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. 10: Concrete bridge detailing

Introduction

This latest article from the CBDG examines the detailing that is particular to a concrete bridge. All details need to consider buildability, durability and maintainability, as well as safety and aesthetic issues[1,2]. The weather is the main source of concern for most external details, with water, often laden with salt, able to reach almost everywhere. Every surface must therefore be detailed on the basis that it will get wet – this requires a positive drainage and drip system for all near-horizontal surfaces. To protect details within the concrete, the characteristics of the concrete must be specified, and this will depend on particular site conditions. Working to these specifications will ensure that the concrete is best suited to its location, in order to provide a low-maintenance working life.

Bridge deck details

Most bridge decks are easily analysed for eccentric traffic loads using grillage or 3D frame analyses, while the torsional and distortional warping behaviour of box sections should be designed using finite element (FE) analyses or other models[3]. Moment rounding (where the peak elastic moment on a knife-edge support is reduced to take account of the actual support width and bridge deck depth) is generally ignored by most structural engineers, but with bridge spans over about 50m, it can be a significant effect reducing the peak support moments by 5–10% (Figure 1). Shear lag (where the actual longitudinal bending stresses in a top or bottom slab are higher than the traditional engineer's bending theory over the webs and lower further away from the webs, due to shear deformations in the slab) should also be considered, with most codes allowing an effective width of flange (each side of the web) of at least 0.1 times the distance between points of contraflexure in the span. In reality, this means that it never actually affects typical midspan regions. It can reduce the effectiveness of flanges in the support regions, but only for widths (each side of the web) of more than 2–3m. In these cases, longitudinal stresses are calculated using a reduced set of section moduli, though the axial prestress is still applied to the whole section.

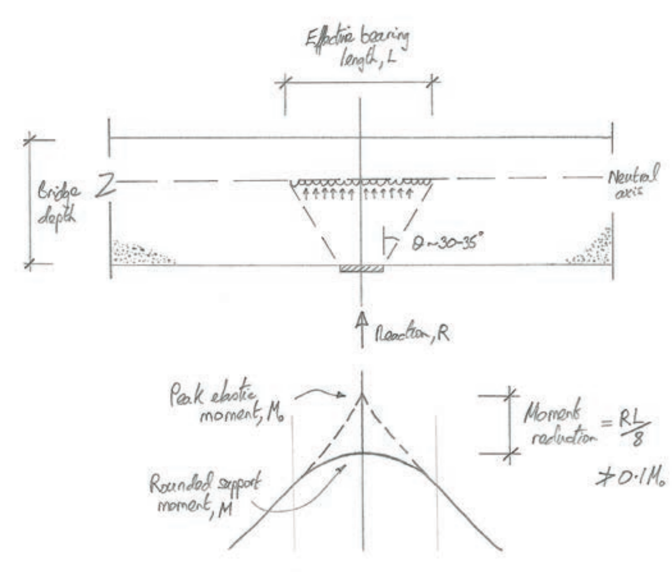


Figure 1
Support moment rounding

The combined analysis of bending and shear within slabs or webs is well described in most codes. However, it was noted in the third article in this series[4] that the haunching of slabs can help with three areas – it aids the flow of concrete during casting, provides an area where internal cables can be located, and controls the longitudinal shear stresses, allowing a better pattern of reinforcement to be detailed. Care should be taken with variable depth bridges, as they exhibit two effects that are not seen with constant depth. Curvature of the bottom slab creates an out-of-plane force that must be resisted by transverse bending in that slab. In the more extreme case of a linear-haunched beam, there is a significant point load at the kink in the bottom slab, which will need a transverse beam as the slab is unlikely to be strong enough (Figure 2). However, this inclined bottom slab also creates significant benefits. The vertical component of the longitudinal force in that slab acts to counter the shear forces, known as the 'Resal effect'. So, with post-tensioned variable depth girders, the inclined prestressing cables and bottom slab provide a large shear relief, enabling thinner webs to be used (Figure 3). EC2 incorporates a variable angle truss analogy for shear design that allows engineers to select any angle between 22–45°. For thick (or lowly stressed) webs, where concrete crushing is not critical, the optimum solution is to minimise the area of shear reinforcement by using a low angle, but no lower than 22°. However, for most bridges where self-weight is crucial and, therefore, where the thinnest webs are used, the concrete crushing is critical and the best method is to use a high angle, i.e. the traditional 45°. The third article also notes the various issues, including web thickness, around using either internal or external cables.

