frequency is a very effective way of determining the force in free lengths of external tendons and can detect very small changes in this.

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Strain Monitoring

8.18 Recommendations on methods and devices which can be used to determine strain in concrete are given in BS 1881: Part 206. Common types of strain gauges are mechanical, electrical resistance, vibrating wire (acoustic) and electrical displacement transducers. Another method of strain measurement is by optical fibres which are bonded to the surface to be monitored. Strain changes in the optical fibre cause leaks of light. Micro deflections of the fibres are therefore accentuated by winding them with a spiral wire (micro-bending) and strain can be measured as a function of changes in light attenuation in the received signal. Sensors can be attached externally to a structure or set into grooves cut into the concrete.

8.19 Strain gauges are able to detect very small movements. Vibrating wire gauges are typically sensitive to movements of $3x10^{-6}$. This corresponds to a concrete stress change of only 0.1MPa or, with a typical 150mm gauge length, a movement of only 0.0005mm. Strain gauges are therefore able to detect small stress changes and also, when placed appropriately, very small movements of cracks or joints.

8.20 Vibrating wire gauges are preferred for high sensitivity and long term stability. They cannot, however, operate at high enough sampling rate to be used for monitoring strain under live load. Electrical resistance or the more recently developed optical type are preferred for this.

8.21 Analysis of segmental structures will typically show that joints will open at around half ultimate load. This has been confirmed by tests on various structures including beams taken from the bridge at Ynys-y-Gwas after it failed (Woodward, 1991). The approach therefore offers the possibility of detecting loss of prestress in segmental structures long before it becomes critical. In larger concrete bridges, this means the minimum prestress level required to give adequate ultimate strength would result in joints being open under permanent load. Some structures have been "strengthened" following crack opening including under dead load when assessment showed they had adequate ultimate strength. In smaller structures with higher live load ratios, it may be necessary to monitor under live load to obtain adequate warning.

8.22 In a segmental structure with unbonded tendons, the first joints to open can be identified by simple structural analysis and instrumentation can be confined to these. However, tests show that even whole broken tendons in grouted ducts can re-anchor in distances as short as 1m. This could result in tendon failure being missed if it is concentrated in a joint other than the one being instrumented. It is therefore likely to be necessary to instrument more joints in structures with tendons which are, or could be, bonded. This is likely to include structures with mortar protected external tendons or other external tendons even though the assessment of these structures may assume they are unbonded.

8.23 In monolithic structures, the onset of cracking can be significantly delayed by the tensile strength of concrete. Normally in highly stressed critical sections, warning will still be adequate but the effect should be included in calculations when assessing the likely warning of failure. Similar issues can potentially arise in glued segmental structures as the glue has significant strength and may be stronger than the concrete. This could result in cracks forming first other than at the joints. However, this is not known to have happened in practice and cracks away from joints would be easier to detect by eye. Tensile strength of mortar/concrete adhesion could also delay the onset of joint opening in structures with in situ joints. However, experience indicates the opposite effect can be apparent and such joints have been seen to start exhibiting non-linear behaviour when the stress is still in excess of 1MPa compression.

8.24 Temperature effects can cause problems. Temperatures should be monitored in combination with strain monitoring and installation of control gauges in unstrained areas has also been found to be helpful.

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9. REPAIR

General

9.1 This Chapter deals with two aspects of repair of post-tensioned concrete structures. First the "preflex method" is described. Secondly, advice is provided on grouting of voids in tendon ducts.

"Preflex" method

9.2 This is used to repair damage resulting from impact or blast. In most cases, accidental damage is caused by over-height vehicles striking deck soffits. This type of damage is likely to be localised and amenable to repairs if no damage is done to the stressing wires. Loads are applied to the bridge using lorries or skips to provide either a distributed load across the deck or a concentrated load above individual members. This method is particularly suitable for the repair of deck soffits. When concrete has deteriorated on the top of the deck, the structure would be jacked upwards from steel beams installed below. In continuous decks props should be applied at appropriate locations to induce desired load effects.

9.3 By preloading the damaged member prior to repairing, the area of the repair will be placed under increased tension. Therefore, once the repair is completed and it has reached an adequate strength, the removal of the preload will induce a compressive stress in the repair material. Ideally, this compressive stress should be representative of that in the undamaged structure. However, even where this is considerably less, any cracks induced in the material under live load will close again as soon as the load is removed. In this way, the repair material should work effectively with the existing concrete and it should improve the future performance of the structure under live loading. An application of a crack bridging surface coating should be considered to enhance durability of the repaired structure.

Materials

9.4 Requirements for repair materials are contained in BD 27 (DMRB 3.3, not applicable in Scotland).

Void grouting

9.5 The need to grout voids in ducts is a matter of judgement, and the decision on whether or not to recommend it will generally depend on the vulnerability and importance of the tendons and the practicality of accomplishing the task.

9.6 Small, isolated dry voids pose less of a durability problem than large voids containing exposed tendons, leakage to atmosphere, dampness and corrosion. Wholly ungrouted tendons are potentially vulnerable, and because they are unbonded may lead to a lower assessed capacity for the member. Moreover, a fracture will affect the tendon along its whole length because re-anchoring cannot occur.

9.7 When a structure or member contains relatively few tendons, each one is likely to be structurally important. This may not be the case when there are many tendons. In addition, when there are many tendons, void grouting may be impractical because of the scale of the work. It should be noted that the PTSI may only detect a small proportion of the voids present and as relatively little may be achieved by grouting just these it could be necessary to carry out further investigations to identify and subsequently grout a more significant proportion of the voids present in the structure.

9.8 Some ducts may be inaccessible because of their position in the structure. Long ducts present the additional difficulty that multiple vent points would be required for void grouting. Ducts that are ungrouted or contain very little grout are potentially easier to grout than ducts where the void contains narrow passages which inhibit grout flow along their length.

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9.9 It is recommended that a trial is undertaken before a void grouting method is applied widely.

9.10 An alternative to void grouting that has been adopted in some structures is to provide small duct-monitoring ports in selected typical locations. The access holes are plugged but the duct is left ungrouted and available for re-inspection, periodically, through the port.

9.11 Research on the rapid corrosion of void grouted strands has suggested as a possible cause a difference in the electrical potential in the strands caused by environmental differences due to the adjacent dissimilar (new/original) grouts (O'Reilly, Darwin & Browning 2013). Designers proposing void grouting schemes should ensure they are aware of the latest developments on this subject and consult the Overseeing Organisation as necessary.

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10. STRENGTHENING

General

10.1 Strengthening techniques can be divided into two groups; active and passive. Active methods, such as the use of additional prestress, actively stress the structure. Passive methods, such as plate bonding, increase the strength but they are not stressed, and therefore do not affect the stress state in the structure, until it deflects under live loading or changes length due to long-term deformations.

10.2 The choice of strengthening method will depend on many factors which will include the nature of the problem which has led to the decision that strengthening is required and the design criteria adopted for the strengthened structure. Other factors include the availability of space for strengthening works which extend outside the original structure or access for strengthening works inside the structure. Another key factor is the necessity, or otherwise, to keep the structure in service for most, or all of the period whilst the strengthening is undertaken. The factors which are likely to decide the choice of strengthening method are considered for various types of bridge deck.

Design Criteria

10.3 The selection of appropriate design criteria is a fundamental requirement for strengthening works.

10.4 The design of strengthening for concrete highway bridges and structures using external and unbonded Prestressing is in accordance with Eurocode 2.

Additional Prestress

10.5 Additional prestress is the most versatile and popular means of strengthening existing post-tensioned bridges. It can be used to increase ultimate strength and improve serviceability behaviour in both flexure and shear. Indeed, in some cases where remedial works were judged necessary to close up existing cracks, additional prestress was effectively the only possible solution.

10.6 In most cases, the additional post-tensioning has been external to the original concrete. This is because it is extremely difficult to drill additional ducts for main longitudinal cables in an existing structure. Where additional vertical prestress has been used to increase shear strength, internal tendons have sometimes been used. However, even these relatively short ducts are difficult to drill particularly in structures which have longitudinal tendons in the webs. Additional prestress should only be added after full consideration of the effects on the whole structure. It is recommended that a monitoring system is installed to enable monitoring of the structure during the installation of the additional prestress.

Anchorages and Internal Fixings

10.7 Modifying an existing structure to accommodate additional anchorages and deviators is both difficult and expensive. It is therefore sensible to minimise the number of such items in any strengthening scheme. Hence there will be a tendency to adopt solutions which favour straight tendons, continuous over the full length of individual spans or even the whole structure.

10.8 Concrete block anchorages formed behind internal and end diaphragms will permit the inclusion of additional reinforcement, and the new anchorages will be stressed directly onto the existing structure. A disadvantage of this arrangement is that there is seldom sufficient space between end diaphragms and ballast walls and major modifications are inevitably required.

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10.9 Alternative approaches to anchoring additional longitudinal prestress include anchoring onto existing diaphragms or onto the sides of webs. The former will require substantial anchor plates glued and bolted to the concrete, as it is unlikely that the existing concrete will be reinforced to take the additional forces. Attaching anchorages to the sides of a web can be undertaken using fabricated steelwork. If the additional prestress is only on one face, a substantial moment will occur in the steelwork connection and the concrete web. If this approach is used, the plates will probably have to be attached to the concrete using high-tensile prestressing bars. An alternative is to anchor to new transverse steel beams which span between the existing webs so that the fixing is only required to resist shear.

10.10 Where these intermediate anchorage systems are used, the prestress force and eccentricity can be varied as required by stopping off tendons. Where longer cables are used, the effective pressure can be adjusted by deviating the cables. The construction techniques which can be used for the deviators are essentially the same as those used for anchors. Deviators can be located in existing or new concrete diaphragms. Alternatively steel deviator beams spanning between webs can be introduced or individual steel deviator saddles can be fixed to the existing concrete members.

Tendon Profiles

10.11 Deflected tendons should be considered in cases where the structure is deficient in shear resistance as well as bending. The shear component of such cables will often lead to a more economical solution than the provision of separate vertical prestress. However, if the shear strength is adequate, it is often more economical to use straight tendons. The quantity of prestressing required with straight full length tendons is likely to be greater than with other methods but the saving in anchors, fixing and deviators more than compensates.

10.12 In many structures which have been strengthened, only the midspan moments have been a problem. In these cases, the parasitic moments and the higher friction losses resulting from using deflected cables tend to make this arrangement less efficient. However, particularly in constant depth sections, it may be necessary to deflect the tendons upward over the support to avoid overstressing the bottom flange.

10.13 In most cases, modification of the structure to form anchorages and deflectors for prestressing tendons will be expensive and therefore the free lengths in remedial work will often be greater than would be used when designing new structures. A consequential problem is the increased tendency for tendons to vibrate. Therefore designers should be aware of the need to check for resonance and of the need to make provision for damping.

10.14 Another approach which can be used to increase prestress is to provide profiled cables in new concrete attached to the existing concrete. This approach could be particularly advantageous if the original concrete section as well as the prestress were inadequate.

10.15 Vertical prestress can be provided by external tendons when they cannot be accommodated within the webs. However it is important to check that flanges and webs are able to accommodate any additional imposed stresses; particular difficulties may be encountered when dealing with box structures with inclined webs. Top anchorages will also be vulnerable to de-icing salt attack so that particular attention should be given to the detailing and waterproofing.

10.16 A major difficulty in installing additional longitudinal prestress is in accommodating it within the existing structure. This is relatively straightforward in major box girder bridges but extremely difficult in small voided decks.

