Concrete Bridge Design and Construction series

This series is authored by the Concrete Bridge Development Group (CBDG)

The group aims to promote excellence in the design, construction and management of concrete bridges. With a membership that includes owners, designers, academics, contractors and suppliers, it provides a focus for the use of best practice, innovation, training initiatives and research and development. Further information on the CBDG can be found at: www.cbdg.org.uk

No. 3: Prestressing for concrete bridges

Introduction

This article from the Concrete Bridge Development Group's technical committee examines the prestressing for a concrete bridge. In concrete bridges, for deck spans more than 20-30m, prestressing should generally be used. As described in last month's article[1], prestressing enhances the capacity of a member that is weak in tension, but strong in compression. It effectively creates a new material that is strong in tension. Prestressing allows a bridge to be more economical, with lighter and more slender members, which improves the appearance. The sections are generally compressed under permanent effects, which give greater durability due to the reduced incidence of cracking. The sections also behave elastically with greater stiffness, allowing deflections to be more easily controlled. A prestressed bridge has much less steel to be handled, which reduces congestion, leading to easier and quicker concrete placing.

Prestressing basics

Prestressing is an active system that opposes externally applied loads and actions with a set of internal forces. The designer has to fully understand this range of actions, and the difference between loads and imposed deformations. Only a brief introduction to prestressing can be given in this article and designers should consult other texts for more detail (see References and further reading section).

For the design of the concrete deck section, the top slabs are governed by traffic loads and transverse bending effects, the webs by shear and torsion at the supports (but by minimum requirements for concreting at midspan), and the bottom slabs/heels by the layout of the prestressing cables and by any compressions at the supports (or the minimum thickness for concreting at midspan). Self-weight

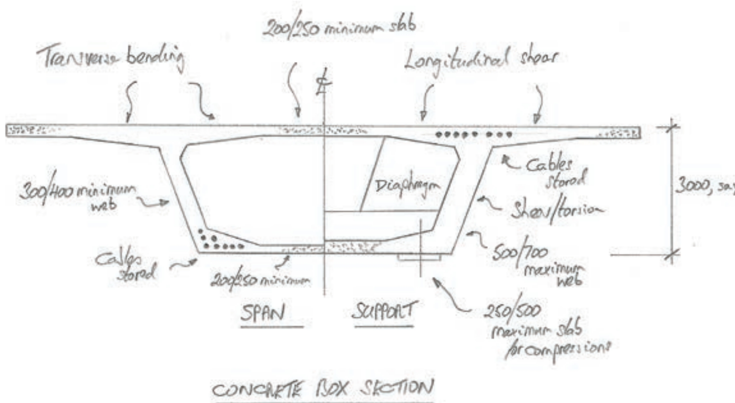

Figure 1
Concrete box section sizing

dominates many bridge designs and the areas of concrete should therefore always be kept to a minimum. It is particularly important to minimise the web concrete in a bridge, as these areas are also inefficient for the prestressing. For most bridge widths up to around 20m, it would generally be best to have only two webs, though with precast beam solutions, the number of webs is greater, as will be seen in future articles. However, for efficiency, the number of webs should still be minimised by spacing the beams as far apart as possible. For the design of box girders, which will generally be used for all spans over 30-40m, single cells are much preferred as they are far easier to cast than multi-cell boxes. Boxes are very efficient in distributing eccentric traffic loads, though some care is needed with the analysis of torsional/distortional warping.

In order to achieve the sufficiently high covers that are needed for durability, the minimum slab thickness is about 225-250mm, though 200mm is often seen in less aggressive environments. Making proper allowances for how the concrete flows during the casting process (and how longitudinal shear is controlled) will often

