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Annex L
Strengthening of concrete structures using external post-tensioning

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Annex L
Strengthening of concrete structures using external post-tensioning

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Summary: These design guidelines deal with the strengthening of concrete structures using external post-tensioning and have been developed on the basis of current knowledge and practice. The technique has been found to provide an efficient and economic solution for a wide range of structures types and conditions. In addition to increasing strength, external post-tensioning can be used to improve the serviceability behaviour of existing structures and to delay or prevent the onset of cracking.

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1 DESCRIPTION

1.1 Introduction

The design guidelines presented here deal with the strengthening of concrete structures by external post-tensioning and are based on current practice and experience. The method has been used in many countries and has been found to provide an efficient and economic solution for a wide range of structures types and conditions. It has been applied mainly to bridges and the technique is growing in popularity because of the speed of construction and the minimal disruption to traffic flow.

In addition to increasing strength, external post-tensioning can be used to improve the serviceability behaviour of existing structures and to delay or prevent the onset of cracking. It can also be used to reduce or close pre-existing cracks. The increased stiffness provided by external post-tensioning can reduce in-service deflections and vibrations. In addition, the stress range can be reduced and the fatigue performance thus improved. The presence of a deformation or sag can be reduced or removed by using an appropriate tendon lay-out.

The method is generally applicable to a wide range of structure types although its use has been primarily aimed at strengthening bridges and bridge supports. There are many examples of its application around the world including steel truss bridges, steel girder bridges, steel box girders, prestressed and reinforced concrete bridges. Many case studies have been reported in the UK and elsewhere and some examples are provided in Appendix A. The use of external post-tensioning as a means of strengthening is particularly useful for short span bridges where under-strength is the result of changes in the specified design highway loading. External post-tensioning is a convenient technique for raising capacity by up to 30%.

For application of the technique, these guidelines should be used in conjunction with a recognised code of practice for the design of concrete structures and structures with external and unbonded prestressing, eg, EC 2, BS 5400: Part 4 (British Standards Institution 1990), BS 8110 (British Standards institution 1985), BD 58 (Highways Agency 1994b).

1.2 Scope

The technique involves applying an axial load combined with a hogging bending moment to increase the flexural capacity of a beam and improve the cracking performance. It can also have a beneficial effect on shear capacity.

Recommendations are given on the types of structure that can be strengthened using external post-tensioned tendons and the factors that need to be taken into account in the design, installation and maintenance of external tendons.

1.3 Definition

The term external post-tensioning is applied to that class of prestressed structure where prestressing bars or tendons are placed outside the member cross-section and the prestress force is transferred to the member through end-anchorages and deviators. Where the technique is applied retrospectively to strengthen an existing structure, it is recommended that the installation system be devised to ensure inspectability of the post-tensioning components. Consideration should also be given to future re-stressing and tendon replacement.

1.4 Case studies

A number of case studies of strengthening using external post-tensioning are included in Appendix A. These are useful in providing general ideas on the use of the technique and to identify criteria that might be used in the selection of systems and the detailing of anchorages and deviators appropriate to particular situations. They are not intended to provide specific design details for general use.

1.5 Preliminary investigation

Before external tendons are applied to a structure a preliminary investigation should be undertaken to ensure that it is suitable for application of the technique due to the additional forces that are being introduced into the structure. This should include a comprehensive justification for use of this method of strengthening. The investigation should include:

- diagnosis of the problem and clear identification of the purpose of the strengthening works;
- identification of the source of defects, eg, design faults, under-design, deterioration, loss of conventional prestress;
- detailed analysis to determine the existing stress distribution and the level of strength increase required;
- detailed analysis to determine the effects of imposing a new stress distribution on the structure, including local effects at anchorages and deviators;
- for bridges, detailed inspection and assessment of the sub-structure to ensure that the bearings, abutments, piers, etc, can take modified load distribution;
- detailed condition survey of structure: this should include the determination of existing material properties, location of reinforcement, etc, if not reliably known;
- future durability and maintenance requirements;
- methodology for installation and stressing of the post-tensioning system, including sequence and control of load application;
- access requirements for installation of anchorages and deviators, and stressing of bars or cables.

The additional stress imposed in the structure will induce a certain amount of movement, both in the longitudinal and vertical directions. Depending on the sequence of stressing, distortion of the structure might also occur. It is important that these movements do not damage the existing structure. when applied to bridge decks, bearings and expansion joints should be examined to determine whether the expected movements can be accommodated. Seized bearings and faulty joints should be replaced as part of the strengthening works.

An economic evaluation should be carried out to compare this technique with other methods of rehabilitation or strengthening, including complete replacement of the structure. Account should be taken of user disruption costs and loss of income, future maintenance costs, etc.

1.6 Structure types

1.6.1 Beams

External post-tensioning as means of strengthening is applicable to a wide range of structure types although it has been mostly used to increase the flexural capacity of beams.

External post-tensioning can be used to improve the performance of any kind of beam, be it timber, reinforced concrete, prestressed concrete, steel or composite. The tendons can be straight or draped using deviators depending on the particular requirements. Various profiles can be adopted to suit the required combination of axial load and bending as shown in Figure 1 (Xanthakos 1996). Cable combinations can also be adopted. Additional compressive members can be introduced if the extra compression stresses imposed on the beam exceed the maximum allowable design stresses.

The technique can also be used to increase the shear strength although vertical prestress is more appropriate for correcting shear deficiencies (see Section 1.8).

Tendon lay-out for box beams can be straight or profiled as shown in Figure 2. Each lay-out has advantages and disadvantages in terms of structural efficiency and ease of installation. There are no general rules: the preferred alignment should be determined on the basis of design criteria and economics of installation.

1.6.2 Slabs

There have been no reported cases of concrete slabs being strengthened using external post-tensioning. The main problem is the installation of tendon anchorages on the soffit of a slab which might prove difficult due to the low depth of construction. The technique could be used to replace prestress loss in prestressed floor- or deck-slabs where, for example, corrosion has taken place.

1.6.3 Trusses

The technique has been used to strengthen steel truss bridges since the 1950s and various configurations have been employed. Individual members can be strengthened by applying concentric prestress on highly-stressed tensile members. Groups of members can also be post-tensioned in the same way. For example, the tension chord can be stressed with a single long concentric cable. A truss can be strengthened by applying a draped tendon to the truss as a unit. The cable is fixed to the top of the truss at the supports and slopes down to the bottom of the truss at mid-span. The tendon can be placed below the bottom chord using a King-post to provide the required eccentricity at mid-span.

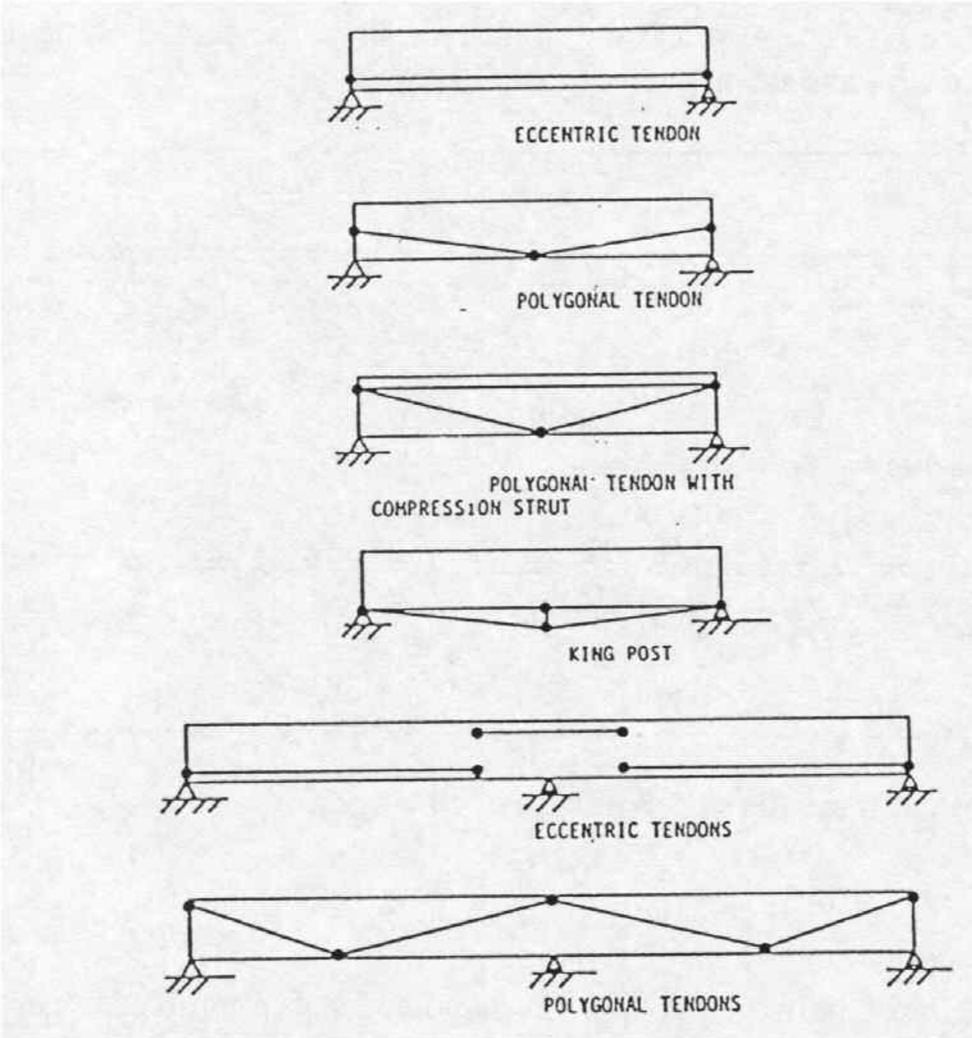

Figure 1 Different tendon configurations (from Xanthakos 1996)

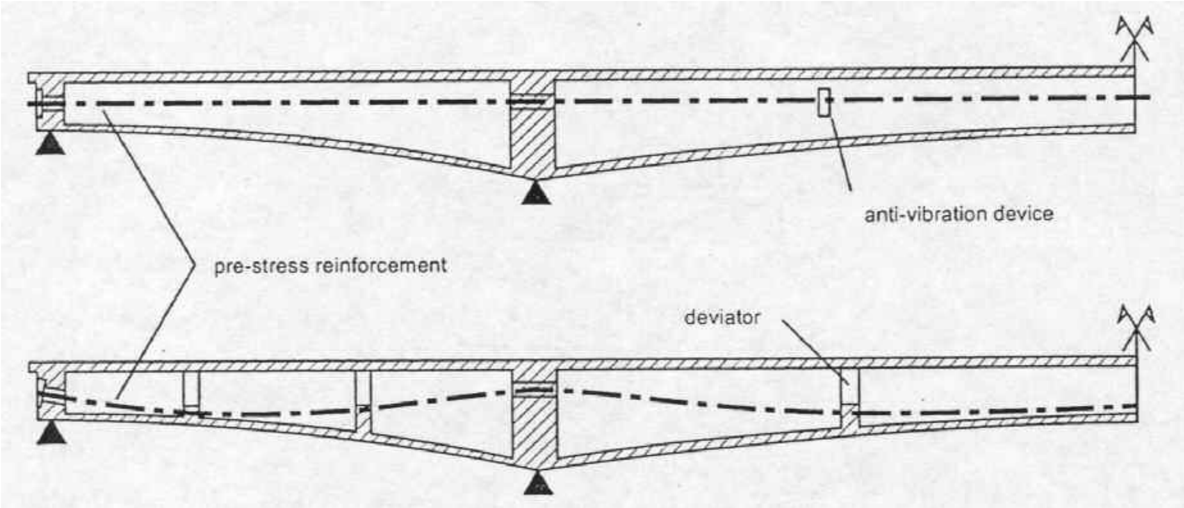

Figure 2 Tendon profiles for box girder.

