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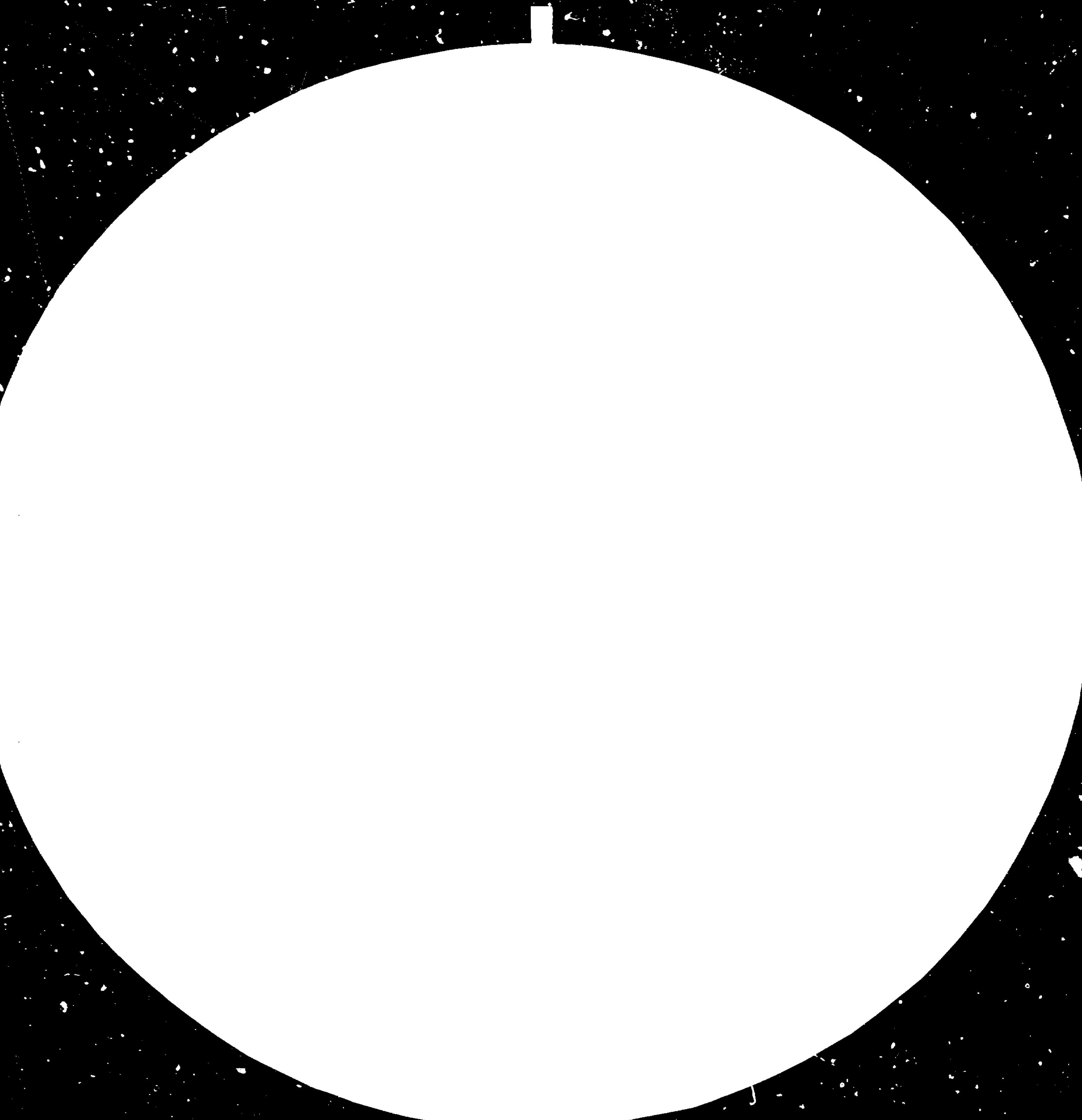

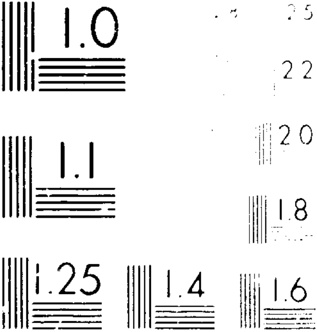
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POST-TENSION SEGMENTAL BRIDGES:
NEVILLE HEWITT BRIDGE - ALBERT ST, ROCKHAMPTON *

by

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CONTENTS

1 INTRODUCTION

This paper discusses the design and construction of the Neville Hewitt Bridge across the Fitzroy River in Rockhampton. The design features twin post-tensioned concrete box girders with a constant depth of 2.8 m and major spans of 71 m.

1.1 Rockhampton Transportation Study

In January 1971 Cabinet approved the undertaking of the Rockhampton Transportation Study. Consulting Engineers, Rankine and Hill were engaged to carry out the work.

The major recommendation of the Report was the early construction of a new bridge across the Fitzroy River on the alignment of Albert Street and the construction of a new access highway from this bridge to join the Pacific Highway at the northern outskirts of the city. This proposal catered most effectively for anticipated traffic flows with 45,000 vpd assigned to the new bridge in 1991.

The recommendation to construct a bridge at Albert Street was accepted and route design was commenced by the District Engineer for the Main Roads Department. Design of the bridge was undertaken by the Special Projects Section of the Department.

