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Strengthening, Repair and Monitoring of Post-Tensioned Concrete Bridge Decks

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Strengthening, Repair and Monitoring of Post-Tensioned Concrete Bridge Decks

June 2014

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Summary:

This Advice Note provides guidance on practice and strategies for the strengthening, repair and monitoring of post-tensioned concrete bridges.

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VOLUME 3 ROAD STRUCTURES: INSPECTION AND MAINTENANCE

SECTION 3 REPAIR AND STRENGTHENING

PART 1

BA 43/14

STRENGTHENING, REPAIR AND MONITORING OF POST-TENSIONED CONCRETE BRIDGE DECKS

Contents

Chapter

1. INTRODUCTION

General

1.1 A series of Standards and Advice Notes have been prepared on the Special Inspection of grouted post-tensioned concrete bridges. This Advice Note deals with the strengthening, repair and monitoring aspects of such structures. Other relevant documents in the series include NRA BA 50 Planning Organisation and Methods for Carrying Out Special Inspections, NRA BD 54 Post-Tensioned Concrete Bridges Prioritisation of Special Inspections and the Standards on assessment and design methods in Volume 3 of the NRA DMRB.

1.2 The types of strengthening that may be employed depend primarily upon the particular form of bridge deck, the bridge location and the extent and source of the damage. Strengthening works may be designed to aid both the longitudinal performance and the transverse characteristics of a deck.

1.3 There is limited experience in Ireland of the application of bonded and unbonded forms of prestress for strengthening works. Conventional grouted post-tensioned systems have been added to existing post-tensioned sections and remedial stressing has been used on an externally post-tensioned concrete box structure, to counteract severe cracks in the anchorage zones and deflector diaphragms.

Mutual Recognition

1.4 The strengthening of road structures will normally be carried out under contracts incorporating the NRA Specification for Road Works (NRA MCDRW). In such cases products conforming to equivalent standards and specification of other states of the European Economic Area and tests undertaken in other states of the European Economic Area will be acceptable in accordance with the terms of the 104 and 105 Series of Clauses of that Specification. Any contract not containing these Clauses must contain suitable clauses of mutual recognition having the same effect regarding which advice should be sought

Scope

1.5 This Advice Note gives guidance for strengthening, repair and monitoring of grouted post-tensioned concrete structures. Case studies of the strengthening techniques used in several countries are given in Appendix A.

Implementation

1.6 The Advice Note should be used forthwith on all schemes involving the strengthening, repair and monitoring of post- tensioned road structures on national roads, including motorways. The National Roads Authority must be consulted to obtain technical acceptance for the proposed works before detailed design proceeds.

2. STRENGTHENING STRATEGIES

General

2.1 Strengthening techniques can be divided into two groups; active and passive. Active methods, such as the use of additional prestress and preflexing, 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.

Design Criteria

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

2.3 Information on the design of concrete road bridges and structures with external and unbonded prestressing is contained in NRA BD 58.

Additional Prestress

2.4 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.

2.5 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.

Anchorages and Internal Fixings

2.6 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.

2.7 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 room existing between end diaphragms and ballast walls and major modifications are inevitably required.

2.8 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.

2.9 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

2.10 Deflected tendons should always 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.

2.11 In many structures which have been strengthened, only the mid-span 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.

2.12 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.

2.13 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.

2.14 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.

2.15 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.

Cable Stays

2.16 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 tower 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 very long span bridges that this solution is likely to be viable.

2.17 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

2.18 External steel plate bonding is not used in Ireland. Guidance on the use of externally bonded fibre reinforced polymers can be found in NRA BD 85.

Elastic Supports

2.19 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.

Preflex Method

2.20 The primary use of the preflex method has been for the repair and strengthening of pretensioned members suffering from the effects of damage caused by impact. Indeed, this is seen as the best use of the method, where small sections of prestressed members have to be repaired or replaced and the prestressing cables remain intact.

3. CHOICE OF STRENGTHENING

General

3.1 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.

Beam and Slab Decks

General

3.2 All the strengthening methods considered in Chapter 2 can be used for this type of structure, but it is unlikely that cable stays will provide an economic solution.

External Tendons

3.3 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 significant increase to the service load capacity.

3.4 Strengthening schemes based upon 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 of 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 roads 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.

Additional Elastic Supports

3.5 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

3.6 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. External steel plate bonding is not used in Ireland. Guidance on the use of externally bonded fibre reinforced polymers can be found in NRA BD 85.

Additional Beams

3.7 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

3.8 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 i.e. work was undertaken for serviceability rather than strength reasons.

3.9 Cable stay techniques could be appropriate for box girder decks, however, their sheer 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

3.10 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 but, provided the cracks were not too numerous, this should not be unmanageable.

