Proceedings of the Institution of Civil Engineers
Bridge Engineering 162
December 2009 Issue BE4
Pages 167–187
doi: 10.1680/bren.2009.162.4.167

Paper 900006
Received 18/05/2009
Accepted 24/08/2009

Keywords:
bridges/concrete structures/piles & piling

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Design and construction of Clackmannanshire Bridge, Scotland

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The Clackmannanshire bridge at Kincardine, Scotland is not only one of the largest bridges in the UK, but is also the second longest incrementally launched bridge in the world. The 1188 m long bridge provides relief to Kincardine on a route that crosses the Firth of Forth immediately upstream of the existing Kincardine Bridge. The environmental sensitivities of the Forth estuary have shaped almost every aspect of the scheme, including route selection, structural form, construction methods and programme.

1. INTRODUCTION

This new crossing of the Firth of Forth carries a three-lane carriageway for the A876 trunk road on a 1188 m long bridge, all as part of a £120 million design and construct project that was built on behalf of the Scottish ministers. Initial studies and the specimen design for the project were developed for Transport Scotland by Jacobs, in the period from 1998 to 2005, and tender documents were released to three design and construct tenderers in August 2005. Morgan-Vinci was then awarded the lump sum contract for the works in February 2006, with a start on site in June 2006. The company appointed Benaim as the designer of the main bridge crossing of the firth, Fairhurst as the designer of all the other highway and bridge works, Gifford as the independent checker and Alexander Associates as the environmental consultant. Jacobs was retained by the client as their consultant to oversee and monitor the works as the engineer.

The 6·4 km route, from the M876 connection in the south at the Bowtrees roundabout to the A977 connection at Kilbagie, acts as a relief road to Kincardine. The route crosses the firth at a 40° skew, which was chosen to minimise the environmental impact on the significant areas of mudflats and salt marshes. The project is part of Transport Scotland's twin bridge strategy for Kincardine. Completion of the new bridge will allow refurbishment of the existing crossing, which was built in 1936 and is now in need of substantial repair.[1]

2. INITIAL STUDIES

2.1. Route selection

Four initial route options were developed in the period from 1998 to 2002 and a preferred route was selected that was upstream of the existing bridge. The closure of the Kincardine power station in 2002, however, allowed a further route to be considered that took the skew crossing from the Higgins' Neuk area on the south of the firth to the upstream end of the power station site on the north. This was the preferred route that was then taken to the public inquiry and consultation in 2003. The feedback from the inquiry was generally very supportive of the scheme and of the chosen route.

The relevant landowners were consulted throughout the process and agreements were reached with them all prior to the lodging of the formal road orders.

2.2. Environmental challenges

This region of the firth is defined as a special protection area (SPA), as well as having sites of special scientific interest (SSSI) on both banks and Ramsar sites on the south. It was important to maintain the water quality and marine habitats in the firth, and to protect the mudflats and salt marshes for a variety of migratory birds. To mitigate environmental impact, Jacobs proposed that construction should be restricted to a period including no more than two consecutive winters. This had been determined as the period beyond which the migratory birds might not return to their nesting grounds if unduly disturbed. The proposed mitigation measure was accepted by Scottish Natural Heritage (SNH) and the client, and was ultimately adopted as a contract requirement. It was also specified that all launching activities should take place from the less environmentally sensitive north shore, within the previous coal yard of the old Kincardine power station (see Figure 1).

SNH and the Scottish Environmental Protection Agency (SEPA) were consulted at all phases of design development and were party to the approval of the environmental impact assessment (EIA). The EIA was created by Jacobs on the basis of the specimen design and was later amended, with the approval of SNH and SEPA, by Morgan-Vinci to suit its value engineering changes.

In particular, the Kennet Pans local wildlife site on the north of the firth was extended by putting an 8 ha area of the old power station on the upstream side of the bridge back to the firth. This created a significant environmental benefit to the area, to mitigate the impact of the bridge on the south side.

The visual appearance of the bridge was considered throughout

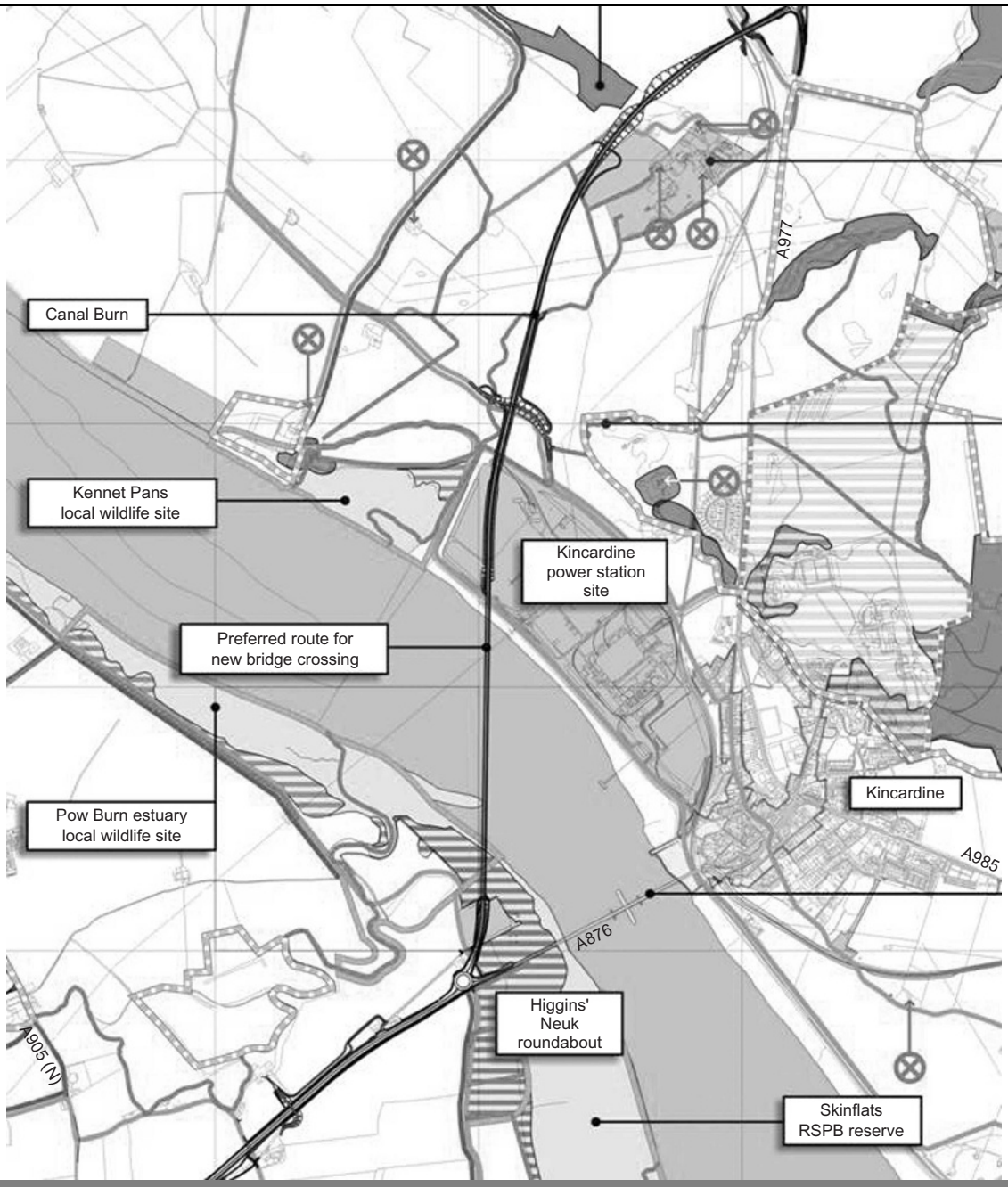

Figure 1. Plan layout

the tidal range, which varied from +2·95 m to −2·35 m. The resulting bridge is a low-level crossing over 26 spans, each of which is typically 45 m long. Its slender deck was incrementally launched from the north side of the firth and has a curved soffit that rests on single circular columns, all of which were designed to minimise visual impact.

The environmental sensitivities of the Forth estuary have thus shaped almost every aspect of the scheme, including route selection, bridge design, construction methods and programme.

2.3. Client's requirements

Transport Scotland required a high-quality bridge with a low visual impact. A low-level crossing was thus specified with 26 spans of 36 m, 10 × 45 m, 53 m, 65 m, 53 m, 11 × 45 m and 36 m. The 65 m central span was required to have a navigation clearance width of 60 m and height of 9.2 m above the high tide, and the four central piers were needed to carry the ship impact load from a 750 t vessel. The deck layout was to incorporate a wide single 2+1 carriageway 11 m wide, two 1 m wide hardstrips, a 2·5 m wide footpath/cycleway and a 1 m verge. This gave a total deck width, including parapet edge beams, of 17·5 m. The early conceptual designs prepared by Jacobs were for a very shallow deck of around 2 m depth to give a ribbon-like appearance but this was ultimately increased to 2·8 m for practical construction and maintenance reasons. A doubly curved soffit was required for the deck cross section. It was to be supported on single, circular columns of constant diameter – to avoid the 'forest of columns' effect that might otherwise result from twin columns at each pier location, especially owing to the 40° skew of the crossing. The crossing was to be straight in plan and to have a large radius curve in elevation, of around 29 000 m (see Figure 2). The bridge was to be designed to carry HA and 45 units of HB live loading.