The most complex areas in bridges can all be analysed using struts and ties. Not only is the concept simple, but it ideally suits the linear pattern within which reinforcement is fixed. FE analyses have their role in some circumstances, and can help the designer select the best struts and ties, but they can also suggest a greater degree of precision than is really the case. Reinforced concrete is neither homogeneous nor a linearly elastic material – cracking produces a material that is generally much better represented by the struts and reinforcement ties of this simple method. EC2 also points the designer in this sensible direction and Schlaich and Schäfer[5] have an excellent summary. Diaphragm areas are generally the location where many forces are at play – they are the culmination of bending, shear and torsion in the beam, provide the lateral stability to the section, carry the reactions down to the piers, and are often the locations where prestressing cables are anchored. They also provide access, during construction and for maintenance, and are therefore penetrated by openings as well. Nevertheless, it is relatively simple to draw struts and ties that resist all these actions (Figure 4). Once these actions have been combined (using the co-existent not the maximum effects) the designer must detail the reinforcement so that it can be easily fixed. 3D sketches, drawings or models can help enormously with this process (Figure 5). The areas above bearing locations are detailed in a similar way to prestressing anchorages. There

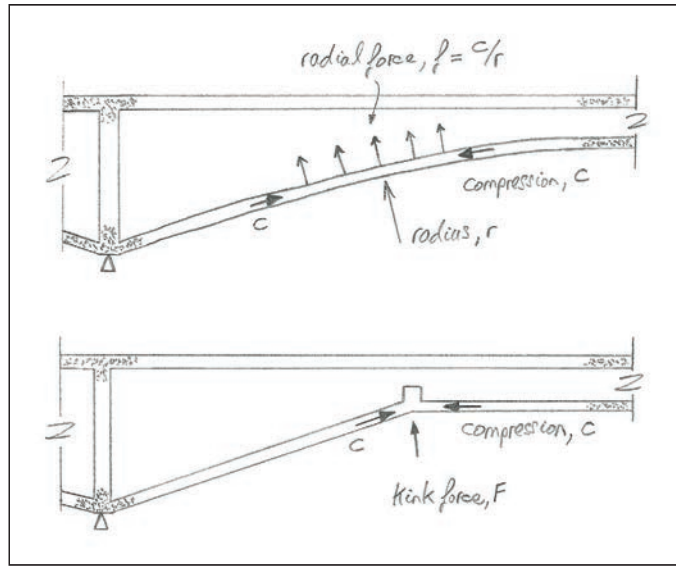


Figure 2 Variable depth beams and bottom slabs

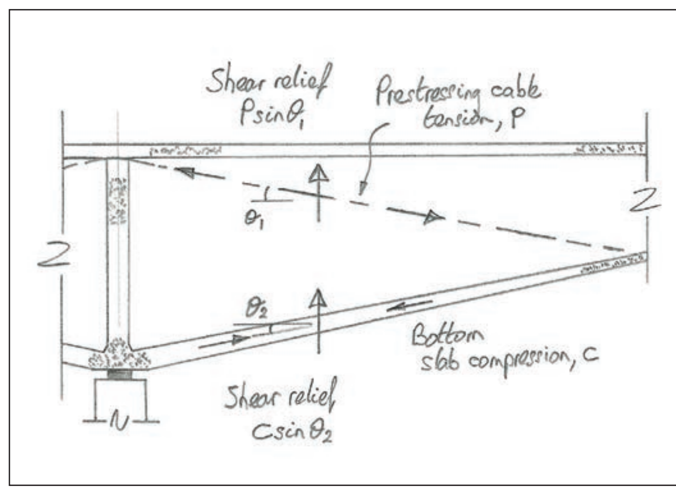


Figure 3 Variable depth beam shear reliefs

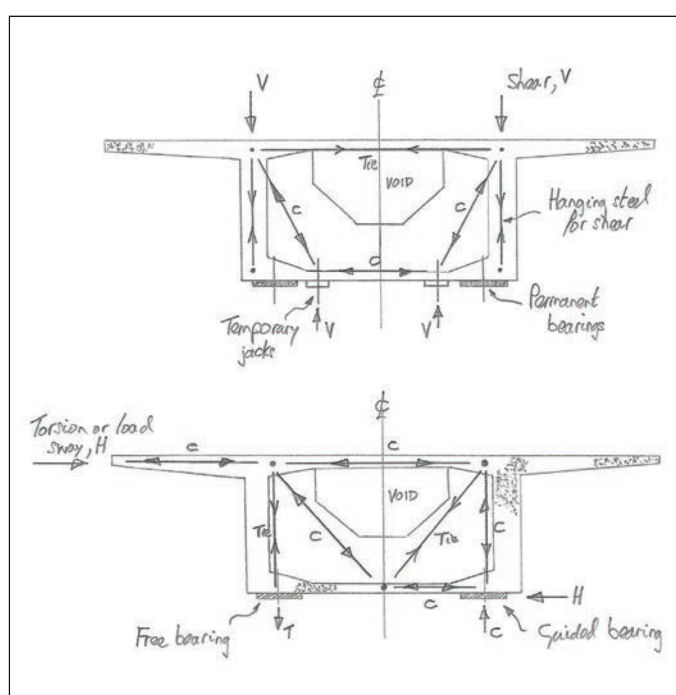


Figure 4 Diaphragm strut-and-tie analyses

are bursting forces above the bearing that may need a block of concrete reinforced in both directions and the overall equilibrium is then carried by struts and ties into the bridge – the same principles apply to pier tops (Figure 6).

Other significant areas of detailing are described in the literature. Suffice to note that the reader should take adequate reference from these sources. In particular, the detailing within seismic zones needs careful consideration – the location of plastic hinges in the system is chosen by the designer and this hinges are designed and detailed to fail in a controlled manner, which needs well-anchored reinforcement and contained concrete. Similarly, the detailing for major vehicle or ship impacts needs a careful analysis of the energy absorption and ductility characteristics of the system.

Reinforcement details

