SEGMENTAL AND STAGE CONSTRUCTION OF PRESTRESSED CONCRETE BOX GIRDER BRIDGES

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Prestressed Concrete Bridges of uniform or near uniform cross-section are admirably suited to construction by precasting segments and post-tensioning. This form of construction, however, introduces additional design and construction problems - problems of carrying flexure, shear and torsion across joints - the most suitable type of joint, - problems introduced by superelevation and constructional tolerences. As the length of the structure increases, the effective prestress is reduced markedly until it is no longer economical to stress in one operation. The method of stage stressing will overcome these problems, however more complex analysis and design is required. Solutions to these problems are discussed with particular reference to several large prestressed concrete box girder bridges recently constructed in Australia.

The use of prestressed concrete for medium span structures has increased dramatically over the last decade, and now is the predominant material of construction used in the 15m to 40m span range. Geometrics imposed on the bridge designer by the freeway and highway designer have forced the change from straight simply supported bridges to bridges with complex geometry. The hollow box girder bridge is admirably suited to these conditions and has the additional advantage of its aesthetically pleasing lines, a highly desirably property for urban structures.

The simple prestressed concrete box girder has evolved rapidly into the sophisticated bridge with many more design and construction developments. Two of these developments, segmental construction and stage construction will be reviewed. Design and construction problems and solutions to these problems will be discussed with particular reference to recent bridges constructed in Brisbane, Australia.

Two structures will be discussed within this paper.

(a) Nyanda Overpass
(b) Roma Street - South Brisbane Rail Link

Nyanda Overpass

Nyanda Overpass (1) is a 388.3m long elevated prestressed concrete box girder bridge carrying four lanes of traffic over an interstate railway line. The bridge replaced an existing level crossing. Although the natural surface levels suited a tunnel structure under the railway, the railway line being on the crest of a small ridge, the extensive cut required for the approaches seriously affected access to an adjacent heavy engineering works and existing surface streets. The additional construction problems of maintenance of railway and road traffic, and in fact, tunnelling under three railway lines added weight to the final decision to use an elevated structure. The minimum vertical clearance over the railway line of 5.2m had to be provided, and as this was above the crest of the ridge the structure had to be quite long in order to keep the approach grades below 5%. The layout of the overpass is shown in Figure 1.

The superstructure has a basic span of 36.6m with variations to allow for railway and surface street clearances. A multicell hollow box spine with cantilevers was selected for the superstructure, the depth of the box being 1370mm. (Figure 2). The small temporary construction depth allowable over the railway line precluded the use of falsework or erection trusses necessary for insitu construction and therefore, precast segmental construction was selected. To reduce site operations the boxes were cast full width with the exception of the pier diaphragm units. The maximum weight of the units (45 tonnes) was determined by the available lifting equipment. With the exception of the diaphragm units all joints were nominally 125mm wide unreinforced. A uniform external mould was designed to reduce the casting problems.

The use of precast segmental construction offered the following additional advantages:

(a) the units are manufactured under factory conditions, with the resulting higher standard of quality.

(b) at the time of prestressing the units, the losses due to creep and shrinkage are those relevant to fully cured concrete, and will generally be less than for an equivalent insitu design.

(c) Construction time can be reduced as the precasting can be carried out simultaneously with substructure construction.

The superstructure is anchored at each abutment with all longitudinal movements accommodated at a single expansion joint in the middle of Span 6. This expansion joint is designed as a hinge and carries transverse shear.

With the total length of the superstructure between abutments and the hinge being 169.75m and 218.55m respectively friction losses in the cables as the prestress is applied were excessive, and residual prestress after all losses was at an uneconomically low level.

For this reason the superstructure was designed to be constructed in a number of self contained stages as shown in Figure 3.