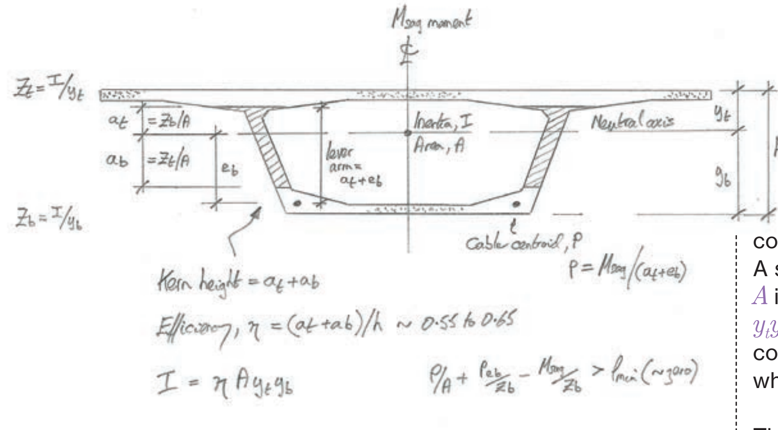

Figure 2
Prestressed section properties

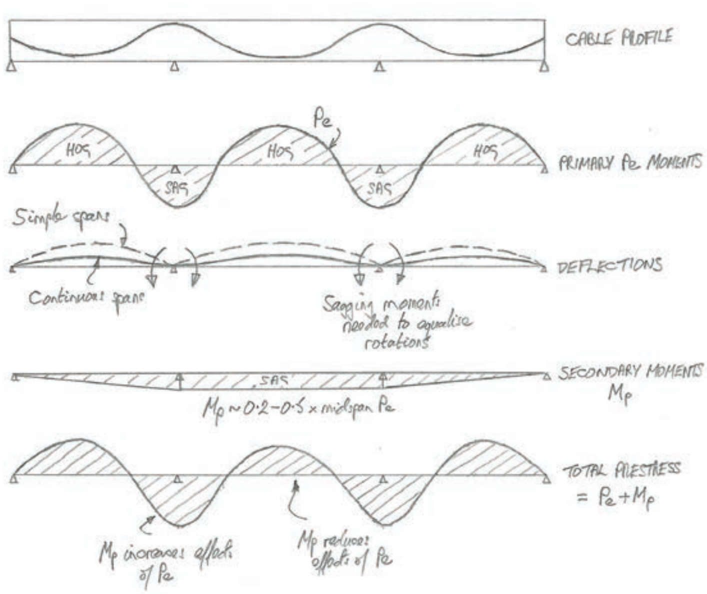

Figure 3
Secondary prestress moments on a 3-span bridge

determine the thicknesses of the section in the regions where the webs meet the slabs. These transition zones between webs and slabs are also the areas where prestressing cables can be located. All these basic section parameters, which are often governed by the practicalities of the construction rather than any detailed analysis, can therefore be sized quickly by an experienced bridge engineer (Figure 1).

The section properties can be calculated using any relevant software, but the designer must concentrate on the kern height and efficiency of the section. The kern height is the area within which a prestressing force can be applied without causing the section to go in to tension. The top kern height a_t is Z_b/A and the bottom kern height a_b is Z_t/A (Figure 2 defines all these parameters). The efficiency of the section η is defined as $(a_t + a_b)/h$ – it defines how effective a section is in carrying the prestress. It is 33% for a rectangle (i.e. the well-known middle third rule) and 55-65% for typical I-beams or box girders. It needs to be as high as possible to avoid wasting prestress by compressing too many webs, or creating webs that are too thick. A simple way to calculate the inertia I is to note that $I = A\eta y_t y_b$. A is easy to calculate, η is usually about 60% and the product $y_t y_b$ is insensitive to exactness. These section properties are vital components in calculating the required prestress force for the whole span.

The prestress force (P at eccentricity e from the neutral axis) at each section carrying a moment M is then determined using $P = M/lever\ arm$, where $lever\ arm = (a + e)$, which would be $(a_t + e_b)$ for M_{sag} and $(a_b + e_t)$ for M_{hog}. Alternatively, if the moment range were critical, then P is defined by $M_{range}/(a_t + a_b)$. Cables zones can thus be calculated for the whole span. This use of kern heights is the same as examining the extreme fibre stresses under the critical set of moments M using inequalities such as:

$$P/A +/- Pe/Z +/- M/Z > zero$$

These equations are all based on the assumption that the section remains fully compressed under all moments i.e. that the minimum concrete compression f_c is zero. The equations can easily be adjusted to suit either a higher positive value of f_c (to allow for residual effects of warping or temperature difference), or a lower negative value of f_c (to accommodate some allowable tension).