Good reinforcement detailing is covered by many documents and, indeed, modern codes tend to outline key areas of recommended detailing[6,7]. This article therefore covers the particular issues relevant to concrete bridges. The minimum bar size is usually H12, although H10s can be used in some nominal areas, whereas structural reinforcement should generally consist of H16s or H20s. In heavily loaded areas, these bars might become H25s or H32s. H40 bars would usually only be used in large beams or columns, or in pads/pile caps. H50 bars should not generally be specified, as they are too heavy to fix. The best bar pitch is generally 150mm (or possibly 125mm in more heavily loaded areas) as 100mm is too close together to accommodate good placing of the concrete around laps, while 200mm may be too far apart for adequate control of cracking. An old rule of thumb for minimum steel is, 'If in a fix, use half inch at six', which in today's parlance is H12s at 150mm, and still a wise rule. The requirements to limit early thermal cracking are also generally covered by this rule for most thin sections.

Sections that are more than about 500mm thick will need slightly more steel, with H16s at 150mm being typical. This is the minimum steel needed within the surface zone to prevent yielding and limit crack widths. Another excellent rule of thumb is to never use bars that are more than 1/10 of the section depth, i.e. do not use H32s in a 200mm slab – the maximum bar size should be 20mm. The fourth article in this series[8] describes the various options at construction joints, where it is possible to use either starter bars and laps, or couplers. Some couplers can have a relatively poor fatigue performance and should only be used in areas close to repeated traffic loads after careful consideration.

Post-tensioning details

As noted in *Prestressing for concrete bridges*, the most common strand is a superstrand, having a 15.7mm diameter and an ultimate strength f_{pk} of 1860MN/m^2. Typical cables for bridges may be formed from 12–37 strands, and the four most common are designated as 12/15, 19/15, 27/15 and 37/15mm. Internal cables are placed within corrugated plastic or galvanised steel ducts having internal diameters of 80, 100, 115 and 130mm respectively. External cables are placed within slightly larger high-density polyethylene (HDPE) ducts. After stressing, all ducts are usually filled with a high-performance cementitious grout, in accordance with the Concrete Society's Technical Report 72[9] in the UK. Single-end stressing is always preferred as it uses less labour and fewer jacks, but the viability depends on the friction losses, especially for internal cables. Most cables less than about 50m long can be stressed from one end, while over this length, it is generally necessary to stress the cable from both ends. Internal cables can be up to 150m long, whereas external cables can be much longer (up to 400m in some cases) as they have less friction. The third article gives further detail about the various losses in the prestressing force that will determine the optimum cable lengths.