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Cable Stays

10.17 Another possible approach to strengthening existing post-tensioned bridges is to install cable stays. These can be considered as analogous to either elastic supports or additional prestress with an unusually large eccentricity. The major limitation on the use of conventional cable stays using a mast is the geometrical problem of installing the mast and stays without interfering with the carriageway. This would require the installation of new cross members at the stay position. In small bridges, the transverse span of these would be almost as great as the span of the bridge. It is therefore only in longer span bridges that this solution is likely to be viable.

10.18 An alternative approach could be based upon King or Queen Post Trusses. Such techniques could be used if there is no problem with clearance under the bridge, using a combination of stays and a strut at midspan.

Plate Bonding

10.19 Requirements and advice on strengthening highway structures using externally bonded fibre reinforced polymer can be found in BD 85 (DMRB 1.3.18).

Elastic Supports

10.20 Elastic supports perform a useful function when it is expected that a structure will suffer from a process of gradual deterioration or where "hard" supports may induce undue stresses in the structure under load. This type of support can be used to support the soffit of the deck directly from the piers or abutments. The method can also be used for box structures through the provision of a prestressed concrete or steel frame "coat hanger" truss within the box cells.

BEAM AND SLAB DECKS

10.21 All the strengthening methods considered above can be used for this type of structure, but it is unlikely that cable stays will provide an economic solution.

External Tendons

10.22 Additional prestress can be provided by installing external tendons between the beams. This is likely to entail installing additional concrete or steel transverse members to anchor the tendons. It may, however, be possible to use the existing diaphragms, but some strengthening might be required. Where deflected tendons are used, similar local strengthening may be required for the deviators. Additional prestress is likely to be most economical way of strengthening whenever there is a requirement to make a significant increase to the service load capacity.

10.23 Strengthening schemes based on additional prestress can often be completed with minimal disruption to traffic, although some schemes may require extensive work at the ends of the bridge to make space for the anchorages and this is likely to require closures. However this could be done on a lane by lane basis. Similarly where new concrete is to be added to an existing structure, it is unlikely to be safe or practical to undertake the work over live highways and staged lane closures will be required. A full (or one span) closure is likely to be required for stressing. Once the anchorages and deviators are installed, the stressing operations can be quickly completed.

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Additional Elastic Supports

10.24 Another possible strengthening approach is to use additional elastic supports. The major limitation of this is the need for ground space on which to build them. However supports attached to the piers and supporting the beams quite close to their ends can be effective for relieving hogging moments. This approach is useful when it has been found that the use of additional prestress would over-stress the concrete. It has also been used in structures where precast beams are too close together to permit the inclusion of additional prestress. It should be possible to install this type of strengthening with minimal disruption to traffic.

Plate Bonding

10.25 Plate bonding is a simple way of strengthening beam and slab decks and has the advantages of minimal requirement for space and clearance. The major limitation is the fact that, although a very efficient method of increasing ultimate strength, it is not very effective when considered against the conventional serviceability design criteria for prestressed concrete. It may, however, be argued that this is not a very logical basis for rejecting the approach when a structure shows no signs of distress unless of course there is a reason to anticipate that there will be a continuing deterioration in serviceability. Possible reasons for further deterioration could be continued loss of the original prestress or increasing loads.

Additional Beams

10.26 It may be possible to introduce additional beams between existing widely spaced members in order to relieve them of some load.

BOX GIRDER

External Tendons

10.27 A review of case histories has revealed that the use of additional prestress has been the most utilised method of strengthening. Also structures were generally strengthened to correct excessive cracking, ie. work was undertaken for serviceability rather than strength reasons.

10.28 Cable stay techniques could be appropriate for box girder decks, however, their size tends to make other strengthening methods difficult. The high dead load to live load ratio which results from the scale of these structures has particularly discouraged plate bonding which has tended to be used only to increase ultimate strength under live load. In major box girder bridges, the prestress required to avoid tension under permanent and environmental loads is sufficient to give adequate ultimate strength. In contrast to the problems of applying other strengthening measures to large structures, the major difficulty with installing additional prestress tends to be the lack of space for the work. These problems tend to reduce as structures get larger. Since the work will take place mainly within the box, this type of structure can be strengthened with minor disruption to traffic either below or on the bridge.

Preflexing and Crack Injection

10.29 Preflexing and crack injection are often used in association with additional prestress. This method has been found useful where wide cracks exist in the soffit before strengthening. It is necessary for a bridge to be closed to traffic during loading and injection.

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IN SITU VOIDED SLABS

10.30 A variety of shapes, most commonly circular or rectangular, have been used to form voids in deck slabs. In terms of structural behaviour, a voided slab bridge (particularly one with rectangular voids) is essentially the same as a multi-cellular box structure. However, for the purposes of choosing a strengthening method, there is an important practical distinction between box girder bridges in which normally access is provided to the inside of the box and voided slab structures which do not have such access. Occasionally, it may be possible to create access for the purpose of undertaking the strengthening works, which could involve dissolving polystyrene void formers. The voids in most bridges of this form are too shallow to enable major strengthening work to be undertaken from inside. This makes the use of additional prestress in this type of structure difficult.

10.31 An alternative may be to install additional tendons below the soffit. Whilst this is only possible in sagging moment regions some advantage may be taken of the parasitic moments to relieve stresses at supports for continuous structures. It may also be possible in bridges with not more than three spans to transfer more of the support moments to midspan by lowering the intermediate supports relative to the end supports. A more serious difficulty in applying external prestress below the soffit is that the clearance may be compromised. The tendons are also more vulnerable to damage, particularly where the bridge is over a highway. This would limit the strengthening options to elastic supports or plate bonding but the considerations are essentially the same as for beam and slab bridges.

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11. NORMATIVE REFERENCES

11.1 Design Manual for Roads and Bridges

BD 21 The Assessment of Highway Bridges and Structures (DMRB 3.4.3)

BD 27 Materials for the Repair of Concrete Highway Structures (DMRB 3.3)

BA 35 Inspection and Repair of Concrete Highway Structures (DMRB 3.3)

BD 44 The Assessment of Concrete Highway Bridges and Structures (DMRB 3.4.14)

BA 54 Load Testing for Bridge Assessment (DMRB 3.4.8)

BD 58 Design of Concrete Highway Bridges and Structures with External and Unbonded Prestressing (DMRB 1.3.9)

BA 58 Design of Concrete Highway Bridges and Structures with External and Unbonded Prestressing (DMRB 1.3.10)

BD 63 Inspection of Highway Structures (DMRB 3.1.4)

BD 79 The Management of Sub-standard Highway Structures (DMRB 3.4.18)

BD 85 Strengthening Highway Structures Using Externally Bonded Fibre Reinforced Polymer (DMRB 1.3.18)

BA 86 Advice Notes on the Non-destructive Testing of Highway Structures (DMRB 3.1.7)

11.2 Manual of Contract Documents for Highway Works (MCHW)

Volume 1: Specification for Highway Works HMSO (MCHW 1)

11.3 Other publications

BS 1881: Part 206: Recommendations for determination of strain in concrete.

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Annex A Example Risk Assessment Report

A1 The example in this annex is given primarily to illustrate application of the risk assessment processes. Whilst it also illustrates an acceptable report format it is not intended to be prescriptive on this aspect and assessors may supplement the content of the report to suit particular cases.

M99 Blue Bell Knoll Farm Accommodation Bridge

This bridge was built in 1972 to carry a farm access road over the M99. The deck is a 3 span segmental post-tensioned box girder, tied down at the abutment supports by Macalloy bars. The main span is 42.5m with 12m side spans.

Summary of Findings From Risk Review

As built information – there is a full set of as-built drawings available.

PTSI Reports – The 1998 PTSI3 report is available. This report also includes the 1996 PTSI1/2 report as an appendix. The PTSI3 report contains a section on risk assessment which describes the risk of failure as low. Critical Sections are identified in the PTSI reports.

The latest Principal (April 2008) and General (April 2012) Inspection reports are available.

Load Assessment – The structure has not undergone a load assessment as accommodation bridges were not included in the assessment and strengthening programme.

Recommendation – Although there has not been a load assessment, as-built information, previous PTSI reports, and recent inspection reports are available so it is considered there is sufficient information to proceed with a risk assessment.

Primary Risk Assessment

Age

The bridge was built in 1972 so although it is not in the most vulnerable age group, it would not have benefitted from more recent improvements in materials and practice. The deck waterproofing system is known to be a modern and reliable one but there may be other issues which will be dealt with under later headings.

Form

The deck is a 3 span continuous non-composite box girder with transverse segmental joints and longitudinal prestress. This arrangement alone would suggest a medium risk. However, in addition, the PTSI reports and as-built drawings indicate that the anchor spans are tied down at the abutments by tensioned Macalloy bars. This type of arrangement is considered high or very high risk. Failure modes could develop through damage to the longitudinal post-tensioning system or failure of the Macalloy bars.

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Vulnerable Details and Materials Hazards

1 **Segmental Joints** – this deck has 100mm wide segmental joints filled with in situ concrete. Segmental joints are well known points of vulnerability. If the joints are poorly constructed or deteriorated, water and chlorides can reach the tendons where they pass through the joints.

Event – deteriorated or poorly constructed joint allows water and chlorides to reach the tendons causing corrosion and fracture of tendons.

Likelihood – Medium. Although they are vulnerable, wider concrete filled joints of this type are easier to construct than narrow mortar joints, which are more vulnerable.

Consequences – High – deterioration of the post-tensioning system with possible loss of capacity and eventually structural failure.

Risk level – **High**

Risk Management Measures – Monitor the joints at routine inspections for signs of leaks, and for transverse cracks which could be an indicator of tendon failure.

2 **Expansion Joints** – leaking expansion joints allow water and chlorides onto the end anchorages for the longitudinal area and also onto the bearing shelf where the Macalloy tie-down bolts are exposed.

Event – water and chlorides penetrate the longitudinal tendon anchorages and the Macalloy tie-down bars resulting in corrosion.

Likelihood – Medium. The joints were leaking at the time of the 1998 PTSI3 investigation. The latest PI and GI also shows signs of leaking. However the bridge does not carry a public road but a farm access road which is not routinely salted.

Consequences – High – deterioration of the post-tensioning and tie-down systems with possible loss of capacity and eventually structural failure.

Risk level – **High**

Risk Management Measures – Ensure the installation and maintenance of watertight expansion joints. Carry out intrusive investigations of the anchorages and Macalloy bars to determine their condition.

3 **Tendons close to the upper surface of the deck at interior supports** – the tendon profile is draped with the tendons running close to the surface over the interior supports. At these locations the tendons are more vulnerable to water and chlorides if they penetrate the deck waterproofing and top slab.

Event – water and chlorides penetrate the longitudinal tendons resulting in corrosion.

Likelihood – Low. The 1998 PTSI3 investigation found the tendons in good condition with few voids. A reliable deck waterproofing system was installed at the time of construction. There are currently no signs of leaks except at the expansion joints. The road over the bridge is not routinely salted.

Consequences – High – deterioration of the post-tensioning system with possible loss of capacity and eventually structural failure.

Risk level – **Medium**

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Risk Management Measures – Maintain the water management systems. Monitor the top slab for signs of water penetration that might indicate a failure of deck waterproofing (ie inspect from inside the box – see item 15).

4 **Anchorages Concealed in Joints** – the anchorages for the longitudinal post-tensioning system are buried in the ends of the box beam webs adjacent to the expansion joints (see item 2) and obscured by ballast walls. The anchorages were not investigated in the 1998 PTSI3 because it was not considered that the traffic disruption necessary to carry out the excavation was justified. Thus, the condition of the anchorages remains unknown.

Event – water and chlorides penetrate the longitudinal tendon anchorages resulting in corrosion.