1.2 Preliminary Design

A four lane facility with divided carriageways was the basis of the design with each 8.6 m wide carriageway providing a cycle lane of 1.4 m and two traffic lanes of 3.5 m and 3.7 m. In addition a footway of 1.8 m width was to be provided on the upstream side of the bridge. The overall length of the structure was fixed at 400 m giving the same waterway area as the existing bridge for a design flood having a 1 in 200 year return period.

In the selection of bridge type, 3 alternatives were studied:

(i) 30 m spans using I section beams.

(ii) 50 m spans using post-tensioned boxes lifted into place on the piers and an insitu deck (weight of boxes would have been approximately 250 tonnes.)

(iii) 71 m span precast, full section boxes, post-tensioned.

Costing of the preliminary designs showed no clear preference for any of the alternatives although there was a slight advantage for the 50 m spans. However, the 71 m span alternative was chosen because it has only one pier in the main river channel and, with fewer piers overall, offered a lesser flood risk during construction as well as minimising construction problems. The heavier loads at the piers have the added advantage of greater flood resistance. The arrangement selected consists of two separate box girders in close proximity with a common central parapet.

The final bridge adopted was a continuous concrete box girder, 2.8 m deep, fixed at the North Abutment. It has four main spans of 71 m and two end spans of 58 m giving a total of 400 m. In addition, an insitu reinforced concrete portal spans Victoria Parade on the south bank. The general arrangement of the bridge is shown in Fig. 1.

1.3 Design Load

The structure was designed for NAASRA HS20 live load as given in the 1970 Bridge Design Specifications increased by 10%. Such a loading was the forerunner in mass, but not distribution, to the T44 load specified in the current NAASRA. In addition, the structure was checked for a 200 tonne Abnormal Load.

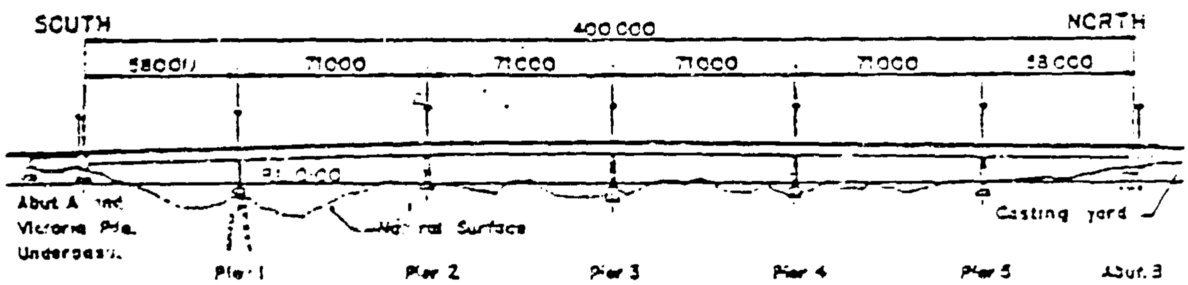


Fig. 1 General Arrangement

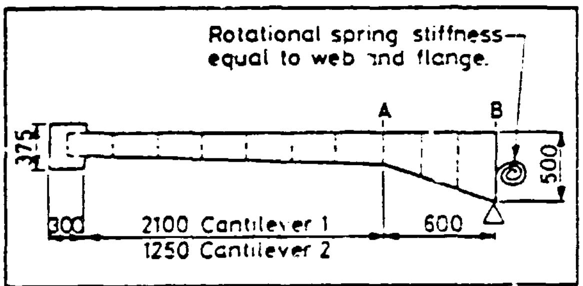
2 SUPERSTRUCTURE DESIGN

2.1 Cross section - General

Studies conducted by Special Projects and by others, including O'Connor (1972) show that the optimum section for concrete box girders is that based on minimum thicknesses of webs and flanges and varying the overall depth. For longer spans, variable depth is essential but for 71 m spans, economies can be achieved in formwork by keeping a constant depth and increasing the bottom flange thickness over piers. Due to congestion of ducts and anchorages in the pier region, local widening of the webs is also necessary.

2.2 Cantilever Design

Maximum weight saving on the cross-section is achieved by using the widest possible cantilevers. Hence an accurate analysis of these, utilising the stiffening effect of the edge beam, is essential. The cantilever profile is determined by the wheel spacing of the design axle, which represents quite accurately real truck loadings. A thin section is used to carry the outer wheel, and a strengthening fillet is used once the cantilever is wide enough to accommodate both wheels of one axle. A finite element model was used to analyse the cantilever for the HS20 design load which has wheel loads of 72 kN increased 30% for impact giving 94 kN. The model used realistic web rotational stiffnesses, and a comparison of the finite element results (on which the design was based) and the simple NAASRA (1970) code formula are given in Fig. 2.



Section	Cantilever	F.E. analysis (a)	Code(b)	Ratio $\frac{b}{a}$
A	1	30·0	68·2	2·27
A	2	21·9	46·5	2·12
B	1	82·3	115·4	1·40
B	2	47·2	62·7	1·33

Fig. 2 Comparison of Design Bending Moments for Cantilevers

2.3 Shear Lag at Piers

Elementary calculations showed that shear lag may be a problem over the piers. The simple engineering theory of bending indicates that bending stress is proportional to the distance from the neutral axis. This is accurate enough for the design of stocky sections. For sections with wide thin flanges, the shear stiffness of the elements is small enough that bending stresses drop off in the flanges further out from the webs.