To ease future maintenance, it was also specified that, for a post-

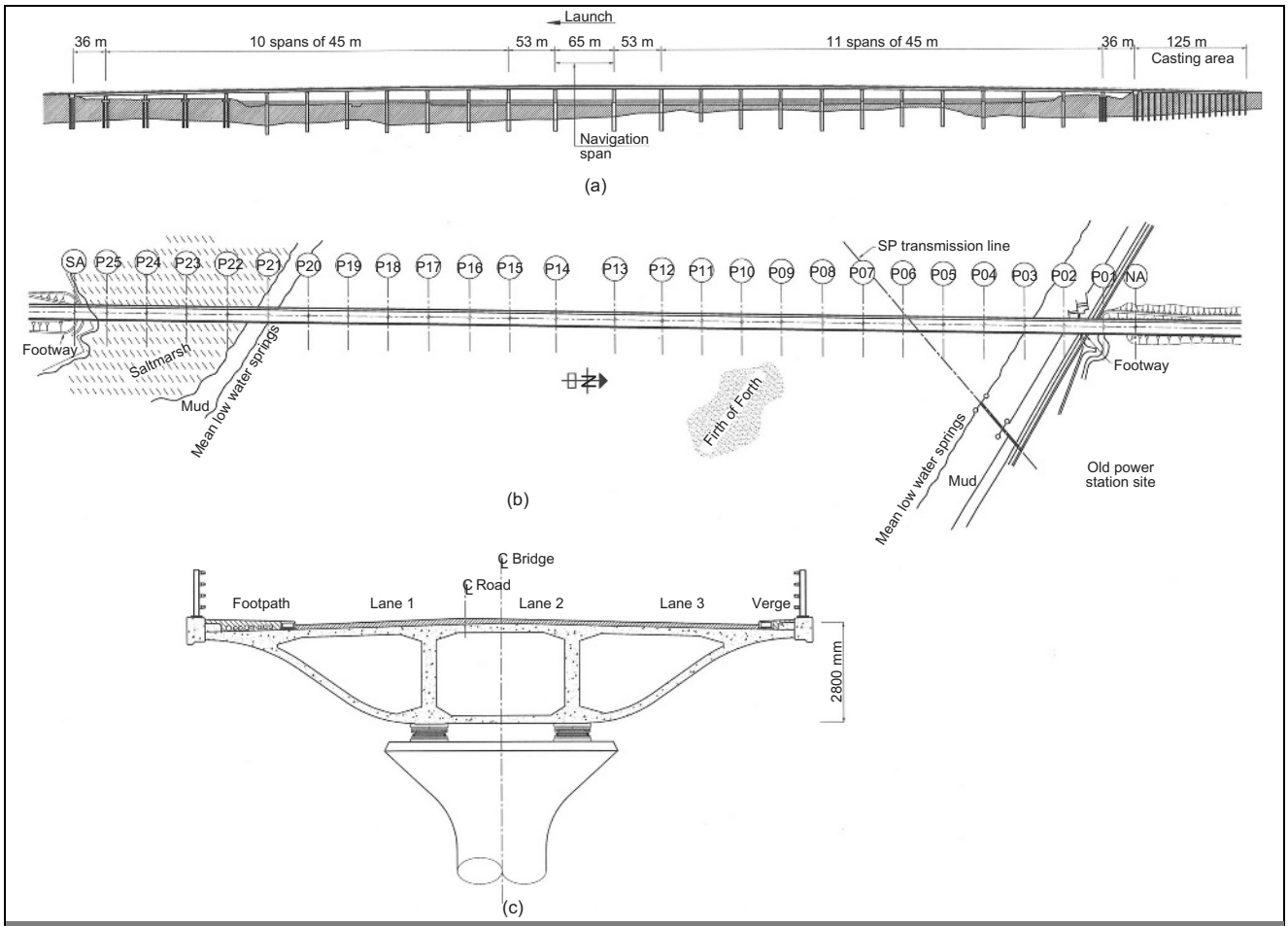

Figure 2. General layout: (a) elevation; (b)plan; (c) deck section

tensioned concrete solution, prestressing tendons be replaceable, and that 25% of the tendons could be removed without requiring any traffic restrictions.

3. TENDER DESIGN

The essence of successful design and construct projects is to take the collective expertise from previous schemes and to develop it further, in order to optimise the chosen solutions. Benaim worked closely with the design department of Morgan-Vinci to consider a range of solutions including incremental launching, precast segmental, steel-composite plate girders and steel-composite box girders. All solutions were priced and programmed to suit the client's requirements, particularly the need for a doubly curved soffit. It was clear that a three-cell, incrementally launched, concrete box girder was the optimum solution, and that it could be launched, within the programme constraints, from the north side only.

Both Morgan-Vinci and Benaim had previous and very recent experience of major launched bridges in the UK. Benaim had designed two incrementally launched bridges in Ireland – the 310 m long Broadmeadow Estuary Bridge[2] and the 450 m long Blackwater viaduct.[3] Both were unusual in having longer spans than would normally be launched, at 69 m and 58 m respectively. They were thus launched with the use of temporary props at midspan, and this enabled them both to be launched as reinforced concrete structures.[4,5] They were prestressed on completion, with external cables that were only anchored at the abutments. This use of external cables also enabled the deck sections to become partially prestressed – a condition allowed in BD 58[6] whereby the section is fully compressed under permanent loads, but where the live loads are shared by both the prestressing and the longitudinal reinforcement in the deck.

Morgan-Vinci had also just constructed the two approach viaducts of the Channel Tunnel rail link (CTRL) Medway crossing,[7] each 650 m long, and designed and constructed the 1025 m long CTRL Thurrock Viaduct.[8] All these structures were classically prestressed with internal cables, both for launching and on completion. The casting techniques and methods used, and the key items of jacking equipment, were all to become vital components of this new crossing.

The tender design thus became a 2·8 m deep concrete box girder with a doubly curved soffit. The marine environment and the 45 m spans determined against the use of temporary midspan props and thus the section could not be launched as reinforced concrete. It was thus felt at this stage that a classic prestressing solution was appropriate. It therefore became a fully prestressed section with a series of straight, internal launching cables, and profiled, external continuity cables. Grade 60 concrete was used throughout, although this increased to grade 70 in the slender areas around the 65 m span. This need for higher-strength concrete to control the compressive stresses began to indicate that the section was too heavily prestressed, and this became a motivation later to move towards partial prestressing.

For the typical spans, the 3·75 m diameter piers, a size specified

by the client, sat on single 3 m diameter bored cast in situ piles. This solution eliminated the group of piles and thus the need for a pilecap. Not only was this a good step forward in material savings, but it was a large step in eliminating the temporary cofferdam that would otherwise be needed in this marine location. The 3 m diameter pile was sized for both strength and stiffness, with the launching condition being critical. In any event, the main reinforcement in the pile was relatively modest, at less than 1% of the pile area. The deck was continuous throughout with guided bearings at every pier. It was fixed longitudinally on the four central piers, where the ship impact load was also to be carried. At this stage, these four piers had a group of three 3 m diameter piles within a circular pilecap of 15 m diameter. This pilecap was offset from the pier centreline to create the full 60 m shipping clearance all the way down to river bed level.

The use of the 3 m single piles had been developed with Fugro Seacore at this stage, and it was their potential ability also to create single bored piles of up to 6 m diameter that allowed a late revision to the tender scheme. This amended the group of three 3 m diameter piles at the fixed piers to a single 6 m diameter pile, with no pilecap.

4. DETAILED DESIGN

4.1. Value engineering

Once the contract had been awarded, it was possible to optimise the scheme further. This was partly through a development of ideas between Benaim and Morgan-Vinci, partly through further discussions with the client and Jacobs, but also by close dealings with the major subcontractors, particularly Fugro Seacore and Freyssinet.

The deck was amended to incorporate a wider section of flat soffit, which was easier for the launching process. It was also decided to make the launching cables external, as opposed to internal – this allowed the partial prestressing clauses of BD 58 to be invoked, resulting in a saving in prestressing of over 50%. But it also enabled much of the prestressing to be taken off the critical path, allowing a faster and easier casting cycle. With less prestressing, it was also possible to reduce the concrete grade, although grade 60 was kept for considerations of shear capacity and early-age strength requirements.

Much discussion took place regarding the appearance of the piers, the pier tops, and their interaction with the deck soffit. Morgan-Vinci employed Yee Associates as its architectural advisor, and many meetings were held with Jacobs and their architectural advisor to resolve this delicate area. The size of the pier top was governed by both the need for the two permanent bearings, at 4 m centres transversely, and the need for the launching bearings/jacks in the longitudinal direction. The pier shaft diameter was modified to be 2·5 m for the typical piers and 3 m for the four central piers. The larger piers were needed to resist the ship impact loads and were also visually correct for the longer spans. The transition in both directions from the circular shaft to the elliptical pier top was finally considered through three-dimensional (3D) visualisations and 1:50 scale models, resulting in a very elegant shape that Morgan-Vinci has been able to turn into reality (see Figure 3).

Figure 3. Model of pier and deck

The ship impact load was reduced in this period from about 15 MN to 11 MN, and the four central piers were considered as being pinned to the deck, which allowed a greater transfer of load to other piers. These factors allowed the single 6 m diameter pile for the central piers to be reduced to around 4·5 m. It became apparent from discussions with Fugro Seacore that a significant cost in re-tooling their drilling equipment could be achieved if the pile diameter was limited to 3·85 m. At some extra cost and density in pile reinforcement, it was decided to adopt this size.

A further advantage of these amendments was to allow the pile casings to be used as the cofferdams for the construction of the pier shafts. The typical 2·5 m piers thus sat within the 3 m pile casing, and the 3 m central piers sat within the 3·85 m pile casing. This was a significant saving for the safety of the marine operations, as well as being faster.