Likelihood – Medium (see item 2).

Consequences – Low – deterioration of the anchorages and post-tensioning system. Failure of the anchorages is not expected to result in structural failure as bonded tendons should re-anchor.

Risk level – **Low**

Risk Management Measures – Ensure the installation and maintenance of watertight expansion joints. Carry out intrusive investigation of anchorages.

5 **Concealed Macalloy Tie Down Bars** – The as-built drawings show a line of eight vertical post-tensioned Macalloy bars tying down the end of the deck to each abutment. The bars appear to be exposed where they pass between the abutment and the deck, at the bearing shelf. This area is vulnerable to water and chlorides leaking through the expansion joints. Weephole drainage to the bearing shelf is shown on the drawing. These details would be difficult or impossible to inspect without intrusive investigation (see also item 14).

Event – corrosion and fracture of the Macalloy bars.

Likelihood – Medium (see item 2).

Consequences – High – fracture of the Macalloy bars could result in uplift of the deck ends, structural instability and possible collapse.

Risk level – **High**

Risk Management Measures – Ensure the installation and maintenance of watertight expansion joints. Carry out endoscope or intrusive investigation to examine the condition of the Macalloy bars where they are exposed at the bearing shelf.

Condition Hazards – External

6 **Cracking and Spalling** – The latest Principal and General Inspections record minor cracking and spalling at various locations on the deck. Cracking and spalling can allow penetration of water and chlorides and can also indicate corrosion of reinforcement and loss of prestress.

Event – general structural deterioration, water and chlorides penetrate the longitudinal tendons resulting in corrosion.

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Likelihood – Low. The 1998 PTSI3 investigation found the tendons in good condition with few voids. A reliable deck waterproofing system was installed at the time of construction. There are currently no signs of leaks except at the expansion joints. The road over the bridge is not routinely salted.

Consequences – High – deterioration of the post-tensioning system with possible loss of capacity and eventually structural failure.

Risk level – **Medium**

Risk Management Measures – Repair spalls to prevent further deterioration of exposed reinforcement. Monitor development of cracks at routine inspections.

7 **Minor cracking to deck surfacing** – cracks allow water and chlorides to penetrate beneath the carriageway surfacing. They can then penetrate the deck if there are similar defects in the waterproof membrane. Cracks in the surfacing can also reflect cracks in the underlying structure.

Event – water and chlorides penetrate the deck and the longitudinal tendons near the top surface resulting in corrosion.

Likelihood – Low. The crack is currently minor and the road is not routinely salted.

Consequences – Low – could contribute to deterioration of the post-tensioning system but unlikely to itself result in structural failure.

Risk level – **Low**

Risk Management Measures – Monitor surface cracking at routine inspections. Investigate the cause of the crack when the bridge is next resurfaced.

8 **Expansion Joints De-bonded and Split** – see item 2.

9 **Water seepage at abutments and north pier** – seepage at the abutments is due to leaking expansion joints (see items 2 and 8). Water seepage at the north pier would be a concern if water was seeping through the deck. However the Principal Inspection photograph suggests the staining is due to a defective drain.

Event – water and chlorides penetrate the longitudinal tendons in the beam above the north pier resulting in corrosion.

Likelihood – Low. The staining appears superficial. The road is not routinely salted.

Consequences – High – deterioration of the post-tensioning system with possible loss of capacity and eventually structural failure.

Risk level – **Medium**

Risk Management Measures – investigate whether the staining is due to a drainage defect or leakage through the deck. Repair any drainage defects.

10 **Bearings** – The latest Principal and General Inspection reports say “Evidence of water seepage to bearing shelf. Bearings not visible. Probably subject to corrosion”. It is not possible to carry out a visual inspection of the bearings. SMIS records suggest the bearings are elastomeric plain rubber pads. Note that the Macalloy bars are also vulnerable in this area but this has not been recognised by the inspector (see item 14).

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Event – Deterioration, damage and unexpected movement of the bearings.

Likelihood – Low. Bearing type is not susceptible to corrosion and the road is not routinely salted.

Consequences – Low – long term deterioration of the bearings.

Risk level – **Low**

Risk Management Measures – if the Macalloy bars undergo intrusive inspection, inspect the bearings at the same time.

Condition Hazards – Internal

The hazard listed is taken from the 1998 PTSI3 site investigation report.

11 **Soft grout was found in 3 out of 11 ducts investigated and partial voids were found in 2 out of 11 ducts investigated**. However there was no significant corrosion of tendons, which were found to be in good condition. Voids and soft grout can allow water and chlorides that enter the duct to contact the tendons causing corrosion. In the event of wire and tendon fracture, soft grout and voids can hinder reanchorage.

Event – water and chlorides penetrate the longitudinal tendons resulting in corrosion.

Likelihood – Low. The 1998 PTSI3 investigation found the tendons in good condition with few voids. All ducts investigated were dry. A reliable deck waterproofing system was installed at the time of construction. There are currently no signs of leaks except at the expansion joints. The road over the bridge is not routinely salted.

Consequences – High – deterioration of the post-tensioning system with possible loss of capacity and eventually structural failure.

Risk level – **Medium**

Risk Management Measures – As the structure appears to be in good condition with no sign of deterioration of the segmental joints, it is considered there is no need for further intrusive investigation of the longitudinal tendons at this time. The need for intrusive investigation should be reviewed if the condition deteriorates.

History Hazards

12 **Maintenance of Water Management Systems** – there is evidence that the expansion joints have been leaking in the long term, allowing water onto the bearing shelves at the abutments (see items 2, 8 and 9). There is also a drainage problem at the north pier (see item 9).

13 **Use of De-icing Salts** – The bridge carries a farm access track which is not a public road, so the position on use of de-icing salts is unclear. However, it is unlikely that it is salted as often as public roads if at all. Any chloride contamination is likely to result from spray from the road under (M99). Materials tests indicate that chloride contamination in this structure is at low levels. These factors would tend to mitigate the risks posed by some of the other hazards listed.

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14 **Bridge form not fully understood** – Macalloy tie-down bars are shown in the as-built drawings and in the PTSI reports. However they are not mentioned in the inventory system "Description of Structure" and "Articulation" fields which state "Two Andre Rotoflon bearings per abutment. Fixed above the piers and free to move above the abutments." Hence maintenance engineers and inspectors may be unaware of the existence of the Macalloy tie-down bars (see 5) and they will not be inspected and maintained.

Event – corrosion and fracture of the Macalloy bars.

Likelihood – Medium (see item 2).

Consequences – High – fracture of the Macalloy bars could result in uplift of the deck ends, structural instability and possible collapse.

Risk level – **High**

Risk Management Measures – Ensure the "Articulation" field on inventory system records the presence of the Macalloy tie-down bars and that maintenance engineers and inspectors are aware of their presence. Carry out endoscope or intrusive investigation to examine the condition of the Macalloy bars where they are exposed at the bearing shelf.

15 **Inadequate inspections** – The inspection summary for the last Principal Inspection that took place in April 2008 states "The internal inspection was unable to be carried out due to coordinating the opening of the hatches with a third party. The hatches are deck waffleboard covers and require a HIAB to lift them off, however information on previous reports suggest no immediate concerns within the internal sections." Therefore it appears there has been no internal inspection of the box since the previous Principal Inspection in June 2001. Internal inspections must not be omitted as they are an essential risk management measure for items 1, 3, 6, 7 and 11.

16 **Recommendations from previous PTSIs have not been implemented** – The 1998 PTSI3 report recommended "Repair or replace leaking expansion joints as a matter of urgency". However the joints still appear to be leaking (see 2, 8, and 9). The report also states "Consider construction of inspection galleries at the ends of the bridge to allow for ease of future maintenance and inspection". It is not clear if this was considered but it has not been carried out. There remains a need to facilitate viewing of the bearing shelf area so that the bearings and exposed lengths of Macalloy bars can be inspected. The report also recommended (as a low priority) a load assessment to examine sensitivity to loss of prestress and failure of the Macalloy bars. This has not been carried out.

Load Assessment Hazards

17 **Structure has not been assessed** – As the structure is an accommodation bridge, it was not within the scope of the assessment programme and so has not been assessed. Sensitivity to loss of prestress and loss of Macalloy tie-down bars is unknown. Although loss of longitudinal prestress is not likely to be a problem at present due to the good condition of the longitudinal post-tensioning system, the condition of the Macalloy bars is unknown (see item 5).

A load assessment was recommended by the 1998 PTSI3 report (see item 16). The need for assessment should be reconsidered following investigation of the condition of the Macalloy tie-down bars.

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Risk Management Plan

The structure was built in 1972 so lies in the second highest risk age band. The structural form has a very high risk of brittle failure. A number of hazards have been identified. Most are not considered to present an immediate risk to the integrity of the structure. However, the unknown condition of the Macalloy tie-down bars could present a significant risk and investigation of their condition should be given a priority. Some risk management measures are proposed to minimise the long term deterioration of the structure and provide assurance for the continued operation and management of the structure.

The proposed risk management measures are:

- **Carry out endoscope or intrusive investigation to examine the condition of the Macalloy bars where they are exposed at the bearing shelf**. If the Macalloy bars undergo intrusive inspection, inspect the bearings at the same time. (H)
- Maintain water management systems. Monitor the top slab for signs of water penetration that might indicate a failure of deck waterproofing. Ensure the installation and maintenance of watertight expansion joints. (H)
- Monitor the segmental joints at routine inspections for signs of leaks, and for transverse cracks which could be an indicator of tendon failure. (H)
- Ensure the Articulation field on SMIS records the presence of the Macalloy tie-down bars and that maintenance engineers and inspectors are aware of their presence. (H)
- Internal inspections must not be omitted as they are an essential risk management measure. (H)
- The need for assessment should be reconsidered following investigation of the condition of the Macalloy tie-down bars. (H)
- Investigate whether the staining on the north pier is due to a drainage defect or leakage through the deck. Repair any drainage defects. (M)
- Repair spalls to prevent further deterioration of exposed reinforcement. Monitor development of cracks at routine inspections. (M)
- Monitor carriageway surface cracking at routine inspections. Investigate the cause of the cracks when the bridge is next resurfaced. (L)
- Carry out intrusive investigations of the longitudinal tendon anchorages. (L)

Risk Rating Calculation

Age Factor – The bridge was built in 1972 so from Table 3.5, $F_A = 4$.

Bridge Form Factor – 3 span continuous non-composite box girder with transverse segmental joints and longitudinal prestress. This arrangement alone would suggest a medium risk with $F_F = 8$ (Table 3.6). However, in addition, the PTSI reports and as-built drawings indicate that the anchor spans are tied down at the abutments by tensioned Macalloy bars. It is not known if the deck is tied down for live load only or dead load and live load so assume the worst case until shown otherwise. $F_F = 12$.

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Vulnerable Details and Materials Factor – The following vulnerable details have been identified with reference to Table 3.7:

- Segmental joints (wide in situ concrete)
- Expansion joints that allow passage of water
- Tendons close to the upper deck surface at interior supports
- Anchorages concealed within joints
- Concealed Macalloy tie-down bars

As there are 5 vulnerable details, $F_V = 5$.

Condition Factor – The following condition hazards have been identified with reference to Table 3.8:

- Concrete cracking – various locations and directions
- Minor cracking to deck surfacing
- Expansion joints debonded and split
- Water seepage and staining at abutments and north pier due to leaking joints
- Minor spalling and exposed corroded reinforcement
- "Evidence of water seepage to bearing shelf. Bearings not visible. Probably subject to corrosion" (quote from GI/PI report). This affects the north and south abutment bearing shelves where the Macalloy holding down bars are also concealed.
- In 3 out of 11 of the ducts investigated in the PTSI3, grout was found to be soft.
- In 2 out of 11 of the ducts investigated in the PTSI3, voids were found.

