Construction and related design aspects of a large span concrete arch bridge

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Introduction

Whereas the construction of large concrete arches had become uneconomical in comparison with structures such as cable-stayed girder bridges, the successful application of sophisticated construction techniques such as the suspended cantilever and related methods has since the late 1960's led to the design and construction of various large span concrete arch bridges1,2,3,4,5,6. Due to the nature of the interaction between the permanent structure and temporary erection and bracing structures these techniques require a greater degree of expertise than the conventional forms of construction such as formwork supported by centering from the ground. The associated savings in the cost of temporary works and labour are such that for sites with suitable founding conditions the large span concrete arch has again become competitive with other structural configurations in bridge construction.

In the Republic of South Africa the experience and 'know-how' gained in the design and construction of the Van Stadens and Gouritz River Bridges1,2,3 has been applied in the planning and design of the construction procedures of three concrete arch bridges which were recently completed in the Tsitsikama on a new section of National Route 2 along the southern coastline of the Cape.

The most important factors controlling the final choice of structure for the Bloukrans Bridge, including the topographical and geological features of the site, design criteria, aesthetic and environmental considerations have been described in the accompanying papers. Although the structural details and configurations of the temporary works for the three bridges varied due to the differences in the topography of the sites and the size and details of the bridges, the principles of the construction techniques were similar. In the case of Bloukrans Bridge, the largest of the three, a suitable construction procedure was analysed in detail during the design stages, details of which were included in the tender documents to assist tenderers with the preparation of their submissions. Certain important design features which were considered advantageous for the proposed erection procedure were incorporated in the design of the permanent structure, great care being taken to achieve this without seriously affecting the appearance and economics of the design. These construction procedures were adopted by the successful tenderer.

Although all three bridges were constructed by the suspended cantilever method, the details described and diagrams used in this paper apply mainly to Bloukrans Bridge.

Description of the permanent structures

The leading dimensions and descriptions of the permanent works are given in the accompanying paper on the Bloukrans Bridge.

Construction methods

The construction procedures of Bloukrans Bridge are described in more detail in a reference6. A brief summary is however given here.

The arch was constructed in stages by means of the suspended cantilever system using temporary suspension and tie-back cables and temporary pendulum towers of which the final configuration at the stage prior to closure, is shown in Fig 1. Segments varying in length from four, five to six m were constructed on travelling formwork carriages which cantilevered out from the previously constructed segments.

The deck was constructed in 19 m stages from both embankments using underslung casting girders which were supported at the columns (Fig 1). Each stage was fully prestressed from the stage construction

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joint situated 3.8 m beyond the leading support. The supporting travelling formwork consisted of inter-connected 'off-the-shelf' structural steel girders, timber joists and laminated timber formwork.

The columns were constructed in 3 m lifts using climbing formwork with steel liners (Photo No 1).

Construction of the bridge proceeded symmetrically from both banks. The first phase, shown in Fig 2(a), included the foundations, abutments, columns on the banks, the initial arch stages and the approach deck spans. Construction of the arch rib, being on the critical path, was proceeded with immediately upon completion of the arch foundations. During the second phase, construction of the deck was interrupted above the tall columns on the arch foundations for approximately eight months until the arch had been closed and the first pair of columns on the arch, had been completed. A typical stage during this phase is shown in Fig 2(b).

During the third phase, the deck over the arch was constructed symmetrically from both sides until closure above the crown of the arch (Fig 2(c)).

**Temporary stabilization requirements**

During the construction stages the various portions of the permanent structure required stabilization against the effects of self-weight, wind, temperature, construction loads and possible structural instability due to buckling. The necessary support, bracing and anchorage systems included:

1. Longitudinal and transverse stabilization of the deck against horizontal wind forces. This was achieved by temporarily stressing the deck at each end to the approach abutments by means of prestress cables (Fig 3). Temporary packers were required between the deck and the abutments to retain the expansion gap openings required for the completed structure at each end. The abutments were stabilized by means of rock anchors. A second function of this tie-back system was to provide anchorage for the third temporary suspension cable of the arch by utilizing the deck as a temporary tie-member (Fig 1).