4.2. Geology

This area of the Firth of Forth is underlain by alluvial and glacial deposits over rock of the coal measures series of carboniferous age. These complex and variable layers of mudstones and sandstones were known to be faulted. A major fault, the Kincardine ferry fault, was known to traverse the firth in a NW–SE direction and was believed to cross beneath the bridge about 350 m from the south abutment, near pier 18. The fault was believed to down-throw the strata to the north by about 185 m. To the north of this fault, the strata are of the lower coal measures series comprising a sequence of sandstones, siltstones and mudstones with thin coal seams. South of the fault is the passage formation that is predominantly sandstones with thin siltstone and mudstone bands and a few coal seams. None of these thin coal seams were known to have been worked in this area. A second area of faulting around pier 4 was apparent from irregularities in the borehole data, and the pile casing for this pier did have to be sunk to lower depths than had been expected (see Figure 4).

The unconfined compressive strengths (UCS) of the sandstones were in the range from 10 to 50 MN/m^2, indicating them to be moderately strong. The weathered mudstones were in the range from 1 to 10 MN/m^2, indicating them to be very weak to moderately weak.

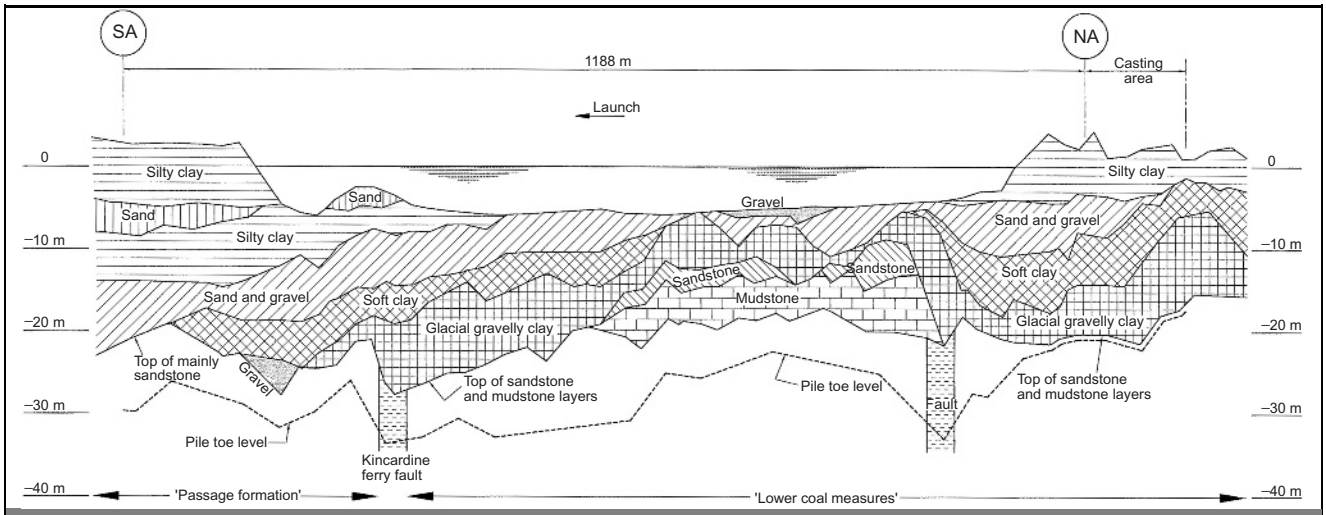

Figure 4. Geological section of the crossing

The upper deposits consist of medium dense to dense sands and gravels over soft clays above stiff, gravelly glacial clay. In some areas, soft clay and sand overlies the upper sands and gravels. Extensive marine site investigations had been carried out in 2004 by Fugro Seacore, but it was felt important to carry out further boreholes, both to complete the package of data in all areas and at all levels, and to ensure that there was an individual borehole at each pile position. Further investigations were therefore carried out by Fugro Seacore in August 2006.

River bed levels vary down to about −6 m, giving water depths from zero to about 9 m.

4.3. Single piles

Continuous flight auger (CFA) piles of 750 mm diameter were used to support the north abutment, the casting area, and pier 1, which was on the land. Bored piles of 1·5 m diameter were used to support the south abutment and the four piers that were in the mudflats, piers 22 to 25. The particular design innovation with regard to the piling was, however, the use of 3 m to 3·85 m diameter single piles in the firth to support the 17·5 m wide deck. This was a substantial step forward in bridge building techniques, especially given the complex and variable geology beneath the firth. Benaim had used single piles previously for viaducts in Kuala Lumpur[9] and Hong Kong[10] and so, with the combination of Fugro Seacore's marine expertise in reverse-circulation drilling into hard rocks, the choice of single piles was clear. The main advantage was the elimination of the pilecaps and resulting cofferdams, which would always be difficult to construct in the water (see Figure 5). By definition, the single piles also have a smaller pile area than the equivalent group of piles, by around 25%. This is simply attributable to the fact that the bending moments are carried in pile bending rather than push–pull in the pile group.

The piles were designed to carry the vertical load by a socket action within the rock, with part of the load carried by skin friction and part by end bearing. Settlements were also considered by an assessment of the deformation of the rock mass. The design parameters were determined so as to eliminate the need for pile testing, which would have been very difficult in the marine environment and unrepresentative as the ground was so variable.[11,12] Sonic logging tubes were installed in every pile

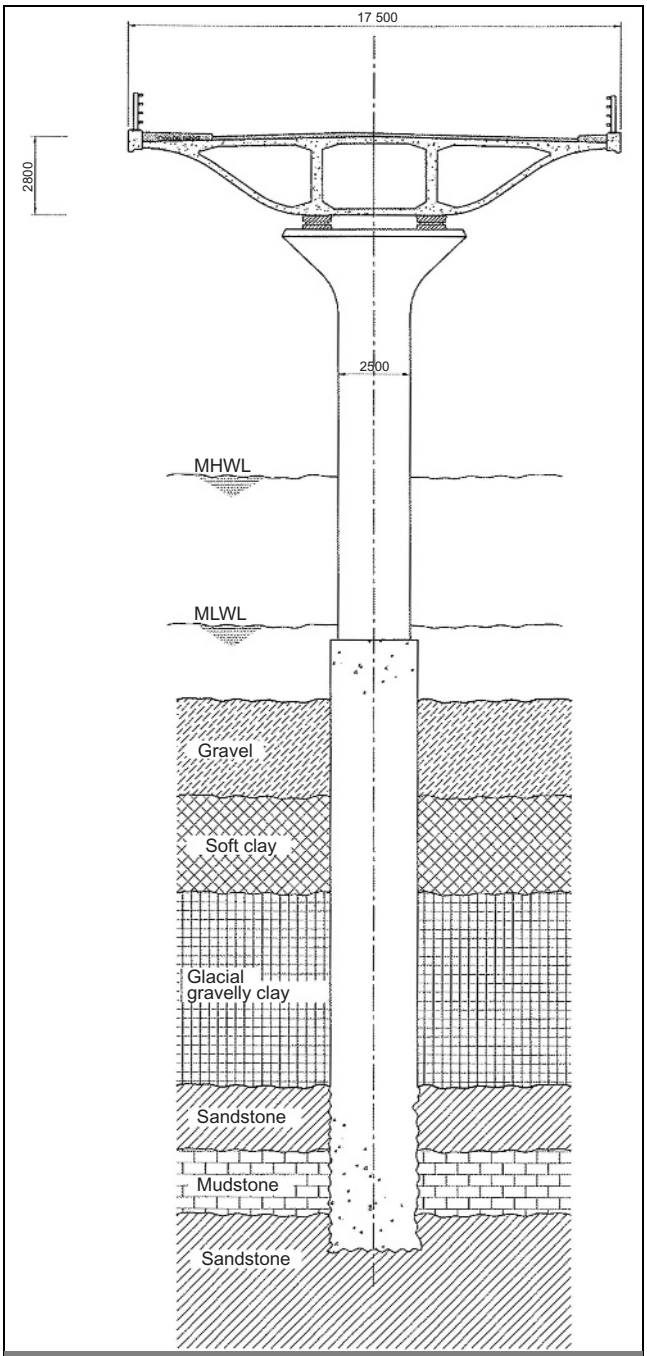

Figure 5. Typical deck section (dimensions in mm)

with the provision for some of the tubes to accommodate a further bore, in case it was felt necessary to core through the pile toe and rock interface for testing. This was done at pier 2 and a good bond was found. A bespoke design was therefore carried out at each pile to determine the toe level, the reinforcement cage length and for Fugro Seacore to determine the length of casing required (see Figure 6). In general, the piles were cased down to the rock, with the load capacity measured from below that level, but in some instances it was necessary to case through the weaker mudstones in order to support the bore, and the pile toe levels had to be adjusted to suit.

In reality, with maximum pile stresses of 3·5 MN/m^2 and a philosophy of always having the pile toe in the moderately strong sandstone, the factor of safety for the load was always well over 3, and load testing was not required. The rock socket for a pile entirely in sandstone was about 4 m, whereas for one in the mudstones, it was about 14 m. Each pile varied between these broad limits to suit the exact layering of the strata at each location, and toe levels varied between −22·5 m and −34·5 m.

In order to control any differential settlements during the launch, each pile was also designed for a 'no slip' condition on the shaft friction.[13–15] This condition was the determining factor for the rock socket length – the expected settlements under the full loads were then around 10 mm and about 5 mm under the launching condition. Elastic shortening of the piles would add about 3 mm to these figures. The recorded settlements of the piers during the launch were indeed in the 5 to 8 mm range, thus justifying the design process.

The typical 3 m piles were also sized to limit the horizontal pier top deflections under the launching loads to less than 50 mm. The typical pile was first modelled with finite element analyses to confirm that simple plane frame models with ground springs could then be used for the actual design. The variable ground springs above the rock were sized using the coefficient of variation of horizontal subgrade reaction, n_h, where n_h is multiplied by the depth below river bed level to give the subgrade reaction. Values for n_h of 1·5 (in MN/m^2 per m) were used in the soft clays, 3 in the stiff clays and 10 in the medium dense to dense gravels. For the rock, the models used constant sub-grade reactions of 200 MN/m^2 for the mudstones and 500 MN/m^2 for the sandstones (see Figure 7). Checks were made to ensure that the soil bearing pressures were consistent with the ground movements. Sensitivity analyses were also carried out to vary the springs by up to 50% and the results showed that the pile bending moments were affected by less than 10%. Moments were added to the system to allow for pile tolerances of ±75 mm in position and of 1:75 in verticality. The ultimate bending moments in the pile peaked at around 35 MN m, only needing a single layer of T40 bars at 200 mm centres (see Figure 8).