As there are 8 condition hazards, $F_C = 8$.

Consequence Factor – Data from the TRADS traffic data website shows that the two way annual average daily traffic (AADT) flow on the M99 near the bridge site is 52435. With reference to Table 3.4, this gives a consequence factor $F_Q = 3$.

Risk Rating

$R\% = 100[((4F_A + F_F + F_V + F_C) F_Q) - 6]/254$

$R\% = 100[((4 \times 4 + 12 + 5 + 8) 3) - 6]/254 = 46\%$

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Annex B Case Studies

B1 Post-tensioned I-Beams

B1.1 M3 Motorway Bridges

During the construction of ten 3-span overbridges on the M3 motorway, between Basingstoke and Hawley, a construction defect was found which led to the demolition of a number of the post-tensioned I-beams and the extensive repair of others. The main beams were precast, by a specialist sub-contractor, in five segments. The segments were erected on temporary supports, connected with 100mm in situ concrete joints and then post-tensioned. The basic bridge had spans of 12.8m, 36.9m and 12.8m with beams at 1.8 to 2.1m centres and a 250mm thick composite prestressed deck slab.

The main prestressing cables consisted of 2 No. 19mm Dyform strands which were winched through 100mm diameter corrugated ducts. The bottom cables were stressed to a load of approximately 390 tonnes prior to the casting of the in situ deck slab. The upper cables were then stressed in a similar manner to the lower ones, but with the prestress acting on the full composite section. During stressing of the cables, a series of cracks were detected in the central spans, which generally started at the intermediate diaphragms and followed the line of the ducts. The cause of the cracking was confirmed by gamma-ray radiographs to be flotation of the ducts which, at worse, were 450mm out of position.

During 1970-71 six of the least affected beams were strengthened by the provision of external remedial post-tensioning. The remaining seven affected beams were demolished. The strengthening work involved the casting of concrete onto each side of the web, which was then prestressed by 25 or 32mm diameter longitudinal Macalloy bars positioned at a suitable eccentricity to compensate for the out of position cables. Additional 20mm diameter vertical Macalloy bars were threaded through holes drilled in the deck and top and bottom flanges of the beams. These bars helped to provide good structural interaction between the old and new concrete.

The anchorage blocks were formed using several transverse Macalloy bars, as well as ordinary transverse steel, which were necessary to link the two halves of the repair with the main beam. During the drilling operations for the insertion of the transverse bars the main cables in two of the beams were cut by the drill and these beams, therefore, had to be demolished.

These mistakes underlined the need for close supervision of work of this nature. The bridges which had been strengthened in this manner were tested using a series of 21 tonne gravel lorries positioned on the deck to simulate the effects of standard HA loading. The load positions were varied to produce maximum longitudinal and transverse bending moments in the main span. The deflections measured at each loading stage were in close agreement with those predicted and the recovery after the loads were removed was extremely good.

B1.2 Bridge W18, Belgium

This three span bridge in Belgium was constructed in 1960. It has a main span of 52m and two 26m side spans. The deck consists of five variable depth in situ beams. It was externally prestressed with 48 or 56/7mm diameter wire cables protected by cement mortar. The deck was not waterproofed and the mortar proved inadequate to protect the cables from de-icing salts and industrial pollution. The sand layer under the footpaths acted as a water trap and the worst corrosion arose in the cable beneath these areas.

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In 1974, one of the outer cables broke. Strengthening of the structure was undertaken in 1975-76 when all the longitudinal prestress was replaced with BBRV cables protected by grouted ducts. Special anchors were used because the new prestress was of a completely different form from the original and normal anchors did not fit the available rectangular space. However, in contrast to strengthening works on internally stressed structures, no significant additional structural concrete was required.

B2 Post-tensioned Double-T Beams

B2.1 Wangauer Bridge, Austria

Wangauer Bridge in Austria was built between 1962 and 1964. The 391m long continuous structure consists of 11 spans of 28m and 2 spans of 41.25m. It was constructed from a post-tensioned double T-section with compression slabs above the supports. A standard depth of section of 2.2m was used which was as required for the 41m spans. This meant that in the shorter spans there was an insufficient level of prestress.

The required level of prestressing was never attained due to the weight of the pavement layer, the underestimate of creep and shrinkage losses at the tendon centroid and the effects of temperature gradient. Over a number of years, the bridge exhibited an increasing crack pattern, with the widest cracks particularly evident at the construction joints. An assessment of the structure showed it to be suffering from a deficit in prestressing. Therefore, the application of additional external post-tensioning presented itself as an ideal strengthening method. Initial considerations about the profile of the additional tendons led to the decision to use straight cables and thereby excluded the possibility of friction losses at deviation points. This also meant that detailing was simpler and saddles were only required to transfer minimum forces to the structure. The prestressing was mounted externally to the T-sections, with 4 No.12/5mm diameter 7 wire VSL type stands positioned on the outside face of each web and contained within a 90mm diameter polyethylene duct which was filled with cement grout after stressing. Saddle points comprised pipe clamps bolted to the outside of each web.

Anchorages for the new cables were embedded in the old concrete using specially reinforced end beams. Transverse prestressing cables were used to transfer the forces from the new longitudinal tendons to the section. The 3 No. 7/5mm diameter 7-wire strands exerted a transverse prestress of approximately half the additional force in the longitudinal direction.

B3 Post-Tensioned Single Cell Boxes

B3.1 Pont de Lacroix Falgarde

This bridge was built in 1960-62 and has a main span of 60.5m with side spans of 30m. The continuous deck is a variable depth single cell box structure, constructed in situ using the balanced cantilever method. Prestress is applied in the longitudinal direction by Freyssinet cables consisting of 12 No. 8mm diameter strands and in the transverse direction by 12 No. 7mm diameter strands.

During an inspection in 1975, significant cracks were noted in the web and bottom flange near the centre of the main span. An investigation of the structure revealed no evidence of the cracks growing wider with time. However, the inclination of the cracks in the webs led to worries over the shear capacity of the structure. Calculations revealed that the links in the structure were sufficient to give adequate shear strength without the need for any contribution from the concrete. It was concluded that, despite the significant cracks in the web, there was no risk of a shear failure.

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Concerns over fatigue in the continuity cables were raised when the passage of a 26 tonne lorry was found to open the cracks by nearly a millimetre. As a result of this discovery, instruments were installed to measure the change in strain in the tendons under load. The measured strain due to the passage of a 22 tonne lorry suggested a stress change of only some 16N/mm² and from this it was calculated that the fatigue strength was adequate. It was, however, realised that this low stress range must have resulted from the tendons sliding over a considerable length in the ducts, showing that the grouting was very poor. A general strength assessment concluded that it was safe to keep the bridge in service whilst permanent repairs were arranged, strengthening being required for serviceability reasons only.

Various causes of the cracking were identified. Some local cracking was due to the reinforcement at the anchorage positions being inadequate to spread the prestress into the section. This was exacerbated by excessive numbers of cables stopped at the same position in the span. However, more importantly, neither temperature difference nor the redistribution of moments due to creep were considered in design whilst prestress losses due to friction and relaxation were under-estimated. The validity of the analysis used in the assessment was confirmed by the rather unusual approach of measuring the reactions on the abutments using flat jacks.

Strengthening of the structure was completed in 1977 and achieved by the addition of external Prestressing cables. The required increase in prestress was achieved by means of 12 No. 15mm diameter strand cables; eight in the main span and four in each side span. Although the new cables were external to the concrete section they were protected by ducts and grouted in the normal way. The use of straight main tendons was made practical by the extreme haunching of the bridge which meant that cables, located as close as possible to the bottom flange at midspan, were in a reasonable position in the support section. Also, since the existing links were adequate, the enhanced shear capacity provided by deflected tendons was not required. The cables were anchored in massive new transverse reinforced concrete beams provided at the abutment and on either side of each of the pier head diaphragms. The pier head diaphragms were strengthened with a total of 8 No. 32mm vertical Dywidag bars in each diaphragm.

The repair work on the structure included resin injecting the cracks. The cracks were also instrumented and this, and other instrumentation provided, was used in load tests performed after the completion of the work. Static tests were performed with enough 26 tonne lorries to represent the French design loading. The behaviour of the structure was entirely satisfactory with no evidence of the loading causing damage.

B3.2 Viaduc du Magnan

The bridge carries the A8 motorway near Nice, France, and has a total length of 486m including three 120m spans. It was built between July 1973 and December 1975, the relatively long construction period reflecting the fact that it is of cast in situ balanced cantilever construction. Unusually for an in situ structure, no continuous secondary reinforcement was provided. During an inspection in 1980 it was noted that 8 or 9 of the joints between the segments near the centre of the 120m spans were opening. In the worst places the opening extended some 1.5m up the web and the worst opening was approximately 1mm. The cause of the problem was identified as the lack of consideration of temperature difference in design combined with under estimation of creep redistribution by a factor of two to three.

Additional prestress was applied between 1982-1984 and designed to put the structure back in compression throughout. All the cables were straight and 8 No. 12/15mm diameter strands ran the full length of the bridge, with an additional 4 No. 12/13mm diameter strands provided in the midspan region of the three long spans. The cables were anchored in substantial new reinforced concrete anchorage blocks which were attached to the existing structure after drilling the necessary holes. Although the volume of concrete in each pour was relatively small, 4 to 12m³, it was pumped for convenience of access.

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In order to make the repaired structure truly monolithic, it was decided to preflex the deck then resin inject the cracks and finally apply the additional prestress. However, because the heavily trafficked bridge could not be closed for the full repairs, the prestress was temporarily stressed to 70%. Later, when a second structure had been completed and the bridge could be closed, the tendons were destressed, the load applied and the cracks injected before the tendons were stressed to their final load of 80% of the characteristic strength. Since there was a significant period between the two stages of stressing, corrosion protection was required. It was therefore decided to use galvanised strand because of the need to destress the strands and the doubts about the effectiveness of using grease protection with such long cables. Concerns over the safety of construction personnel working with unducted cables led to the provision of rings round the tendons at 1m centres near the anchors and 4m centres elsewhere.

In common with other "strengthening" projects undertaken in France, extensive instrumentation was used, including many strain gauges and displacement transducers across the joints. The instrumentation confirmed that, after completion of the work, the structure was behaving in an essentially monolithic fashion as intended. However, it was still not possible to verify exactly if the stress state was as assumed in the design of the strengthening work.

B3.3 Boivre Viaduct, France

Boivre Viaduct is located about 180 miles southwest of Paris. The seven span, 290m long, continuous prestressed concrete bridge was one of the first incrementally launched structures in France. Temporary towers and provisional stays were used for placing the deck elements. The single-cell, post-tensioned concrete box girder deck had a constant depth, along the centre-line, of 2.5m and a width of 13.41m. The five interior spans of 43m and the two exterior spans of 35.8m were each constructed from 13 No. precast prestressed sections.

Additional longitudinal prestressing tendons were provided over the supports in order to ensure continuity over the entire length of the viaduct. In addition to the longitudinal cables in the top and bottom slabs, transverse cables were located in the top slab throughout the length of the bridge and vertical tendons in the webs for 11m of either side of each of the piers. Constructed in 1971, a survey in 1978 uncovered a pattern of cracks, up to 6.4mm wide, in the deck.