M should be taken as the critical combination of either the maximum values (under full traffic load combinations), or the minimum values (under permanent or construction loads, or from any reversed traffic loads taken from the relevant influence lines). Due allowance should then be made for the prestressing forces, which will drop over time from higher values at the time of jacking, to lower values after the effects of creep, shrinkage and relaxation have occurred. A tabular list of the stress history at every section should always be shown at the end of the design process, partly as a check and partly as a clear means of seeing all the stresses in the section at all stages. Magnel diagrams exist to allow the prestressing to be designed for both the minimum and maximum compressions in the section. However, though they are useful in understanding the overall parameters, these diagrams are generally too cumbersome for the design process and any issues with maximum compression will not be solved by adjusting the prestress force, but by increasing the concrete area or strength.

For all indeterminate structures (continuous bridges, for example), the prestressing primary moments Pe also produce secondary, or parasitic, moments M_p. These moments are needed to ensure the compatibility of rotations at each support position – they are generally very significant sagging moments that vary linearly between supports (Figure 3). Their values must be iterated with the required values of Pe at each section, as M_p is dependent on the Pe profile of the whole structure. The calculation can be carried out using numerical integration of Pe with the flexibility method, or equivalent prestressing loads with the stiffness method. Designers should appreciate the effects of this moment, as M_p can be used to real benefit in the design, by allowing the engineer to transfer moments between support and midspan sections. When bridges are built in stages, creep of both the self-weight moments and M_p occurs, though it tends to work in opposite directions, negating the need for too much precision in the calculation of the creep factor.

Figure 4
Pre-tensioning bed (Banagher Precast Concrete)

Pre-tensioned bridges

With pre-tensioning, the prestressing steel is stressed first and the concrete member is then cast around this steel (Figure 4). It has the advantage that as the prestressing is embedded in the concrete, there is no need for any grouting. The prestressing consists of individual strands, each made up from seven spirally-wound steel wires. The most common strand is a low relaxation superstrand, having a 15.7mm diameter, an area of 150mm^2 and an ultimate strength f_{pk} of 1 860MN/m^2. As the strands are generally straight, the system usually only allows straight beams to be produced.

Standard precast, pre-tensioned concrete beams have been used for years in many countries. Various shapes and depths are produced, suitable for a range of spans from 5-40m (Figure 5). They are generally cast off-site in precast factories and transported to site. Typical beams weigh 5-60t and are generally erected by crane. Smaller beams are placed adjacent to each other and the space between the beams filled with concrete to form a solid slab, with typical spans varying from 5-25m. Larger beams are spaced 1-2m apart, permanent formwork is placed between the beams and an *in situ* deck slab is cast over the top. Typical spans in this case vary from 15-35m, though up to 40m is possible. Various arrangements of bespoke pre-tensioning also exist, allowing much larger I, T or U-beams to be cast. These beams are then spaced 2-4m apart, with beam weights of 40-200t, and spans of up to 60m can be used. For very long viaducts, whole span precast units (with spans of 35-50m) can be cast and pre-tensioned in purpose-made factories on site. Such units might weigh up to 1 000t and be erected using special transporters and gantries.

The prestress and self-weight loads are carried on the precast beam while all other loads (finishes and traffic loads) are carried on the composite section i.e. including the top slab. The top surface of the beam is suitably prepared and has projecting reinforcement so that the slab and beams act together. The prestress is applied to the ends of the member by bond action between the strand and the concrete, resulting in a length over which the force is transmitted (of 500-1 000mm). Elastic deformation, creep and shrinkage losses (all of which are high due to the early transfer of the prestressing onto young concrete) combine with steel relaxation losses to give long-term stresses in the prestressing of 1 000-1 100MN/m^2, or 55-60% f_{pk}. De-bonding of some of the strands is often used at the ends of the beams, so as not to either overstress the bottom fibre or put tension into the top fibre.