This effect is aptly called shear lag and is worst in regions of high moment gradient, such as the region between the piers and the point of contraflexure, approximately one-fifth of the span away.

A finite element model of this region was developed using the Stardyne program on the CDC 6600. Very significant shear lag was discovered as predicted by the simple rules not in the NAASRA (1976) code. Fig. 3 shows the ratio of actual bending stresses to the maximum stress in a top flange over a web.

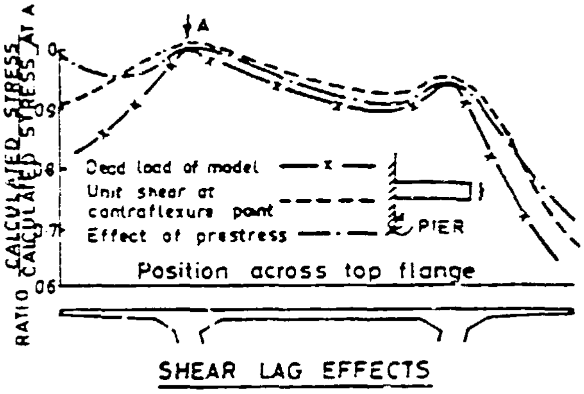


Fig. 3 Ratio of calculated stress to calculated stress over the inner web.

Fortunately, the distribution of stresses induced by the draped prestressing cables had a similar transverse distribution and hence the combined shear lag effects of dead load and prestress differ by only a few percent from the simple bending theory. A different prestress arrangement using straight cables in top and bottom flanges would not necessarily balance the dead load shear lag, and careful analysis would be required.

2.4 Distortion and Warping of box cross section

A box girder distorts due to asymmetric live loading. This distortion causes additional longitudinal warping stresses which add to the normal bending stresses.

This effect can be analysed using the "beam on elastic foundation" (BEF) analogy, where the cross sectional distortion stiffness is represented by the foundation modulus of the analogous beam and the warping stiffness of the box is represented by the bending stiffness of the analogous beam. A full description of this method is given by Wright et. al. (1968).

The effects of diaphragms and changes in box cross section are easily modelled by this method and maximum box distortion, with resulting transverse bending moments, calculated. These are then added to the local plate bending effects of wheel load to design the transverse reinforcement of the box.

The worst longitudinal warping stresses are caused by one eccentric design truck. The section can accommodate these stresses as the normal bending stresses are only half that caused by the full design load of two trucks on a span.

The box has a high transverse distortion stiffness and does not need internal diaphragms to prevent distortion.

2.5 Longitudinal Bending Design

Having proved that shear lag of dead load and prestress was "balanced", the longitudinal analysis was based on a simple beam modelling the variable stiffness of the box section to determine bending moments, and the simple bending stress theory.

Design of a prestressed concrete box girder is heavily dependent on the construction sequence and method. Various methods of construction were considered, including

(i) Precast segments erected on a truss and stressed together in 71 m lengths extending to the dead load contraflexure point, 0.2 of the span past a pier.

(ii) Cast insitu on falsework, but stressed in sections as in (i) above.

(iii) Cast in sections at an abutment casting area and "rolled out" into position - the incremental launching method.

It was considered that method (i) would be most economical due to the advantages of "factory" production in fixed steel forms using steam curing and daily production in easy-access working areas. Several contractors did consider incremental launching, but this method appears uneconomical unless ground access beneath the structure is quite difficult, as in mountainous terrain. Also, the superstructure must be specifically proportioned and designed for incremental launching, with a much smaller span/structural depth ratio than normal (e.g. 12 to 15 instead of 25 for this design).

Jointing of precast segments can be done in three basic ways:

(i) Glued (epoxy) joints of a few millimetres thickness, requiring match-casting of all segments and very careful profile control on pre-set soffit forms.

(ii) Narrow mortar joints 75 to 100 mm wide without reinforcement.

(iii) Wide insitu concrete joints (400 mm) allowing full lapping of longitudinal reinforcement.

All three methods have been used widely. This Department's experience has lead it to favour the latter method because of the additional strength of the fully reinforced joint in case of unexpected overloading, and its ability to cater more easily with normal construction tolerances.

The construction method and sequence having been determined, the longitudinal design proceeded by analysing each construction stage including final completion and proportioning the prestress accordingly.

It is possible to build such a structure with a wide range of final bending moment profiles. However, the effect of creep under dead load is to alter the bending moments back towards those occurring in the simple continuous beam on unyielding supports. Therefore, if the structure is built to achieve this moment profile, creep will not alter support reactions and the stresses will not change with time. Stress and deflection calculations can then be made with confidence.

To achieve the "correct" final bending moments, it is necessary, at each new construction stage to transmit the correct shear and bending moment to the cantilever end of the previous stage.

Since the cantilever end of each stage is tilted up slightly (as it has no end shear load) the next stage can be built up at a slight angle so that, on lowering, at the leading pier, the correct moment and shear is transmitted. See Fig. 4.