Typically, ducts are spaced at two diameters, while the minimum cover to internal ducts is also generally a duct diameter. The alignment of all ducts should follow as smooth a profile as possible to avoid excessive friction losses. Profiles usually consist of straights and radii as these are the easiest to set out. Minimum radii are dependent on the cable size, but vary from 5–10m for internal cables and approx. 2.5–5m for external. There should also be a minimum length of straight duct immediately behind each anchorage, of 1.0–1.5m. There are significant in-plane radial forces at these curves, which for internal cables are usually carried by the concrete in compression, although every curve needs to be checked and tied back with reinforcement, if needed. There are also out-of-plane spalling forces at these positions, though these are generally resisted by the existing web or slab reinforcement found in bridges. External cables are straight, except at deviator positions, where the in-plane radial forces are carried into the body of the section, using compression struts, tension reinforcement or beams.

Post-tensioning anchorage zones

Individual strands might be stressed to around 20t (0.2MN), but post-tensioning cables require jacking loads of 250–800t (2.5–8MN). These enormous forces need to be safely applied to the bridge within anchorage zones. Many will be within the length of the bridge, while some will also be at abutment ends. Once stressed and grouted, all these anchorages need to be protected with wax or grout. The abutment anchorages need particular attention as they can be exposed to the weather. External cables are usually left accessible, whereas internal cables are generally capped with a further layer of reinforced concrete. Both cable types are then waterproofed and sheltered within an access chamber. The practicalities of construction also determine much of the layout as the jacks are of a significant size, e.g. 300–800mm in diameter. The width of concrete needed for the anchorages and the space needed for the jacks can therefore determine many of the concrete dimensions in these areas. The stresses behind anchorages can be close to the concrete strength and therefore the bursting issues are dominant. Besides the need for any bending or shear steel in these areas, there are three types of additional anchorage zone steel – bursting (or splitting), equilibrium and spalling.