2. Lateral stabilization of the tall columns against horizontal wind forces until they were connected to the deck structure: This was achieved by longitudinal and transverse structural steel bracing girders between columns, stressed to the columns at suitable levels (Photo 1) and temporary rock anchors stressing the foundations to the rock formation. The same system of bracing was utilised to stabilize the tall columns at the cable anchorage points during construction of the arch and to reduce their effective buckling length during the stages when these columns supported the temporary pendulum towers for the arch suspension system (Fig 1).

3. The arch suspension system: During the construction of the arch, the two halves were constructed concurrently from both banks. Each curved cantilever was progressively supported at regular intervals, by...
inclined stay-cables, to limit the bending moments at any point to acceptable limits (Fig 1).

In the transverse direction the suspended half-arches resisted lateral wind forces as cantilevers. The effect of the suspension cables on the transverse stiffness of the whole system was not significant.

4. Arch closure system: A special closure operation requiring a system of thrust blocks, prestress cables and jacks was required to lock the two half-arches temporarily together at the crown until the closure section had been completed.

5. Stabilization of the arch during construction of the deck spans over the arch was effected by the application of counter-balancing ballast in order to limit the bending moments to acceptable limits.

One of the most important features of the deck bracing and the suspended cantilever systems is the requirement of sound anchorage conditions for the rock anchors. Care was taken in the design, installation and monitoring of the rock anchor systems to ensure an adequate level of safety at all stages.

Rock anchors and rock grouting

The rock formation at the site consists mainly of near vertically bedded quartzitic sandstone alternating with weaker bands of shale and argillaceous sandstone. The strike of the bedding planes is practically due east-west parallel to the bridge. The rock is fractured by various joint systems. Core drilling and test grouting results indicated that the joints were generally closed and tight but that open joints existed on the bedding planes at the contact zones between the argillaceous sandstone and shale horizons. In order to provide lateral confinement of the vertically bedded strata and to consolidate the rock mass as a whole thereby ensuring competent anchorage zones, these zones were pressure grouted according to a detailed grouting programme.

Generally grouting proceeded in three m stages from the surface inwards, using a pattern spacing of two m at the surface. The grouting was carried out using neat cement and water with w/c ratios starting at 8:1 and, where necessary, gradually thickening down to 1:1.

Grouting pressures varied depending on depth from the surface and the amount of 'take', but the grouting pressures were limited to the following for the depth stages indicated:

1. 0 — 3 m  150 kPa — 200 kPa.
2. 3 — 9 m  500 kPa
3. 9 — 15 m  500 kPa etc.

In the large areas of the springing foundations the grouting was carried out in primary and secondary grouting stages in which the primary holes were spaced at approximately four m at the surface and the secondary stage holes positioned in between the primary grouting pattern, giving a two m grid. The depth of holes under the foundations were determined from idealised compression zones down to a theoretical stress level of 0.2 MPa. These zones were determined taking the laminated nature of the rock into consideration. The holes varied in depth between a minimum of six m for the column foundations up to a maximum of 30 m at the arch foundations. A total quantity of 350 t of cement was injected on the site doing 12 000 m of percussion drilling in rock. The largest quantity of cement was injected into the contact zones between the argillaceous sandstone and shale horizons of the west bank.

Rock anchor holes were drilled and tested for water tightness after completion of the cement-grouting operations in a particular area. The optimum length of rock anchors for each anchor group was determined from a stability analysis of the rock mass. A minimum global factor of safety of 2.5 against sliding of the rock mass on possible failure planes was assumed. For this analysis it was furthermore assumed that an anchor would fail from midway down the fixed length. Due to the laminated nature of the rock and the direction of strike of the bedding planes, a fixed length of nine m was specified. As these discontinuities are roughly parallel to the direction of the bridge, each group of rock anchors was fanned out at angles varying from 5° to 15° in plan in order to extend the anchorage zone across as many discontinuities as possible. However, in the stability analysis, the additional restraint due to friction or rock interlock was neglected on the theoretical slip planes on the sides of anchorage rock mass.