1.7 Transverse post-tensioning

External post-tensioning can be used in the transverse direction to increase the transverse stiffness in the same way as conventional post-tensioning, to improve the load distribution across a deck or floor. The method can be convenient for replacing conventional transverse prestress lost due to creep or corrosion. The same post-tensioning techniques are applicable.

The technique can be used to repair arch bridges which have developed cracks in the spandrel walls due to lateral pressure through the fill material.

1.8 Vertical post-tensioning

Longitudinal or draped tendons can be used to improve the shear capacity of beams. Where flexure is not a problem but a significant improvement in shear strength is required, the application of vertical prestress might provide an effective solution. Vertical prestressing tendons or bars can be applied to the web of an I-beam. For concrete box sections, they can be installed either inside or outside the box. Because of the short length required, the post-tensioning force can most easily be provided using high strength bars with the required prestress produced using fine threads. As prestress losses tend to be high, provision should be made for re-stressing without the need for too much disruption. Where stress is applied to one side of a web only, account should be taken of induced bending stresses and the possibility of cracking.

Vertical prestress can also be used to provide tie down at supports. For example, in earthquake regions where possible lifting forces might occur, vertical prestress can be retro-fitted to existing bridges and buildings to improve support details. It can also be used to improve defective joints.

2 DESIGN CRITERIA

The design and detailing requirements of strengthening schemes using externally unbonded tendons should follow the appropriate national or European standard for external post-tensioning. For example, BD 58 and BA 58 (Highways Agency 1994a, 1994b) gives for the requirements of externally post-tensioned concrete bridges in the UK. This determines the allowable value for the tension stresses induced in the concrete. For strengthening schemes, the class of structure used should be as in the original structure unless agreed by the Technical Approval Authority. In some cases it may be appropriate to change this. For example, if a conventional prestressing system is being completely replaced by external post-tensioning, it may be appropriate to change the class to comply with the requirements of BD 58.

2.1 Limit states

The design of strengthened structures should be considered for the serviceability and ultimate limit states in accordance with the relevant design code.

2.2 Partial safety factors

The values for partial safety factors should be as specified in the appropriate part of the relevant design code (EC 2, BS 5400, etc). Where comprehensive testing has been carried out to determine in-situ material strengths, the nominal design characteristic values can be modified on the basis of statistical analysis of the test data, subject to the agreement of the Technical Approval Authority.

2.3 Flexure

Flexural capacity should be determined on the basis of strain compatibility using the assumptions prescribed in the appropriate design code. Simplified equations developed for determining the flexural capacity of section with bonded reinforcement and prestressing cannot be used, as the strain (and hence the force) in unbonded tendons does not increase in proportion to the assumed strain distribution. For unbonded tendons, the increase in strain at ultimate is less than for bonded tendons and is usually not sufficient to reach yield. Failure is likely to be through crushing of the concrete. However, the overall behaviour of the structure should remain ductile with extensive cracking and excessive deflections being apparent before yield.

Accurate evaluation of the flexural capacity of beams with unbonded tendons is difficult because the load in the tendons is a function of the overall behaviour of the beam, rather than just depending on the strain distribution at a particular section. Secondary effects, such as changes in the cable eccentricity as the beam deflects and increase in prestress force with deflection, are difficult to quantify. Loss of eccentricity can be reduced by careful choice of deviator position.

As a conservative approach, the strain in unbonded tendons can be assumed not to increase above the initial value due to prestress after all losses and including a partial factor of 0.87. In particular cases, some increase can be allow3ed for:

> (1) In slabs and beams, the strain in the mid-span region of cables which are within 0.1*d* of the soffit at mid-span and which do not extend beyond the supports may be taken to increase by 0.0005, with no additional calculation.
>
> (2) The strain in the tendons at failure may be calculated from a non-linear analysis of the structure. If this is done, checks shall be made to ensure that conventional “conservative” assumptions, such as ignoring the tensile strength of concrete, do not have the effect of increasing the tendon strain and hence the ultimate strength.

Tendons and reinforcing bars that are anchored within a distance of *d/2* of the section being considered shall be ignored. However, within *d/2* of a simply-supported end, all prestress that is anchored beyond the centre-line of the support can be considered effective.

2.4 Shear

Shear capacity should be determined on the basis of the appropriate design code. However, many of the codified equations were developed empirically from tests on beams with fully bonded tendons and may not be applicable to unbonded tendons. The shear rules in EC 2 do not differentiate between bonded and unbonded tendons and can therefore be used. Comparisons with test results have shown that they produce conservative results.

Other less conservative methods can be used subject to the approval of the Technical Approval Authority.

Optionally, sections with unbonded tendons can be checked for shear by considering them as concrete columns subjected to an axial load. The external load should be taken as the tendon force after all losses multiplied by a partial factor of 0.87.

2.5 Lateral bending

Where possible post-tensioning bars or cables should be placed symmetrically about the cross-section to avoid lateral bending moments. The installation sequence of the post-tensioning system should be devised to minimise the lateral bending moments that can be induced.

2.6 Vibration

Fatigue failure of prestressing cables due to vibration should be considered. Checks should be carried out to ensure that resonance does not occur. For bridge applications, excessive vibration of external post-tensioning tendons can occur if the fundamental natural frequency of the tendons is in the range 0.8 to 1.2 times that of the bridge. Vibration control can be incorporated either by limiting the length of free cable or by utilising anti-vibration devices.

2.7 Prestress losses

Loss of stress due to creep, relaxation and friction should be taken into account. Appropriate values of friction coefficient are given in the appropriate national or European design codes. Losses due to creep should be calculated from creep movement between anchors and other fixed points in the tendons.

In the absence of more exact data, friction losses may be calculated using the values of friction coefficient in Table 1.

Table 1: Friction values used to determine prestress losses for unbonded tendon.

Coefficient of friction	Steel tube	HDPE tube
Lubricated strand	0.18	0.12
Lubricated wire	0.20	0.14
Non-lubricated strand	0.25	0.15
Non-lubricated wire	0.27	0.17

2.8 Design of anchorages and deviators

The strength of structures with unbonded tendons is dependent on the anchorage and deviators where present. This is particularly significant as the failure mode of these elements may be brittle. Design of end blocks, anchorages and deviators should be based on unfactored tendon strength. Where serviceability checks are required, the design service load in the tendons shall be taken as the tendon load before long term losses. For deviators restrained in reinforced concrete, consideration shall be taken of the tensile and splitting forces generated.

If tendons are not adequately restrained, the deformations of the concrete between deviators can have a significant effect on the moment applied by the tendon to the concrete section. In addition, inadequately restrained tendons may vibrate excessively and be susceptible to fatigue failure.

To avoid these problems external tendon should be restrained transversely relatively to the concrete section at centres not exceeding 12 times the minimum depth of the beam between fixing points. For bridge applications, if the spacing between points where the tendons are held in position laterally exceeds 12m checks should be made to ensure that the first natural frequency of the tendons vibrating between fixing points is not in the range 0.8 to 1.2 times that of the bridge.

In the absence of test results or other investigations justifying smaller values, the radius of curvature of tendons in the deviators should not exceed the values in Table 3.

Table 3: Minimum radius of curvature.

Tendon (strand number-size)	Minimum radius (m)
19-13mm and 12-15mm	2.5
31-13mm and 19-15mm	3.0
53-13mm and 37-15mm	5.0

For singe tendons, the deflector in contact with the tendon shall produce a radius of not less than 5 times the tendon diameter for wire, or 10 times the diameter for strand. The total angle of deviation should not exceed 15^o.

3 EXECUTION

3.1 General

Good quality workmanship is essential to the construction of an effective and durable strengthening system. Construction should be by suitably qualified and experienced staff. It is recommended that external post-tensioning strengthening systems are installed by accredited contractors. An example of appropriate accreditation is the CARES certification scheme in operation in the UK. This is outlined in the Concrete Society's Technical Report No 47 (1996) describing the quality system requirements for the supply and installation of all post-tensioning systems.

Installation should be based on clearly marked reference points located using the exact dimensions of the structure. The design of the post-tensioning system may need to be adapted to meet actual on-site conditions. If conditions are different from those assumed in the design and changes are required for effective or efficient installation, they should be agreed with the design engineer prior to installation. Acceptable tolerances should be specified in the contract documents. Correct alignment of cables, anchorages and deviators within the defined tolerances is essential for proper performance of the strengthening system and to prevent damage to ducts, tendons, deviators and anchorage assemblies. To ensure proper installation, the design engineer should be present during installation, as minor variations in geometry may be required to overcome problems during installation.

The surfaces on which anchorage and deviator systems are to be mounted must be in good condition. Actual locations should be approved by the design engineer. Any required surface preparation should be contained in the contract documents.

3.2 Tendon lay-out

The number and position of tendons is determined from the design criteria used for the structure. Careful consideration should be given to the stress distributions at critical sections. The two main criteria are that the section will not crush or buckle under the applied prestress force and, for concrete, the section will not crack (or crack widths will be limited depending on the class of the structure) due to the applied moment. Account should be taken of possible loss of eccentricity when positioning tendon deviators.

Tendons can either have straight or draped profiles, depending on the design requirements. A draped tendon is made of straight segments stretched between specially fabricated deviators, usually made of steel or concrete. Shorter straight lengths of cable placed at different positions and eccentricities can also be used.

Tolerances for the alignment of bars or cables should be specified in the contract documents. Any variation beyond these tolerances must be approved by the deign engineer.