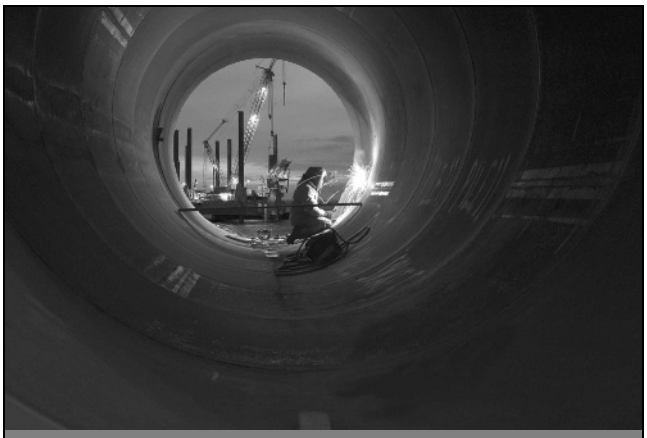
Figure 6. Welding of the pile casings

Although in one sense the pile analysis was very simple – it was just a stick in the ground – in another sense, great time and effort was used to ensure that a careful site investigation had been carried out, that it was bespoke to every pile and that the results were not too sensitive to the actual assumptions made.

The four central piles were constrained to be no more than 3·85 m diameter, which as noted previously was less than would have normally been chosen in the design. The ship impact load case was critical and, owing to the 40° skew of the bridge to the firth, the load could be applied in any direction. The deck was able to share the load to the other fixed piers though, with about half the 11 MN impact load being transferred from the pier that was hit. This behaviour generated peak ultimate moments of around 120 MN m, which needed three layers of T40 bars at 150 mm centres.

4.4. Deck section

The doubly curved soffit of the deck was created by using a three-cell concrete box girder. The main webs of the box were vertical and at 4 m centres, and the outer webs that form the soffit shape were fully utilised in all the structural actions. The 4 m spacing was chosen so as to create a good distance between the bearings and a good space within the central cell of the section. Both the launching bearings and the permanent bearings sat directly beneath the webs of the box to avoid any moments caused by eccentric reactions (see Figure 9). The central cell incorporated the majority of the prestressing and thus had to be made wide enough to house most of the cables and anchorages, and to allow easy striking of the inner mould through the previously cast units.

All slabs were made 225 mm thick, which was the minimum to suit the cover requirements of BS 8500[16] and the need to maintain a good effective thickness in bending. The vertical webs were kept to a thickness of 400 mm throughout, with a slight increase to 450 mm local to the piers. There was a strong desire to keep all these thicknesses down, not only to reduce weight during the launch but also to ease the manufacture and use of the mould panels. These compressions and shears in the section were thus controlled using grade 60 concrete.

The pier diaphragms were made to serve several purposes – to carry lateral shears and torsions, to carry eccentric bearing reactions when the bearings were being replaced, to anchor all the prestressing cables and to provide a length over which the cables were able to lap each other. The diaphragm in the centre cell was 3·5 m long with a central walkway, whereas the outer cells had a 3·5 m long transverse beam placed at the top of the cell, which then provided anchorage for half of the launching cables and allowed full access through the outer cell.

The continuity cables were typically only provided in the centre cell and these were profiled by deviators at third points in the

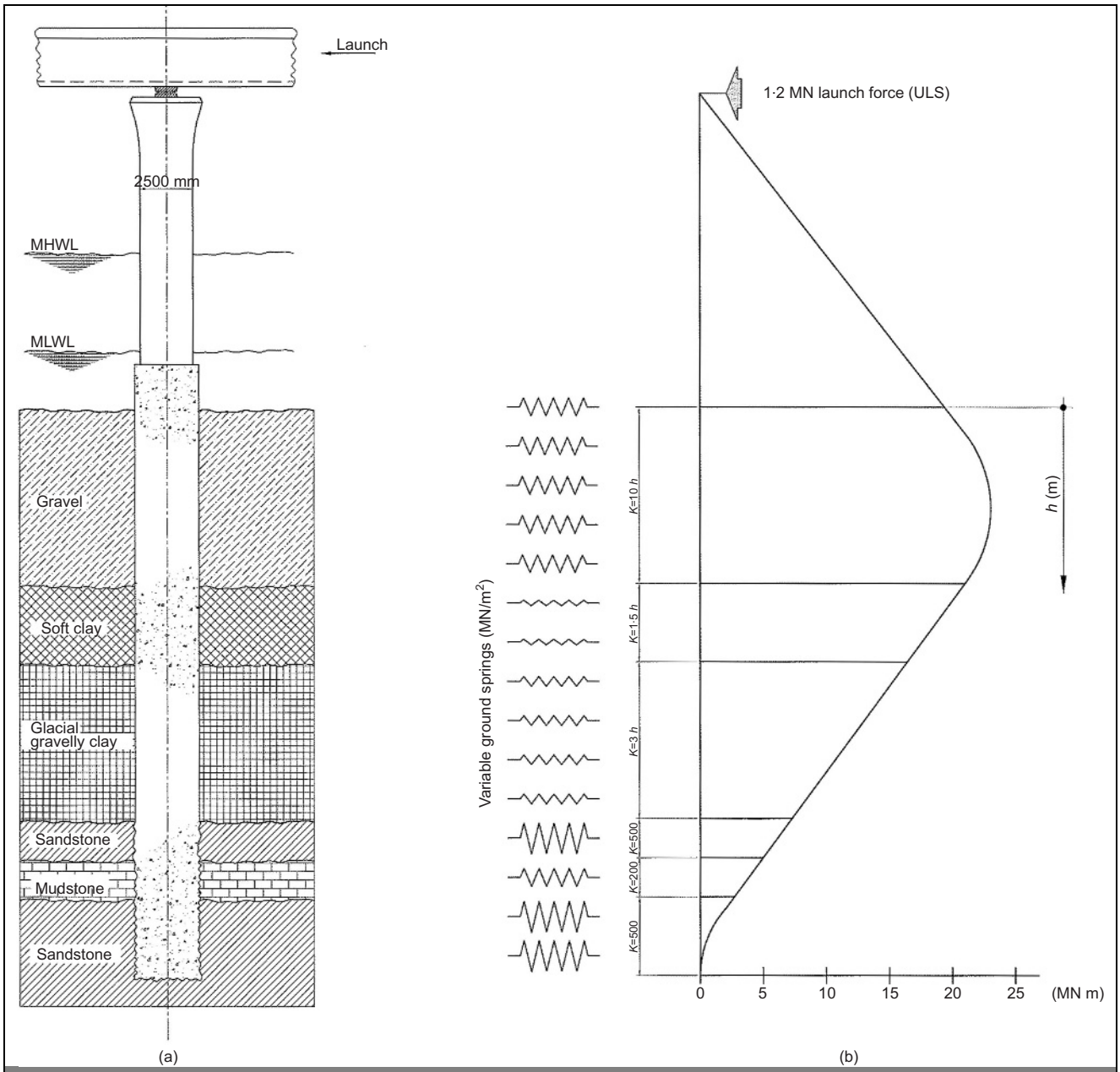

Figure 7. Single pile layout and moments: (a) longitudinal section; (b) launching pile/pier moments

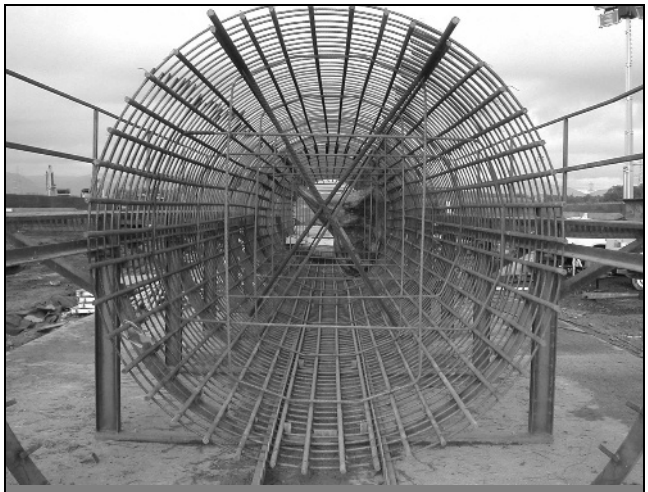

Figure 8. Typical pile reinforcement cage

span, which provided reaction points to carry the loads back in to the webs of the box (see Figure 10).

4.5. Partial prestressing

The second major innovation was the use of partial prestressing in the bridge deck. By using external prestressing cables throughout, – that is, for both the continuity cables and the launching cables, it became possible to share loads between the prestressing and the longitudinal reinforcement in the deck; passive reinforcement that would otherwise be ignored. This approach highlighted inconsistencies in the concrete codes. BS 5400 part 4[17] allows class 3 (cracked) members and gives tensile stress limits for construction; BD 24[18] does not allow class 3 members at all and BD 58 gives crack width limits for the service condition but not during construction. A departure from standards was therefore sought to allow a crack width limit of 0·25 mm to be used during the launch condition. This was granted on the basis that the 0·25 mm limit would be adequate for reinforced concrete and the durability of the prestressing

Figure 9. Launching details

tendons would not be compromised because they were external (see Figure 11).