Each anchor had a working load of 1.8 MN with a safety factor of 2.0 against failure. The anchors were subjected to a strict test procedure which extended over a period of two months and which included an initial overstress of 25 per cent. Hereafter the anchor forces for the tie-back system were monitored on a regular basis to ensure that the residual prestress force did not reduce below the minimum specified working load of 1.8 MN. This required that all anchors used for the temporary suspension system had to remain ungrouted over their free lengths. The free lengths of these cables were corrosion protected by a double protection system comprising:

1. a PVC sleeve filled with "nuclear grade grease" and
2. pressure injected cement grout around the PVC sleeves.

Although these cables were only required for the construction period, they are generally beneficial to the stability of the permanent structure and rock banks and were therefore not destressed. The specification for these cables called for a protection system to last 120 years.

Cantilever construction of the arch

General considerations

The advantages of the suspended cantilever method — The advantages of the suspended cantilever method with respect to more conventional construction methods of centering for arch construction can be briefly summarized as:

1. Economic advantages due to a saving on material, labour and construction time. Of these the most significant is the saving on material due to the effective use of concrete in compression and high tensile steel cables in tension, and by utilizing portions of the permanent structure as part of the composite temporary works system. Further savings can be effected if the temporary suspension cables can be re-used as prestressing cables in the superstructure.
2. Structural advantages resulting from a reduction in axial creep and shrinkage deformation in the completed arch. Although the arch is placed in compression at an early age by the action of the inclined stay cables the axial forces increase gradually as the length of the arch rib increases until closure is reached. The residual creep on the arch rib after closure due to the dead load of the arch itself is therefore reduced considerably. The same applies to the residual shrinkage. Due to the ageing of the arch rib during the cantilever construction, the loads due to the construction of the superstructure are applied to...
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matured concrete with ages varying between one year and three months. The total creep due to this load is therefore also significantly reduced.

The stresses and deformations of the arch can furthermore be closely controlled by the careful adjustment of cable forces. Due to the early application of compression in the arch, cracking due to differential shrinkage and temperature during the construction stages is also reduced.

3. The distribution of residual long-term bending moments in the arch can be improved by preflexing the arch prior to closure by means of controlled adjustment of the stay cables.

Control of forces and deflections — The arch is, however, subjected to flexural stresses during construction which have to be limited to ensure acceptable levels of safety. The extent to which these forces and deflections can be controlled is determined by various factors viz:

1. The distance between cable groups which is a function of the maximum free cantilever capacity of the arch at the particular section.
2. The size of individual cable units,
3. The length of construction segments which is affected by the position of the arch diaphragms, the volume of concrete that can be placed in a single pour and the size and weight of the travelling formwork carriage, and
4. The configuration of the suspension system in general which is affected by the position and height of the temporary tie-back towers, and other anchorage points.

The configuration of the system developed is shown in Fig 1 and Photo 2.

Analytical requirements — The analysis of the construction stages of the arch must cover all conditions during the construction operation. The analysis therefore has to be carried out in great detail down to the individual stressing operation of each cable.

As the cantilever half arch is extended, additional cables are required while the forces in some cables, installed during an earlier stage, are reduced or in some cases a cable may become redundant. Consequently the configuration and indeterminacy of the structural system changes as many times as there are changes to the cables. The number of structural systems that are required to be analysed is dictated by the size of the structure, the structural (stiffness) properties of the arch, its flexural capacity as a cantilever and the number of cables required.

It is therefore extremely important at the planning stages of the temporary works to optimize these requirements in terms of the economics of the total system, the practical construction requirements and structural safety.