Post-tensioning is normally applied through bars or cables stressed using conventional hydraulic equipment. Prestress can be applied using more unconventional techniques provided due consideration is given to Health and Safety regulations. For example, stress in a tendon can be developed by anchoring a straight tendon in place and imposing a deflection at mid-span. The deflection is then retained by fixing the deflected point. Prestress can also be developed by applying an upward load to the structure to reduce overall deflection prior to anchoring the tendons or bars.

For reinforced or prestressed concrete beams, tendons can be installed either beneath the bottom flange or on opposite sides of the webs, depending on the design requirements and accessibility. In box girders, the tendons are usually placed within the box to keep them out of sight and to provide some degree of protection against corrosion.

3.3 Anchorages

Tendons can be anchored at different parts of the structure, eg, in end blocks, diaphragms, on webs or on flanges. Whenever possible, the prestress should be applied to existing structural elements. The anchor block and connection should be designed for the characteristic strength of the tendons. For strengthening systems, this can be reduced to the design force at ultimate plus 20%, subject to the approval of the Technical Approval Authority.

Careful detailing is required to minimise stress concentrations in the existing structure. Normally, drilling through or welding existing structural components to steel webs or concrete webs or flanges is required. This can be problematic if the structure is already under-strength. However, deviators and anchorages can normally be installed in less critical areas. Local stiffeners may be required at anchorages and deviators.

In the design of the new anchorage block, stress concentrations in the supporting system, whether it is of steel or reinforced concrete, must be taken into account and this should include consideration of eccentric loading resulting from the stressing sequence. Detailing of the reinforcement in concrete sections should be such that cracking is avoided or limited in width.

The anchors should, as far as possible, be positioned symmetrically about the vertical centre-line of the beam cross-section to avoid inducing permanent out-of-balance forces to the structure. The sequence of prestressing should be carefully chosen to minimise the out-of balance force during stressing. Out-of-balance forces can be avoided altogether by using two or more jacks simultaneously.

When a number of anchors are required on a web or flange, it is recommended that they are staggered to reduce stress concentrations. Sufficient space should be provided behind the anchors to accommodate the stressing jacks and the pushing and cutting machines. Where such space cannot be provided, an intermediate stressing anchorage can be used. However, this would lead to extra cost due to the requirement for additional anchorages.

Anchors should, as far as possible, be placed in regions where existing stresses are low to avoid over-stressing. Where tensile stresses in deck elements adjacent to anchor blocks exceed the maximum allowable, they can be strengthened by local prestressing. Bursting forces within concrete anchor blocks can be controlled by placing sufficient reinforcement in the block and appropriate detailing.

Three types of anchoring techniques are normally used. These are end anchor blocks, bonded anchor blocks or diaphragm anchoring.

3.3.1 End anchor blocks

End anchor blocks are the normal method of applying external post-tensioning in new construction. For existing structures, a reinforced or prestressed concrete block or cross-beam is constructed at the end of the beam as shown in Figure 3. The block provides anchorage for the additional cable system and a path to transfer the force into the existing member. It should be designed to distribute the prestress as uniformly as possible and should ensure that no significant concentration of force occurs. If suitable end blocks or transverse diaphragms already exist, it may be sufficient to use these as they are or to thicken them with additional concrete.

End anchor blocks have the advantage that existing webs and flanges are not over-stressed and eliminates the need for complicated connection details. They can also avoid excessively horizontal bending both in the existing structure, and in the new anchor block. However, as access to the ends of the beams is required it may be necessary to modify the support system. Their construction may necessitate closure of the structure for a relatively long period which may not be always acceptable. If existing end blocks are long enough, tendons can be embedded within holes drilled inside the blocks which are subsequently filled with grout although this makes it more difficult to replace of the tendons. Where the member is skewed, careful detailing is required to take account of the angle of the cables relative to the end anchor block. This method is not appropriate for heavily skewed bridge decks.

3.3.2 Bonded anchor blocks

Bonded anchor blocks are fixed to the webs or flanges and can be made from steel or concrete. Installation is either by casting in place, or fixed using bolts or transverse prestressing. The most favourable bonded anchor block is a cast in-situ block, as shown in Figure 4.

For concrete structures, short high strength alloy threaded bars can be used to provide transverse prestressing to fix the anchor block. The magnitude of the transverse prestressing force depends on the nature of the interface and the friction between the surfaces of the existing structure and the new anchor block. Approximate coefficients of friction are given in Table 4. An appropriate value of friction coefficient should be chosen depending on the quality of the surface and surface preparation carried out. It is recommended that the bars are re-tensioned after a few days to counteract the anchorage slip and local creep which can reduce clamping forces by 20% to 30%. For this reason, clamping bars should not be grouted immediately after installation. Inserting the bars inside a greased sheath provides corrosion protection and de-bonding which would allow future re-stressing if required.

Table 4: Value of coefficient of friction.

Surface	Coefficient of friction, μ
steel on steel	0.15 to 0.35
steel on concrete	0.3 to 0.7
concrete on concrete	0.6 to 0.7

It may be possible to fix new anchor blocks to the existing structure without the need for transverse prestressing. Blocks may be attached using reinforcing bars grouted into the existing structure. Where only a small post-tensioning force is required, the anchorage block can be fixed to the existing beams using high strength bolts. The bolts can be self-anchoring or can be fixed into resin filled holes. Anchorage blocks can be fitted to both sides of the web of a beam using short lengths of threaded bars. This eliminates the possibility of the anchorages pulling away from the beams.

The actual method of attachment depends primarily on the magnitude of the prestress force and the safe transfer of this force to the existing structure. The anchorage imposes stresses on the structure for which it was not originally designed and care in devising the installation method is required.

When drilling existing concrete for installing grouted bars or bolts, it is important to avoid damaging the structure. A careful site survey is required to identify the actual position of reinforcement and prestressing tendons. Reliable non-destructive testing techniques should be used, such as a cover-meter survey or radiography. This may require minor modifications to the positioning of the anchor block which should be taken into account in the final detailing.

Tension induced in the existing structure behind the anchor block should be taken into account. Local flexing should also be considered: this can be avoided by placing anchor blocks symmetrically about the web. Additional strengthening may be required, eg, vertical prestressing or bonded plates. The best approach is to place the anchor block as near the end of the member as practicable. This limits the effects of tension and reduces the requirement to move stressing equipment about. Local bending effects can be reduced by installing a series of ties to attach the anchor block as shown in Figure 5.

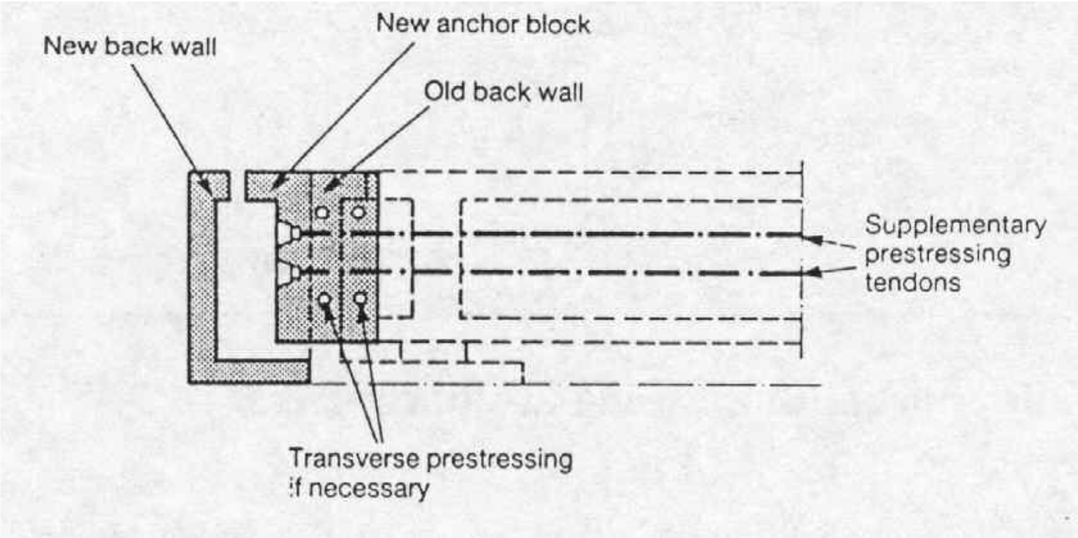

Figure 3 End anchor block.

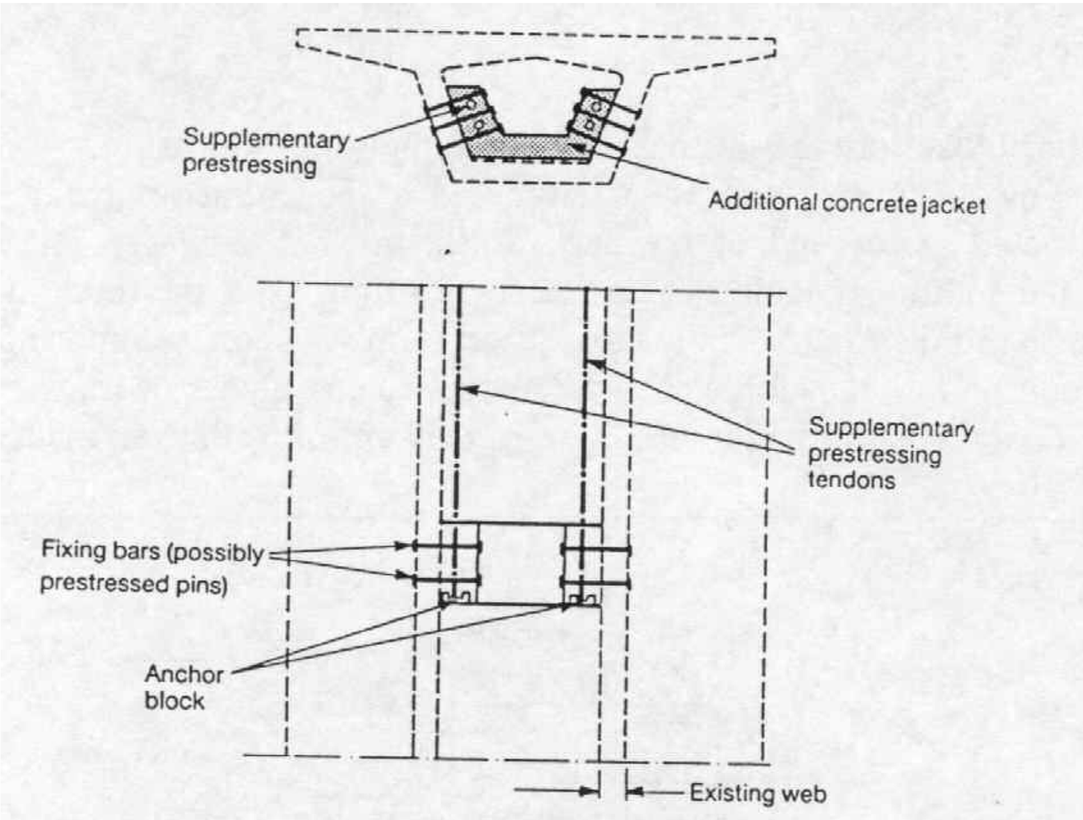

Figure 4 Bonded anchor block using in-situ concrete.