Longitudinal cracks were found between the webs and the top and bottom slabs, almost completely separating the slabs from the webs. In the webs, inclined cracks were found between the top and bottom longitudinal tendons in the vicinity of the anchorages. Transverse cracks were found in the bottom slab behind the anchorages of the longitudinal tendons. The transverse cracking was only evident during thermal fluctuations when vertical cracks were found, at midspan of the interior spans, propagating from the bottom of the webs. Under test loading, deflections were found to be 25% greater than expected and large stress discontinuities were found, with stresses in the bottom slab half those at the bottom of the webs.

Analysis of the original design showed that the longitudinal prestress was either inefficient, inadequate and/or ineffective. There were no draped cables longitudinally and the vertical and transverse cables did not provide the compressive stresses required in the webs. In addition, the longitudinal tendons were anchored in clusters which produced areas of high stress concentrations. As a result, strengthening was considered necessary.

Strengthening of the deck took place between 1982 and 1984. External vertical and horizontal tendons were used to re-establish the integrity of the structure and increase the shear capacity of the webs. Additional longitudinal cables were used to produce the desired horizontal compressive stresses. The transverse and vertical tendons comprise 37mm diameter stainless steel bars, 6 of which were stressed to produce a force of 785kN at each of 156 locations throughout the length of the viaduct. As these tendons were positioned around the outside of the box, the longitudinal cables had to be placed on the inside.

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Ten longitudinal cables 290m long each comprising 17 No. 15.7mm diameter galvanised strands were used to provide a force of 2943 tonnes which induced a compressive stress of 4N/mm² in the section. The cables were threaded through 114mm diameter glass fibre reinforced ducts, which were supported in position by metal supports bolted to reinforced concrete blocks spaced at 4m centres, After stressing the cables, a wax-based grout was pumped under a pressure of 1kg/cm² into the ducts.

New reinforced concrete end anchorage blocks had to be constructed to distribute the longitudinal forces uniformly. End blocks 1m thick were stiffened by three 2m thick beams. Two 1.2 x 1m steel bearing plates were used to support the ten additional anchorages.

Load tests carried out on the bridge after the strengthening work, indicated that the measured deflections were 15-20% less than the computed values. In addition, the transmission of forces from one span to another had returned to normal and stress distributions across the section became linear.

B3.4 Rhone Bridge, Switzerland

The Rhone Bridge at Massongex in Switzerland is a 230m long, six span single cell box girder bridge with a maximum span of 72m. Monitoring of the structure showed the haunched main span to have a midspan deflection of 113mm, which was accompanied by a number of cracks with a maximum width of 0.5mm. As the deflections had increased linearly with time, it was feared that the trend would continue. Assessment of the design indicated that the effects of differential temperature gradients had not been considered. It was also suspected that the existing level of precompression was considerably less than considered in the design. It was therefore decided to install a strengthening system using additional prestressing.

In 1992, eight 12/15.2mm VSL type cables were installed within the box section. The longitudinal cables were installed with draped profiles, using deviator tubes placed in the existing pier diaphragms and two low-point deviator beams in the main span. The strand bundles were pulled through polyethylene ducts and slightly stressed prior to grouting with cement grout. The grout was allowed to harden prior to stressing the cables.

The anchorages for the longitudinal cables were located in prestressed buttresses added to the abutment diaphragms. Stressing of the cables took place from access chambers constructed behind the abutments.

B4 Post-Tensioned Twin-Cell Boxes

B4.1 Great Naab Bridge, Bavaria

This 3-span continuous two cell box structure was constructed in 1953-1954. The post-tensioned haunched deck had spans of 27.2m, 34.0m and 27.2m. Diaphragms were constructed at each midspan point and over the piers. Within two years after construction, deflections of approximately 30mm were found in the central span which caused cracks up to 0.35mm wide to appear in the soffit slab.

Remedial works took place in 1956 when additional external prestressing was applied. The prestress was applied by 16 No. cables comprising 38mm diameter strands with a prestressing force of 600kN. The strands were coated with a corrosion protective paint. Restressing of the cables took place in 1958 and 1959 to reinstate the initial prestressing force of 600kN.

During a principal inspection, in 1979, it was found that all the additional prestressing cables were badly corroded and that one cable had broken. Corrosion was worst where the cables passed throughout the transverse diaphragms.

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Given the concerns about the additional prestress, in 1982 it was decided to replace all the corroded external cables. The cables were replaced by coupled short lengths of strand comprising 7 No. 12.2mm diameter wires stressed to a prestressing force of 600kN. Corrosion protection was applied using a protective cold paste and wrapped with a petroleum bandage. A second polyethylene protective wrapping was also applied. Where the cables passed through the diaphragms, joints were provided on each side to allow pumping of grout into the duct.

B4.2 Los Chorros Viaducts, Venezuela

Los Chorros Viaducts are two parallel 320m long 5 span bridges built in Caracas, Venezuela from 1969-71. The central sections of the post-tensioned, segmental, two cell box structure were constructed using the balanced cantilever method, with 60m long haunched cantilevers. Surveys of the structure showed gradually increasing deflections in the central 120m long span. By 1982 the midspan deflection had reached 410mm and there was a distinct kink in the deflected profile. In addition, cracks up to 1.5mm in width were clearly visible at midspan, which indicated a significant decrease in flexural stiffness at this point.

An analysis of the structure indicated that the excessive deflections were likely to have been caused by higher than expected superimposed dead loads and an underestimation of the redistribution of bending moments caused by creep, shrinkage and relaxation.

The strengthening procedure adopted was designed to close the cracks and to provide sufficient factors of safety against fatigue failure and at the ultimate limit state. In 1988, 12 No. 12/15.2mm VSL type strands were added externally to strengthen the main span. Four draped tendons were positioned along each web, with two on either side of the central web. The continuous cables were deflected through steel tubes set in holes in the diaphragms over the piers and through deviation frames in the main span.

The cables were passed through polyethylene tubes and the prestressing force was applied from both ends before grouting with a cement grout. The cables were anchored in the side spans by reinforced concrete buttresses stressed into the webs. Two transverse RC struts helped to resist the transverse forces which arose from the eccentric force applied to the outer webs. In addition, short lengths of prestressing bar stressed through the webs provided the necessary normal force to ensure that the force was transferred to the webs.

B4.3 Mur Bridge, Austria

Mur Bridge near St Michael, Austria was built in 1973-74. The five span continuous, post-tensioned, two-cell box girder structure had a maximum central span of 105m. At the middle of the centre span, a deflection of 90mm was discovered shortly after the bridge was completed. By 1987 this deflection had increased to160mm, resulting in poor surface water drainage.

An analysis of the existing structure indicated that the long term deflection would increase to approximately 220mm and the maximum tensile stresses would lie within the range 4 to 5 N/mm². Although this was not seen to present a problem structurally, it was decided that remedial measures should be taken in the late 1980's to reduce the high accident risk. Therefore, external post-tensioning cables were used to strengthen the central span.

Prestressing cables were positioned on the inside of the box cells, at the bottom of the section at midspan and at the top of the section over the central two piers. Four 16/12.7mm diameter strands were installed on each side of both box sections, making a total of 16 cables. Each cable was covered with polyethylene sheath and then sealed with a durable corrosion protection material. Stressing of the cables took place after sealing.

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The eccentric forces created by the longitudinal cables on the webs produces additional bending moments which would have imposed severe stresses. In order to carry these additional forces, horizontal steel tubes and four strand cables were used as compression and tension members between the webs.

The cables were anchored in staggered anchorage strips in order to reduce the problems of stress concentration. These strips were connected to the webs by means of horizontal stressed bars which helped to transfer the longitudinal prestress to the section. Additional vertical and horizontal prestressing bars were needed to counteract the tensile splitting forces introduced by the longitudinal cables into the webs as well as the top and bottom slabs. Both the vertical and horizontal bars were anchored on the outside of the section.

B5 Post-Tensioned Multi-Cell Boxes

B5.1 Pont De Fives, France

This bridge near Lille has spans of 20.4, 30.8 and 16.7m and was built in 1955. It has 8 post-tensioned beams each precast in three elements, with RC box beams on either side. The longitudinal prestress was applied using 12 No. 7mm diameter wire cables. In the transverse direction the cables comprised 12 No. 5mm diameter wires. The completed structure was effectively a multi-cellular box and prestressed transverse diaphragms were provided at approximately 6.5m centres.

In 1966, cracks were noticed in the vertical joints in the beams. As a result of this an inspection, which included the use of gamma radiography, was undertaken. This revealed that less than half the prestressing ducts had been properly grouted. Samples of the steel were taken and these revealed corrosion and embrittlement. There was concern about the possibility of wires breaking without warning. It was therefore decided to install acoustic detectors. The sensitivity of these had to be reduced to prevent their sounding the alarm during the passage of heavy vehicles. However, some wire breaks were detected and in 1972 the bridge was closed to vehicles over 3.5 tonnes.

Strengthening works, completed in 1977, were designed to ensure safety in the short term. Longitudinal prestress was installed consisting of four No. 15mm diameter strands placed in plastic ducts and anchored in the transverse diaphragms. Vertical prestress was also provided by means of wires anchored to plates on the top and bottom of the bridge. Concrete was added to the top flange before prestressing.

To ensure the long term service of the bridge for “many decades” additional post-tensioning was provided to allow for further deterioration of the original prestress. This was designed to avoid initial over-stressing of the concrete in compression when much of the original prestress was still effective. The cables comprised 12 No. 15mm diameter strands which ran the full length of the bridge. These cables were anchored in new concrete added at each end of the bridge and were deflected. In addition, short (6 to 7m) cap cables were installed over the supports.

In order to install the additional prestress, many holes of 80mm diameter had to be drilled in the original concrete. Access problems made this and other stages of the operation difficult. However, the strengthening works appear to have been successful.

B5.2 A14 Railway Viaduct

Railway Viaduct carries the A14 over the main East coast railway line and a local road in Huntingdon, Cambridgeshire. The 6 span structure opened in 1975. Each span is 32.3m with the exception of span 4, the main span, which is 64.3m. Spans 1 and 2 at the western end and span 6 at the eastern end comprise simply supported precast pre-tensioned box beams that are also post-tensioned transversely. Spans 3 and 5 are of a multi-cell in situ post-tensioned box construction, continuous over the supports of span 4 to provide 16.15m cantilever sections. Within span 4 is a 32.0m precast pre-tensioned box beam span suspended from the cantilever sections by means of half joints. Tie-down bars are located at piers 2 and 5 to resist any uplift due to loading on the cantilever and suspended spans.

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The special inspection of the post-tensioning system in accordance with BD 54/93 and BA 50/93 was carried out in 1994/5. This found voids, moisture and chloride penetration in the ducts of the in situ post-tensioned parts of the structure (spans 3, 5 and the cantilever sections of span 4). Despite this, there was no corrosion of the tendons (Barker, 2010).

Inspections had also discovered cracks developing from the re-entrant angle of the half joints on the cantilever spans. Monitoring between 1995 and 1999 indicated that the crack widths were increasing and in 1998/99, radiographic surveys provided further evidence of significant voids in the ducts (Welsford, 2005).

As part of a management strategy to keep this structure in service, it was decided to install an acoustic monitoring system. There were 36 sensors installed on span 3 and the western cantilever of span 4, to include monitoring of the half joint. The system was commissioned in mid-1998 in conjunction with off-site trials of the monitoring system conducted by the Transport Research Laboratory (TRL) (Barker, 2010).

Around the year 2000 assessments on the half joints were unable to give satisfactory results. A study suggested the most economic option would typically be to replace the structure but at that time there was a plan to realign the A14 potentially leaving the structure redundant within 12 years. Therefore a strategy was developed to keep the structure in service for that period. A proposal to provide an alternative load path should the half joints fail was adopted and so in 2001, grillages of steel beams were installed under the cantilevers of span 4 to provide support to the half joints in the event of failure (Welsford, 2005).