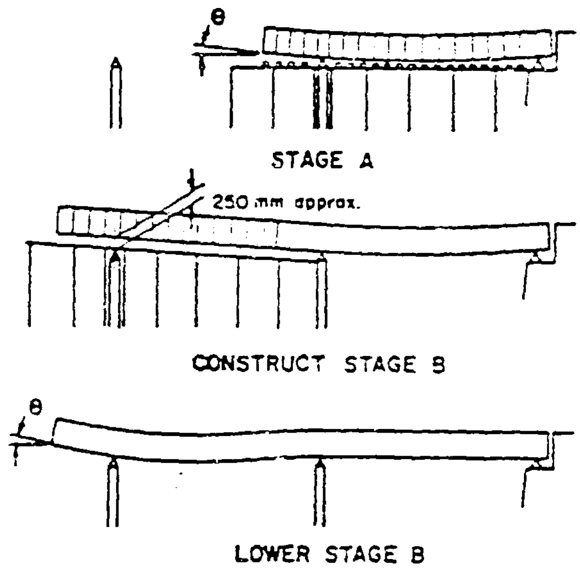


Fig. 4 Lowering of each stage to adjust dead load bending moment profile.

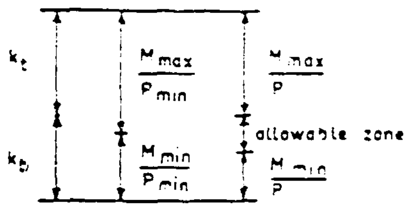
2.6 Prestress Arrangement

Prestress layout should also mirror the dead load moment profile closely to achieve a concordant cable. That is, a cable profile which, on stressing, does not alter the support reactions and hence create "parasitic" moments.

Preliminary design was based on 12.7 mm regular strand with an effective prestress after losses of 60% of ultimate. More detailed calculations later showed that the effects of relaxation and creep, on the widely varying cable stress profile after jacking, was to reduce the stresses to an almost uniform stress profile ranging from 58 to 62% of ultimate.

An allowable zone for the prestressing cables, to give zero concrete stresses, was calculated at each 3 m section as follows. Due to the need for a discrete number of cables, the effective prestress, P is always greater than the theoretical minimum prestress, P min.

$$P > P_{min} = \frac{\text{Maximum variation in moment}}{\text{Depth of kern zone}}$$



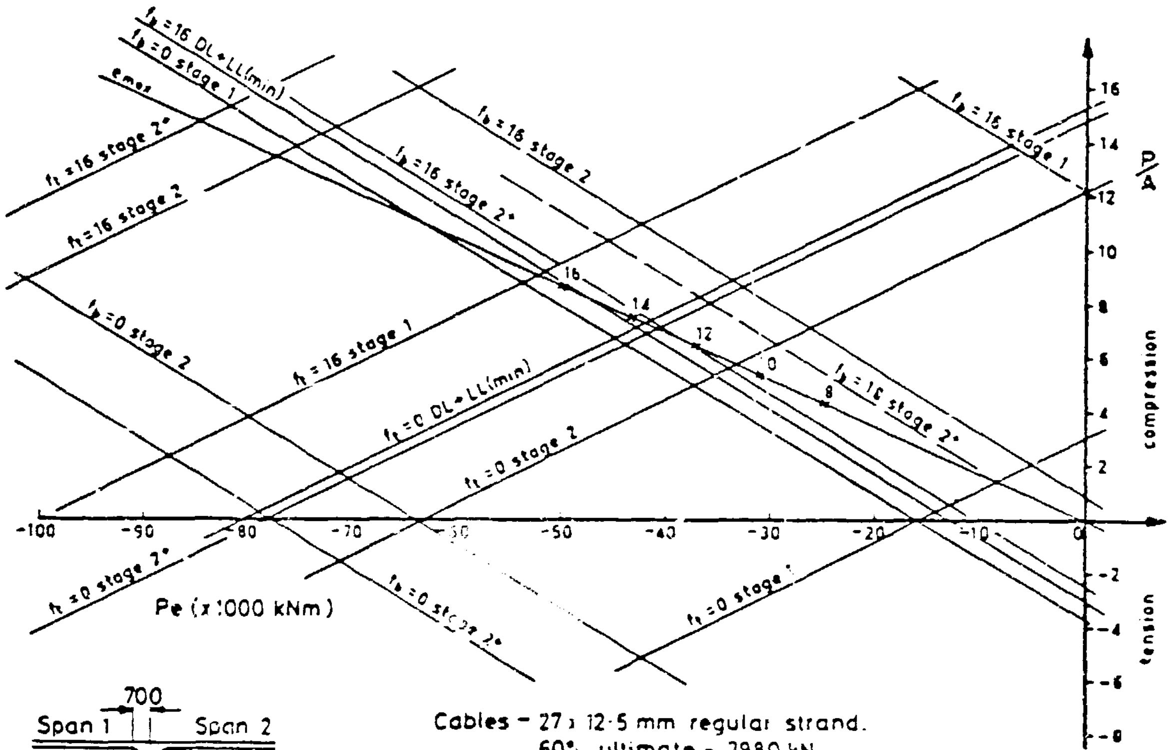
If $P = P_{min}$, the allowable zone diminishes to a point. (kern points k_t (top) and k_b (bottom) are defined as the points at which an applied force would cause zero stress at the opposite face in the absence of other forces or moments.

$$k = \frac{\text{section modulus, } Z}{\text{section area, } A}$$

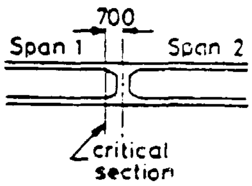
Note that the sign of the bending moment is important and that the allowable cable zone is not necessarily within the kern zone.