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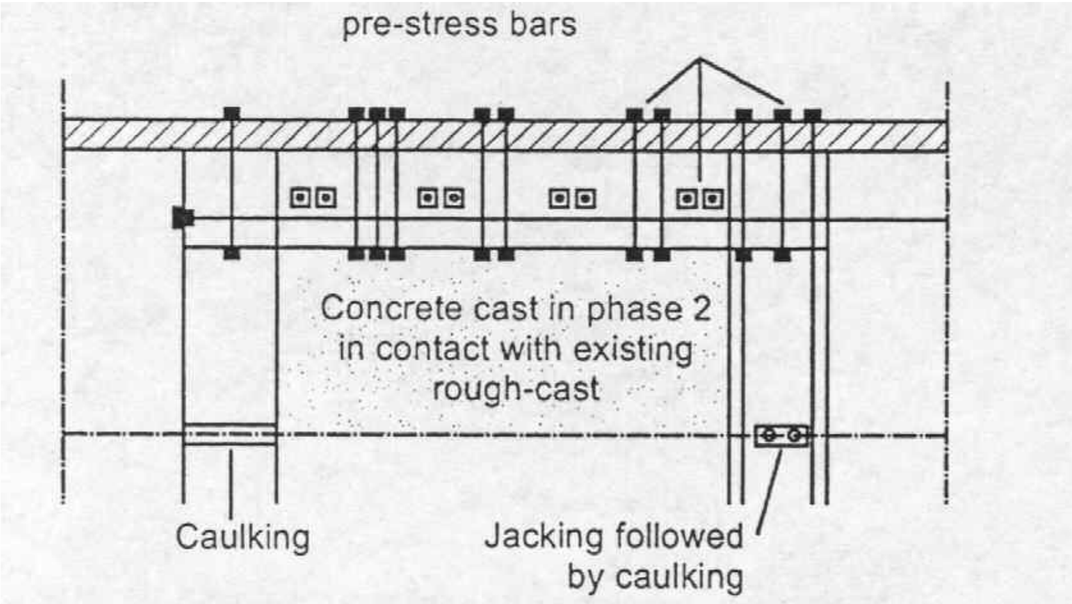

Figure 5 Anchor block attached using a series of ties.

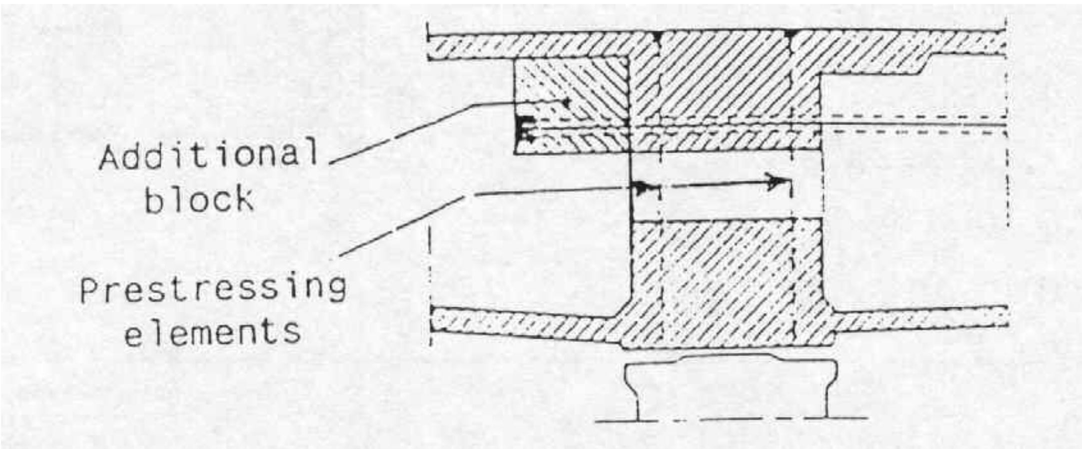

Figure 6 Diaphragm anchoring.

3.3.3 *Diaphragm anchoring*

Anchorages can be formed by drilling holes through existing diaphragms and anchoring the additional cables directly to them. This requires the diaphragms to be strong enough to take the anchorage stress. They should also be properly attached to the deck. If the diaphragms do not have sufficient strength to transfer the prestressing force to the structure, they can be strengthened by prestressing them vertically and/or horizontally or by attaching stiffeners (see Figure 6). When no suitable diaphragm exists, new diaphragms can be constructed to anchor the tendons.

3.4 Deviators

Deviators are purpose-built units designed to provide the appropriate profile to the prestressing cable. They normally consist of steel tubes embedded in concrete, profiled to avoid fretting damage to the tendon and to provide an even distribution of contact pressure: see Figure 7. To improve inspectability, saddle deviators are recommended wherever possible. With this type deviator, the cable passes over or under a steel saddle rather than through a tube which facilitates visual inspection. Figure 8 shows a typical example.

Deviators should be designed for the characteristic strength of the tendons. As for anchorages, this can be reduced to the design force at ultimate plus 20%, subject to the approval of the Technical Approval Authority.

The position and detailing of the deviators should be chosen after consideration of two main parameters: tendon profile and tendon restraint. If necessary the eccentricity can be increased by applying the prestress below the bottom flange of a girder bridge, as shown in Figure 9.

Additional deviators might be required to provide lateral restraint for the cable, either to prevent resonance resulting from traffic vibrations for example or loss of eccentricity due to deflection of the member. Figure 10 shows how a proper distribution of deviators can be used to limit loss of eccentricity.

The failure of a deviator will affect the prestress level along the whole of the deck and care must be taken in the design and detailing. The deviation system must allow for the continuity of the cable corrosion protection system. This can be provided in one of two ways. Ducts (or sheaths) can pass through the deviator without the need for joints. The curvature of the deviator should be kept low to avoid excessive contact pressure between strands and ducts which could damage the ducts. This system can be used for both tube and saddle deviators. Where tube deviators are used, the ducts can be connected to the ends of a deviator tube by coupling sleeves in a similar way as that used at anchorages. A low curvature is also desirable to reduce the friction between the strands and the tubes.

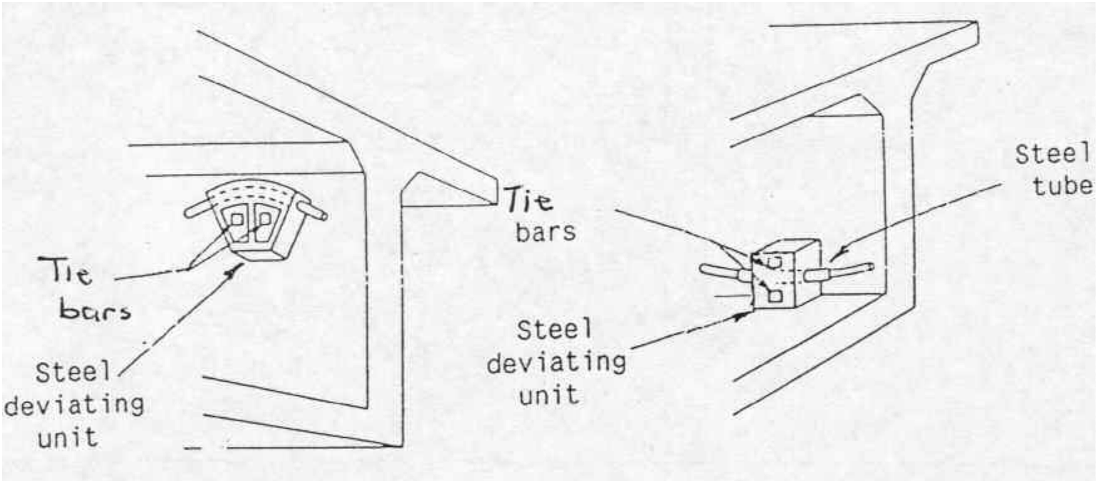

Figure 7 Deviators for cables (from NF P-104, KAK report).

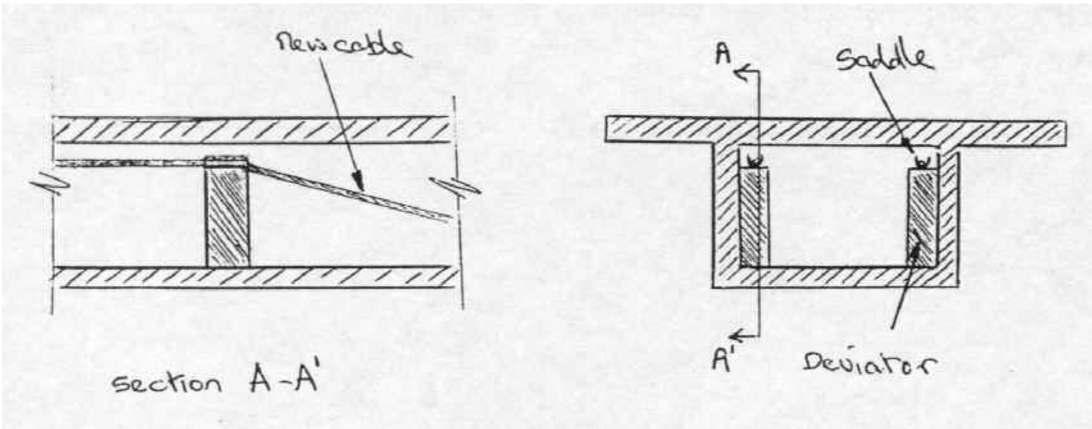

Figure 8 Saddle deviator.