The bursting stresses depend on the

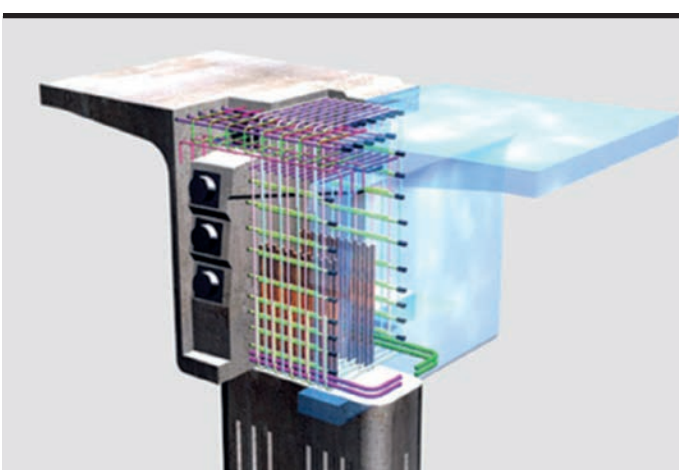


Figure 5 Diaphragm reinforcement model

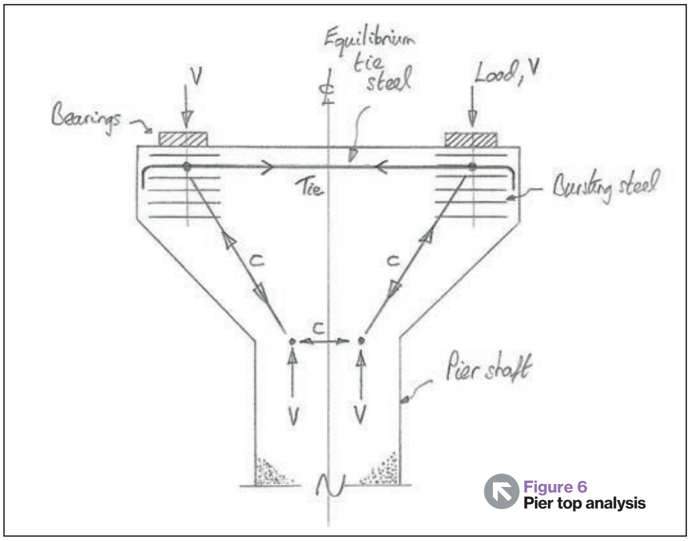
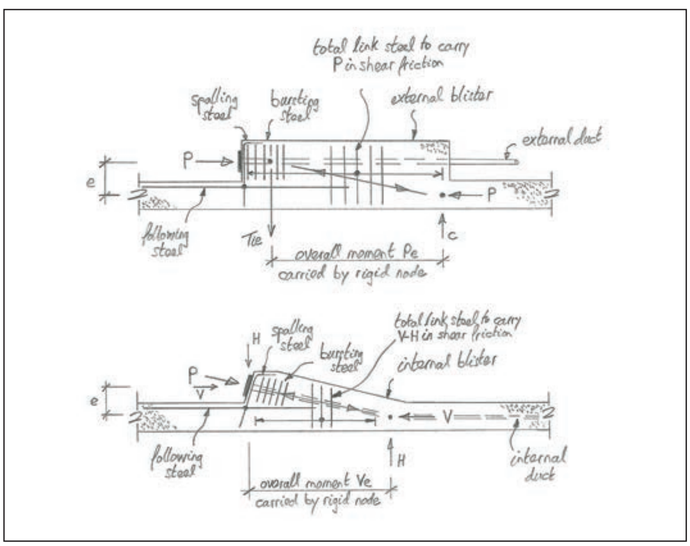