Assessment had shown that in addition to the steel grillage, it was necessary to re-grout some of the tendon ducts in order for the structure to act as fully bonded in the cantilever spans. A programme of off-site re-grouting trials, radiographic testing and site works was successfully completed in 2001/2, increasing the shear capacity of the cantilevers (St Leger, 2005).

In 2008, further assessment work established the tendon section loss that could be tolerated by the structure before theoretical failure would occur. It was decided to extend the acoustic monitoring system with an additional 16 sensors on span 3 and then the whole system mirrored on span 5 and the eastern cantilever of span 4 (Barker, 2010).

The acoustic monitoring system did not detect any wire breaks between 1998 and 2005. Since 2005, there have been about 18 detected wire breaks, a rate which does not give cause for concern.

B5.3 M56 Bowdon View Bridge

The bridge was built in 1971 to carry the M56 Junction 7 westbound exit slip road over the M56. The two span (33m and 38m) deck was made up of precast concrete box beams. There were 6 internal main beams and 2 narrower edge beams, with a depth of 1.75m. Each beam was made from 11 precast segments. Longitudinal post-tensioning tendons were located in the gap ("trench") between each beam, which was packed with expanded polystyrene, except for a gap around the tendons which was grouted after stressing. Continuity was provided at the segment joints using narrow in situ concrete diaphragms that were post-tensioned using tendons in grouted ducts. Wider post tensioned diaphragms were provided at the end supports and the central support was a 1.83m wide post-tensioned concrete cill beam on a reinforced concrete column.

In 2000 a PTSI found some deterioration of concrete and tendons and in 2001 an assessment found the cill beam to be deficient in bending and shear, with the assessed capacity less than dead load. Interim measures were put in place including props to the cill beam, weight limit and speed limit.

In 2006 an investigation was carried out to trial the feasibility of repairs to the concrete surround to the longitudinal tendons. Hydro-demolition was used to excavate one of the trenches containing the tendons over the length of one and a half precast box units. Corroded and broken wires were found and as a result the capacity of the bridge was

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reduced to a single lane at 40T ALL. The material surrounding the tendons was found to be a form of concrete rather than the expected grout. It would have been difficult to ensure compaction around the tendons and this raised concerns about the likelihood of voids. It was recommended that the deck should be replaced, rather than repaired and strengthened.

In 2006/2007 an assessment was carried out using elastic analysis assuming the longitudinal tendons to be unbonded. With the assumption of no corrosion in the tendons the assessed capacity was 40T ALL in lane 1 and 7.5T ALL in lane 2 (although the bridge had already been restricted to a single lane following the 2006 investigations).

In 2007 further intrusive investigations were carried out drilling in from the deck soffit and from the top of the cill beam, and this discovered further corrosion and wire breaks as well as grout voids and water penetration. The deck was rewaterproofed in order to minimise further deterioration and a monitoring system was installed to enable the deck to be managed in service until it could be safely demolished and replaced.

In February 2009 an assessment was carried out using non-linear analysis. This assessed the capacity at 40T ALL in lanes 1 and 2, with no abnormal heavy vehicles and no hard shoulder live loading. This was valid for up to 20% of strand loss due to corrosion. The analysis found that the joints would open at a load below the ultimate failure load and that this would be observable using the installed monitoring system and would give sufficient warning of failure. The region of highest bending effect was the hog region in the central portion of the deck over the cill beam.

In July/August 2009 the bridge was closed as an emergency measure for a period of 6 weeks in response to some alarming readings from the monitoring system.

The monitoring system comprised three elements: acoustic monitoring; strain and displacement monitoring; and visual inspection.

The acoustic monitoring system was made up of 36 sensors fixed to the soffit in a grid of 3 by 12. This enabled remote detection of wire breaks in the longitudinal tendons. The system operated successfully from installation in December 2007 until the deck was demolished, detecting a total of 20 wire breaks (Barker, 2011).

The strain and displacement monitoring system installed in December 2007 used fibre optic technology. The system was calibrated and checked by means of load tests. A review of the output from the January 2009 load test suggested anomalies in the system, particularly in the level of temperature compensation. Confidence in the strain and displacement monitoring system declined, then at the end of June and beginning of July 2009 some alarming readings contributed to a decision to implement an emergency closure. At first it was thought that the bridge would need immediate demolition. However the closure gave an opportunity to uncover two sensors on the surface of the deck, one of which had indicated 2mm movement above the cill beam where flexural cracks were expected to form according to the anticipated failure mode. There was no sign of cracking or displacement and it was concluded that collapse was not imminent. The strain and displacement monitoring system was declared unreliable.

It was decided to replace the strain and displacement monitoring system with a new one based on vibrating wire technology. On the top face of the deck, 12 vibrating wire strain gauges were placed adjacent to the cill beam at the end of each internal box beam, and 24 displacement gauges were placed in pairs at the joints between each internal box beam and the cill beam. On the deck soffit, 12 displacement gauges were placed on the joints between the internal box beams and the cill beam. At each third diaphragm a total of 24 strain gauges were placed in pairs either side of the joints and 24 displacement gauges were placed in pairs across the joints. The system was load tested, calibrated and new trigger levels agreed so that it was possible to reopen the bridge to traffic. There were no concerns with the acoustic monitoring system and this remained in place unchanged. Following reopening both systems remained operational until the deck was demolished.

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The post-tensioned concrete deck was demolished in October 2010 in a planned operation. It became clear from observations during demolition that there was little if any bond between the tendon grout and the concrete. The condition of the tendons was generally observed as good with more than 50% having nothing worse than light surface rust and much of the rest having a heavier rusting with light pitting. However there were examples of localised heavy corrosion with heavy pitting and complete wire breakage. Some tendons had sections with more than 50% section loss, although the average loss of strand appeared to be about 5% which suggests that previous assumptions had been conservative. This suggested scope for a longer term management strategy on similar structures. The lengths of tendons that were lower in the trench tended to be more heavily corroded than the lengths positioned in the upper part of the deck.

Experience of Bowdon View Bridge has shown that a combination of tools including investigation, assessment and monitoring can be used to understand the condition of a bridge and to keep it in service when problems are discovered, until such a time as it can be repaired or replaced.

The most significant learning points from Bowdon View Bridge are:

- A clear understanding of the likely failure mechanism is needed to enable design of a monitoring system.
- The monitoring system needs to be tested at installation and trigger levels for strain and displacement need to be set with care. A low level may result in high numbers of alarms that could mask a genuinely serious situation.
- The use of different types of equipment to measure the same effect is recommended as this enables a check, for example, both strain and displacement gauges.
- Monitoring systems can be affected by temperature and these effects need to be allowed for when interpreting the monitoring data.
- Interpretation of data from sophisticated monitoring systems such as the one on Bowdon View is rarely straightforward and system malfunctions are not unusual. Experience and judgement are needed with knowledge of structural behaviour and monitoring issues, to ensure that appropriate decisions can be taken.

B5.4 M56 Thorley Lane Bridge

This bridge carries the unclassified Thorley Lane over the M56, near Manchester Airport. It has a two span (44m and 40m) continuous post-tensioned deck. Its construction is the same as M56 Bowdon View Bridge, although in this case the box beams are tapered from a depth of 2.44m at the central pier to about 1m at the abutments.

In view of experience at Bowdon View it was decided to reassess Thorley Lane as an external unbonded post-tensioned bridge. This assessment suggested the capacity of the deck was less than the permanent load effects when normal load factors were applied. Only when material factors were removed and nominal loads were applied did the assessment show some live load capacity at the ultimate limit state, but this could not be considered a valid assessment loading.

However, in situ stress testing has shown that there is a significant residual compressive stress in the tensile zones, suggesting there is a low risk of collapse due to loss of prestress. Further, static load tests that were carried out to supplement the assessment suggest that the calculated strains at critical locations were at least three times the measured effects. It was concluded from this that the structure was capable of carrying 7.5T ALL on two lanes. The structure remains open with load mitigation interim measures in place, in the form of restriction to a single lane with a signed weight limit.

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During 1999 refurbishment works exposed about half the tendons. They appeared to be in reasonable condition but with some slight corrosion and section loss. However the most crucial areas were not exposed and although as a result of this refurbishment Thorley Lane was considered in better condition than Bowdon View, concerns remained over deterioration of the tendons due to corrosion. Therefore an acoustic monitoring system was installed in April 2009. A strain monitoring system was also installed in April 2009. This includes remotely monitored vibrating wire strain gauges on the south span and electrical resistance strain transducers on the north span.

The case of M56 Thorley Lane Bridge illustrates how a combination of assessment, width and weight limit, in situ stress testing, load testing, strain monitoring and acoustic monitoring can be used to keep a structure in service.

B5.5 A4 Hammersmith Flyover

Hammersmith Flyover in west London, constructed in 1961, is 623m long with sixteen spans varying between 28m and 43m and is one of the earliest examples of a post-tensioned segmental viaduct in the UK. It has dual two lane carriageways and carries the A4 over local roads and railways.

The deck is a three-cell box beam of varying depth, 1.981m at mid-span and 2.743m at supports and formed of segments 2.591m long which alternate with 0.305m thick precast cantilever units acting as diaphragms to the spine beams and supporting 200mm pre-cast slabs forming the outer deck. In situ concrete joints between these precast units are 76mm thick so that together the units make up a 3.048m module when post-tensioned together. Thirteen modules make up each 42.7m main span. The flyover was built continuously from the western end, each span being post-tensioned individually.

There are four tendons, one either side of (but external to) the inner webs. At mid-span, the tendons run in 254mm grouted ducts in the bottom slab of the spine beam. At 7.62m either side of mid-span, the strands are directed upwards via saddles to run externally along the webs, protected by a grout casing tied into the webs by reinforcement. Each tendon has minimum ultimate strength of 83.9 tonnes and comprises 16 No. 29mm strands of nineteen wires.

The tendons are arranged in overlapping groups, each passing through two spans and anchored at both ends in the top flanges of the beam on the far side of the extreme piers of the pairs of spans affected giving 50% more pre-stress at the piers than at mid-span.