Stress conditions were checked at each 3 m section for all construction stages using modified Magnel diagrams. Due to symmetry and repetitive construction process, only stages one and two had to be checked in minute detail. Later stages then repeated the construction moment profiles quite closely.

The modified Magnel diagrams were developed in the Special Projects design office and graphically represent every moment, code stress condition and geometric limit applicable to a cross section. The diagrams are formed by taking the basic stress equations at each construction stage



Cables - 27 x 12·5 mm regular strand.
60% ultimate = 2980 kN.



Z_t = 5·24 m³
Z_b = 4·22 m³
Area = 5·52 m²
$\bar{y}_t$ = 1·25 m
e_{max} = 1·04 m

Minimum Prestress $\frac{P}{A}$ = 7·1 MPa
∴ Required P = 7·1 x 5·52 x 1000
= 39 200 kN
= 13·2 cables

APPLIED MOMENTS (kNm)	
Stage 1 Dead Load	15 850
Stage 2 Dead Load	63 590
Stage 2 + Box moving	77 920
Final D.L. + Secondary D.L.	64 100
D.L. + max Live Load	61 200
D.L. + min. Live Load	80 100

Fig. 5 Modified Magnel diagram for section at Pier 1 in Span 1

$$f_t = \frac{P}{A} - \frac{Pe}{Z_t} - \frac{M_1}{Z_t} - \frac{M_2}{Z_t} \quad \text{——}$$

and re-arranging in the form

$$\frac{P}{A} = \frac{1}{Z_t}(Pe) + f_t - \frac{M_1}{Z_t} - \frac{M_2}{Z_t} \quad \text{——}$$

where P = prestress, A = cross section area
f_t = stress at top, Z_t = section modulus
e = eccentricity, M_1, M_2 —— various applied bending moments.

Plotting this line in $\frac{P}{A}$, Pe space gives various lines of slope $\frac{1}{Z_t}$ corresponding to the limits placed on f_t (e.g. $f_t = 0$ or $f_t = 16$ MPa). A similar formulation for f_{bottom} completes the basic diagram. Lines corresponding to various eccentricities pass through the origin and all the intersections on the diagram have visible properties which enable easy checking of the accuracy of the plot.

The allowable region is marked off, the minimum possible prestress is obvious, the limits of eccentricity are clearly shown, and initial and final prestress can be checked.

A typical Magnel diagram is shown in Fig. 5.

Due to limitations in anchorage positions, a truly concordant cable layout cannot be achieved, but calculations showed that the "parasitic" bending moments were quite small.

2.7 Deflection Calculations

Extensive calculations were carried out to ascertain all the stage construction and final deflections due to dead loads, prestress parasitics and creep. The construction profile was preset to cater for these. Design values of elastic modulus were assumed, and then corrected after careful measurement of elastic modulus on test cylinders and a load test on the first stage measuring the displacements under a 40 tonne box.

Due to the age of the concrete at this stage, a high value of E = 42700 MPa was measured in the actual structure in bending, compared with cylinder measurements averaging 39000 MPa.

2.8 Bearings

The bridge is a continuous bridge fixed at Abutment B with sliding bearings at the remaining piers and abutments. The fixed bearing is a mild steel bearing designed for the required loads. The sliding bearings are pot bearings with teflon on stainless steel sliding surfaces. They have runner bars to prevent lateral movement. They are supplied by the manufacturer to a performance specification.

2.9 Expansion Joints

The expansion joint at Abutment A is a finger joint catering for approximately 225 mm total movement. Only small movements occur at Abutment B and a Wabo joint has been used. Both joints were installed after bituminous surfacing was complete.

2.10 Services

The bridge has street lighting, internal power for maintenance and added provision for decorative lighting. Telecom have installed ducts to utilise the crossing and electricity authorities plan future installations.

2.11 Substructure

Design of the substructure is fairly conventional. Pier 1 and Abutment B are supported on 1.2 m cast insitu reinforced concrete piles using sacrificial liners. Abutment B has 16 piles with a maximum length of 14.5 m. Pier 1 has 10 piles with a maximum length of 20 m. The remainder of the substructure consists of spread footings onto moderately weathered mudstone.

2.12 Roadworks and Drainage

Roadworks and drainage from Denison Street on the southside is also included in the contract. Pavement consists of 150 mm of bituminous concrete and 350 mm of cement stabilised crushed rock on a clay subgrade.

Drainage was designed for a 1 in 10 year intensity for the collector system and 1 in 50 year intensity for the main drain.

3 CONSTRUCTION

In January 1978 a contract was let to Pearson Bridge (Qld) Pty. Ltd. for $4,645,623 to construct the bridge and associated roadworks.

Construction of the project can be divided into five major sections:

- Access across river to foundations and to facilitate erection
- Foundations and pier stem construction
- Precasting operation
- Erection falsework and method of construction
- Miscellaneous operations

3.1 Access across River

A major problem was providing an access system to foundations which could also be used for erection. Several alternatives were considered.