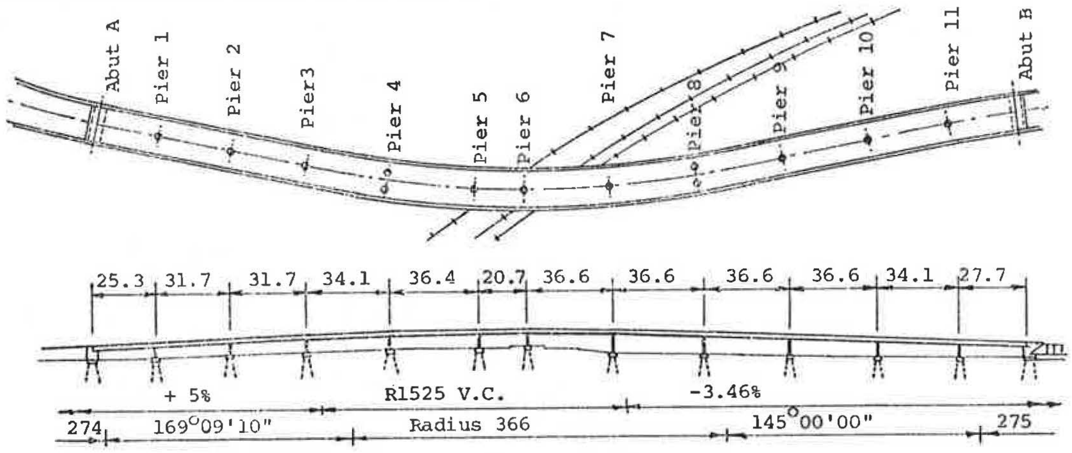
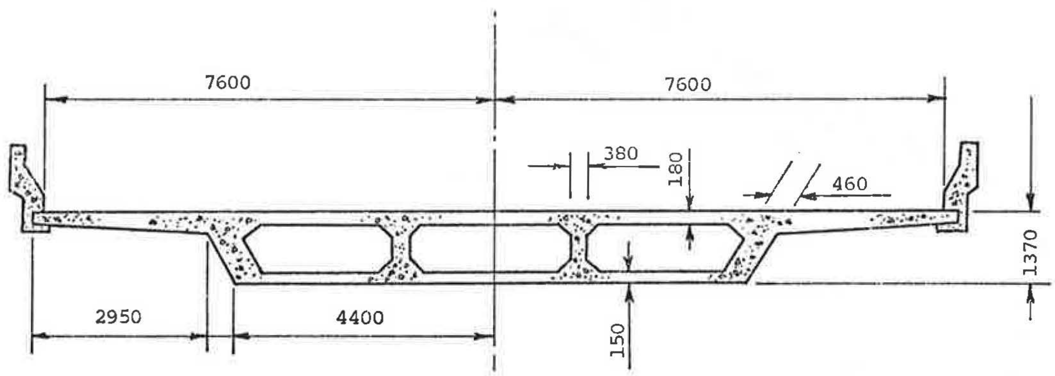


FIGURE 1



Typical Section

FIGURE 2

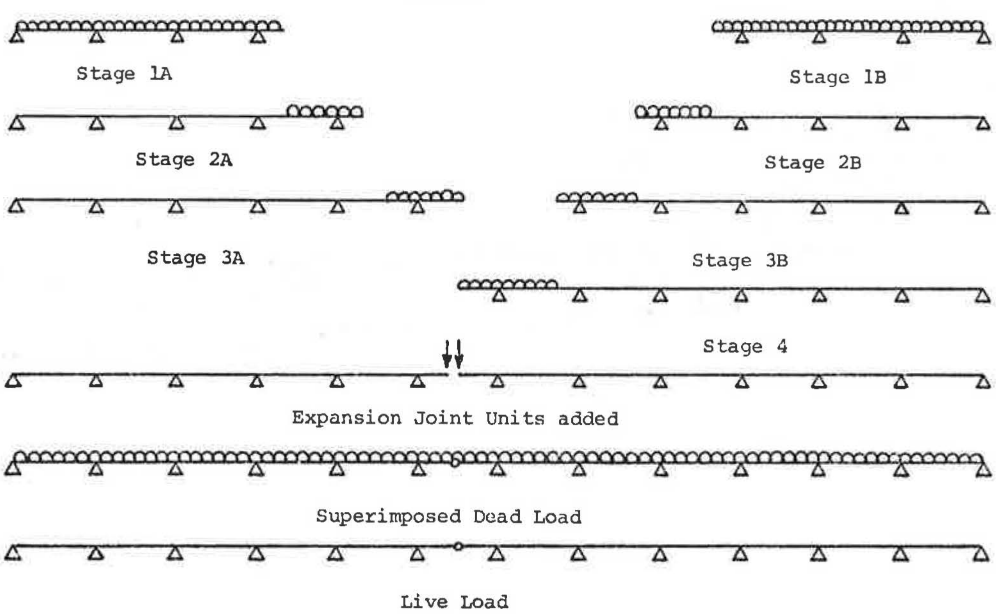


FIGURE 3

Roma Street - South Brisbane Rail Link

The city of Brisbane is located on the Brisbane River which splits the city into two. The city is served by a number of road bridges, however there has been only one railway crossing for many decades. This existing railway crossing carries suburban railway traffic to the western suburbs and western towns. More recently the population expansion has been to the south and south-east of the city, thus imposing heavy demands on public transport. Brisbane has for many years been served by two major railway stations - Roma Street serving the suburban lines on the north side of the river, and all intrastate lines, and South Brisbane serving the suburban lines on the southside of the river, and the interstate line to Sydney. Suburban and intrastate lines are narrow gauge while the interstate line is standard gauge.