The straight launching cables were chosen to give an average axial stress on the section of 2·5 MN/m^2, which was about half the stress that would have been required to fully compress the section. This was provided by eight 19/15 mm straight cables, four in the centre cell and two in each of the outer cells. These cables provided an axial stress only, which was necessary to accommodate the full moment range that every section experienced during the launch cycle. On completion of the launch, a further set of profiled continuity cables was installed, which consisted of six 19/15 mm cables, all located in the centre cell. The launching cables were typically two spans long and the continuity cables were typically three spans long. The length of the launching cables was chosen to have no more than 50% of the cables lapped at any pier diaphragm and to give a gradual increase in the axial compression as the bridge moved away from the casting area. The longer spans around the centre of the bridge had more continuity prestressing, using up to eight 37/15 mm cables. The details of these arrangements were developed through close working between Benaim, Morgan-Vinci and Freyssinet.

The continuity cables were sized so as to keep the section fully compressed under the long-term permanent loads. This was achieved, in combination with the launching cables that were left in place to become permanent cables, with a very simple cable profile that used deviators at third points in the span. The final design to carry the live loads was thus created through tuning of the reinforcement, not the prestressing. It was actually found, however, that the reinforcement needed to carry the launching loads was sufficient to carry the majority of the live load cases. Ultimate load cases were generally not critical with the section usually being sized by crack width limits. In these cases, the section was treated as a reinforced concrete member with an applied force and moment from the prestressing. Crack widths in the long term, with HA live loads only, were limited to 0·25 mm on the top surface and 0·15 mm on the soffit, and during the launch a limit of 0·25 mm was used throughout.

Shear lag calculations were prepared using finite element models for a typical span and these were used to guide the placing of the longitudinal reinforcement into sensible panels. The top surface was critical throughout, whereas the soffit only needed marginally more reinforcement than would normally be provided. Over the whole of the central cell, the longitudinal reinforcement in the top slab was typically T25 bars in the top surface and T20 bars in the bottom surface, both at 150 mm centres. Away from the webs, these bars dropped to T16 and T12, respectively (see Figure 12).

This efficient use of the passive longitudinal reinforcement enabled the section to be less heavily compressed. The average axial stress on the section was 4·5 MN/m^2, which was reasonable for a bridge of this nature. If it had been fully compressed to carry all the loads, the axial stress would have reached 7·5 MN/m^2 and nearly 10 MN/m^2 in the longer spans, which in the current authors' view were too high. The use of partial prestressing reduced the tonnage of cables by 50%, with an increase in the tonnage of deck reinforcement of only 20%. There was 500 t of prestress saved and 500 t of reinforcement added, which equated to not only a material saving but also a saving in the time, and ease, of fixing in the casting area (see Figure 13).

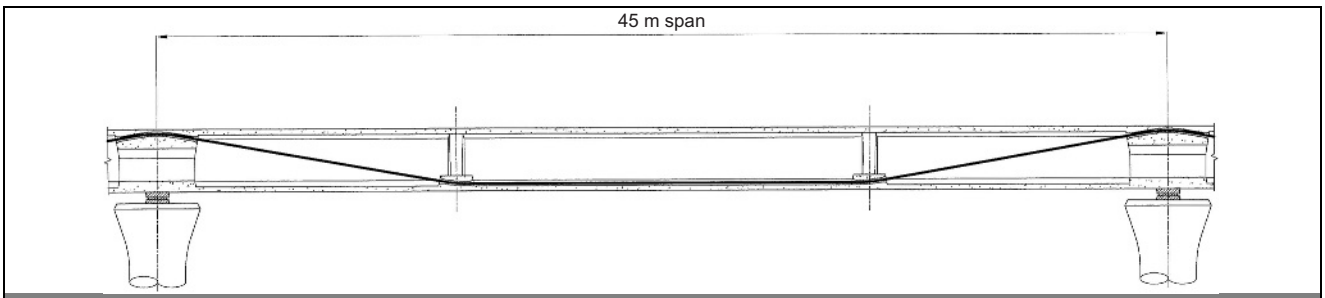

Figure 10. Typical prestressing layout. Continuity cable: longitudinal section

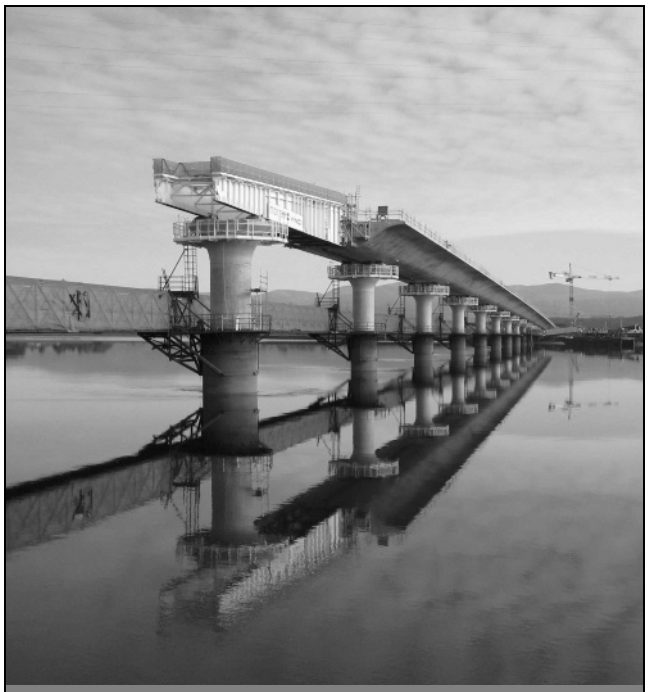

Figure 11. Launching of the deck

5. CONSTRUCTION

5.1. Programme

Three bidders – Morgan-Vinci, Balfour Beatty and a Bilfinger Berger-Mowlem JV – were invited to prepare design and construct tenders between August 2005 and December 2005. The Morgan-Vinci tender was accepted in February 2006 and design commenced straight away, though the formal contract award was not until April 2006 with a start on site in June 2006.

The site phase ran from 5 June 2006 to 31 August 2008, which was a 27-month construction contract over two winter seasons, with completion of the main bridge before the end of August 2008 when the third winter season was deemed to start (see Figure 14).

5.2. Casting area

The 125 m long casting area used two skidding beams that were located under the two main webs of the box girder. These were supported, through 20 m of soft alluvial clays, on 130 750 mm diameter CFA piles that were drilled into the top of the hard sandstone, some 20 m down. Control of settlements under the casting loads was critical and the pile loads were determined so as to limit the settlements to less than 2 mm. The top surfaces of the skidding beams were finished with adjustable steel plates and a second stage pour of micro-concrete, to achieve tolerances of less than 0·4 mm. This determined effort at the start of the works, together with the use of specially formulated silicone greases, ensured that the steel-on-steel friction in this area was kept to less than 15% (see Figure 15).

It was envisaged at the start that the 32 500 t deck would be pushed from the rear by the two jacks that had been used on the CTRL launched bridges. These hydraulic jacks were clamped to the sides of the skidding beams and had a total capacity of 1200 t. A further 600 t of reserve capacity was to be provided by lift-and-push jacks at the north abutment. The required friction resistance was assessed at every launch and was monitored so as to predict the final forces required (see Figure 16). In the end, with low friction in the casting area and frictions of less than 2% on the launching bearings, the maximum push needed once the deck was moving was less than 1200 t. A third 200 t jack was installed at the rear of the deck for the last two launches, just to get the deck moving, that is to overcome the initial static friction, which was always about 20% higher than the moving friction.

The highest expected horizontal reaction of 1800 t was carried by all the CFA piles beneath this whole area, simply acting in bending. It was found that the number of piles needed to limit the settlements in the area could readily carry this horizontal load using about 1% reinforcement – this only required T25 bars at 200 mm centres in the piles. The bending moments in the piles were assessed using similar ground springs that had been used for the marine piles, and the moments had disappeared, even in these soft clays, within a height of about 8 m.

The construction of the deck in 45 m long units on a two-week cycle was an extension of what Morgan-Vinci had successfully