For the section from pier 5 to pier 2, it was felt that only two possibilities were viable, and they were causeway or bridge. The section from pier 2 to southern abutment appeared to be capable of construction from the water or from a bridge.

The decision made was to construct a low level access bridge full width of the river along the bridge centre line between the pier stem positions, the influencing factors being:

(i) Removal of a causeway and possible delays to construction due to wash outs.

(ii) Dangerous situation of having floating plant in the vicinity of sharp rocks not visible except on extreme low tides.

(iii) Separate foundations would not be required as the bridge was to be supported on the erection falsework foundations.

The access bridge installed was in ten metre spans consisting of three 760 UB's complete with bracing and timber decking, and designed to support a 45 tonne crawler crane working to maximum capacity. As mentioned, the bridge was supported on the erection falsework foundations which were installed progressively from the end of the access bridge.

3.2 Foundations and Pier Stem Construction

(1) Northern Abutment

Foundations for the northern abutment consisted of 16 1200 diameter encased cast insitu piles founded on moderately weathered mudstone. These were constructed by preboring through the overburden approximately three metres, standing the casing in the hole and driving to initial refusal with a bell-mouth helmet and K22 diesel hammer. The piles were then excavated using a 4 tonne Benoto hammer grab and then alternatively redriven and re-excavated until acceptable founding material was reached. Sockets and bells at the pile toes were hand excavated where required.

On several piles a reverse circulation drill rig was used to excavate the socket below the casing. Troubles were encountered with this in the form of blockages which were due to the highly fragmented state of the insitu rock.

Once excavated, the reinforcement was placed, prefabricated using stainless steel spacers welded to the cage to maintain position, and concrete poured.

(2) Pier 5

Pier 5 foundations consisted of twin spread footings each with a 3700 x 1700 x 11.5 metre high pier stem. As this pier was located on the river bank in the tide zone, the area was filled to above high tide and the footings excavated through the fill. Founding level was approximately 5 metres below top of fill, the founding material being moderately weathered mudstone.

The excavation was carried out conventionally in open cut using a hydraulic excavator, hand trimming and some drilling and blasting. Footings were then constructed in the usual manner.

The pier stems were poured in two lifts each, the first lift being a make up section approximately 2 metres high and the second lift of 9.5 metres was poured using a ply faced steel backed 4 piece form which was used for all the pier stems.

(3) Pier 4

Although similar to pier 5 in arrangement, pier 4 was located in the river between two islands and the founding level was approximately 6.5 metres below MHWS. Natural rock surface level in the footing area varied by 3 metres.

To construct these footings a sheet pile cofferdam was installed from the end of the access bridge. Sheets were driven using a spudded template frame and had an average rock penetration of 700 mm.

Excavation of each footing was done using a backhoe lowered into the cofferdam, with a grab and by hand. Construction of the footings then followed generally along the lines of pier 5.

(4) Pier 3

This pier was similar to pier 4 in arrangement and location except that founding level was 8.5 metres below MHWS and natural rock surface level across the cofferdam width of 8 metres varied 5 metres. To overcome this problem, the high areas were drilled and blasted from a barge down to a depth of 2 metres above founding level. The area was then drag-lined from the end of the access bridge and the cofferdam and structure constructed as for pier 4.

(5) Pier 2

This pier was sited wholly on an island with the founding levels being within the tide range. A simple sandbag and mass concrete cofferdam was installed, the excavation carried out using drill, blast and grab and the structure completed as before.

(6) Pier 1

Pier 1 foundations consisted of ten 1200 diameter encased cast insitu piles, a single pilecap at mean tide level with precast skirts, and twin pier stems as for the other piers. This pier is situated in the main river channel and access was provided by constructing the access bridge past the pier location. A template frame was set up on spud piles and the 1200 diameter casings pitched through fixed guides. These casings were then alternatively driven and excavated until acceptable founding material was reached. Driving was done with a bell-mouth helmet and K32 diesel hammer, and excavation carried out using a 4 tonne Benoto hammer grab and 4 tonne 1120 diameter rock chisel. As for the north abutment, sockets and bells were hand excavated to the required depths. Good seals were obtained on all piles, the only trouble experienced being one distorted pile toe. On all piles, driving shoes were attached to the outside of the casing. Reinforcing cages and concrete were placed conventionally in the dry.

To construct the pilecap, a sacrificial soffit was used supported from the piles, the soffit level being 4 metres below MHWS but 0.8 m above MLWS. Thus, the soffit, side forms and reinforcement were placed during periods of low tide and concrete poured on a falling tide to give enough time to complete the pour. The pilecap was poured in two sections, the construction joint being vertical along the bridge centreline. Each pour was approximately 60 cubic metres.

Pier stems were then constructed in a similar manner to previous foundations.

(7) Southern Abutment

The southern abutment was supported on a spread footing and construction of the abutment and associated overpass and overpass rear wall was carried out using open cut excavation and normal methods for formwork, formwork support and concrete placing.

3.3 Precasting Operations

The bridge is composed of cellular box sections a total of 274 sections for both bridges. It was decided to cast these units on site and in the upright position. Choosing the site for the casting yard had to be tied in with the erection method as handling the units and transferring them from yard to bridge was of prime importance.

A casting yard running at right angles to the bridge centreline with gantry tracks running up to the outside edge of the bridge was chosen. Units could be transferred from the stacking area onto the erection falsework or onto stage 1 of the completed bridge quite easily. Access from the back of the abutment, where the ramp was steep and turning space limited, was not required for erection.