The design for the 2km link was commissioned in 1972 and construction is currently in progress. The link consists of 642m of elevated vaiduct, a 132m steel tied arch across the river, two through bridges over surface streets of 60.1m and 29.2m respectively, 117.4m of embankment, and 111.4m of cut and backfill tunnel (see Figure 4).

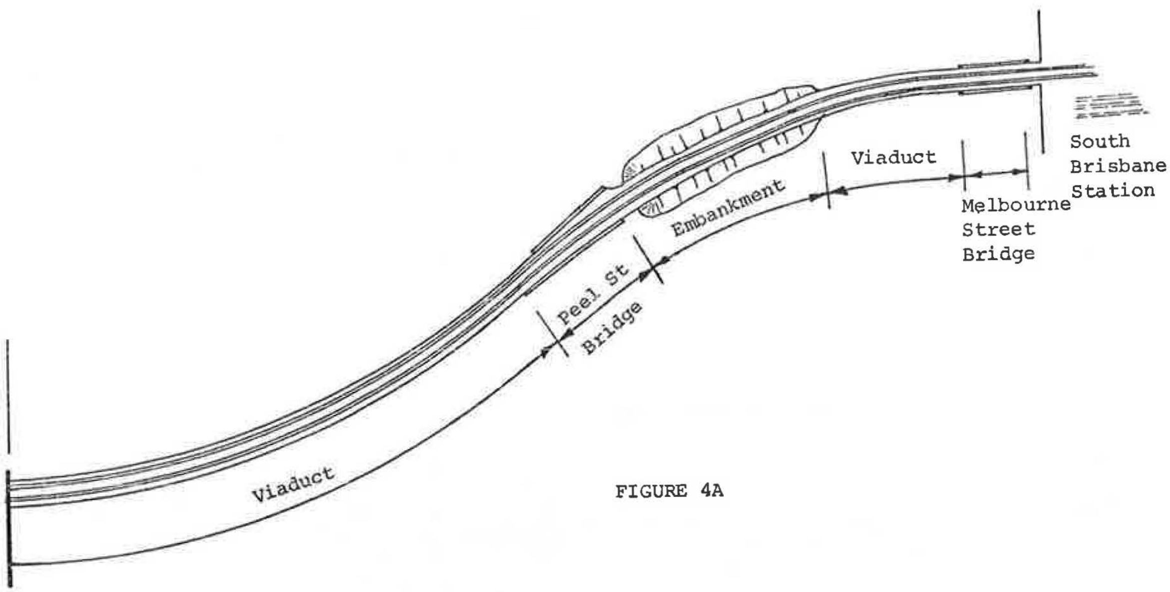


FIGURE 4A

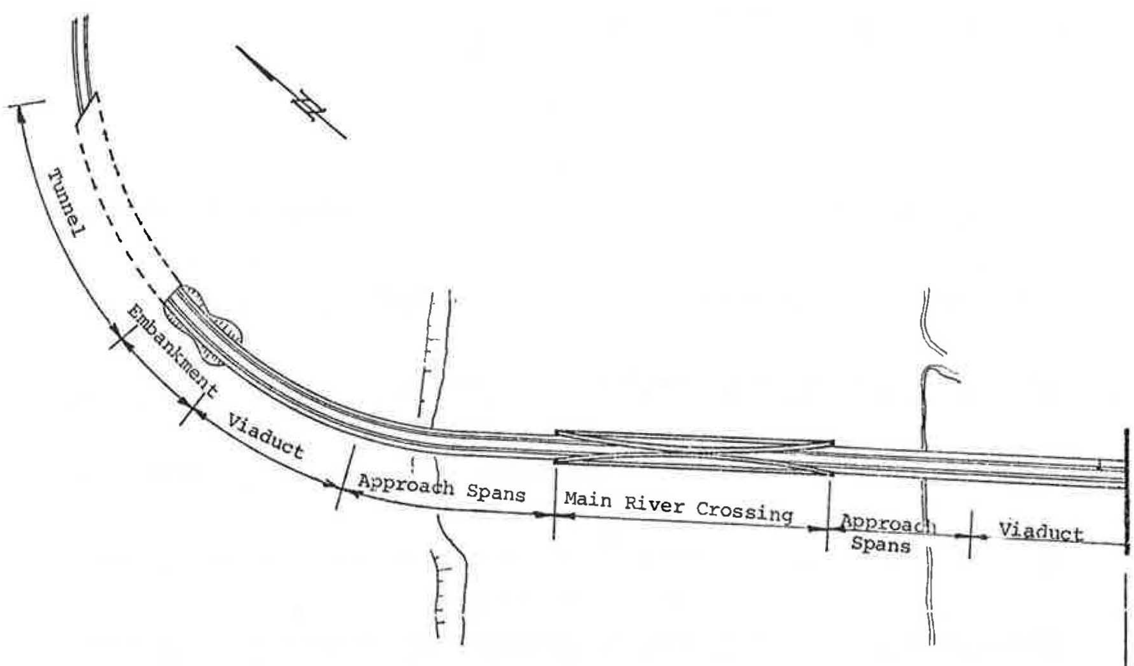


FIGURE 4B

This paper will be concerned with the design aspects of the elevated viaduct sections.