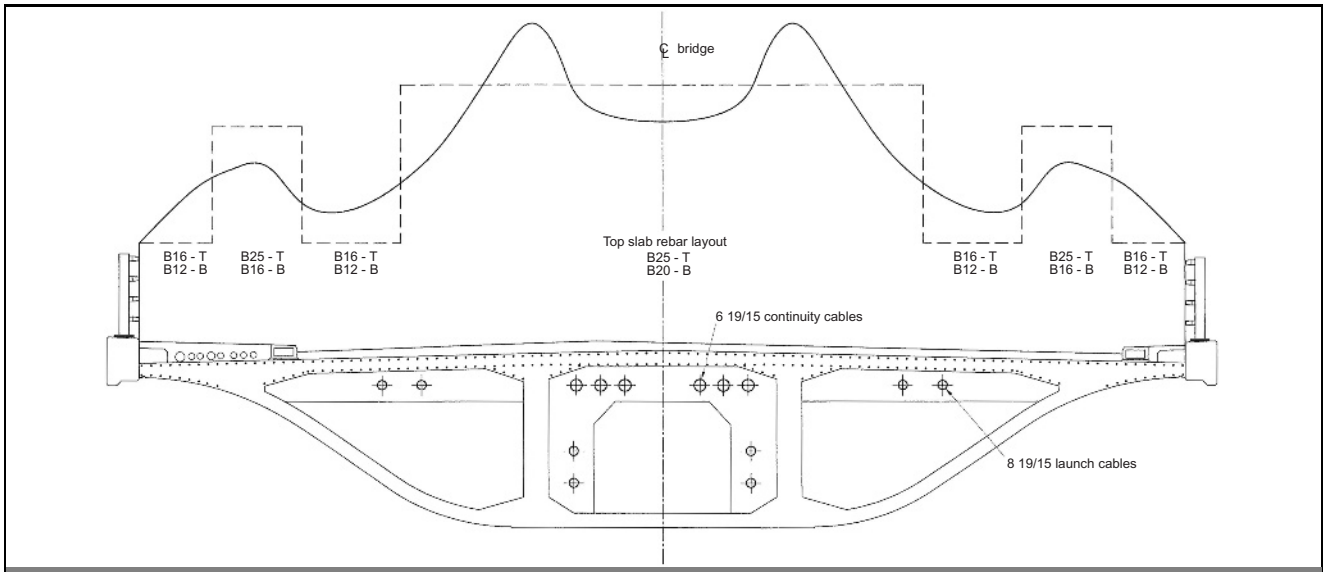

Figure 12. Shear lag and reinforcement distribution

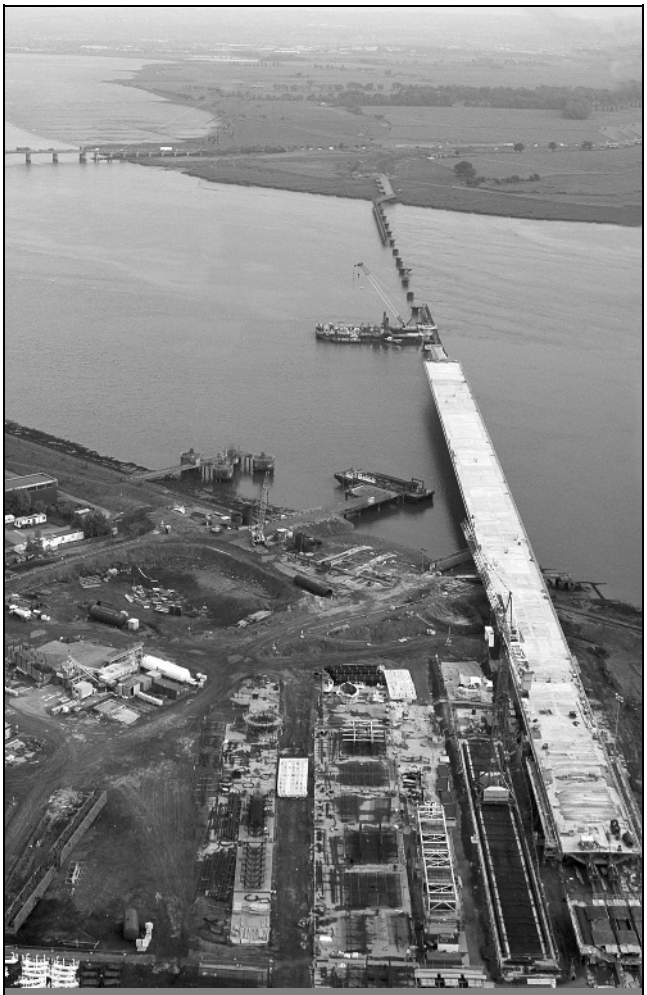

Figure 13. Casting area layout

achieved on the three launched bridges for the CTRL. The advantage of a two-week cycle was to take the launching activity away from the critical path. This was possible at the expense of a longer section of casting area and mould. The deck was cast in two sections with the webs and bottom slab as a first pour, and the top slab as a second pour. The span was cast in three lengths of 11 m. The 12 m long pier unit, which had a 3·5 m long diaphragm and all the prestressing anchorages, was cast as a single pour at the rear of the casting area. It was then launched into position midway through the cycle to allow it to be joined to the remainder of the span. This process not only took the complex pier unit off the critical path, but it also ensured that it was a week old by the time it was launched forward. The installation of the launching prestress was also moved forwards a span, to take it well away from the casting activity. The pier unit was thus at least a week old by the time the launching cables were stressed.

A purpose-built and hydraulic steel formwork system was procured for the deck. The soffit and outer side shutters slid up and down the casting length and were stripped sideways, whereas the internal shutters were lifted in from the sides (see Figure 17). The final pieces of shutter were then extracted through the box cells, and in particular had to be collapsible enough to be withdrawn through the pier unit diaphragm opening (see Figure 18). The base of each deck web was cast on a steel plate that slid over the steel plate cast on top of the skidding beams – this plate fell away as the deck moved away from the casting area. Over the remainder of the skidding beams, the deck was supported at approximately 15 m centres by temporary launching bearings.

The whole area was laid out in a factory format with dedicated sections for storage of materials, steel fixing jigs for the reinforcement cages, and shutter preparation. The area was served by a movable Potain MD 600 tower crane that could lift 25 t at 20 m radius – one of the biggest tower cranes in the UK (see Figure 19).

The considerable planning and efforts to optimise the layout of the casting area enabled the overall programme to be met within a safe and well-controlled factory environment.

5.3. Temporary works

A gangway was installed between all the pile casings, which gave safe and reliable access for men, materials, services and equipment to all the marine sections of the works. This gangway was also used to provide concrete access to the piles and piers, pumping concrete by up to 700 m out over the firth.

The launching nose was a 35 m long, twin-plate girder that was attached to the front of the deck with prestressing bars. It was used to control the moments in the concrete deck. Even with this nose, the moments in the first two spans were around 25% higher than in the typical spans (see Figure 20). As the critical case for the launching was not ultimate capacity, but crack control, it was decided not to increase the prestressing or the longitudinal reinforcement in this area, but to reduce the bar centres. The standard bars at 150 mm centres were therefore interlaced with T12 bars at 150 mm centres, to give a bar pitch of 75 mm. These small bars at close centres were successful in

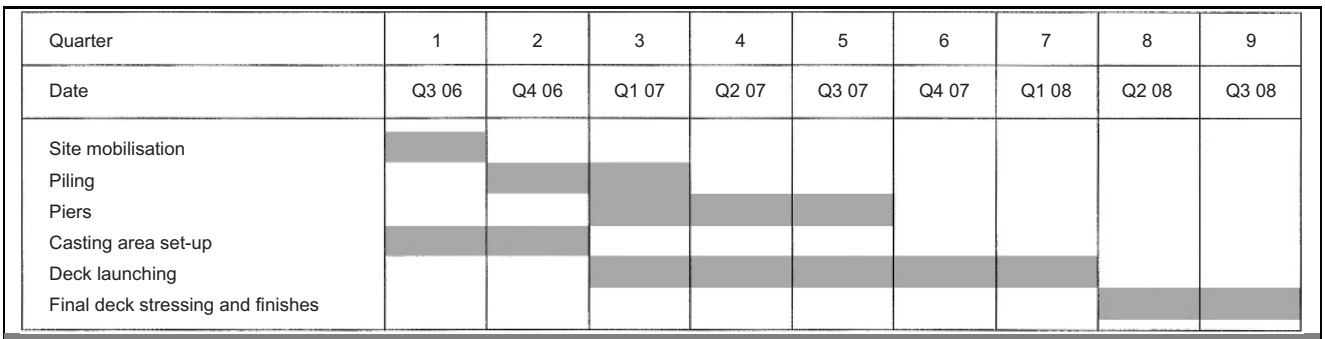

Figure 14. Overall programme

Figure 15. Casting area details

controlling all cracking in the area, without being a hindrance to concrete pouring.

The three longer spans were reduced during the launch such that no span was ever greater than 45 m. Three temporary piers were formed using the same 3 m diameter single piles as used for the permanent piers. The piles were drilled into the sandstones, and the casings filled with concrete and reinforcement up to river bed level. Above this level, the 35 mm thick steel casing became a temporary steel tubular pier. A concrete pier top was cast on a precast slab to make a composite connection to the steel casing. The pier top incorporated provision for all the launching bearings and jacks (see Figure 21). The temporary piers were demolished by lowering the concrete pier top down from prestressing bars in the deck on to river barges, and by hydro-demolition of the steel casing from within the casing, thus avoiding the need for any divers.

The two main pushing jacks each had a 600 t capacity, operating at a maximum pressure of 400 bar. They clamped with flat jacks on to the sides of the skidding beams and reacted against the bottom of each main web of the deck. Each stroke was 1·2 m and the 45 m launch took about 5 h (see Figure 22).

The lower halves of the launching bearings were cast on the pier tops and faced with a stainless steel sheet. During the launch, the 20 mm thick neoprene launching pads, faced with Teflon on their lower surface, were fed through between the deck soffit and the launching bearings. Jacks fore and aft of the launching bearings were positioned to be used in the event of any difficulties with the bearing. Control systems were in place at every pier during the launch to monitor the placing of the launching pads and the deflections of the pier tops.

A steel jetty was installed to serve piers 22 to 25 in the region of the mudflats on the south bank. The jetty was used for piling equipment to access the cofferdams in this area, within which traditional pilecaps and groups of four 1·5 m diameter bored piles were installed.