Figure 6
Pier top analysis



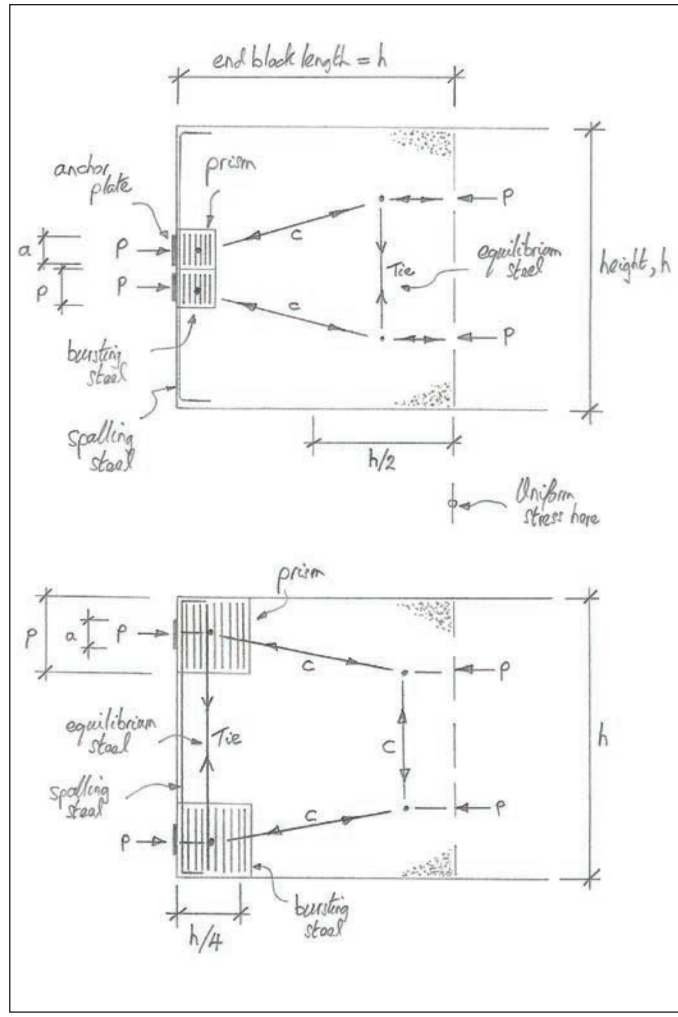


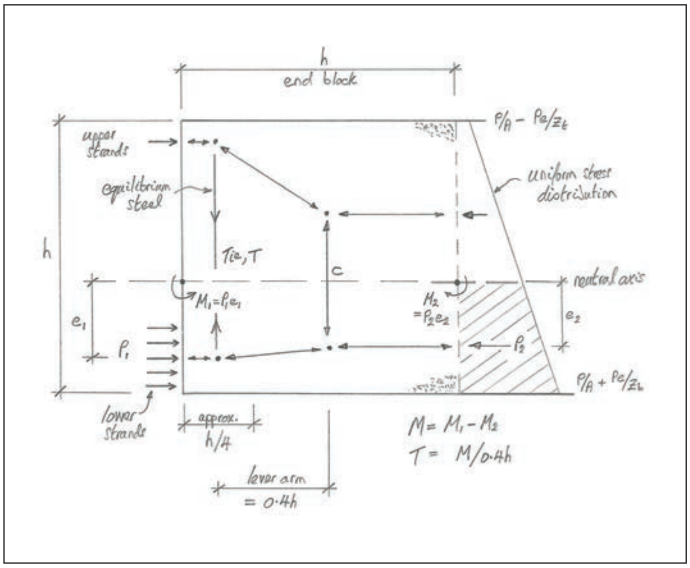
Figure 8
Post-tensioned blisters

Figure 7
Post-tensioned end zones

size of the anchorage a with respect to the size of the concrete prism p to which it is applied. Much research was done on these stresses in the 1950s and 60s, which showed the bursting force $T = 0.3P\,(1-a/p)$, where P is the prestressing anchorage force. The bursting force therefore varies from zero (for a uniformly applied load) to $0.3P$ (for a point load). CIRIA Guide 1[10] uses almost exactly the same equation and is still the best guidance note for this whole topic. Reinforcement to resist this bursting should be applied over the length of the prism, in both directions if needed. This is often in the form of spirals directly behind each anchorage. Equilibrium steel is the additional steel that might be needed to transmit the prestress force from the anchorage prisms into the section, again in both directions. The CIRIA guide shows a deep beam method based on the equilibrium of a square end block. Figure 7 show two configurations that highlight the differences between bursting and equilibrium. Equilibrium can also be analysed using struts and ties, assuming that the prestress spreads out at no more than 30–35°. Finally, there is the spalling steel. This is a nominal requirement on the end face to accommodate any surface tensions, and the CIRIA guide suggests that steel should be provided to cater for $0.04P$.

These anchorage zones are the most highly stressed areas of the bridge and are often the most heavily congested. It is therefore important both to understand all the actions, so that only the correct and co-existent loads are considered, and to detail the areas with great care; otherwise they can become unnecessarily congested and prone to poor compaction, whereas well-compacted concrete is vital. Smaller, well-anchored bars such as H12s and H16s are much more likely to provide better resistance than larger bars. It will generally be necessary to consider the stressing sequence in the area, as well as all stages of construction, as more critical conditions than the long-term might be found in these short-term load cases. A judicious choice of the order in which the cables are stressed will reduce the overall anchorage zone reinforcement. Further discussion on these issues can be found in a paper by Bourne[11].

Where cables are anchored along the length of the bridge, as opposed to an end face, there are additional effects at these blisters. The same issues of bursting, equilibrium and spalling still apply, but the designer should also take account of any corbel action using a strut-and-tie analogy, especially for external blisters. Blisters, which are generally only found in box girders, are best placed at the intersection of a web and slab, i.e. where there is sufficient rigidity of the node in both directions to carry the overall moments. Even though the compression from the anchorage passes into the section, there will always be a tension near the anchorage due to the continuity of the section. Accepted practice is that well-anchored following steel should be provided at this interface to carry $0.25–0.40P$ (Figure 8). Prestressing cables can also be