The viaduct has been constructed in a number of sections, these being:-

Section	Span (m)
A	22.5, 25.9, 22.9
B Approach Spans	28.2, 33.9, 33.9
C Approach Spans	33.45, 33.9, 28.2
D	19.1, 30.0, 23.0 22.1, 27.6, 20.0, 20.0
E	7 spans @ 20.0, 1 @ 22.0
F	20.22, 25.0, 20.22

Twin single cell box girders were used for section A, D, E, and F each girder carrying one track, while a single multicell box was used for the longer approach span sections B and C. The boxes had a constant depth of 1830mm, with a spine width of 2500mm for the single cell boxes and of 6500mm for the multicell box. (see Figure 5).

Variations in overall width due to curve widening were accommodated by varying the lengths of the cantilevers. The superstructure was cast insitu and post-tensioned with high capacity cables. The insitu superstructure was selected because it offered a number of advantages:

(a) Only a small proportion of the viaduct crosses surface streets and interruption to road traffic flow would be minimal
(b) Shuttering is simple
(c) Heavy lifting equipment is not required.
(d) Previous experience had shown that insitu construction in urban areas was less costly than precast segmental construction.

Section A, B, C and F were cast and stressed full length, while the stage method of construction was used for sections D and E. The stages are shown in Figure 6.

Segmental Construction

The method of segmental construction offers a number of advantages over the insitu method.

(a) Falsework and shuttering is considerably lighter
(b) Considerable economy in formwork design - particularly for uniform sections
(c) Factory casting conditions allow better quality control and thus higher strength concrete may be used
(d) The majority of the shrinkage has occurred at the time of erection

Contra these advantages there are a number of disadvantages:

(a) Heavy site lifting equipment is required for erecting the units
(b) Effects of horizontal and vertical alignment are difficult to accommodate
(c) Casting facilities must be available in close proximity to the site or transport costs will be excessive.