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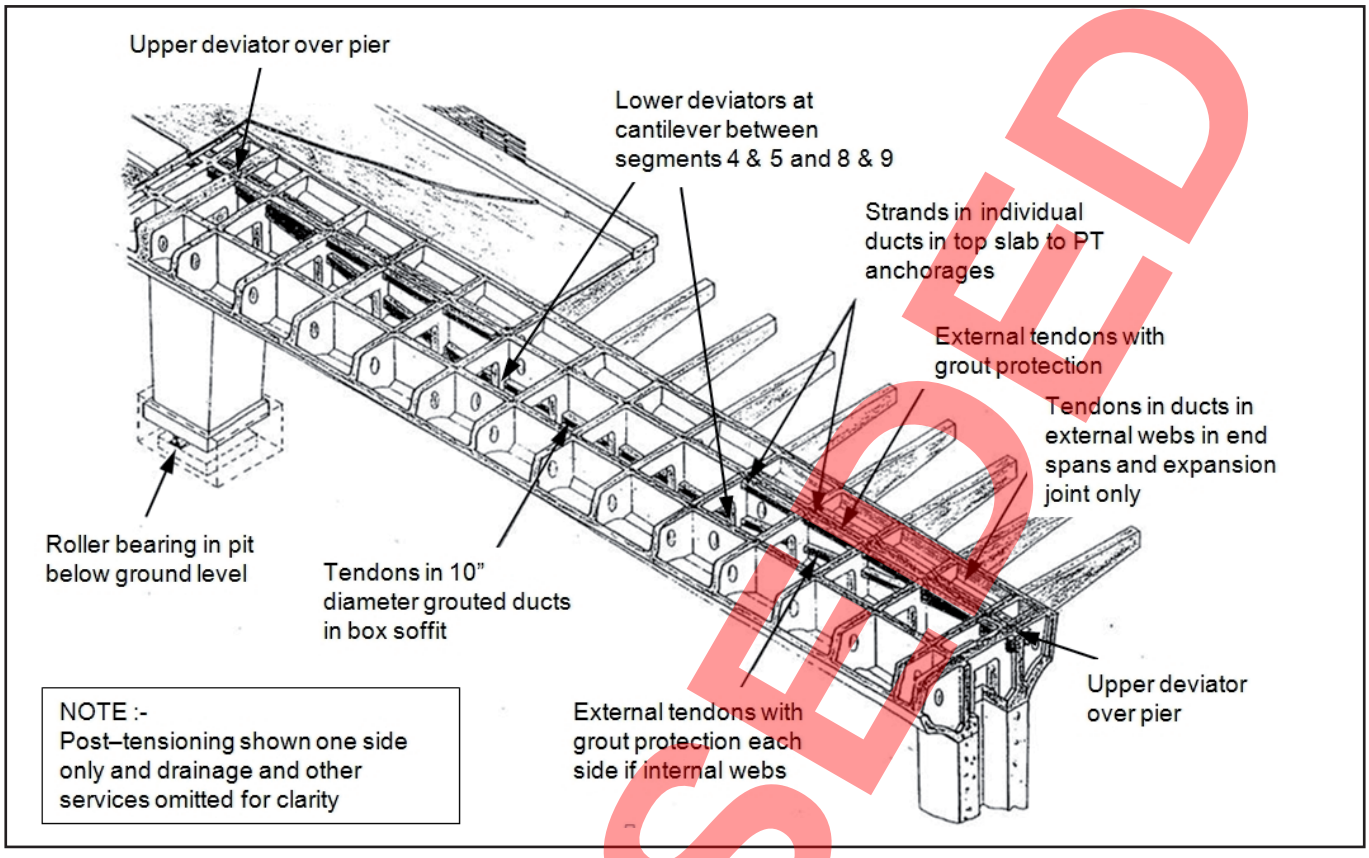

Figure B5.5.1 – Sectional view of post-tensioning layout in a typical span

PTSI (BA 50/93) commenced in the mid 1990's, finding significant defects including corrosion and fracture of wires. A monitoring system using vibrating wire strain gauges was installed with gauges fixed at critical locations to detect changes in stress due to deterioration of the post-tensioning system. However the change in strain resulting from the loss of individual strands was virtually undetectable.

In 1999 a load assessment of the structure concluded that despite significant corrosion, there was at least 10% spare capacity for HA loading. It failed the assessment for HB loading and abnormal loads were excluded.

The flyover was re-waterproofed in 2003 to reduce ingress of chloride rich water.

In 2007 a desk study and review of previous PTSI was undertaken and a technical plan developed for further PTSI.

In 2009, intrusive investigations at locations previously inspected identified the extent of further corrosion. At new locations, investigations assessed the severity and extent of deterioration, carried out corrosion testing and took concrete and grout samples for chemical testing to determine the likelihood of corrosion. Severe deterioration was found principally where the upper strands pass from their individual anchorages through the top flange of the box sections from the 50mm diameter ducts into the cantilever units and at the joint either side of the cantilever unit adjacent to the lower deviator locations. Exposures of the anchors showed them in good condition with little evidence of corrosion but 'black' water was seeping from the strands indicating corrosion due to water travelling between the wires.

In many places, formwork that should have been removed was found. It was used to form pockets at the anchorages to give access to grout the strands from above. Flexible filler board and hessian wadding used as stop ends during grouting was also left in place. These held moisture around the tendons and acted as wicks to draw moisture into the centre of the tendon. These defects were not visible without breaking out the tendon casings. It is certain that

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this poor workmanship contributed to the corrosion of the tendons. Use of absorbent material as stop-ends was also widespread at the lower deviator locations where box-outs left for grouting the mid-span ducts have led to corrosion.

A load assessment completed in 2010 incorporated the actual section losses found by exposing the tendons at critical locations. The results showed that despite the corrosion, the structure passed 40t ALL but the spare capacity given by the utilisation factor had reduced to 0.05 for ULS bending at the piers. Sensitivity checks indicated the results were sensitive to changes in structure condition, especially the post-tensioning tendons. Whilst this gave reassurance that the structure remained fit-for-purpose, visual inspection was limited to specific locations and not sufficient to be confident that the condition of the structure was fully understood.

The strategic importance of the flyover and the lack of capacity within the surrounding network to provide diversion routes meant that closing and re-constructing the flyover was not an option. The condition of the structure was deteriorating and plans had to be made to manage the structure until strengthening or replacement. The biggest unknown was the deterioration rate of the post-tensioning system.

In June 2010 an acoustic monitoring system was installed in the eastern section (9 spans) of the flyover to record wire breaks. The eastern section was visually in poorer condition than the western and it was decided that a deterioration rate derived for the east structure could conservatively be applied to the west.

The pattern of wire breaks broadly matched the observed condition in the PTSI with concentrations of wire breaks in the upper tendons at anchorages and at the lower deviators. In first the 3 years of operation, 719 wire breaks were logged.

To generate a deterioration model an assumption on the number of remaining effective wires was necessary. The assessment model showed the pier head locations as critical and the model was used as the baseline for condition and capacity. The ULS bending capacity at each pier in respect of the original prestress was assumed directly proportional to the number of remaining effective wires and further loss in capacity would be proportional to wire breaks recorded at that pier.

The Projected Date was defined as when the capacity of the structure to support 40t ALL drops to the point that there is zero spare capacity. The output from the acoustic monitoring was used to adjust the numbers of effective wires remaining and the rate of deterioration. It was noted that the breakage rate increased as temperatures increased in the first half of the year and reduced during autumn and winter. The model indicated that if deterioration continued at the same rates, the most critical span would reach capacity by November 2011. Three other spans would also reach capacity around the time of the London Olympic Games in 2012. The flyover formed a vital link in the Olympic Route Network and it was essential it be available during the games.

A two stage plan was initiated. Firstly ‘interim’ measures to provide strengthening at critical pier locations followed by a permanent scheme to strengthen the entire deck. Propping and jacking from below at the lower deviator locations either side of four piers provided sufficient moment relief over the piers to push the Projected Dates sufficiently beyond the Olympics to enable preparation for the permanent works.

The first pair of jacking towers was installed in November 2011 and 50t load jacked into each position. At the same time, because of the unexpectedly high rate of wire breaks the acoustic monitoring system was extended into the west structure (commissioned April 2012).

Concurrently with the Phase 1 strengthening, further PTSI work was undertaken at locations identified by the 2009 inspections as needing more detailed investigation and by the locations of high wire break activity from the acoustic monitoring. Wire breaks were detected while exposures to the grout boxes were made and work was suspended due to safety concerns. A borescope survey was instigated to determine strand condition and, although not conclusive, revealed voids and hessian packing.

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During the course of these inspections all strands in one tendon at a pier head were discovered to be completely severed by corrosion where they emerged from the ducts from the anchorages in each of four adjacent segments. There are 24 strands present at this location and all 8 strands that were anchored in the top slab on the same side of the upper deviator were severely corroded.

Following this discovery on 23rd December 2011 the flyover was closed immediately since if this was repeated in the remaining three tendons anchored at that position, the flyover would not theoretically be able to support its own weight. No restrictions on traffic below the structure were imposed since close examinations showed no signs of distress or joint opening under live loading.

Immediately following closure, further inspections examined the other upper anchorage locations. These showed corrosion was not as extreme and the remaining 16 strands were in better condition. After a re-assessment the flyover was re-opened with traffic restricted to one lane in each direction and a 7.5t weight limit.

As an alternative to intrusive investigations to determine the condition of the strands, radiography was trialled. There was uncertainty in interpretation of images and a definitive verdict was not always possible without intrusive investigation. Radiography required lengthy planning, could only be done when the flyover was closed to traffic, and required many tonnes of protective lead shielding. It was concluded that the pace of the project was too fast for the technique to be deployed extensively.

In situ stress measurements were also made, repeating tests that had been undertaken for the 2009/10 assessment. These gave higher results than the 2009 figures. The increase was attributed to thermal effects and demonstrated the need for caution when monitoring at a point in time without taking into account the behaviour of the structure and the influence of daily and seasonal changes.

It had become apparent from these examinations and re-assessment that propping would not provide the intended medium term solution pending the implementation of a permanent strengthening scheme. An initial strengthening scheme was devised over the 5 most critical piers which was to become part of the permanent strengthening for the full length of the flyover as a second phase. The requirement was to provide sufficient additional post-tensioning to ensure the safety of the flyover until November 2013. Starting in January 2012, the scheme was implemented within 5 months.

The initial strengthening comprises 10 new external post-tensioning tendons over the five critical piers anchored above and below the top flange of the main spine box beam using reinforced concrete anchor blocks. The top slab of the box required strengthening to spread the forces from the anchorages. A 230mm reinforced concrete slab was dowelled into the top flange to which a 1.25m high reinforced concrete vehicle restraint system was attached to protect the tendons.

The second phase will strengthen the remaining sections of the flyover and supplement the initial works as part of a major refurbishment to restore a serviceable working life. The design will remove reliance on the original post-tensioning system by replacing the corroding tendons with a system of external, inspectable and replaceable tendons. Because of space limitations within the deck, some short pier capping and midspan tendons will be located on the external faces of the main spine box. The acoustic monitoring system has been supplemented by a series of displacement gauges across critical joints. They are capable of providing warning of loss of compression and are sufficiently sensitive to detect opening of joints under live load. Work is expected to be complete in summer 2015.

Text and diagram on Hammersmith Flyover by courtesy of Transport for London.

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B6 Externally Post-Tensioned Single Cell Box

B6.1 A3/A31 flyover, Guildford

The A3/A31 flyover at Guildford was constructed during the period 1973-1974 to carry the A31 slip road over the dual carriageway A3 trunk road. The two span single cell box girder bridge was constructed from precast segments, post-tensioned with unbonded, polypropylene coated external tendons. Each tendon comprised 10 No. 19mm diameter 19 wire CCL strands. The main 50m east span was prestressed using 24 No. tendons, 16 of which carried through to the west abutment. Additional bonded tendons were grouted in ducts within the top slab over the pier.

An inspection of the structure, carried out in 1978, revealed cracking of the concrete in the end anchorage blocks, the midspan deflector, the main span anchorage unit and the pier diaphragm. An assessment of the deck design showed that reinforcement stresses and theoretical crack widths were excessive at the positions where cracking had already occurred.

At the west anchorage block there was only a marginal factor of safety for the block to resist the effects of redistribution of the longitudinal prestressing forces into the deck. Remedial measures involved the installation of six 40mm diameter vertical Macalloy prestressing bars with anchorage plates top and bottom of the block. The vertical clamping forces exerted by the additional prestressing increased the existing horizontal tensile stresses. To counter this, two horizontal 40mm diameter Macalloy bars were installed.

The midspan diaphragm deflector unit was inadequate to resist the vertical loads existing at the saddle points, thus creating a tendency for the diaphragm to separate from the bottom slab and webs and lift the top slab. To counteract this effect, additional reinforced concrete clamping blocks were concreted on each side up to two thirds of the height of the diaphragm. The blocks were prestressed vertically by 32mm diameter Macalloy bars to reduce the uplift on the top slab. Transverse 20mm diameter bars were stressed through the webs and blocks and through the diaphragm.