5.4. Piling

The construction of the single piles out in the Firth of Forth was a major challenge requiring the use of a sea-going, jack-up barge operated by Fugro Seacore. The 1450 t *Excalibur* was 60 m long and 30 m wide with eight jack-up legs. It carried a Demag PC 1200 crane with a lifting capacity of 180 t at a radius of 20 m, as well as their own reverse-circulation drilling tool (see

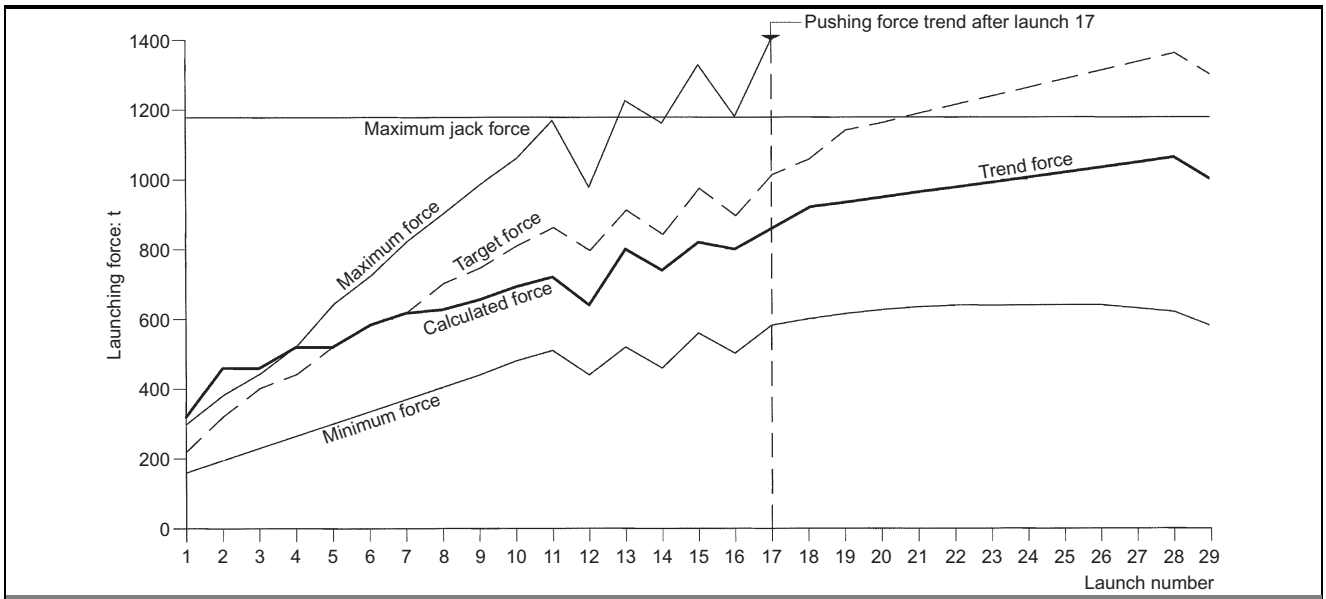

Figure 16. Typical jacking force assessment

Figure 17. Internal deck shutters

Figure 18. Internal diaphragm shutter

Figure 23). A significant problem for Fugro Seacore was transportation of the barge. This had to pass upstream of the existing Kincardine bridge; a structure which, although still a trunk road bridge, had ceased to operate as a swing bridge in 1988. It was not possible to undertake a one-off bridge opening to facilitate the barge movement, and therefore the team had to utilise the existing clearance of 9·2 m. The barge movement was only possible, with centimetres to spare, during a total of four spring low tides in April. Fortunately the movement was successful, as failure would have meant six months delay to the piling operation.

The 3 m to 3·85 m diameter pile casings were delivered by road with lengths between 21·5 m and 32 m, and a maximum weight

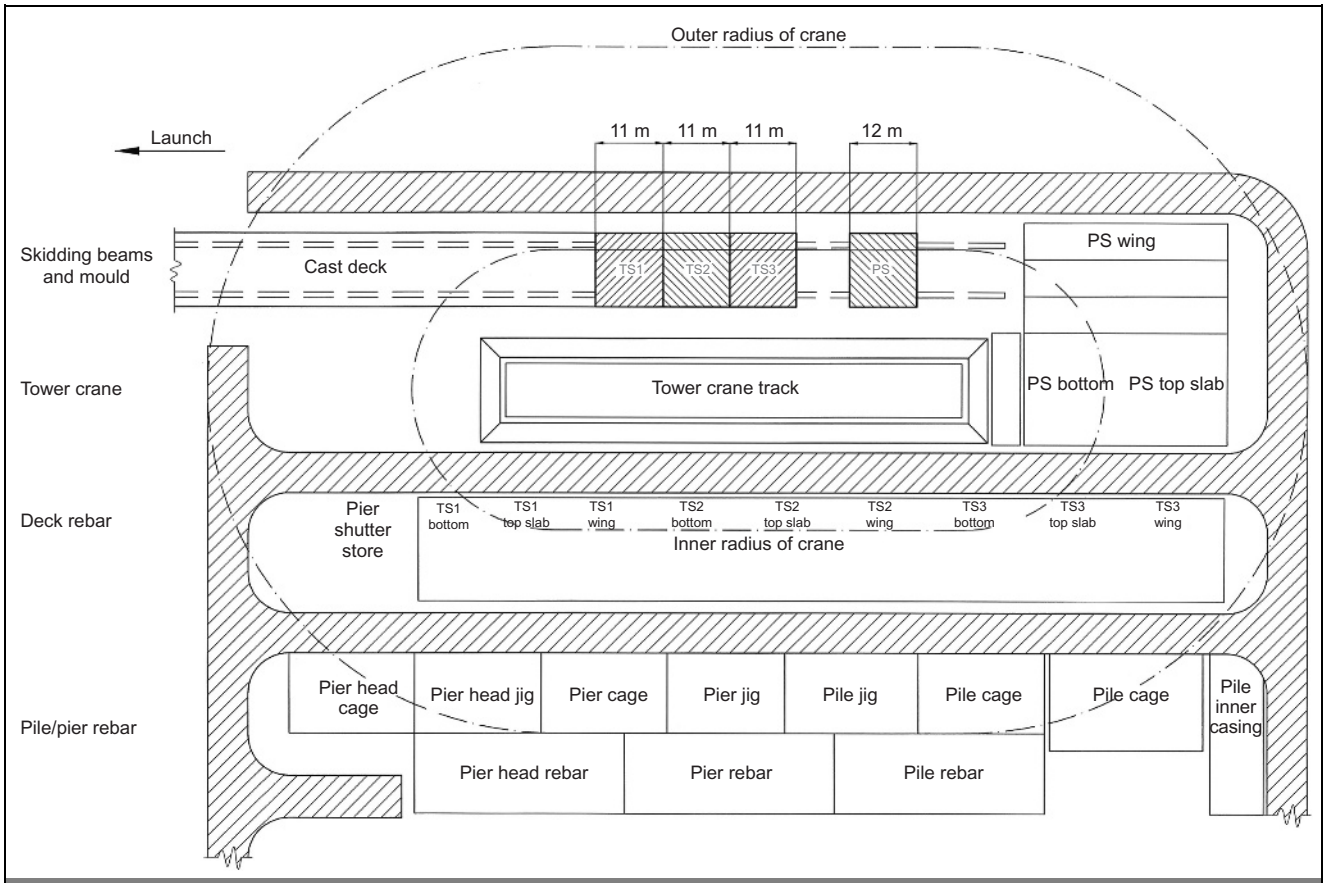

Figure 19. Casting area plan

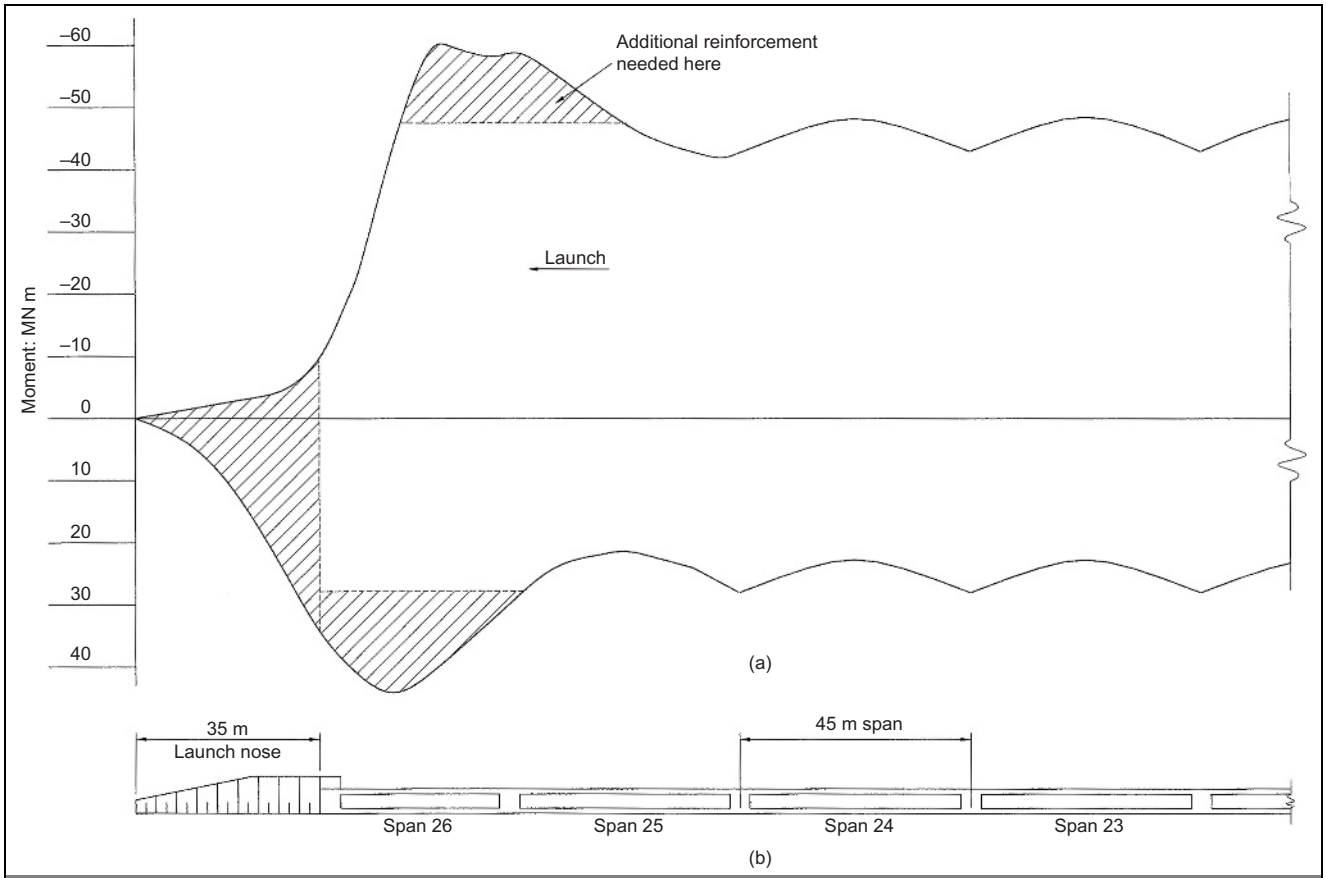

Figure 20. Moments due to launching effects: (a) launching moment envelope; (b) deck/nose elevation