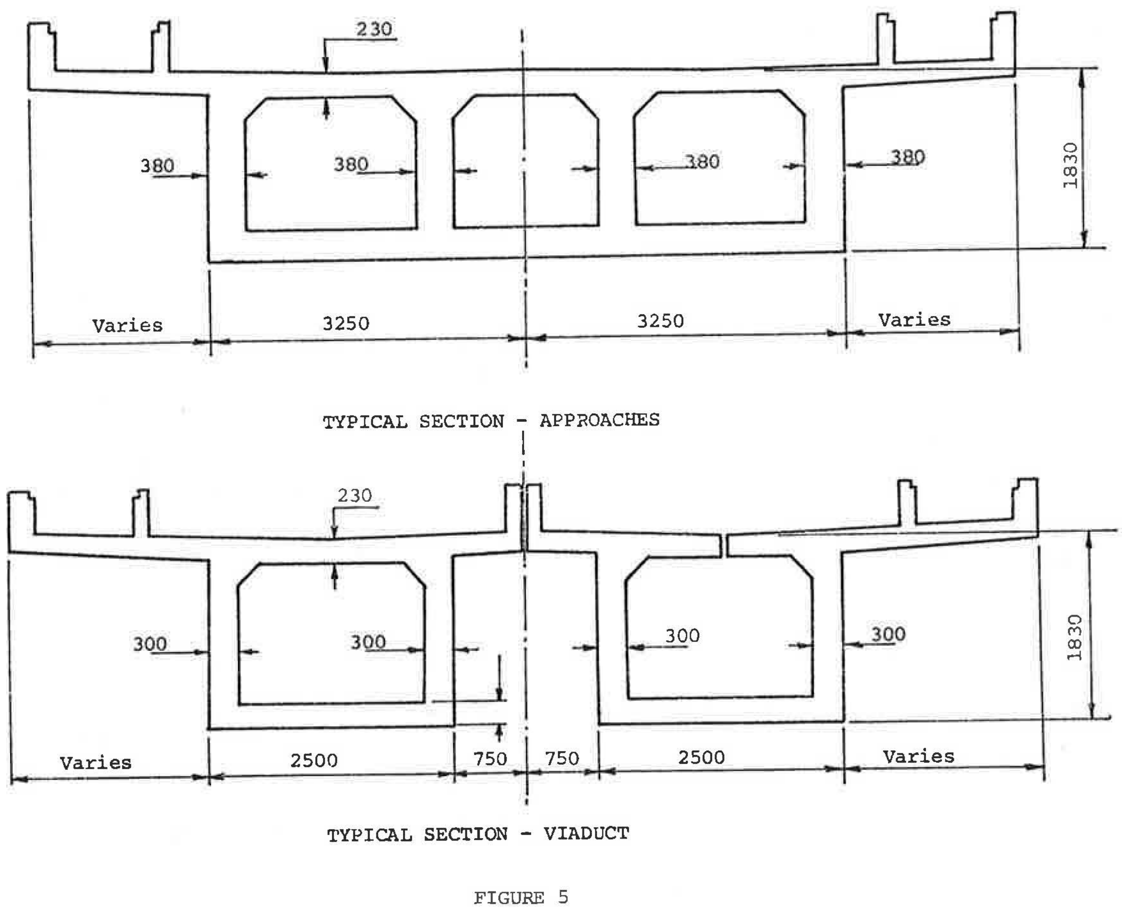
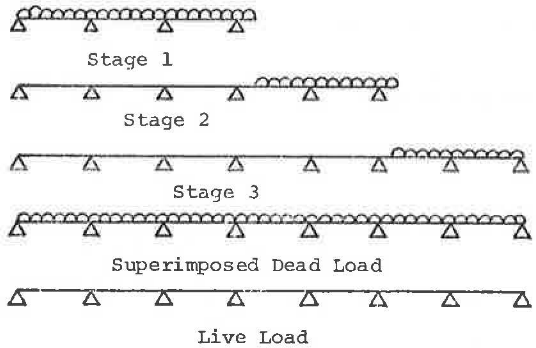
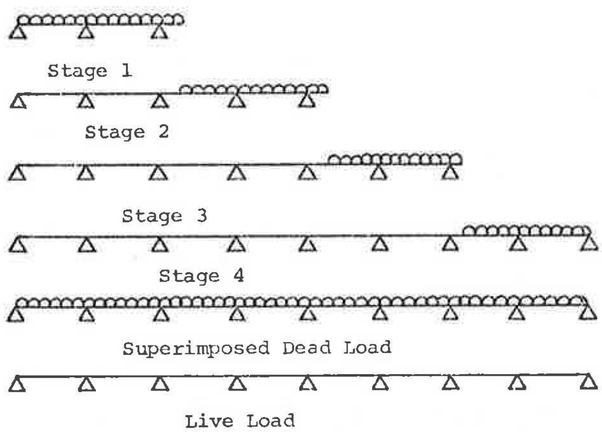


FIGURE 5



SECTION D



SECTION E

FIGURE 6

Casting of Units

The precast units may be cast either vertically or "right way up". The advantages of casting vertically are:-

(a) Easier to place concrete
(b) Finishing off is easier since only one end of the cross-section need be finished, and this will be sand blasted prior to final assembly.
(c) Storage requirements are less
(d) No windows and chutes are required to enable placing of the concrete in very deep sections.

Advantages of cast "right way up" are:-

(a) Easier to place formwork, since all the soffit forms are fixed
(b) Easier to place reinforcement
(c) Requires less formwork
(d) No turning mechanism is required to rotate the segments to the horizontal.

Both methods of casting are used extensively and it would appear to be purely a matter of preference by the individual contractor which method is used. For long structures with a unifrom cross-section it is often economical to design special purpose formwork. Hydraulically controlled steel formwork has recently been developed in Germany. This formwork may be stripped by only one semi-skilled operator, and formwork is collapsed and retrieved in a matter of minutes.

Joints

Joints between units may be either reinforced or unreinforced. The reinforced joint will allow diagonal tension to be carried across the joint, however, in order that account may be taken of the continuity of reinforcement, a positive tensile splice is required. This may be achieved by the welding of mild steel reinforcement either by a butt weld (with costly preparation) or by fillet welding to an angle of equal tensile strength. The size of the joint in this instance will be determined by the length of fillet weld required to transmit the tensile force in the bar. As welding of cold work reinforcement is not generally permitted, a positive screw type connection would be required to splice cold worked bars. The cost of these connections will far exceed the benefit gained by using cold worked reinforcement in lieu of mild steel reinforcement. The plain unreinforced joint is simpler to form and cast, but cannot be relied on for the transmission of tensile stresses. Joint thicknesses typically vary between 75m and 150mm, however the larger joints have shown a tendency to crack under shrinkage. Although these cracks close on tensioning they do initiate leakage paths for grout and provide an initial crack which is easily propagated by principal tension.