Standard precast beams, produced in a factory, can therefore be of high-quality, with a proven record of durability. These precast beams can be quickly erected on site and are therefore particularly useful when bridging over live roads, railways and waterways, where interruptions to the traffic must be minimised. See CBDG CPS 4[2] and CBDG TG 13[3] for further details. Hybrid schemes also exist where an initial phase of pre-tensioning is then augmented with post-tensioning at a later phase.

Post-tensioned bridges

With post-tensioning, the concrete member is cast first and the prestressing is applied afterwards. The range of possible bridge types and construction methods is wide, and most concrete bridges over 30-40m spans (and up to 300m spans) will use post-tensioning. Bridges can be formed in to any shape and the most intricate alignments can be accommodated. The same superstrands are used, but they are bundled together to form cables. Typical cables may be formed from 12-37 strands, and be designated as 12/15mm or

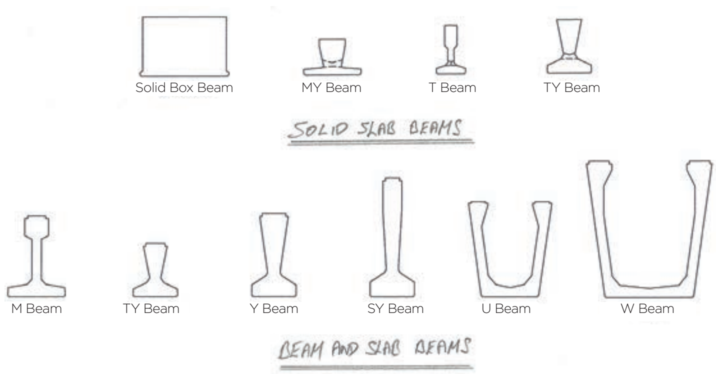

Figure 5
Standard precast beams

Figure 6
Post-tensioning jack stressing a cable (Clackmannanshire Bridge, UK)