Planning of the arch temporary works

Utilization of the permanent works — In order to minimize costs of the temporary works, the completed portion of the permanent structure was utilized wherever possible as part of the composite temporary works. The following parts of the permanent structure were utilized for the temporary works system:

1. The tall columns were designed to support the suspension cables and temporary pendulum towers for the construction of the arch. These columns were also placed on the arch foundations in such a position as to have a stabilizing effect during the arch construction stages (Fig 1).
2. The foundations of the columns on the banks were utilized as temporary anchor blocks for some of the suspension cables of the arch (Fig 1).
3. Similarly the abutments were utilized both to stabilize the deck against wind forces as well as to provide anchorage for the suspension cables of the arch by using the deck as a temporary tie for the third set of suspension cables (Fig 3).

Suspension cables — Another important factor affecting the economics of the system was the type and size of the individual cable units. This had to be carefully considered as it affected both the effective utilization of the prestressing material as well as the geometrical arrangement of the whole system. A considerable saving could also be realized by the prestress wire being re-used after completion of the arch for the prestressing of the superstructure.

For structural reasons symmetry in the transverse direction was essential. The minimum number of even cable units per cable group was influenced thereby and was therefore an important factor to be considered. The optimum size of cables might vary depending on the structure and on the magnitude of forces to be resisted, the number of cable adjustments and other structural details.

There were obvious advantages for both the designer and the contractor if a standard sized cable could be used throughout. A cable consisting of 30 No 7 mm wires with a working load of 1.0 MN at 50 per cent of ultimate proved to have optimum characteristics.

During construction of the arch the suspension cables were protected against corrosion by corrosion preventative grease and polypropylene ducts which were sealed against water ingress. These ducts, which were white, also served as insulation against excessive temperature fluctuations in the cables.

Due to the long length of the cables, provision had to be made for cable adjustments of up to 850 mm which included allowances for the straightening out of the catenary sag of the cables. Allowances had to be made during the initial tensioning stages, when cable forces were relatively low, for the non-linear response of the cables. It was therefore important to use a fully adjustable prestress system whereby accurate extensional adjustments could be made repetitively without damage to the cables. By using intermediate couplers, cables could be made up to the required length by re-using portions of cables which had become redundant during the earlier construction stages.

In planning the geometry of the system it was necessary to strike a balance between the requirement to minimize bending moments in the arch and to minimize the cost of the temporary works. The ideal...
distribution of cables to satisfy the first requirement was a uniform distribution of cables closely spaced along the length of the arch. This would, however, have lead to an uneconomical usage of material and could have complicated the details, especially in the tie-back towers and during the stressing operations.

The most economical distribution of cable groups was achieved by the maximum use of the free cantilever capacity of the arch in terms of bending and shear. For Bloukrans Bridge this allowed three arch segments to be constructed as a free cantilever before support from a suspension cable at the leading end was required.

Temporary towers — The 'critical path' of construction for the bridge was controlled by the construction of the arch. It was therefore most important to be able to proceed with the arch construction as early as possible in the construction programme. This was achieved by utilizing the permanent columns on the arch foundations as tie-back towers for the first three suspension cables, making it possible to complete the first half of each cantilever before the temporary tie-back towers, supported at deck level, were required (Fig 1).

The temporary pendulum towers (Fig 4) could only be erected after the deck had been completed up to the tall columns. It was therefore of great importance to the contractor to minimize the time required for the erection of the pendulum towers. Similar criteria with respect to construction time taken for the removal of the towers was applicable after completion of the arch.

Although a steel structure would have had an advantage in respect of erection time, a design in precast post-tensioned concrete proved to be more economical.

The design of the pendulum tower was therefore based on the requirement of speedy erection and removal. The legs were constructed in hollow precast reinforced concrete blocks with matching contact surfaces. The blocks were post-tensioned together in a vertical position in two stages for each leg. The head section was constructed by precast reinforced concrete segments separated by alternate in-situ reinforced concrete segments all stressed together by post-tensioned cables to provide the required pre-compression for the individual slices to act as a beam supported by the two legs. Full continuity between the legs and head section was provided by post-tensioned cables.