Torsional Strength of Units

The box girder section is used principally for its extensive torsional strength, a property which makes it invaluable for structures with horizontal curvature, and where eccentric live loads produce large torsional moments. Although the webs and to a lesser extent the deck and soffit slabs have physically large dimensions (up to 600mm) their relation to the overall dimensions makes the assumption of thin walls quite valid. Direct shear and torsional shear stresses may then be calculated on the basis of the thin wall theory (2) used for steel box sections assuming the shear stress to be constant across the thickness. Principal tensile stresses should be calculated at the critical points of the webs and flanges as shown in Figure 7.

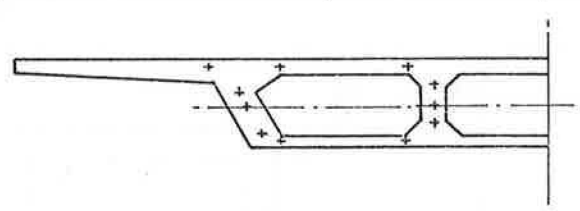


FIGURE 7

On the Nyanda Overpass calculations were made for the co-existing live load longitudinal moments, direct shears and torsion, caused by two lanes, three lanes, and four lanes loaded. The principal tensions were limited to $3\sqrt{Fc'}$. This stress limitation is in itself insufficient, as any principal tension will propogate the existing shrinkage cracks in the joints. The opening of these cracks at the surface and hence the opening of the unreinforced joint can be minimised by ensuring that the orthogonal plane to the direction of the principal tensile stress is always at an angle, preferably greater than 15°, to the joint.

An additional problem in the segmental construction caused by torsion is the transmission of the longitudinal hoop tension across the joints. This is of particular importance at abutments and those piers where a torsional restraint is supplied. In these areas of high torsion it is necessary to provide continuity of the longitudinal reinforcement across the joint, and a full strength welded connection should be used. where the torsion moments are lower it is not necessary to use continuity reinforcement provided the longitudinals are anchored adequately. The unreinforced joint should be checked for its capacity to resist the applied torsion.

Prestressing Ducts

The use of short precast segments of up to 3m in length enables the cable ducts to be straight with all angle changes in the cable profile accommodated at the joint. The straight ducts can be set extremely accurately by inserting steel mandrels through the ducts, and locating them through prepositioned holes in the end forms. This stiffening of the ducting ensures that there will be no movement of the ducting during the placing and compacting of the concrete and as a consequence the parastic angular deviation (previously known as wobble coefficient) is quite low. Site measurements indicated extremely low values of parasitic angular deviation varying from 0.025 to 0.0025 (or 0.003 to 0.0003 per metre in the old terminology).

Horizontal Alignment and Superelevation

Structures set to horizontal curves, reverse curves or transitions provide additional problems at the joints. If the units are adjusted to take up these effects the economies achieved through uniformity of formwork will be largely reduced as each unit will require special attention to the end forms. The other alternative is to cast the units with identical external forms and take up the length variations in the joint. This was the solution chosen for the Nyanda Overpass, however, many problems were encountered during the erection stage. The Nyanda Overpass is on a reverse curve (see Figure 1) varying from 275m radius to 366m radius. The units were set with their centre lines radial and on the smaller radius the nominal 125m joint reduced to 55mm. The change in superelevation produced a sawtooth appearance with a discontinuity of 6mm to which can be added the construction tolerance of $\pm$ 6mm. This effect was even more pronounced on the inside of the curve where the joint width had been reduced. The effect at the tips of the cantilevers is masked by the parapet overhang, however at the spine of the box it is quite noticeable. The change in superelevation gave a discontinuity of 3mm at the main longitudinal cables and 6mm at the slab cables. This discontinuity was particularly important at the slab ducts which were only 19mm thick carrying a 12.5mm strand, thus allowing no tolerance for construction.

Stage Construction

Continuous bridges with more than three spans and total length longer than about 100mm suffer very large friction losses during the stressing operation, and the effective prestress available to resist flexure reaches an uneconomical level. An effective prestress of less than 50% of the Ultimate Tensile Strength is usually considered undesirable. The problem can be overcome to some extent by using low friction ducts and cold drawn strand which has a lower friction coefficient, however this will only marginally increase the economical stressing length.

Coupling

The basis of the stage method is to break the structure into a number of smaller structures with total length of cable restricted to an economical length with regards to friction losses. Each stage is then tied back to the previous stage by coupling the prestressing strands at the previously stressed anchorage. Special coupling anchorages are available from most manufacturers. Since the cables are connected to an anchorage which will be completely enclosed in concrete, it will be possible to stress all stages except the first, from one end only. Thus the economical length for subsequent stages is approximately half that of the first stage.