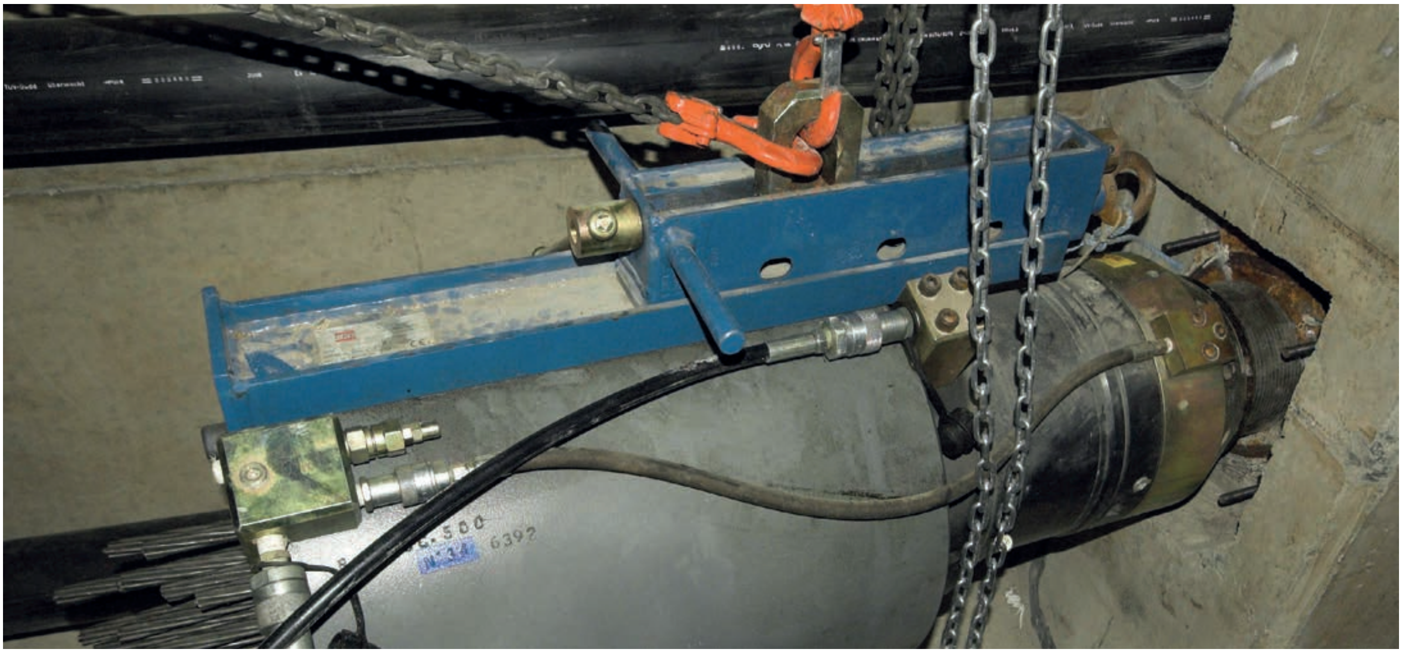

Figure 7
Anchorage blisters inside box (East Moors Viaduct, UK)

Figure 8
Internal cable ducts at pier (STAR LRTS Viaducts, Malaysia)

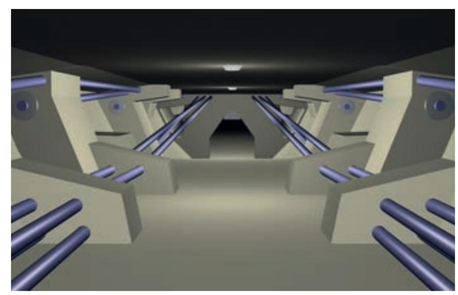

Figure 9
External cable profiles inside box (A13 Viaduct, UK)

37/15mm. Each cable sits inside its own duct which, after threading and stressing, is then usually filled with a high-performance cementitious grout. The specification and application of these grouts needs to be very well controlled in order to achieve a completely filled duct[4].

The prestress is applied to the ends of each cable via a steel anchorage, which is cast in to the concrete. Each strand is then clamped by a set of wedges that locks the strand in to the anchorage. The prestress force is applied to the anchorage with hydraulic jacks, which typically apply loads of 2-8MN, i.e. 200-800t (Figure 6). Elastic deformation, creep and shrinkage losses (all of which are lower with post-tensioning due to the later transfer of the prestressing onto more mature concrete) combine with duct/cable friction and wedge lock-off losses, and with steel relaxation losses to give similar long-term stresses in the prestressing steel, i.e. 1 000-1 100MN/m^2, or 55-60% f_{pk}. The prestress can be finely tuned to suit the required forces at every section. As such, it is common for post-tensioned bridges to have many sets of cables, each starting and stopping in a variety of locations to suit both the construction method and applied loads. Each anchorage is housed within a lump of concrete that transmits the huge forces into the main body of the member – these lumps might be at diaphragms, abutments, pockets or a variety of blisters (Figure 7). The design of these highly-stressed locations will be covered in a future article in this series. As post-tensioned cables follow the bending moments within the member, they also move up and down the section. This inclination of the cables provides a shear force, which acts against the applied shear forces, in turn providing a significant shear relief which reduces the amount of reinforcement needed in the webs.

There are two types of cable configuration – internal cables that are grouted inside ducts, which are within the concrete and bonded to it, and external cables that are also grouted inside ducts, but which are outside the concrete and not bonded to it. Each type provides a three-layer protection system to the cables using grout (or wax), the duct and the concrete. Internal cables are more compact with smaller cables, anchorages and blisters – often using 12/15mm or 19/15mm cables. They can more closely follow the pattern of moments in the member and thus have a better eccentricity and ULS performance than external cables. In the UK, ducts are required to be made from a continuous, corrugated plastic, whereas elsewhere, away from road salts in particular, ducts are often formed from corrugated, galvanised steel. Cables are located in the top slab regions within hogging zones (Figure 8), and in the bottom slab regions within sagging zones, with cables moving up and down the webs in between these regions. These bonded sections tend to be designed as fully compressed under all frequent traffic loads. This is the requirement in both BS 5400[5,6] and Eurocode 2[7]. As a result, these internally prestressed sections are generally governed by SLS, and ULS will not be critical.