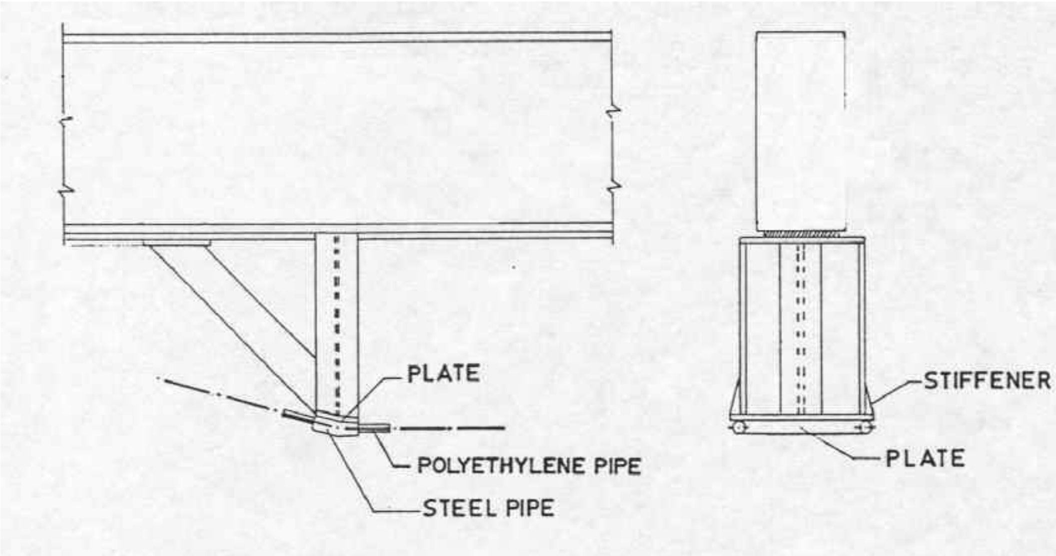

Figure 9 Deviator to increase cable eccentricity below bottom flange.

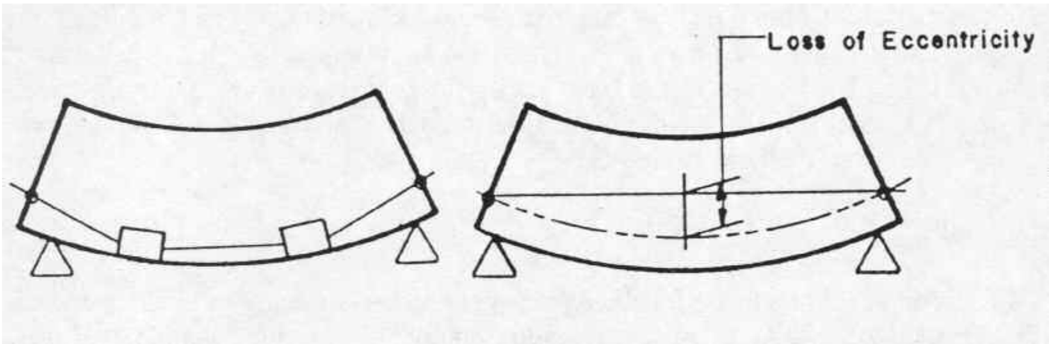

Figure 10 Distribution of deviators to prevent loss of eccentricity (from Muller and Gauthier, SP-120).

To align the individual strands in a cable and prevent tangling and trapping at deviators, an assembly of spacer tubes or separators can be used. However, threading single strands one by one through the tubes can be a lengthy and delicate operation. To minimise this effect, a multi-strand jack can be used to tension all strands simultaneously: see Section 3.7.4.

Construction of both deviators and anchor blocks, particularly in a confined or small space, is a labour intensive operation. These locations are also critical in terms of the subsequent strength of the deck and its susceptibility to corrosion damage. For these reasons, it is recommended that the number of deviators and anchor blocks be kept to a minimum in devising the tendon lay-out.

3.5 Replaceability

Where possible, a length of tendon should be provided behind the anchor to allow re-stressing, de-stressing and replacement of strands when required. This length is also needed when the prestressing force is required to be monitored and no provision is made for load cells.

3.6 Health and safety

As for any stressing operation, the safety of site personnel must be taken into account in devising and carrying out the strengthening scheme. Consideration should be given to the sequence of stressing, eccentricity of loading at all stages, and the consequences of strand failure. Special precautions should be taken to limit the consequences of whipping of prestressing strands, shearing of anchorage or deviator blocks and crushing of concrete.

The risk of damaging the structure during stressing should be minimised. The stressing system must include a method of accurately monitoring the force in the cables or bars. The pre-determined sequence of stressing should be followed in order to avoid over-stressing certain elements. As each stage of stressing is completed, close inspection of anchorages and deviators should be carried out to identify movement and/or damage. The engineer should satisfy himself that any movement or cracking is not indicative of over-stressing.

Additionally, provision should be made for the following:

- Work in confined and/or poorly ventilated spaces
- Work in the vicinity of power lines or other services
- Conduct of grouting and/or injection operations.
- For bridge works, effective traffic management to ensure minimal disruption.

3.7 Equipment requirements

Equipment for the casting anchor blocks and installing anchorages and deviators is standard. Most of the specialist equipment required for stressing is readily available from the suppliers of conventional prestressing systems. The installation of the tendons require the special fabrication of anchorage block and deviators. Details of the special equipment required are given in the following sections.

3.7.1 Tendons

The prestressing force is applied using conventional prestressing materials and usually comprise bars or single or multiple strands. The use of fibre-reinforced polymer (FRP) in prestressing tendons offer significant advantages in terms of higher strength/weight ratio and increased resistance to deterioration. These can be used provided their material properties and durability (both short term and long term) are properly taken into account.

3.7.2 Corrosion protection

The tendons, being external, are more susceptible to corrosion, and often need to be placed in areas of run-off, eg, near joints in bridge decks. The tendons can be susceptible to contamination by bird and bat droppings, eg, inside boxes and along beams. Cases where tendons have been submerged during floods have been reported. Effective corrosion protection is critical to the effective performance of this method. The principles are the same as for new construction using external tendons and reference should be made to the appropriate design code.

The lengths of tendon left at the anchorage to ensure replaceability requires corrosion protection. A suitable system such as a cap filled with grease or wax should be incorporated. Effective corrosion protection should also be applied to protect any transverse post-tensioning bars or tendons used to install anchorage blocks or deviators.

All exposed metal components such as plates, anchorage blocks, deviators, etc, should be painted to provide corrosion resistance and protection in accordance with the relevant design code.

The use of other materials which are resistant to corrosion, such as stainless steel and FRP, can be used without additional corrosion protection. Galvanised strand while not normally used for conventional prestressing can be considered for unbonded tendons. Due account should be taken of the different behaviour of these materials in terms of material properties, durability and anchorage requirements.

3.7.3 Anchorage systems

Proprietary anchorage systems for external post-tensioning systems are produced by a number of prestressing equipment suppliers. These can be very sophisticated, incorporating appropriate corrosion protection systems, but are very expensive. For simple applications, anchorage systems can be purpose built. Where these are used, an effective corrosion protection system for the strand as well as the whole anchorage system must be incorporated.

3.7.4 Stressing equipment

Various stressing jacks are available from specialist prestressing contractors. When a single strand jack is used to tension a cable, the successive tensioning of strands can cause tangling and trapping which can prevent uniform tensioning along the strand. Where individual sheathing of strands is present, this can also damage sheathings at deviators. To avoid this problem an assembly of spacer tubes or separators can be used but this complicates the installation of strands. Another alternative is to place individually sheathed and greased strands inside an HDPE duct which is grouted prior to stressing. However, this has the disadvantage of reducing the inspectability of the tendon. Removal and replacement of individual strands is not possible after installation.

Multi-strand jacks used for conventional internal post-tensioning can be used to stress all the strands simultaneously. This minimises the problems of tangling or trapping of individual strands and eases the installation process as all strands can be installed as a group.

It is recommended that single strand jacks be used where possible because of ease of handling in confined spaces, particularly where re-stressing is anticipated. However, there will be cases where multi-strand jacks are the preferred choice.

An important component of any stressing system is a method of accurately measuring the applied force.

4 QUALITY ASSURANCE

4.1 Acceptance

Inspection should be carried out throughout the course of the strengthening works. On completion, a detailed inspection should be carried out to ensure that all aspects have been completed in accordance with specifications. All anchorage and deviators should be examined to identify any movement or cracks. The cables, ducts, etc, should be inspected to ensure the integrity of the corrosion protection system. Any damaged or incomplete paint-work should be repaired.

At the end of the maintenance period, a further inspection should be carried out to identify any movement or cracks at anchorages and deviators. Load checks should be carried out to ensure that prestress losses are acceptable and the cables re-stressed if required.

4.2 In-service maintenance and inspection

The handling, transport and storage of prestressing assemblies prior to installation should be organised to avoid the physical or chemical damage to the components. Once installed, exposure to the elements and contamination by corrosive elements should be avoided, particularly prior to completion of the permanent corrosion protection system.

The corrosion protection system should be applied as soon as possible after installation is complete. This includes painting of exposed metal surfaces, sealing of ducts, wax injection and grouting.

In order to be effective, the corrosion protection system must be continuous. After installation, all ducts, painted surfaces, etc, should be inspected for defects and any breaches effectively repaired.

The strengthening system should be devised with a view to simplifying as much as possible procedures for inspection, maintenance and demolition of the strengthened structure. The facility for re-stressing and replacing tendons, either because of damage from corrosion or impact or to modify the strengthening scheme, should be incorporated. An inspection and maintenance strategy should be developed as part of the design. This should include an outline of recommended inspection procedures. The requirements for any monitoring system should also be identified.

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Appendix A: Case studies

External post-tensioning as a means of strengthening concrete bridges has been in use since the 1950s and many examples have been reported. A number of case studies are presented here to illustrate how the technique has been applied. These case studies are not intended to provide specific design details for general use. However, they are useful in providing general ideas on the use of the technique and in identifying the main criteria that might be used in the selection of systems and the detailing of anchorages and deviators appropriate to particular situations.

A.1 Clifton Bridge

Clifton Bridge (Driver and Haynes 1998) is a post-tensioned concrete box girder bridge carrying part of the A52 north bound Nottingham ring road over the river Trent. It was first opened to the traffic in 1958 having taken 3 years to build and at the time was the longest prestressed concrete bridge in the country.

The bridge has a main span of 83.8m comprising two 26.7m cantilevers supporting a 30.5m suspended span. Two 38.1 m end spans and three 27.4m approach viaduct spans complete the structure which has an overall length of 242.2m. The end spans and cantilevers are of cast in-situ multi-cell construction and the suspended span and viaduct are of precast beam and in-situ slab construction. The existing prestressing system comprises wire tendons located in the top slab. These tendons were placed in open ducts which were subsequently filled with grout. A general elevation of the bridge is shown in Figure A.1.

A condition survey carried out in 1994 revealed that approximately 25% of the top slab wires had suffered corrosion damage and many of these had fractured. Assessment of the bridge confirmed that even with this level of prestress loss, the end cantilever spans still attained a 40 tonnes assessment load capacity. However, due to the possibility of future loss of prestress precautionary strengthening of the bridge using external unbonded post-tensioning was carried out. The project was very demanding because of the limited access to the hollow sections of the bridge, where headroom was a little as 1.3m in places, and because of the requirement to keep the bridge open to traffic throughout the strengthening works.