The speedy removal of the towers after completion of the arch was facilitated by destressing of the precast head section and removal of the separated concrete sections (slices) by means of the Blondin cable crane. Similarly, the legs were taken down by splitting them at preformed debonded surfaces into blocks which could be removed by the cable crane.

The pendulum towers were supported on mild steel strip hinges which allowed free rotation of the tower at deck level in the direction of the main span. It was therefore necessary to follow a detailed pre-planned procedure during the erection of the towers, to ensure stability at all times and also during the installation of the first suspension and tie-back cables. During these initial stages, it was important to remove the cantenary sag in the cables, thereby obtaining a truly elastic system that would respond elastically under the subsequent application of dead loads on the arch and cable stressing operations.

This was most important for the accurate control of arch bending moments and cable forces and deflections. The towers were initially positioned to allow for subsequent rotations due to the extension of the tie-back cables as the tie-back force increased with progress on the arch.

Analysis of the arch temporary works — The analysis of the arch had to make allowance for each construction operation which would affect the forces in the various members.

A typical construction cycle for one segment comprised the following:

Day 1: Launch carriage and align the external side shutters. Proceed with stressing of the suspension cables (every third or fourth
segment). Place and fix reinforcement of bottom flange and concrete bottom flange.

Day 2: Complete web reinforcement (continue stressing).

Day 3: Move internal tunnel formwork into position and align wall shutters — concrete walls.

Day 4: Fix reinforcement of top flange and concrete top flange.

Day 5: Strip back-shutters and stop-ends.

Day 6: Wait for concrete in top flange to reach 17 MPa (normally 36 - 48 hours) and prepare carriage for launching.

For the analysis some of these operations were combined and the following load effects were investigated for each segment:

1. Segment dead load.
2. Movement of the arch carriage and formwork.
3. Temperature effects on cables and concrete.
4. Individual stressing operations of the cables.

From a construction point it was essential to keep the number of stressing operations to a minimum. In order to minimize restressing of a cable group, the maximum cable force had to be applied at the stage of first stressing.

The maximum force that could be applied at any stage was limited by the capacity of the arch at that stage of loading in terms of bending and shear, the magnitude of force which would be absorbed by the cable at the addition of dead loads subsequent to the stressing operation as well as the effects of temperature.

Fig 5 and 6 show two typical stages of construction. The applied bending moments resulting from the dead loads, the carriage, temperature and the cable forces are shown in conjunction with the bending moment capacity corresponding to the particular stage of construction. It can be seen that the most critical section is always situated at the anchorage of the leading cable in the arch. The reason for this is that the spacing of cable groups was based on the bending...
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The bending moment capacity as referred to above, had to be considered both at the serviceability and at the ultimate limit state. However, for the arch it was found that the limiting strain and stress criteria specified for the serviceability limit state governed during construction.

These criteria were:
1. The maximum tensile steel strain = 0.001
2. The maximum compressive stress in the concrete = 0.33 fcu

A check on the serviceability limit state of cracking showed that the above criteria limited the crack widths to acceptable values.

The above criteria were applied in the compilation of moment-thrust interaction diagrams for various sections of the arch, based on a linear stress-strain relationship and ignoring concrete in tension. These diagrams shown in Fig 7 were used to determine the maximum allowable bending moments for any co-existent thrust acting at these sections and thereby determining the cable-force limits at each stage.

A loading cycle at a particular stage of construction started with the dead load of the first segment at the leading edge after installation and stressing of a cable group, and ended with the cable stressing load case of the next cable group. The cable stressing operation was simulated on the pertinent structural model for that particular stage, assuming zero stiffness properties for the new cable that had still to be installed.

A change in the structural model was required, ie the addition of a new cable group and/or any changes in the stiffness properties of existing cable groups, before the next loading cycle could be applied. The effects of all load cases were accumulated sequentially to provide a continuous stress and displacement check during the construction procedure.