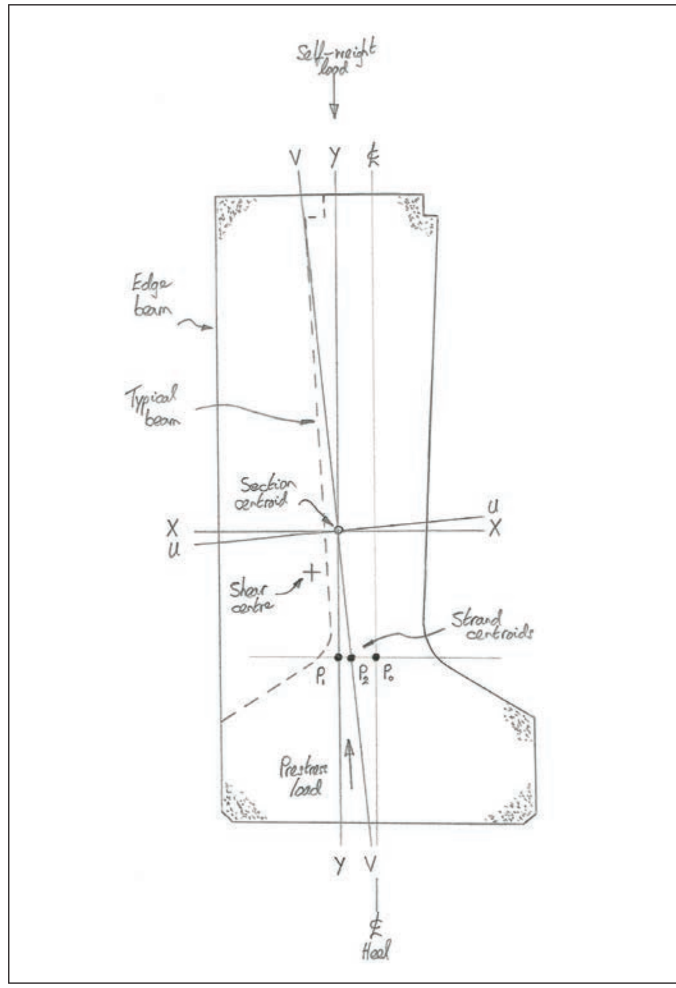


Figure 9
Pre-tensioned end zones

Figure 10
Asymmetric edge beam analysis

coupled, although the more common practice is to lap cables using blisters – this is generally a more reliable and economic method. If couplers are used, then ideally no more than a third of the cables should be coupled at any one location, although EC2 allows 50%.

Pre-tensioning details

Pre-tensioning uses the same strand as post-tensioning, i.e. 15.7mm superstrands. The strands are generally treated in the same way as reinforcement as regards cover and spacing, although EC2 quotes an extra cover of 10mm. Typically, in pre-tensioned precast beams, the cover is 50mm and the spacing of the strands is 40–50mm. The prestress is applied to the ends of the member by bond action, resulting in a length over which the force is transmitted (500–1000mm). Older codes quoted transmission lengths of around 30 diameters, whereas EC2 suggests figures of up to 60–70 diameters. In reality, the figures must be closer to 30–35 diameters, otherwise the 2.6m long pre-tensioned railway sleepers that are used worldwide, where the peak moment is about 400mm from the end, would not work – and they do!

The analysis of stresses in the end zones of pre-tensioned members is not really covered in the codes, but the basic principles of bursting, equilibrium and spalling still apply. However, the strands are more evenly spread out than post-tensioning anchorages and the transmission length applies the loads more gradually. So, the effects are much less pronounced than with post-tensioning. As seen in the CIRIA guide, if the applied loads are uniform, then there is no bursting or equilibrium, and indeed no spalling. Some pre-tensioned beams that are used in integral bridges do have a distribution of strands that closely matches the prestress in the beams, resulting in no particular issues within these end zones. However, most beams would tend to have the majority of strands placed within the lower heel or flange, which will generate equilibrium effects, which will try to split the web horizontally. An International Federation for Prestressing (FIP) paper[12] outlines some guidelines and basically uses the same method as the CIRIA guide, by calculating a resultant moment M at the neutral axis from the applied prestressing forces M_1 and the resulting prestressing forces once they have spread out M_2, i.e. $M = M_1 - M_2$. The total tension force in this deep beam arrangement is $M/0.4h$, where h is the height of the beam (the CIRIA guide uses $0.5h$). The reinforcement to carry this force should be distributed over half the transmission length, i.e. the first 250–500mm (Figure 9). Applying these simple equilibrium rules to a range of pre-tensioned beams reveals equilibrium forces of 0–0.10 times the total prestressing force. However, the American Association of State Highway and Transportation Officials (AASHTO)[13] specifies that reinforcement to carry 0.04 times the total prestressing force should be provided within a zone that is $h/4$ from the end, i.e. over 200–600mm, but the code limits the stress to 140MN/m^2. Overall, the FIP/CIRIA method and the empirical AASHTO rules are likely to generate similar amounts of reinforcement in the same location. Though all the strands are usually released at the same time, the actual order in which they are released can also have a significant effect. As with post-tensioning, small well-anchored bars such as H12s are likely to be the best type of reinforcement.