External cables are generally larger with fewer anchorages. As they sit outside the concrete, they tend to follow more simple profiles and need deviator blocks at all changes of direction (Figure 9). The cables are housed within continuous HDPE ducts, which are 6-10mm thick in order to resist the grouting pressures. These large external cables - often using 27/15mm or 37/15mm cables - need large anchorages and deviator blocks, which can contain considerable volumes of concrete and reinforcement. Sitting outside the concrete, external cables have a lower eccentricity than internal cables and, being unbonded, their ULS performance is not as good as internal cables. However, they do allow thinner webs and many construction methods (such as span by span precast segmental or whole span precast) work very well using external cables, where it is quicker to install a smaller set of larger cables. They also allow the use of partial prestressing (covered in the following section). External cables were first introduced to enable the ducts/cables to be easily inspected, maintained and replaced. However, with the improved grouting technologies introduced by TR 72 in 2010, and indeed its forerunner in 1996, the need for such inspection is significantly reduced. Subject to local regulations, designers should therefore choose between external and internal systems, or a mixture of the two, on the basis of what is best for the design and construction method.

Partially prestressed bridges

As noted previously, partial prestressing is possible when external cables are used. As the cables are protected within the envelope of the concrete, the concrete section can be allowed to crack, as this cracking would have no detrimental effect on the cables. The designer thus has the option to consider a full range of prestressing, from full compression to none (i.e. just reinforced concrete). Further discussion on this topic can be found in the 2012 Milne Medal paper[8], where it is concluded that a high level of prestressing is likely to provide a better set of results. High levels of prestressing provide the benefits described in this article's introduction, whereas low levels of prestressing are not really suitable for spans over about 30m, primarily due to the much higher deflections and ongoing levels of creep (Figure 10). This ability to consider much lower levels of prestressing has arisen because of Eurocode 2, which allows more partial prestressing than BS 5400. In BS 5400, which was augmented by BD 58[9], the section had to remain fully compressed for all permanent loads and then crack widths were checked under frequent traffic loads. Other codes around the world require the section to be fully compressed for frequent traffic loads and then crack widths to be checked under rare traffic loads. However, Eurocode 2 now requires there to be no decompression check at all, with crack widths being checked under permanent loads only. Consequently the design is generally governed at ULS. Theoretically, Eurocode 2 is saying that external partial prestressing is simply a form of reinforced concrete. Practically

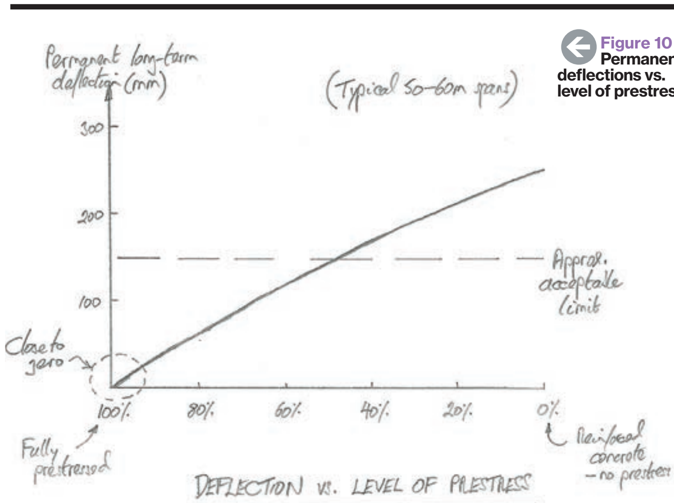

Figure 10 Permanent deflections vs. level of prestress

though, bridges with spans over 20-30m will need a significant level of prestressing in order to make them satisfactory in all regards.

Conclusions

Of all the materials available to a bridge engineer, prestressing is the most challenging as it is an active, not passive, system. One cannot simply add more prestressing steel in order to be conservative, as the addition of prestress is just as likely to be detrimental to the section as is its removal. The designer must therefore calculate all the effects along all sections of the member, and then design the prestress to counter them at all locations. This process needs a determined effort from skilful designers, and throughout, they must consider the critical construction issues, as these affect all subsequent decisions.

References and further reading

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