Analysis had shown that during the earlier stages of construction, the arch rib would be relatively stiff and deflections induced by dead load and cable forces would be small. During these stages the axial thrust was also relatively low resulting in reduced flexural capacity. Consequently force control in the cables was the controlling factor to avoid overstressing of the arch rib. This is illustrated in Fig 5 which shows that the actual bending moments in the arch varied between the positive and negative capacity of the critical sections.

During the middle and later stages, the flexural capacity of the arch was increased due to the higher thrust in the lower regions whereby the bending moments became less critical as shown in Fig 6. However, the deflection of the leading portion of the cantilever was considerably increased and profile control became a determining factor of the cable force requirements.

On the basis of risk considerations the cable capacities were governed by maintaining a factor of safety of two on the guaranteed ultimate tensile strength for the prestressing steel at all construction stages.

The bending moments in the arch at the stage prior to closure is shown in Fig 8. The forces in the cables were adjusted to obtain this particular distribution of bending moments which would, after closure of the arch and removal of all temporary loads, result in the bending moment distribution shown in Fig 9. This distribution was considered beneficial for the minimizing of the long-term stresses in the arch under permanent loads. The resulting bending moment distribution which differed from the 'one-go' bending moments (Fig 9), incorporated an adjustment through preflexing of the structure to allow for the effects of residual shrinkage and creep shortening of the arch rib after closure of the arch. Although a portion of the preflex would be lost due to relaxation, the residual distribution of bending moments for long term loads would still be an improvement on what would have been obtained for an arch theoretically constructed in 'one-go' on 'rigid centering'.

Application of cable forces — The stressing procedures and sequence of installing the suspension cables depended to a large extent on the equipment available for stressing.

Each cable stressing load case had to be subdivided into individual stressing operations in order to determine the actual forces required per jacking operation. The following example may serve to explain the procedure followed:

If it is assumed that a particular cable group consists of 12 cables and that four jacks are used simultaneously to stress these cables to a combined force of P, the group force can be applied in three operations. If the same force is applied in each of these operations ie P/3, then the final forces in the three subgroups of cables, after the last subgroup is stressed, will be different. Subsequent equalizing cycles of stressing are time consuming and not practical. It was therefore required to stress each subgroup to a predetermined force that would result in all subgroups having an equal force of P/3 after the last subgroup had been stressed. This was accomplished by analysing each structural system that existed at a stressing operation with 'unit' cable forces substituted in turn for each subgroup, whereas the actual cable group was represented by the number of cables that had been stressed at that stage.

In this particular case three sets of simultaneous equations had to be solved to obtain the required 'lock-off' forces for the three subgroups. The largest group of cables installed in this way was C8 (Ref Fig 1), the longest group, consisting of 22 cables which were stressed in six subgroups.

Arch construction profile and profile control — Creep effects in the concrete during construction of the arch were disregarded because of the insignificant effects on the suspension cable forces. Similarly the effects of creep on the concrete arch profile were ignored, because the stress condition at a particular section due to bending moment in the arch at that section would be of relatively short duration, fluctuating with the changing bending moments from tension to compression (Fig 5).

The required construction profile of the arch was determined from a

![Graph]( Fig 7)
separate stage-by-stage time dependent analysis, considering all effects after closure of the arch. The required precamber was superimposed on the final required profile in order to obtain a construction profile. The deflection of the leading two or three control points along the arch rib were carefully monitored during each stressing and loading operation in order to correlate the actual deflections with those theoretically calculated.

In order to obtain the correct arch profile at completion of the bridge, each segment had to be accurately set out relative to the previous segments, taking the absolute position of the arch into account at any stage of construction. The effect of temperature on the cables and on the concrete had to be considered at each stage in order to relate all deflections and cable forces to the same datum temperature. Provision had also to be made for the measurement of gradient temperatures in the arch box section as these effects were particularly significant for setting out of the new segments and stressing of the leading cables.

**Arch closure** — The arch closure operation consisted of three main stages, viz:

1. To lock the two suspended half arches temporarily together, thereby eliminating relative displacements between the two systems in order to allow the green concrete in the closure segment to mature without being subjected to excessive strains;
2. To complete construction of the in-situ closure segment, and
3. To transfer load from the temporary locking system and the suspension cables to the completed arch rib.