With pre-tensioned, asymmetric edge beams (Figure 10), it is important for the designer to place the centroid of the strands close to the lateral centroid of the section, i.e. close to P_1 on the Y-Y axis, not on the centre of the heel P_0. This will minimise any lateral deflections (and additional lateral stresses) that would otherwise occur. As the section is asymmetric, the design should actually be carried out using the principal axes (U-U and V-V). It might be assumed that the strand centroid should then be positioned on the V-V axis, i.e. at P_2. However, even though this position causes no prestress deflections about the V-V axis, it still has a component about the Y-Y axis. The self-weight loads also cause deflections about the V-V axis (with a Y-Y component). Taking the prestressing and self-weight loads into account, it can be shown that the correct position of the lateral strand centroid (for zero lateral deflection and stress) is between P_1 and P_2, but generally quite close to P_1. The best guidance therefore is to place the strand centroid at P_1 for any edge beam with a span less than 25–30m. For any longer, more

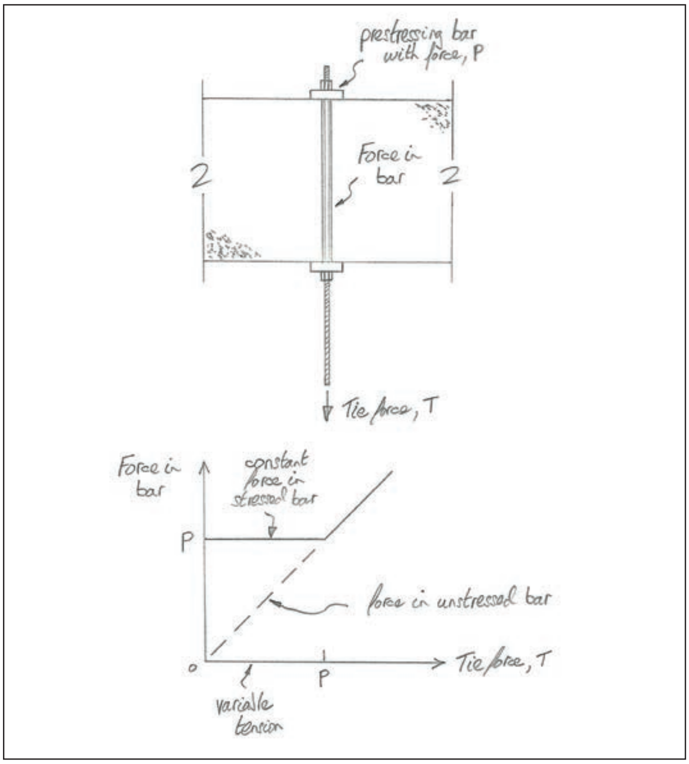


Figure 11
Temporary prestressing bar forces

slender or heavily prestressed edge beams, a proper principal axes calculation should be carried out to determine the optimum position. The vertical behaviour can always be adequately calculated using the X-X axis, and once the beam is cast with the deck slab, it is forced to only deflect vertically anyway. The torsional behaviour of the edge beam is very rigid and even though there might be some eccentricity of the self-weight and prestressing loads to the shear centre, the torsions are very small and the torsional rotations negligible.

Temporary details

As described in the fifth article in the series[14], temporary prestressing bars are frequently used to clamp elements together, providing a quick and easy method of generating stability. The common bars are 26.5, 32, 36 or 40mm in diameter and are stressed to about 70% of f_{pk} (i.e. about 700MN/m^2). All these connections are held rigid by the bars, where the force in the bar stays constant, until the prestressing force is exceeded by the tie force (Figure 11). This is a good example of prestressing, as it shows that the concrete carries the tension, not the prestressing bar. Similarly, bars are used with lifting beams to move precast units, where the clamping action ensures a safer and more rigid lifting process. Bars tend not to be used as part of the permanent works on economic grounds, as their f_{pk} is nearly half that of strands for about the same price and, being rigid, they are much less capable of accommodating the moment variations seen within bridges.

Conclusions

The careful detailing of concrete bridges ensures that the construction process is safe, easy and quick, and the bridge is durable and easily maintained for its entire life. Many details require the designer to only consider the correct co-existent effects, which needs a good understanding of the principles of reinforced and prestressed concrete. As with most design and detailing, the drawing of sketches and the preparation of 3D models will help the designer identify all the relevant issues and create good solutions.

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