The coupling anchorages are necessarily bulky and require the webs to be thickened considerably, to carry the splitting forces at the anchorage. The physical size of the couplers ensures that the centre of gravity of the cables will be close to the centroid of the section and thus the logical location for the coupling anchorages will be at a point of contraflexure, i.e. about one fifth of the span length from a pier. The critical point for stresses at the coupler position will be the "dead" end side of the anchorage as the effective prestress will, in general, be less than the effective prestress at the "live" end of the same anchorage (Figure 8).

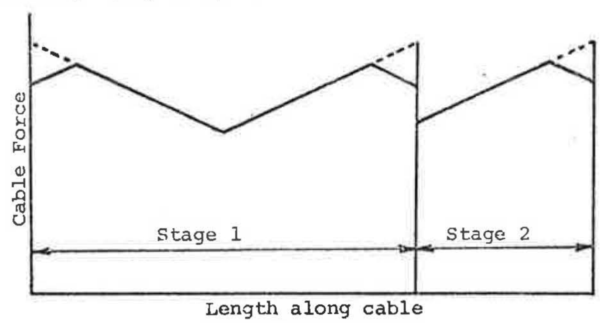


FIGURE 8

Analysis

Each stage must be analysed for self weight moments, shears and torsions, and for the parasitic moments caused by the cable profiles being non-concordant. It is assumed that the structure remains elastic and the principle of superposition may be applied to obtain the final effects. On the completion of stressing the final stage, the structure can then be analysed as a single continuous beam for superimposed dead load and live load effects, and for temperature differentials through the deck slab.

An additional advantage of this method is that as each stage is stressed and becomes self supporting, the erection falsework and formwork can be removed and used for the next stage. This provides a major cost saving for bridges with a large number of spans e.g. sections D and E of the Rail Link viaduct, where seven and eight spans respectively are involved.

Post Tensioning

The V.S.L. System of post tensioning used for both the Nyanda Overpass and the Rail Link viaduct, the Nyanda Overpass using cables of 48 No. 12.5mm super 7 strands with an ultimate capacity of 8830 kN, and the Rail Link using cables of 27 No. 12.5mm super 7 strands with an ultimate capacity of 2970 kN.

Details of the V.S.L. coupling anchorages are shown in Figure 9.

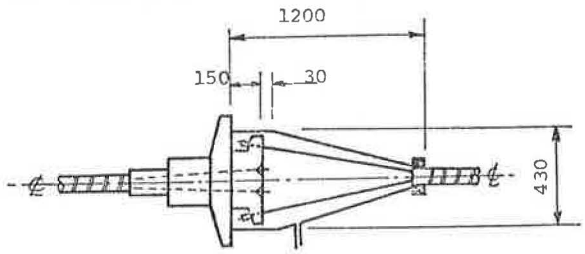


FIGURE 9

The physical size of these anchorages is such that the webs had to be increased in thickness from 300mm to 600mm for the Rail Link and from 450mm and 380mm to 790mm for Nyanda Overpass. With the extensive reinforcement required around these anchorages placing and compacting of the concrete in the webs is difficult and care must be taken on site to ensure adequate compaction of concrete is achieved. In the case of the Nyanda Overpass structure, recesses were left in the webs to allow concrete placement of the anchorages and to facilitate connection of the unstressed strands to the coupler. The recesses were then filled with concrete after the second stressing operation.

When geometric layout requires large variations in span lengths, or precludes the use of shorter end spans, it may not be possible to use the main longitudinal prestress to resist entirely the applied longitudinal moments, and additional capping cables may be required in the deck and soffit slabs. In both the Nyanda Overpass and the Rail Link small cables with four and seven 12.5mm super 7 strands respectively were used. Anchorage of these capping cables will normally occur on the inside of the box, although it is possible to anchor on the outside surfaces of the slabs and infill after stressing. This latter technique has the disadvantage of reducing the effective thickness of the slabs considerably, and this will generally far outweigh the disadvantage of forming anchorages on the inside of the box. Stressing the small cables from within the box is a relatively simple procedure requiring only adequate lighting and more particularly adequate ventilation during the stressing operation. These cables may be stressed from both ends to reduce the effects of friction losses.

Deflections

The stressing of the cantilever on each stage will result in an upwards deflection of the tip of the cantilever as the centroid of prestress will be predominantly above the centroid of the section. To ensure an aesthetically pleasing joint and to prevent an unsightly kink at the joint it is necessary to set the formwork to allow for these deflections and rotations. Allowances must be made, of course, for the final camber profile.