Because the future extent of corrosion was not known, the feasibility studies carried out by Nottinghamshire County Council assumed different levels of prestressing loss. The use of multi-strand tendons as the prestressing material was preferred over high tensile alloy bars because of the high forces required and because the tendons could be deflected to positions of maximum eccentricity. Assumptions of 100%, 75% and 50% prestress loss resulted in the requirement for an additional 22, 13 and 7 No 19-wire strands for each web, respectively. Out of the three options studied at the feasibility stage the option with 22 strands was used in the detailed design as shown in Figure A.2. Serviceability stresses resulting from additional external unbonded prestress combined with the existing prestress were not found to be a limiting criteria. Maximum stress under total dead and live loading was 15 N/mm^2 with no tensile stresses occurring, ie, a class 1 structure according to BS 5400: Part 4.

The tendons used consisted of individually greased and sheathed mono-strands in a 140mm HDPE duct. A nominal load was applied to take out the slack and the duct was grouted before final stressing. Spacer tubes were not used and the tendons were stressed using a series of multi-strand jacks.

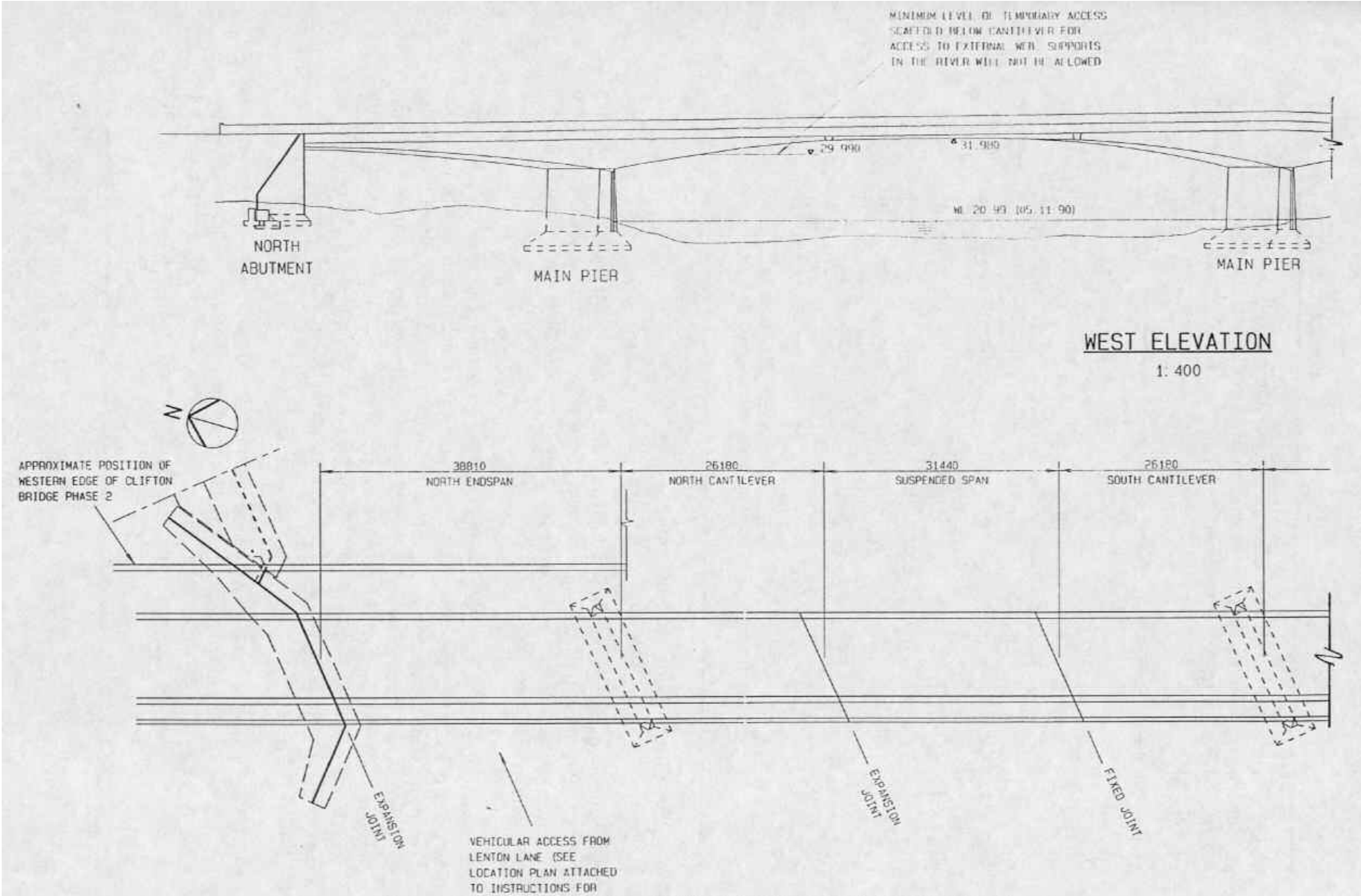

Figure A.1 Strengthening of clifton bridge: general arrangement.

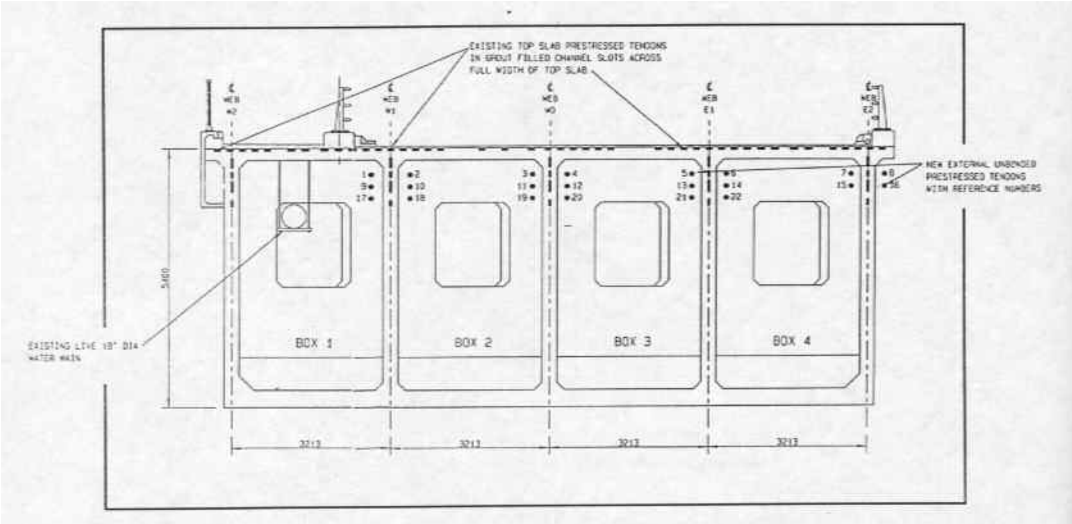

Figure A.2 Clifton Bridge strengthening: position of tendons.

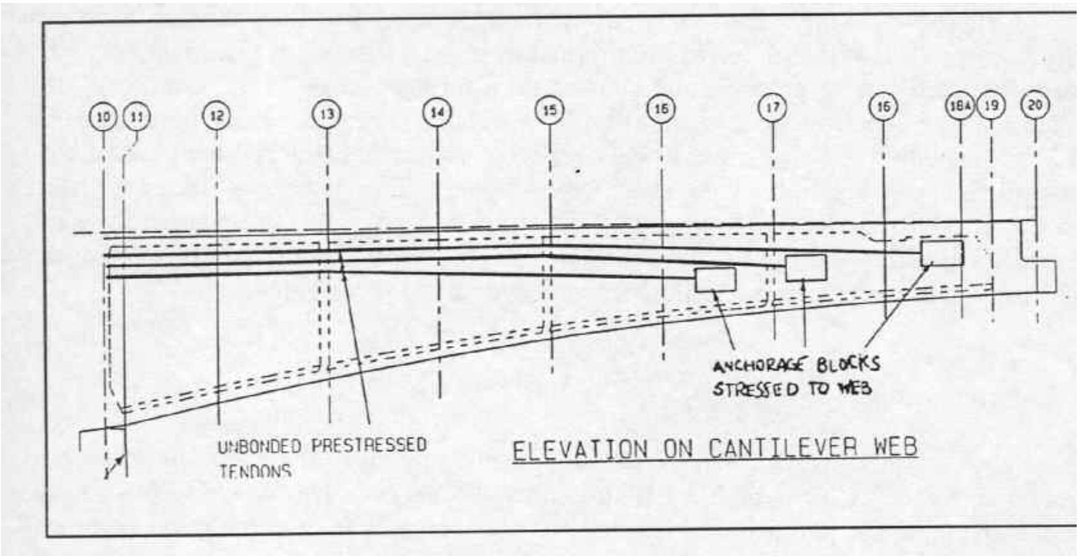

Figure A.3 Clifton Bridge strengthening: position of anchor blocks.

The prestressing forces are transferred to the webs via concrete anchorage blocks cast onto opposite sides of each internal web and attached using prestressed high tensile alloy steel bars. The position of the anchorages is shown in Figure A.3. The transverse prestressing bars were greased and shrink-wrapped to provide corrosion protection and to de-bond them from the concrete so that further load could be applied if required. The reinforcement in the anchorage block consisted of a cage of 16mm bars and a 16mm spiral bar around the anchorage tube. The anchorage tubes were made of galvanised steel with a 350mm square, 50mm thick end plate.

The tendons are deflected using galvanised steel tubes bent to radius and resin-grouted into position at existing diaphragm positions where possible. For larger deviation angles, additional deflector blocks were connected to the existing webs using prestressing bars similar to the anchorages. In shear critical sections the tendons were deflected to provide a vertical component of prestressing force to enhance shear capacity.

To avoid applying unbalanced forces to the eastern outer web at the anchor block it was necessary to provide tendons external to the box on the east elevation. This was aesthetically acceptable as the presence of an adjacent bridge carrying the south bound ring road hides the tendons from general view. Additional prestressing was not required on the western outer web as it mainly carries pedestrian loading and the existing tendons over the web are not affected by corrosion.

Before drilling the webs to install the transverse prestressing bars, the location of existing tendons were detected by drilling 20mm holes in the web and inserting a probe similar to a cover-metre. The accuracy of the detection is claimed to be within 1mm. This ensured that existing internal tendons were not damaged during installation.