The temporary locking system had to be designed to withstand the following actions without subjecting the maturing concrete in the closure segment to excessive strains:

1. Temperature effects due to temperature variation relative to the temperature at locking of the two half arches.

**Arch**
- Uniform temperature change = +15°C, -5°C
- Gradient temperature change of ±5°C, ±5°C (ie reversible)

**Cables**
- +20°C, -10°C
- Differential temperature between the two suspension systems = 5°C.

2. Wind loads acting on the whole structure as specified for the particular construction stage. (Refer the accompanying article on Bloukrans Bridge)

3. The dead loads of the closure segment.

It was an essential requirement that up to the actual concreting of the closure segment, all operations could be reversed and repeated if necessary without risk of damage to the structure.

After completion of the cantilevered construction of the two half arches, a closure gap of 2.6 m remained with 12 No 600 × 450 mm concrete struts cast as monolithic appendages at the inside corners of the cellular compartments of each half arch. These struts protruded across the closure segment to leave a 100 mm gap between opposing struts (Fig 10). The struts had two ducts each to accommodate temporary prestressing cables. The two formwork carriages were moved forward to bridge the gap of the closure segment, without however making contact. Each carriage was clamped to the leading edge of the lower flange of each half to avoid steps being formed due to the deflection of the carriages when subjected to the load of the fresh concrete.

The arch closure operation was planned to be carried out during a period when temperatures would be low and stable (concrete and steel temperature between 10°C and 15°C) with preferably no wind. From a pre-planned programme, foregoing weather observations, and experimental work on quick setting cement cubes (Ciment Fondu), it was established that the operation had to start at midnight to ensure that the locking of the two half arches would be completed before a rising temperature cycle started.

After all preparations had been carefully checked beforehand and all
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adjustments to the suspension cables had been completed, the two half arches were carefully aligned by stressing the crossed over prestress bars (Fig 10). Although an adjustment of 50 mm could be achieved with this system it was only necessary to apply a 15 mm adjustment. After locking the two half arches in the vertical direction by means of the crossed over prestress bars, all longitudinal prestress cables were installed in the temporary concrete struts, together with a temporary mechanical jack between each strut. After expanding the jacks to make full contact, one cable in each strut was stressed to a force of 0.57 MN. Hereafter the gap between each strut was filled with a quick setting cement morter (Ciment Fondu) leaving the mechanical jacks exposed in recesses. This operation had to be completed by approximately 3h00. After approximately 3.5 hours when the mortar filling had achieved a 15 MPa compressive strength the mechanical jacks were removed by destressing and the recesses filled with the same quick setting mix. When a strength of 30 MPa was reached on the original filling after approximately five hours, the remaining cables in the top struts were stressed to 0.57 MN each. At 35 MPa the remaining cables in the lower struts, which required a greater prestress force, were stressed to 1.115 MN each.

At this stage the arch was fully locked and construction of the closure segment could be proceeded with in the conventional manner. When the concrete in the closure segment had reached a compressive strength of 20 MPa, the quick setting cement mortar filling between the temporary struts was removed by drilling and breaking out in a symmetrical sequence thereby transferring the combined compressive force of the temporary locking cables to the cross section of the arch and eliminating excessive tensile stresses developing in the young concrete.

The destressing and removal of the suspension cables was then proceeded with following a detailed sequence to ensure a gradual symmetrical transfer of force from both sides without over stressing the arch or cables. The specified procedure ensured also that the Pendulum towers remained approximately vertical until they could be stabilized by temporary bracing.

When cable C8 (Refer Fig 1) had been reduced to 10 cables in temporary locking cables at the closure segment were destressed and removed.

The planning, design and analysis of the Bloukrans Bridge (continued from page 173)


Fergusson, P. M., and Breen, J. E. Investigations of the long concrete column via frame subject to lateral loads. ACI, SP 13, paper 4, 1965.


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