In-Situ Voided Slabs

3.11 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.

3.12 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 road. This would limit the strengthening options elastic supports or plate bonding but the considerations are essentially the same as for beam and slab bridges.

4. REPAIRS TO ACCIDENT DAMAGE

General

4.1 This Chapter deals with accidental damage resulting from vehicle impact or terrorist blasts. In most cases, 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.

Method

4.2 The "preflex method" is most useful when repairing damaged elements. 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 "propping" should be applied at appropriate location to induce desired load effects.

4.3 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 induce 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 suitable protective surface coating (the National Roads Authority's requirement for surface impregnation to concrete are contained in the NRA Specification for Road Works, clause 1709) and possibly an additional crack bridging surface coating should be considered to enhance durability of the repaired structure.

Materials

4.4 Requirements for repair materials and hydrophobic impregnation are contained in NRA BD 27,, NRA BD 43 and NRA MCDRW 5500.

5. MONITORING

General

5.1 The type and extent of defects in a structure may dictate an increased frequency of inspections and monitoring of critical sections. Where monitoring is considered, the frequency of readings and the measurement of environmental conditions should be carefully planned. Where joints in segmental structures occur at critical locations or there are known defects in the vicinity, regular monitoring will be required.

5.2 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 considered unsafe.

Acoustic Emission Methods

5.3 In this method the structure is monitored for internally generated pulses. A sudden release of elastic strain energy due to brittle fracture process, friction and fluid turbulence generate stress waves which propagate throughout the structure. These wave packets of vibrational energy are usually referred to as Acoustic Emission (AE), and they can be measured and analysed using acoustic emission test methods. The band width of the stress waves depends on the velocity and magnitude of the associated fracture events.

a) In the case of major fracture events associated with catastrophic failure, the frequency range spans several orders of magnitude, down to a few 10's Hz.

b) In the case of intermediate size fracture events, which usually precede failure, the predominant frequency range is around 1 KHz to 100 KHz.

c) In the case of the micro-fracture events associated with stable crack growth in metals, the frequency range is around 100 KHz to 10 MHz.

5.4 As the stress waves propagate through the structure the lower frequencies are less attenuated making the detection of large events possible over long distances and small events possible over much shorter distances. Background noise also limits the range of detection.

5.5 AE testing has been used in France to monitor a number of post-tensioned bridges by detecting the acoustic energy released as wires fracture. Experience in UK indicates that it would be difficult to detect single wire fractures and even where a measurable signal is produced it would be difficult to distinguish this from other sources of noise.

Methods for Strain Measurement

5.6 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 leakages 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.

6. REFERENCES

6.1 National Roads Authority Design Manual for Roads and Bridges (NRA DMRB).

NRA BD 2 Technical Approval of Structures on Motorways and Other National Roads

NRA BD 21 The Assessment of Road Bridges and Structures

NRA BD 54 Post-Tensioned Concrete Bridges Prioritisation of Special Inspections

NRA BA 50 Post-tensioned Concrete Bridges. Planning, Organisation and Methods for Carrying Out Special Inspections

NRA BD 43 The Impregnation of Reinforced and Prestressed Concrete Highway Structures using Hydrophobic Pore-Lining Impregnants (DMRB 2.4)

NRA BD 58 Design of Concrete Highway Bridges and Structures with External and Unbonded Prestressing

6.2 NRA Manual of Contract Documents for Road Works (MCDRW)

Volume 1: NRA Specification for Road Works

6.3 Case Studies

Vikram Pakrashi, J. K. A. F., Julie Harkin, & Nanukuttan, S. (2013). Monitoring and Repair of an Impact Damaged Prestressed Bridge. Proceedings of the ICE - Bridge Engineering, 166 (1), 16–29.

7. ENQUIRIES

7.1 All technical enquiries or comments on this document or any of the documents listed as forming part of the NRA DMRB should be sent by e-mail to infoDMRB@nra.ie, addressed to the following:

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…………………………………………
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APPENDIX A CASE STUDIES

A1. Post-tensioned I-Beams

A1.1 M3 Motorway Bridges, UK

During the construction of ten 3-span overbridges on the M3 motorway in the UK, 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 on work on 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.

A1.2 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.

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.

A2. Post-tensioned Double-T Beams

A2.1 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.

A3. Post-Tensioned Single Cell Boxes

A3.1 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. 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/mm^2 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.

A3.2 Viaduc du Maghan

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 pour was relatively small, 4 to 12m^3, it was pumped for convenience of access.

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.

A3.3 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 each 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.

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/mm^2 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/cm^2 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.

A3.4 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 tostressing 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.

A4. Post-Tensioned Twin-Cell Boxes

A4.1 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. Re- stressing 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.