The amount of longitudinal prestress that could safely be applied to the deck was restricted by how much local additional tensile stress could be tolerated on the webs. It was not possible to compute these stresses as the condition of the existing internal tendons was not known. Tests were carried out to determine the existing prestress levels but these indicated less horizontal compressive stress and more vertical tensile stress than expected. The procedure adopted was to apply the external prestress while continuously monitoring the stresses in the webs. Stress application would be suspended when the additional tensile stress in the web reached $2N/mm^2$. The sequence of stressing was worked out to ensure that the resulting out of balance forces would not induce excessive stresses in the super-structure. Other monitoring was carried out during the stressing, including movement at the expansion joints and vertical movement at the cantilever half joints.

A total of 2000kN was applied to the tendons when the agreed additional stress limit was reached: this was about two-thirds of the original intended force. This effectively replaced the identified 25% loss of prestress and allowed for a further loss of 25%. The monitoring indicated that the cantilever half-joints lifted by about 20mm and the bride joints open by some 3mm. The live end anchorage strand over-lengths were not trimmed but covered with a grease filled cap bolted to the back of the bearing plate. The anchorage tubes were filled with a protective grease rather than grout. This makes it possible to re-connect the jacks and check the prestress load or to apply further load. It also allows the safe removal of the tendon load either to replace the tendons or to decommission the bridge.

A.2 Kingston Bridge

Kingston Bridge was constructed in the late 1960's and opened to the traffic in May 1970. It forms part of the M8 Glasgow Inner Ring Road and consists of two adjacent 20.8 m wide three pin asymmetrical portal frame structures with side spans connected to the north and south structural approaches. Each three span (62.5, 143.3 and 62.5 m) super-structure is made of a three cell in-situ concrete post-tensioned concrete box girder constructed by the balanced cantilever method. Figure A.4 shows a general elevation of the bridge. The north pier is pinned at the top and bottom and the south pier is built-in at the top and pinned at the bottom. Half-joint connections to the north and south structural approaches are restrained vertically but free to move horizontally (Carruthers and Couts 1993).

In the late 1980's a number of deficiencies were observed as outlined below:

- the north expansion joints tended to remain closed;
- the south expansion joints tended to remain open;

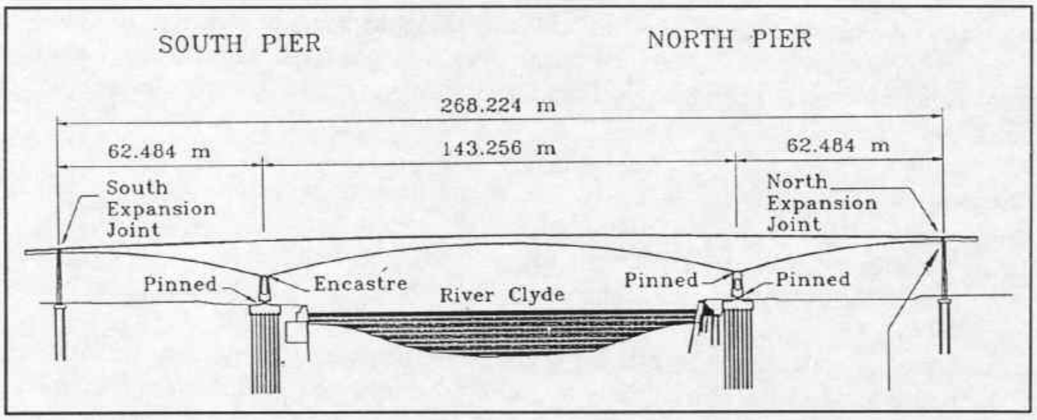

Figure A.4 Strengthening of Kingston Bridge: general arrangement.

- the north pier walls were out of longitudinal alignment by 165mm;
- the concrete between the north pier and the bridge deck was crushing; crushing was also occurring at the base of the north pier;
- the design capacity of the rocker bearings at the top of the north pier walls were exceeded, resulting in slip of the pier walls relative to the decks;
- the centre span deflection had increased by around 300mm at its mid point.

A subsequent investigation indicated the need to secure to horizontal movement of the bridge and to strengthen the main span. Among measures taken to retrofit the bridge was to modify the three pin structural system to a two pin one. This was achieved by eliminating two of the bearings at the bottom of the pier walls and adding a bearing at the top of southern pier. This measure together with the requirement to strengthen the main span meant increasing the existing 35MN prestressing force in the decks to 90MN (Hayward 1996).

The additional prestressing force is applied to each deck using 26 tendons, each up to 200 m long. The position of the tendons and anchorages are shown in Figure A.5 (Hutchison and Collings 1997). Since there is no room alongside existing tendons, the new tendons are routed through the box void on the underside of the top slab. The tendons are anchored on either side of the internal webs and to the bottom slab by means of precast concrete anchor blocks connected to the webs using short pre-tensioned Macalloy bars. Details of the anchorages are shown in Figure A.6. Part of the voids at either end of the bridge are filled with mass concrete to counterbalance uplift resulting from substructure retrofit. The external tendons will be anchored to the webs in front of the mass concrete. The proposed strands can be replaced by welding new strands to the end pulling them through their sheathing.

The tendons consist of greased/sheathed mono-strands (7-wire superstrand) with a nominal diameter of 15.7mm. The sheaths are 2mm thick HDPE with nominal OD of 22.2mm. They are applied loose to facilitate replaceability of a single strand within an otherwise stressed tendon. At the anchorages the strand will be contained within a removable grease filled cap. Spacers are provided in the ducts to separate the strands. The tendons can be stressed and re-stressed as necessary using a lightweight portable single strand jack.

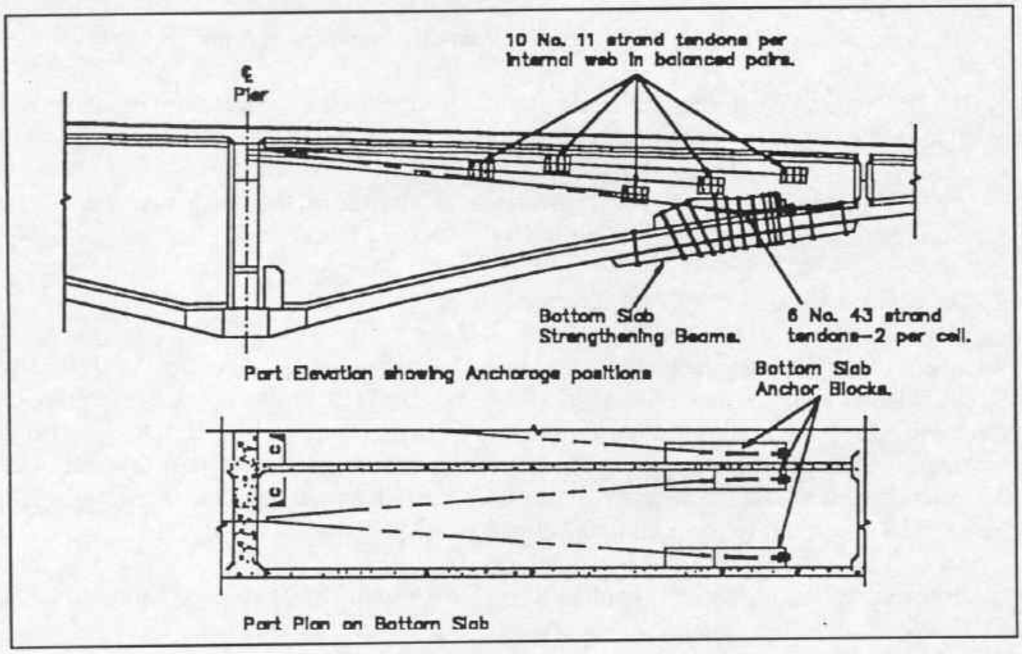

Figure A.5 **Strengthening of Kingston bridge: position of tendons and anchorages.**

During the strengthening two existing lanes over the bridge will be closed. This is required mainly due to the strengthening of the substructure rather than the decks. The two lanes will be ballasted at the ends of the bridge to counteract uplift during the works. Load cells will be placed at the ends of a number of external tendons to monitor prestressing force during and after strengthening.

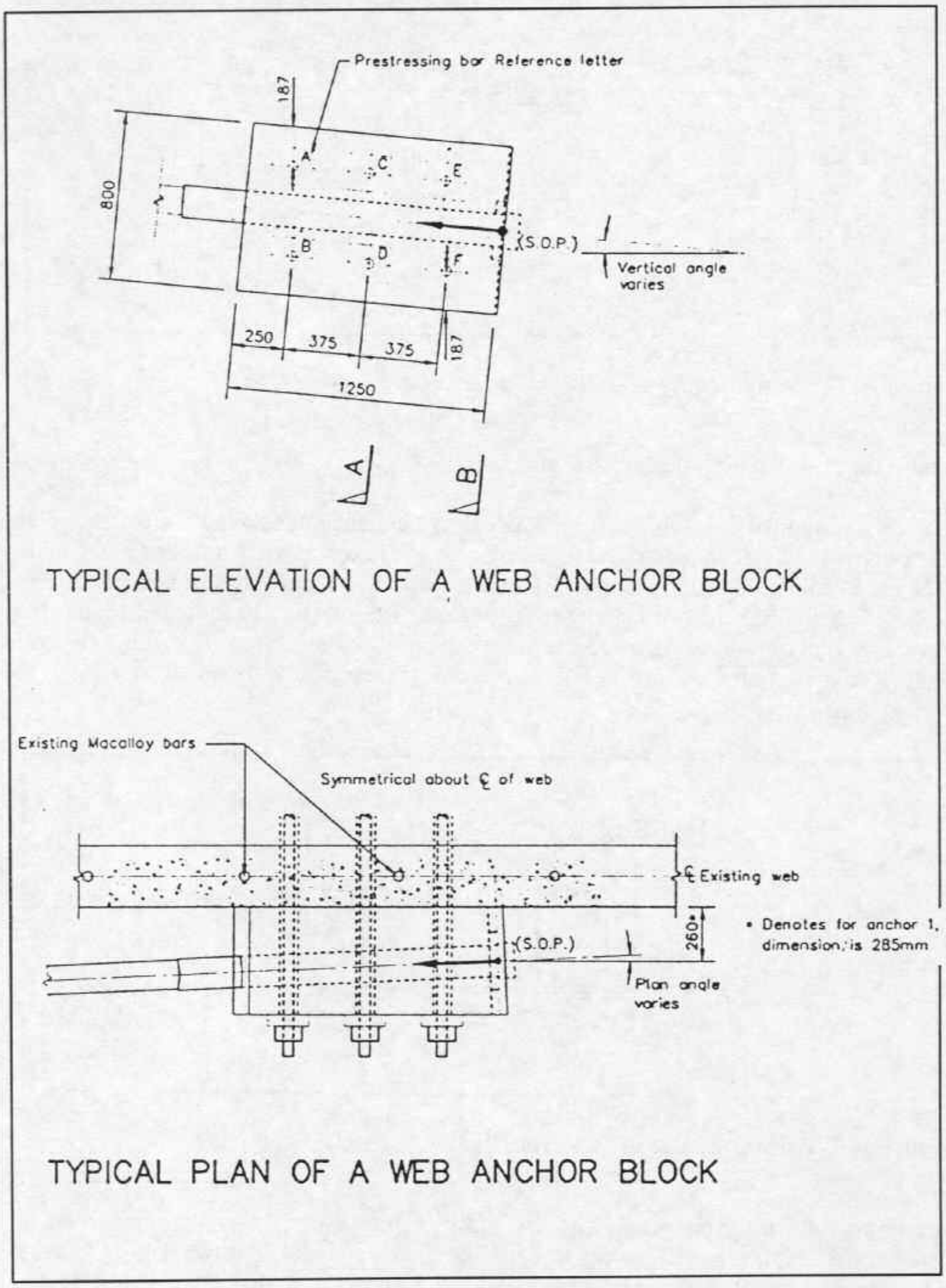

Figure A.6 Kingston bridge: detail of web anchorages.