Grouting

It is almost impossible to produce a water tight seal to the duct connections and invariably some leakage will occur from one duct to the next. This loss can be reduced by grouting ducts in pairs. On Nyanda Overpass the length of the cables caused severe head losses during pumping and a loss of workability at the face of the grout, and the grout pump had to be moved along the bridge to the last point where grout had already vented. This process was continued along the bridge until grouting was complete.

General Features of Design

Shear Lag

The assumption of the theory of bending, that plane sections remain plane, is no longer valid for wide flange box girder bridges. The effect of shear lag over the supports may be calculated from the theory of elasticity (3) or some empirical formula. Tests carried out in England (4) produced the simple formula that the shear lag effect can be accounted for by simply ignoring the cantilever at the supports. Thus the effective cross section is only the spine of the box. The effect of the shear lag is taken from the support to a point one fifth of a span length on either side of the support. This approximation has since been checked against the German code (5) and was found to be in good agreement.

Distribution of Prestress

A more important problem in the vicinity of the supports is the distribution of the prestress, particularly through a solid diaphragm. It has been assumed, conservatively, that the axial component of the prestress is distributed over the whole of the cross section, while the eccentric component is distributed over the reduced "shear lag cross section". This assumption has lead to some structures containing unnecessary additional axial prestress in the vicinity of the supports.

Diaphragm Action

The superstructure may be supported at the piers and abutments either by a single bearing allowing transverse rotation, or by a multiple bearing system providing torsional restraint. The large shears carried by the webs must be transmitted to the bearings through the diaphragm. These transverse bending effects are resisted by prestress in both the Nyanda Overpass and the Rail Link. Detailing the intersections between the transverse cables and the main longitudinal cables requires more than ordinary care as during stressing of either the transverse or longitudinal cables there is a distinct possibility of crushing the ungrouted duct running at right angles. It may be necessary in some instances to provide a more solid duct. 6mm thick steel tubing has been used to carry the longitudinal cables through the diaphragm, in order to resist crushing during the stressing of the transverse cables.

The large capacity bearings used exert concentrated forces of up to 15000 kN to the diaphragm. The diaphragms must therefore be designed to resist the bursting and spalling stresses associated with the application of a large concentrated force. The theories proposed for similar stresses behind post tensioning anchorages may be used to reinforce the diaphragm in the vicinity of the bearings.

Diaphragms at torsion restraint piers and abutments will require heavy reinforcement to carry the applied torsion moments.

When maintenance access is required through the boxes small manholes will be required through the diaphragm, thus complicating the detailing further.

Differential Temperatures

Thermocouple readings taken from existing box girder bridges indicate that, for the climatic conditions of Brisbane (27½°S) an almost linear temperature differential of 11°C can occur between the outside and inside faces of the deck slab (6). This figure was used for design calculation in both the Nyanda Overpass and the Rail Link, although it is expected that the temperature of the outer surface of the deck slab in the Rail Link under 450mm of ballast will be lower than under the 125mm to 150mm of hotmix surface on road bridges.

Thermocouples are being incorporated in sections of the Rail Link to verify the temperature differential.

Large Capacity Post Tensioning Cables

The 48 strand V.S.L. cables used on the Nyanda Overpass at the time of construction were the largest capacity cables to be used in Australia. The bridge contains 120 tonnes of main prestressing strand, and the large capacity cables were adopted because:

(a) There is an economy in minimising labour associated with fixing a smaller number of ducts.

(b) There is an economy in minimising the number of anchorages and couplers

(c) The larger than usual ratio of duct area to strand area indicated that lower friction values could be used, thus decreasing the losses

(d) There is a design economy associated with the force being concentrated in a fewer number of cables, thus allowing larger eccentricity of prestress at points of maximum moment.

There are, however a number of disadvantages in using this system:

(a) Any failure to achieve adequate prestress in a cable (by breaking strands etc.) cannot be rectified by increasing the prestress in other cables, as easily as in a design with a larger number of cables

(b) The jacks are very heavy and cannot be manhandled easily

(c) The large forces at the anchorages require special attention in the design of details to resist bursting stresses

(d) The large anchor plates and couplers require the webs to be thicker than normally would be required for stress considerations.

Despite these disadvantages the use of the large capacity system proved to be worthwhile.

Heavy lengths of cables were pulled through the ducts with an air winch, while for shorter lengths the individual strands were pushed through manually.

Site measurements confirmed the expectation of lower friction values with a measured coefficient of friction of only 0.12, i.e. half of the normal design value.

Conclusion

The prestressed concrete box girder bridge in its many forms provides an aesthetic, structurally sufficient solution to complex crossings. The problems encountered during these designs are no doubt universal, however the solutions discussed are those applicable to the local conditions of construction in Queensland.

Some of the aspects discussed still require further research in order to provide more economical solutions.

Acknowledgements

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