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 (Denso-jet) and wrapped with a petroleum bandage (Densoflex). A second polyethylene protective wrapping was also applied (Densolenband). Where the cables passed through the diaphragms, joints were provided on each side to allow pumping of grout into the duct.

A4.2 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.

A4.3 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.

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.

A5. Post-Tensioned Multi-Cell Boxes

A5.1 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 security 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.

A6. Externally Post-Tensioned Single-Cell Box

A6.1 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 it was reported in July 1994 that the bridge had been closed pending further work. This was because a number of the original post-stressing strands had corroded and broken. There was no evidence that there had been any deterioration of the works carried out in 1982.

A7. Prestressed Concrete Beams
A7.1 Brownsbarn Bridge, Republic of Ireland.

An emergency rehabilitation was required to be carried out on Brownsbarn Bridge situated over the national road N7 near Dublin, following impact damage to its soffit from a low-loader carrying an excavator passing underneath the bridge. The rehabilitation posed a number of theoretical and practical engineering challenges in terms of uncertainty in the stresses, lack of time, liaison among a number of groups working on disparate aspects of the project, the practical execution of the rehabilitation strategy and the health and safety aspects of a number of high risk activities.

The structure is a two-span continuous slab-girder bridge comprising six precast, prestressed U8 simply supported concrete beams connected by a continuity diaphragm. The reinforced concrete piers are integral to the deck and the ends of the bridge are simply supported. The abutments are made of reinforced concrete. The continuity diaphragm is connected to the U8 beams through steel plates of dimension 300x30x1700 (mm). These U8 beams date from one of the earliest of their kind of design in the Republic of Ireland.

The impact damage affected two of the prestressed U8 beams. The edge of the outer beam was damaged in a benign fashion although one of the tendons in the lower row snapped. This was not a major issue because a rapid assessment calculation proved that the beam was well within the safe zones under stability and serviceability conditions with the exclusion of the tendon. An internal beam was more significantly damaged in which the tendons remained intact but the concrete was crushed from the impact.

Estimates of the levels of stresses present in the structure in an undamaged state were made possible through the availability of production drawings. However, uncertainties existed in estimating the stresses in a damaged condition as it is very difficult to model the redistribution of stresses following an unknown impact. The existence of a credible benchmark was absent, unfeasible or error prone and local damage often manifests very little global change in measurable parameters. The local fracture of concrete in a beam leading to redistribution of stresses at and around the affected regions, including neighbouring beams, is extremely difficult if not impossible to deal with. The monitoring of repair is thus often dependent on the detection of sudden, unusual or unacceptable levels of change in stress at critical locations of the bridge from an unknown baseline of stress.

A visual survey provided information regarding the extent of damage. This was followed by a three-dimensional laser scan visualisation that was made available for the bridge. A hammer tapping survey at and near the location of the main damage indicated that the true damage extended beyond the visually superficial regions. This fact was reinforced by carrying out an impact echo survey. The tendons were unaffected by this incident. Structural cracking in the prestressed concrete beams was absent following the damage in an unloaded state or due to the passage of vehicles, qualitatively supporting the fact that the concrete was probably within a linear and compressive zone. Repair had

Instrumentation was carried out on the bridge in the form of the installation of 19 strain gauges at five preselected monitoring points. The monitoring points were strategically chosen so that the interaction of the damaged and the undamaged beams, including the behaviour of gauges at, near and away from the damage, can be probed. Three monitoring points at the centre and at the two ends of the damage were chosen. The centre of the two undamaged beams and the two sides of the damaged beams were chosen as the two other monitoring points. Gauges were installed at the top and at the bottom of the soffit so that the deformations at these two levels could be observed simultaneously.

A preload was applied to either side of the damaged region thereby releasing some of the high prestressing compressive force in the soffit of the beam. The damaged concrete around the area of impact was removed after the application of preload. The preload was removed following the application and sufficient hardening of the repair material. Consequently, the prestrain due to the preloading was released and the hardened repair material was expected to experience some compression. The attempt was to restore some of the lost prestress in the concrete.

Preloading consisted of placing 20 t bales of concrete blocks either side of the damaged region. These were staged in three applications to a total of 120 t. The damaged concrete was then removed from the beam by hydrodemolition. This method of concrete removal was chosen for this project due to its precision and low impact on the existing strands. The repair material chosen was a fibre reinforced spray mortar. It was designed to have a 28 day compressive strength of 70 MPa and was able to take greater tensile force than standard concrete. The load was removed from the top of the bridge after the repair material had gained adequate strength and some amount of the prestress was reinstated following the removal. The strain gauges remained for a further 4 days to allow any further strength gain to be examined.

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