The casting yard, under the gantry was approximately 200 metres long comprising 30 metres material storage area, 65 metres work area and 105 metres stacking area. Outside the gantry, on the north side there was a reinforcement stacking area with its own unloading gantry and on the southside there was a boilershed/workshop/office building and access for concrete placing equipment and concrete delivery.

The gantry was purpose-made with a 15 metre span, 15 metres high and a capacity of 50 tonnes. Its operation was all hydraulic - hydraulic drive motors on each side for gantry travel, hydraulic crabbing operation, and hydraulic ram for hoisting. The hoisting speed of the main ram was quite slow. For faster working, a small three tonne electric hoist was added which handled all the light lifts. Units were lifted using a four point lifting beam and 48 mm Ø helical anchors.

As the upstream and downstream bridges have different cantilevers due to the pedestrian footway, it was necessary to have equipment for two separate casting operations. It was decided to have four complete moulds, each mould having its own reinforcement fixing jig, core and coretraveller except that the core unit used for casting units with raised stressing anchorages was used for both upstream and downstream units.

The external moulds were completely fixed with the exception of the vertical kerb face which was bent to the stripped position and was sprung into the vertical position for pouring

The cores were carried on a counterweighted core traveller which was winched into the mould in a collapsed position. The core was then expanded to final position by using two hydraulic rams. End forms were placed and the sides fixed to the external mould and the form was ready for casting. On stripping, the reverse was carried out. One major problem with the moulds was how to overcome the fact that there were some two hundred and twenty bars protruding through the stopends on each side. This was solved by making steel stopends with 20 mm holes to take the 12 mm bars. Plastic champagne corks with holes punched through the cap were placed on the bars and then pushed into the stopend from the inside. Thus the stopend was sealed and stripping of it was turned into a minor operation only.

Units were stacked up to three high in the stacking area where there was working capacity of 96 units and a maximum possible capacity of 108 units. A single erection sequence required 48 units, thus one span reserve storage was available.

Manufacturing capacity in the yard was nine units a week but had it been necessary to increase this, minor modifications would have brought production up to twelve units per week.

In order to achieve this turnaround, each unit was steam cured to give the required lift out strength of 30 MPa within twelve hours. To steam the units a Presha 50 HP steam generator was installed. The steam covers were built as a complete unit on a pipe frame, so that covering a unit after pouring was quite a simple matter.

To eliminate the use of the gantry for pouring concrete in the casting yard, a Morgen mobile conveyor was used. This unit had an articulated boom and the advantage of being self propelled in any direction. It had a placing capacity of 75 m^3 an hour and a vertical lift of 7½ metres. A swivelled distributing chute allowed a radius of pouring of 14 metres.

Vibration of the concrete in the casting yard was achieved using eight Wacker high frequency external form vibrators on each mould, complimented with internal vibrators. The external units were 200 Hertz 55 volt, with a speed of 6,000 RPM and a maximum centrifugal force of 8,000 newtons, and operating over an area of approximately 1.3 m diameter.

3.4 Erection Falsework and Method of Erection

The superstructure design called for a staged erection sequence commencing at the northern abutment, each stage for each bridge comprising the erection of 24 units from the previously completed section to a distance of approximately 16 metres past the next pier. The method of erection chosen was one which used a falsework system wide enough to take both bridges and moved at the end of each stage by pulling forward onto another set of supports for the next stage erection. This system enabled the erection of both bridges simultaneously and thus reduced the cycle time for an erection sequence.

The falsework is shown in Fig. 6 and consisted of a series of frames spaced at 10 metre intervals along the centreline of each bridge