The main span anchorage unit had inadequate reinforcement to prevent further cracking between the anchor block and the bottom slab and webs. Horizontal prestress was applied to the front of the block and tension cracking at the rear was controlled by 32mm diameter vertical Macalloy bars.

In the east anchorage block, the reinforcement stresses due to the distribution of prestressing forces into the deck were excessive. Eight vertical 20mm diameter bars were passed through holes in the webs and the resulting increases in lateral stress were resisted by two 32mm diameter horizontal bars.

The remedial works were completed in 1982 but in 1994 further serious defects were found. Two of the original prestressing strands were found to be severed and individual wire failures were found in other strands. The cause was corrosion, mainly in the anchorage zones. A temporary closure and propping were necessary.

A permanent strengthening scheme was implemented. This involved the complete replacement of the end pre-cast segments that housed the corroded anchorages. All the main prestressing tendons and deflectors were also replaced (Brooman, 1996). Work was completed by April 1997.

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Annex C Advice on Severe Localised Corrosion (Crevice Corrosion)

Background

C1 Corrosion of steel in concrete structures is caused when the passivating effect of the highly alkaline pore solution present in grout and concrete is compromised. This usually occurs by reduction of the alkalinity through carbonation or the presence of aggressive ions such as chlorides at the level of the steel. The corrosion mechanism is electrochemical with metal ions passing into solution at anodic regions, and, usually, the formation of hydroxyl ions at cathodic regions. There is a current flow between the two zones with the pore water acting as an electrolyte. The specific positions of anodic and cathodic regions, and the corrosion rate, are influenced by a number of factors such as differing concentrations of contaminants, differences in oxygen availability and moisture content. For post-tensioning systems in particular, additional factors such as the presence of voids in the grout and the particular configuration of the steel tendons could also be important. Crevices either between the wires themselves or the wires and any spacers used to separate them, and bimetallic effects between the spacer and the wires could lead to an increased chance of severe localised corrosion.

C2 Examples of severe localised corrosion were found after examination of a bridge demolished following externally inflicted damage. In this structure the individual wires forming a tendon were held apart by a metal spacer. This system is no longer used but a significant number of bridges were constructed in this way. The examination found severe localised corrosion and fracture of prestressing wires, and this was concentrated at and around spacers for the wires. High levels of chlorides were present in the concrete (probably present from the date of construction and possibly derived from the use of chloride based accelerators). Levels of chloride in the grout were lower but still significant and were thought to have diffused in from the surrounding concrete. The post tensioning system used did not incorporate ducts and utilised a removable former, which in itself reduced the degree of protection of the post-tensioning wires. It was thought that the crevices between the spacer and the wire were exacerbating corrosion in the high chloride environment. In some cases there were external signs of the underlying corrosion such as cracking of the concrete; however in other cases there were no external signs. This raised the concern that such localised corrosion would not be detected by the usual post tensioning inspection procedures.

C3 Further research was carried out to reproduce this type of corrosion under laboratory conditions and assess the relative rates of corrosion of the spacers themselves, the wires in the vicinity of the spacers, and on the wires in general. Results showed that corrosion only occurred where chlorides were present and the most severe corrosion was on the spacers themselves. However there was also significant corrosion on the wires both at and away from spacer positions. Some examples of localised corrosion were observed in the crevice between the wire and the overlying spacer lug. At the moment there is insufficient information to show whether the rate of wire corrosion is significantly higher in this position than on the wires generally.

C4 In addition to the laboratory work, finite element analyses were carried out to assess the structural effects of wire corrosion. This indicated that external signs of the underlying corrosion –such as cracking of the surrounding concrete – would only become apparent when the corrosion was well advanced.

C5 The following parameters have been identified as contributing to the possibility of the occurrence of localised corrosion:

a) Tendon type

b) Sheathing of duct

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c) Spacers

d) Voids in duct grout

e) Water in duct

f) Chlorides in concrete

g) Chlorides in duct grout

h) Acidity

Additionally three other parameters may also influence the occurrence:

i) Pre-cast construction

j) Condition of the prestressed elements

k) Joints

C6 The following paragraphs give further information on these parameters.

Tendon type

C7 The bridge that was the subject of the research used prestressing tendons comprising individual spaced wires. For this reason, all wire systems have been identified as being at risk of crevice corrosion, although wire spaced systems, incorporating spacers appear to be most at risk.

C8 Strand prestressing systems use wires spirally wound. No evidence has yet been put forward associating strand with crevice corrosion but their use of wire means that they cannot be eliminated on the basis of current knowledge. Hence they are considered to be at lower risk than individual wire systems.

C9 It is understood that there is, technically, no reason why larger steel sections should not, also, suffer from crevice corrosion, although they are considered to be at lower risk than wire and strand systems.

C10 Further information on early post-tensioning systems is available in CIRIA Report 106 Post-tensioning systems for concrete in the UK: 1940-1985.

Sheathing of duct

C11 Tendons with no prefabricated duct (sheath), i.e. where a removable former has been used to form the duct in the concrete, are seen to be most at risk from ingress of chlorides. Use of a sheath, embedded in the concrete to form the duct through which the tendons are threaded (drawn), provides greater resistance to the movement of chloride ions from the surrounding concrete. Sheaths of metal or plastic are commonly used. A metal duct is, of course, itself subject to the risk of corrosion and a plastic duct is, therefore, considered to provide the best protection to the tendon.

C12 If the chloride concentrations in the surrounding concrete are low, or if the concentrations in the grout are already high, the protection provided by a sheath would have no value in preventing the passage of chloride ions.

Spacers

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C13 The research suggests that the gap between the wire and spacer could provide a crevice in which corrosion could initiate. It needs to be reiterated here that a corrosion initiating crevice could be formed in a gap between metal and any other non-metallic material. Therefore, a risk still remains even when non-metallic spacers are used or no spacer at all. Since the metallic spacers are, themselves, subject to corrosion non-metallic spacers are considered to present a slightly lower risk. Unfortunately, the level of information on the prestressing systems available rarely confirms whether spacers are present or not.

Voids in duct grout

C14 The TRL investigation does not identify poor grouting of tendons as a factor contributing to the risk of the initiation of crevice corrosion. However, it is considered that there is a greater risk of chloride contaminated water penetrating and reaching the tendons where voids exist. Therefore the parameter is included, but with a reduced rating of overall significance.

Water in duct

C15 Water is, generally, a requirement for corrosion of steel to occur, but the amount of water required is very small, particularly for crevice corrosion. This was particularly illustrated by the results for the bridge researched where the ducts were found to be well grouted, yet crevice corrosion of the wires within the grout was found. The small amounts of water required to initiate the corrosion reached the tendons through microcracking in the duct grout. Nevertheless, the passage of water through voids in poorly grouted ducts is still considered as having the potential to increase the risk of corrosion.

Chlorides in concrete

C16 The investigations into the mechanism of crevice corrosion have indicated that a high chloride concentration is probably the most important parameter influencing the initiation of crevice corrosion. Chlorides in the structural concrete surrounding the duct (but not in the duct) may not be able to reach the tendons because of the presence of a sheath. Chlorides in the concrete in the ducts (grout) are discussed in C.19.

C17 Chlorides are measured in a number of ways. It is important to ensure that the values are being compared like with like. For concrete, the results are conventionally recorded as the total chloride ions as a percentage of mass of cement.

C18 The chloride concentrations in decks have often been found to be high but to reduce to much lower concentrations near the tendons.

Chlorides in duct grout

C19 Chlorides in the grout within the duct will be able to reach the tendons regardless of the presence of a sheath. While chlorides in the surrounding concrete have a potential influence on the chloride concentrations reaching the tendons the most significant effect will arise from chlorides immediately adjacent to the tendon, particularly in the grout in the duct.

Acidity

C20 Acidity, measured as a low pH, is associated with crevice corrosion. It is noted that the corrosion itself reduces the pH at the location.

C21 It is not yet clear to what extent the acidity of the surroundings is particularly influencing, even though it is fundamental to the localised effect of crevice corrosion.

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C22 The pH below which protection provided to the reinforcement by the alkalinity of the concrete matrix starts to reduce is 11.5. The pH value of pure water is 7, and this can be defined as the start of the acidic range at lower values. Acidity is, generally, only measured where water has been discovered in the duct.

C23 The extent of alkalinity is considered to be a significant factor governing the inception of corrosion as increasing alkalinity is believed to inhibit the reaction. An acid environment can result in the initiation of crevice corrosion even without the presence of chlorides.

Pre-cast construction

C24 There is a greater possibility of chloride based accelerators having been used in the concrete mix for precast concrete which therefore increases the probability of deleterious high chloride contents. There is also a greater probability of the use of joints. Therefore use of precast construction could slightly increase the risk of crevice corrosion in combination with other factors.

Apparent condition

C25 The bridge that was researched was stated to have been in apparently "reasonable condition for its age" except for the "distress associated with damaged prestressing wires". Crevice corrosion was found at positions that showed some exterior evidence of deterioration immediately local to the points of crevice corrosion. The structure as a whole also exhibited some cracking and rust staining at other positions. It cannot, therefore, be discounted that crevice corrosion can be associated with other forms of deterioration in a structure.

Joints

C26 The discontinuities in precast structures are known to introduce a potential vulnerability to the penetration of moisture and its deleterious effects on the structure. However, the crevice corrosion discovered in the structure investigated was at points remote from any joints. The joints could, therefore, only have had an indirect effect in that particular case.

Risk Management

C27 Where structures are considered to be at greatest risk of severe localised corrosion the following actions should be instigated as part of the Risk Review, Risk Assessment and Risk Management process:

a) Review previous PTSI reports

b) If there is no evidence of localised corrosion, assess whether there should be further testing and inspection. In most cases this will be unnecessary, but should be judged on the basis of structural sensitivity to loss of post-tensioning. A programme of monitoring may be necessary to provide evidence of wire breaks or overall structural condition.

c) If there is evidence of localised corrosion, carry out a further programme of testing and inspection, to determine the extent and severity of the deterioration.

d) Following the inspection and testing, review structural assessment results, instigate monitoring and plan remedial actions.

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Inspection and testing

C28 The fact that corrosion is likely to be found at spacer positions, if they have been used, suggests that this would be the position at which any monitoring should be targeted. However this assumes that the location of spacers is known, which may not be the case.

C29 Visual inspection is only likely to find cracking and rust staining when the underlying corrosion is well advanced. Standard NDT techniques such as half-cell potential monitoring should detect corrosion activity in unlined ducts provided electrical connection to the tendons can be made and the tendons are not shielded by reinforcement. However the localised nature of the corrosion could make it difficult to find without a very close interval survey. Radiography might detect spacers and possibly wire fractures but is expensive and requires stringent health and safety precautions. Where there is a serious risk associated with wire fractures, acoustic monitoring to detect breaks could be installed. Monitoring acoustic emission from the corrosion process rather than wire fracture might also be feasible but is, as yet, unproven for this type of application.

C.30 Invasive inspection using the procedures in Chapter 7 would provide the most definitive confirmation of the presence of localised corrosion and should be targeted at high risk areas such as spacer locations provided their location is known. The fact that localised corrosion only occurs where there are significant levels of chloride suggests that monitoring chloride content in the grout would be the most informative simple technique.

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Annex D List of TPB Contributing Organisations

Association of Directors of Environment, Economy, Planning and Transport (ADEPT)

Department for Regional Development – Northern Ireland

Highways Agency

Transport Scotland

Welsh Government

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