A.3 Friarton bridge

Friarton bridge is a continuous steel/concrete composite box girder bridge which was designed in the 1977. The bridge has nine spans, and carries the M90 over the Tay bridge in Perthshire with a total length of 831m. The main river span is 174m. A general view of the bridge is shown in Figure A.7. The deck consists of two independent longitudinal and transversely stiffened steel box girders with a 200mm lightweight concrete deck slab. The girders are 4.3m wide with depth varying from 2.7m over the approach spans to 7.5m over the supports.

Since the bridge was built, bridge loading specifications have increased considerably. Assessment to current standards indicated that the bridge had insufficient capacity mainly due to hogging bending over and adjacent to the supports and inadequate shear connection between the steel box and the concrete slab. Otherwise inspection showed that the bridge was in good condition with no major defects.

Feasibility studies were carried out to determine the most effective way of strengthening the bridge. The solution adopted is described by Slater (1997). It consisted of adding longitudinal stiffeners to the compression flange at the support positions and post-tensioning the tension flange directly over the supports. The purpose of the stiffeners is to increase the buckling capacity of the compression flange, while the post-tensioning relieved the dead load stressed from the tension flange. The combination of methods allowed the use of relatively light post-tensioning tendons at the top flange level where installation problems existed due to restricted access. Figure A.8 (from Slater, 1997) shows the main strengthening details.

The anchorage consisted of a thick plate welded to two longitudinal stiffeners which in turn are welded to the top flange. The existing longitudinal stiffeners had to be cut back to allow the new plate to be installed and additional transverse plates were introduced to ensure lateral stability of the anchorage. The connections of the anchorages to the top flange are staggered over three ring frames, as shown in Figure A.9, to prevent local over-stressing of the shear studs. Slots were cut into the ring frame top members to allow the tendons to be located as close to the top flange as possible, ensuring minimum eccentricity of the prestress force. Additional bracing of the ring frames was introduced to resist this load effect. The ducts consisted of polyethylene and were filled with wax to protect the tendons against corrosion.

An important factor in the strengthening methodology was to minimise the traffic disruption during installation. Making allowance for the increased dead load and temporary works, it was possible for the unstrengthened bridge to sustain one lane of HB loading or a 150 tonne abnormal load, but not both together. Thus the deck had to be restricted to one lane of traffic and abnormal loads would have to be escorted with no other traffic on the bridge. In addition, to avoid locked-in stresses during welding, live load could not be permitted on the bridge. To minimise traffic disruption, the girders were strengthened consecutively rather than simultaneously.

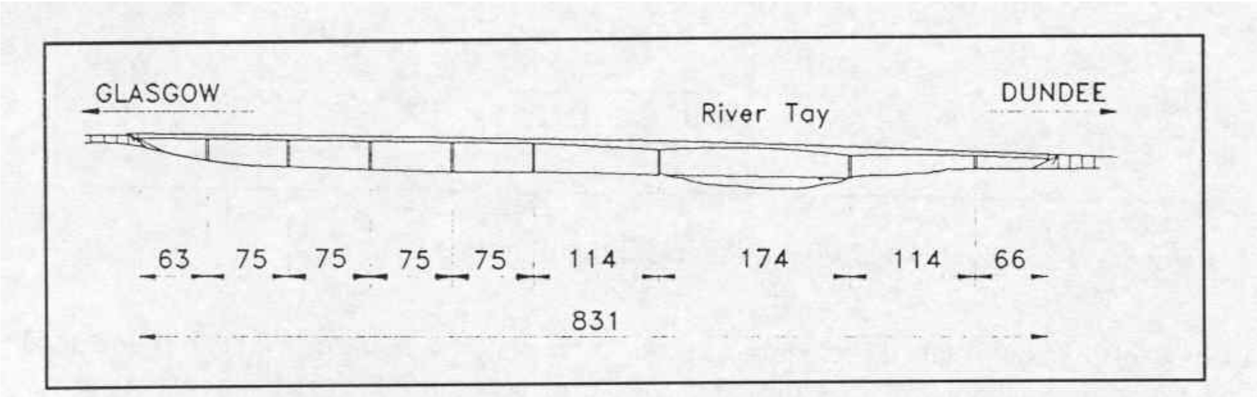

Figure A.7 General elevation of Friarton Bridge.

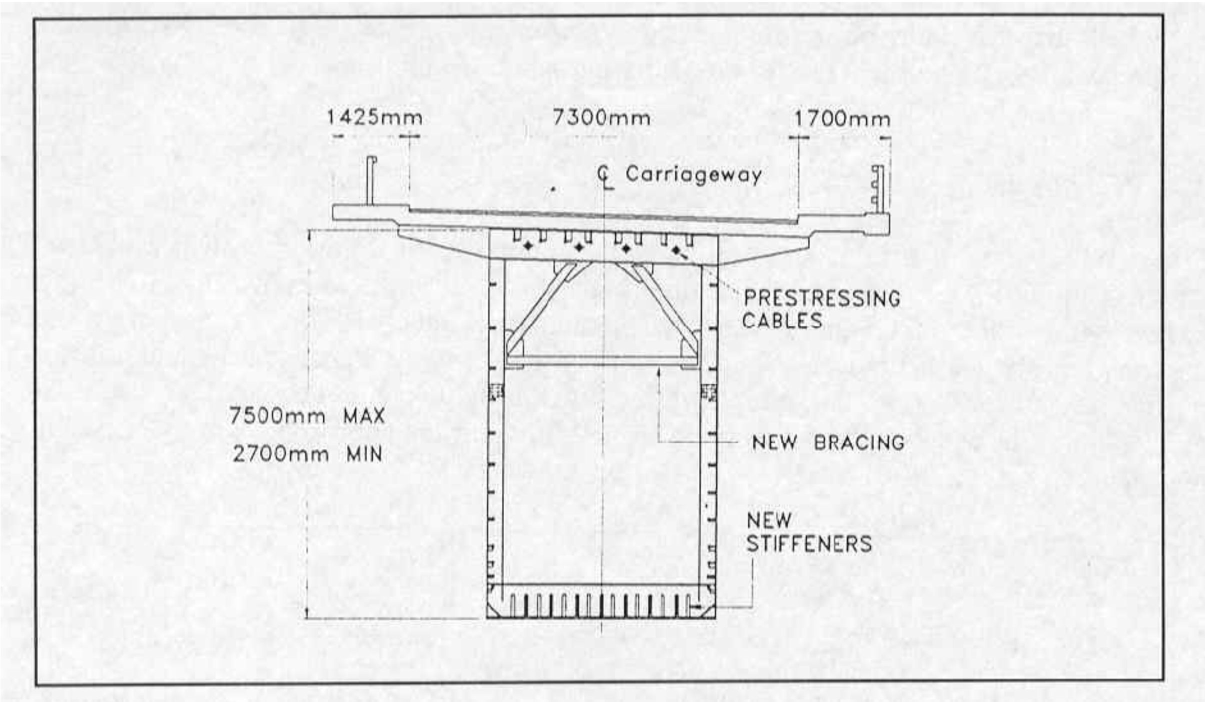

Figure A.8 Main strengthening details (Slater 1997).

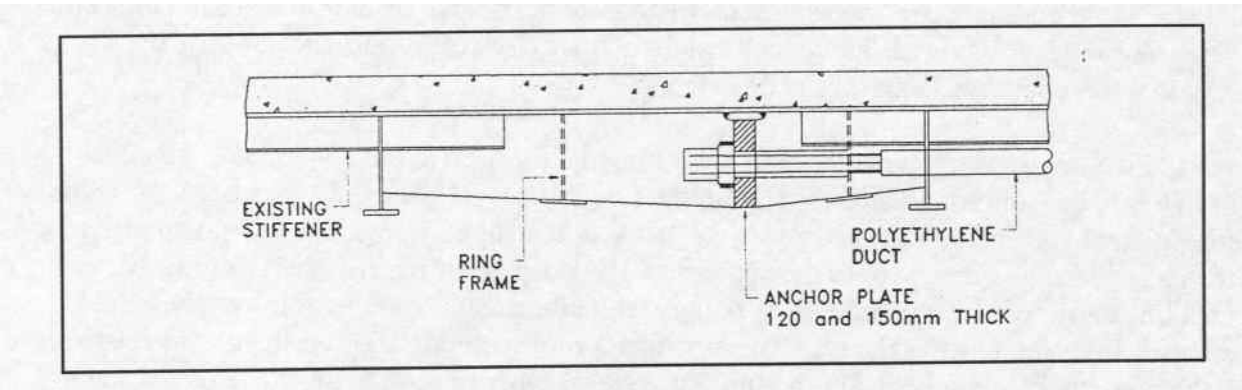

Figure A.9 Anchorage detail (Slater 1997).

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Figures

MAIN TEXT:

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Figure 2 Tendon profiles for box girder bridges - from Anglo-French report.
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Figure 4 Bonded anchor block using in-situ concrete.
Figure 5 Anchor block attached using a series of ties.
Figure 6 Diaphragm anchoring.
Figure 7 Deviators for cables.
Figure 8 Saddle deviator.
Figure 9 Deviator to increase cable eccentricity below bottom flange of steel girder.
Figure 10 Distribution of deviators to prevent loss of eccentricity (from Muller and Gauthier, SP-120).

APPENDIX:

Figure A.1 General elevation of Clifton bridge.
Figure A.2 Position of external tendons in Clifton bridge.
Figure A.3 Position of the anchorages in Clifton bridge.
Figure A.4 General elevation of Kingston bridge.
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Figure A.6 Details of the anchorages in Kingston bridge.
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Figure A.8 Main strengthening details (Slater 1997).
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