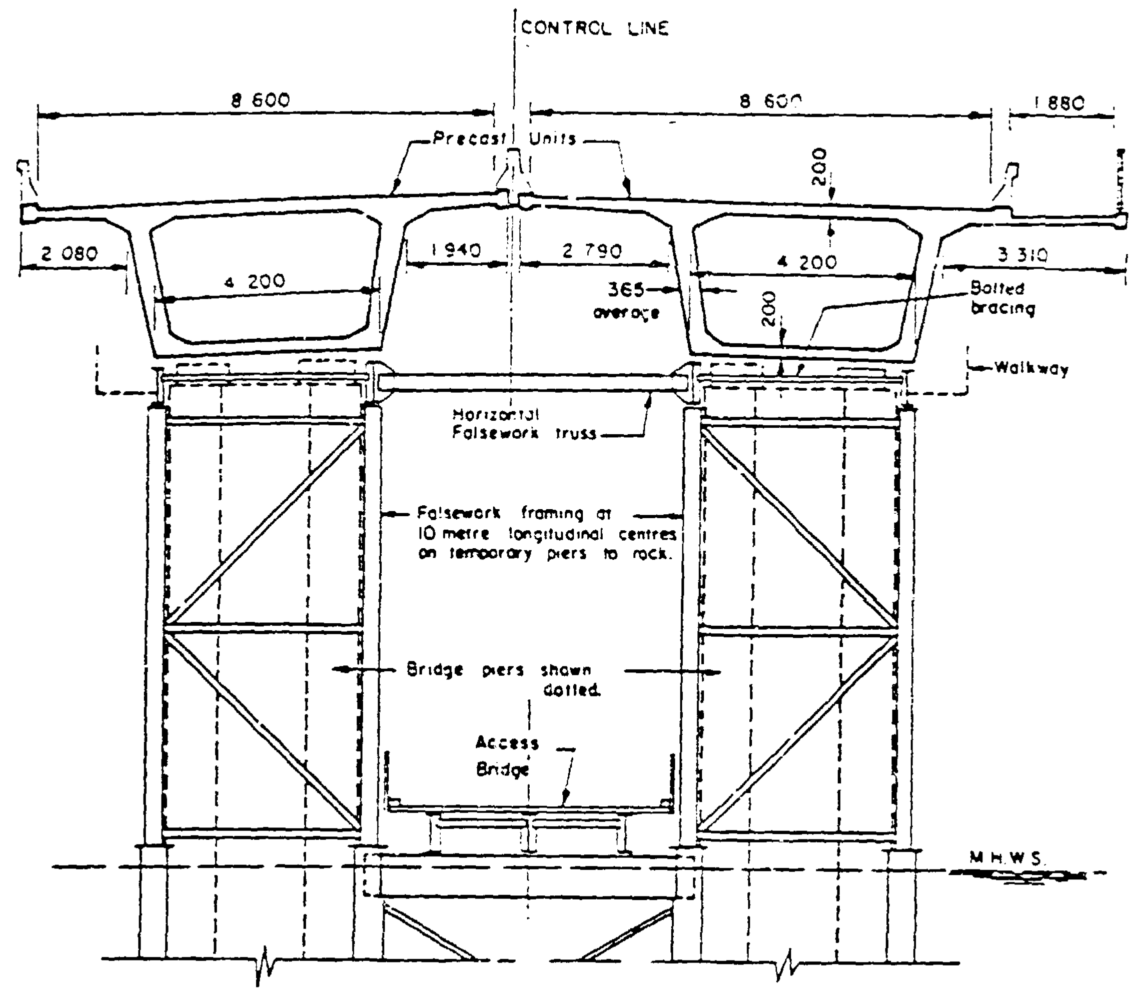


Fig. 6 Typical section showing cross section and falsework arrangement

each frame supporting two continuous 760 UB's 80 metres long. The frames were in turn supported on varying types of pile foundations, the type being dependent on the rock level, depth of water and nature of founding material. The most common type of pile used was a 500 mm diameter 6 mm steel tube dowelled or socketed into rock and concrete filled.

The 4 runs of 760 UB's were joined laterally into a horizontal truss with the centre beams designed to resist flood loading by spanning between the bridge piers, and bolted bracing between the centre and outside beams. The reason for this configuration was that once an erection stage was complete, the falsework "superstructure" was winched forward onto the next stage frames, the horizontal truss passing between the concrete piers and the bolted bracing being successively removed and replaced to enable the falsework to pass the concrete bridge piers.

In general, the erection sequence was as follows:

1. Set up falsework superstructure to level.

2. Erect precast units.

   (a) Transfer units from casting yard onto transfer steelwork using casting yard gantry.

   (b) Slide units across onto completed bridge using skates.

   (c) Winch units along the completed upstream bridge on a rail mounted trolley to underneath a cantilever gantry which was fixed to the end of the completed upstream bridge.

   (d) Lower units onto falsework on upstream side, slide sideways (required for downstream erection only) and move out along falsework using skates.

3. Set units to line and level using packing and sandjacks.

4. Pull cables and construct joints. The closing joint between the new section and the old section and the joints in the staged stressing sequence were steam cured using a portable 15 HP steam generator and a light steel form incorporating an external steam jacket.

5. Stress and grout.

6. Remove packing, lower falsework onto fixed skates set on top of falsework frames and winch falsework forward onto the next stage frames for the next erection sequence.

7. Set falsework to level, move cantilever gantry forward and start next erection sequence.

The design of the bridges called for the erection of each stage above its final position and lowered at the completion of stressing. Thus the final operation to perform in an erection sequence was to lower each bridge into its final position on the bearings. To do this, four 3000 kN hydraulic jacks were set up on frames around the pier, the weight taken, packing removed and the bridge lowered, the maximum distance being approximately 250 mm.

The first erection sequence took approximately 12 weeks, the second 7 weeks and the remainder 6 weeks each, these times being total for both bridges in each sequence.

3.5 Miscellaneous Operations

(a) Parapets

Parapets were constructed using 6 metre long steel forms, one form each for the upstream section, the downstream section and median. Production in this area was intermittent due to other constraints but with the reinforcement fixed in advance, it was possible to pour 6 metres of each parapet per day.

(b) Painting

Painting of the bridge external faces was done from an overhang scaffold, the operation consisting of light sand blast, filling of the larger air holes, priming and final coat application using airless spray. Average production achieved here was 1 week per span of 1 bridge to complete the box soffit, outside face and underside of cantilever. Painting of parapets and handrails was done in a separate pass.

(c) Miscellaneous Trades

Roadworks, drainage, blockwork, landscaping etc. were all done in a conventional manner on a subcontract basis.

4 CONCLUSION

All major works in preparing the bridge and approaches for traffic were completed by the end of June, 1980, approximately 7 weeks prior to the date for practical completion, leaving only minor cosmetic work to be done. The bridge was opened to traffic on the date for practical completion, the 16th August, 1980.

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