Figure 21. Temporary pier details

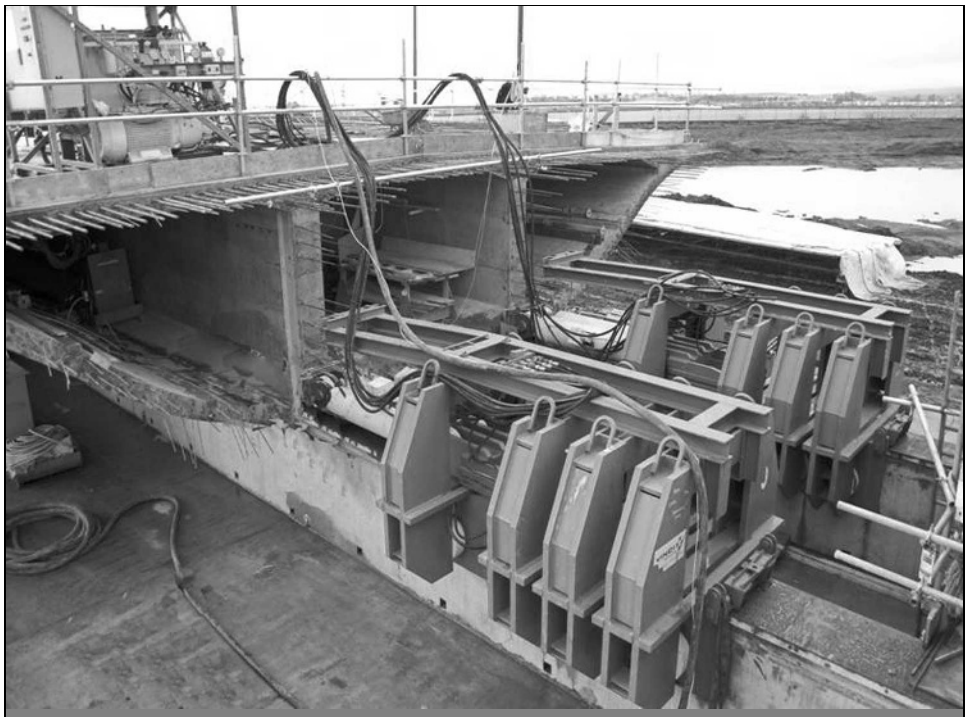

Figure 22. Launching jacks

Figure 23. *Excalibur* jack-up barge

of 80 t. They were taken out on the water using flotation bags. The jack-up barge was first located by global positioning system (GPS) systems and the casing was then held in the moonpool of the barge while the casing position was finally adjusted and checked by land surveying. The 25 mm to 35 mm thick casings were then vibrated down to between −17·5 m and −28 m, with a top level about 1 m above high water at +4 m. The pile was then drilled with reverse-circulation tools under water with the pile arisings being air-lifted to the surface, where they were dumped in an adjacent settlement barge (see Figure 24). The material was then later deposited from the barge at sea; at an approved site near Bo'ness.

The reinforcement cages were spliced into two pieces using up to 15 m long bars giving a maximum lift weight of around 60 t (see Figure 25). The pile concrete was a C32/40 mix with 515 kg of cementitious content, 70% slag replacement, a water–cement ratio of 0·39, water-reducing agents, retarders and pumping aids. The target flow was 650 mm, with 400 mm for up to 10 h. The actual 28 day strengths were in the range 65–80 N/mm^2. The largest single pours were around 350 m^3 for the piles at piers 12 to 15.

An inner casing was also used in the top 5 m of the pile. This 1·5 m diameter core was used to collect the spoiled concrete from the top of the pile pour, thus ensuring that the outer thickness of pile was made from sound concrete and eliminating the need to trim any of the pile within the casing, which was


Figure 24. Drilling of the piles

Figure 25. Lifting of the pile reinforcement cage

deemed to be time consuming and poor practice. The inner core was effectively filled by the overflowing pile concrete, with no detrimental effect on the pile integrity – the system worked very well at all locations (see Figure 26).

The cycle time to install the casing, drill the pile, install the reinforcement, install the new section of gangway and to then concrete the pile was between three and five days.

5.5. Piers

As described previously, the casings for the piles were used as cofferdams for the pier shaft construction, which had a significant safety and programming benefit. The typical 2·5 m pier shafts were built within the casing of the 3 m single pile and the four central 3 m piers were built within the larger 3·85 m pile casings (see Figure 27).

The steel pier shutter was specially designed to fit within the pile casing, which required great care as the notional perimeter gap was only 250 mm. This had to accommodate pile positional tolerances of +/−75 mm as well as normal construction tolerances. The key to the system was the use of a Christmas tree device that could unlock the shutter from the top only and that could be rotated within the pile casing to a position where the pier/pile gap was greatest (see Figure 28). The pier top shutter was also made in steel with small facetted panels that accommodated the double curvature of the shape (see Figure 29). The finish of the pier tops has been particularly

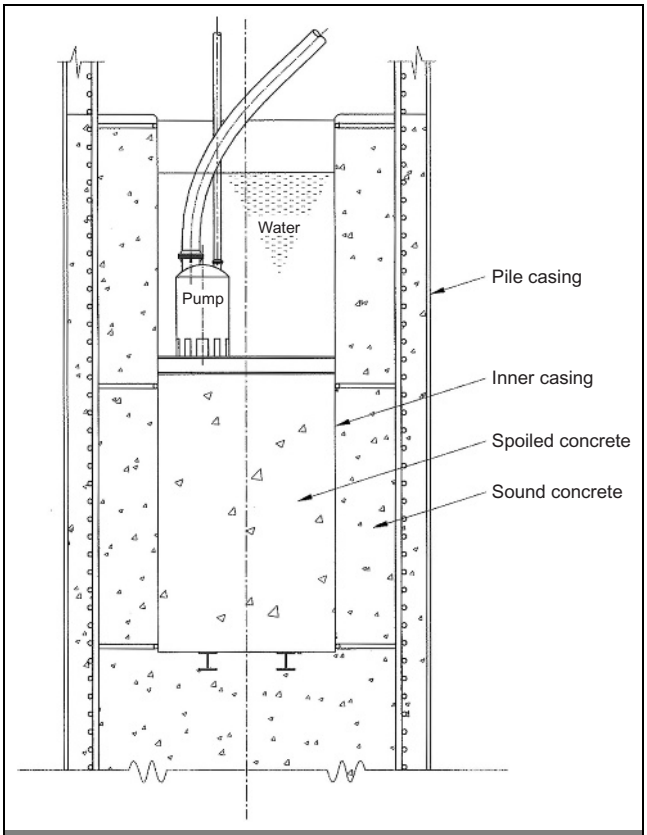

Figure 26. Inner casing to pile tops, section

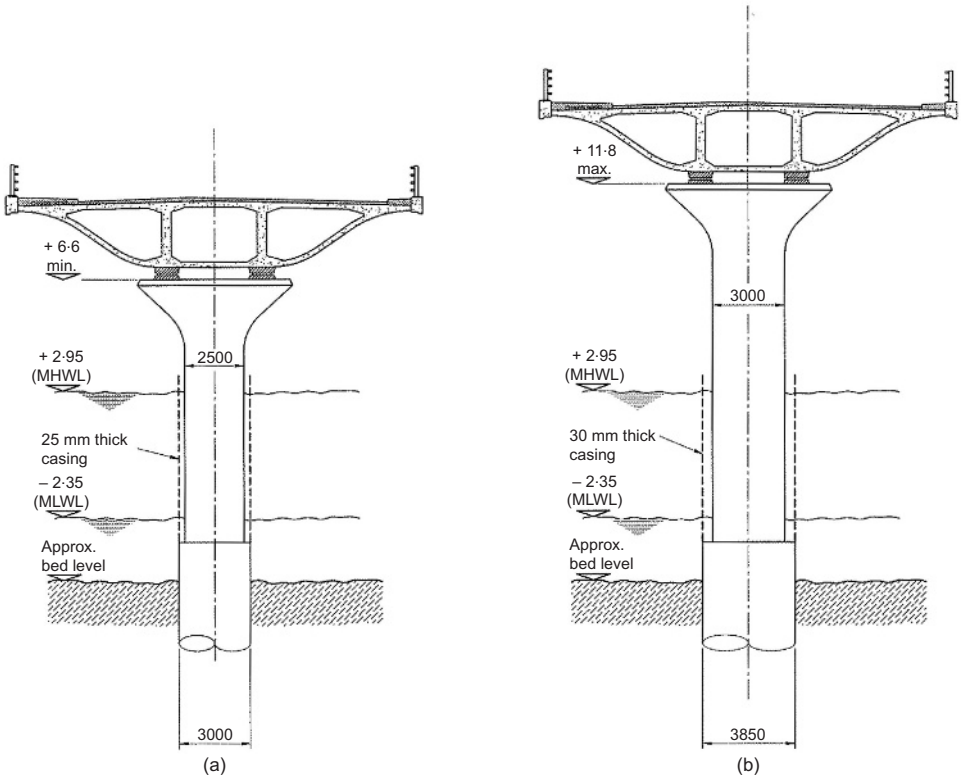

Figure 27. (a) Standard piers; (b) ship impact piers (dimensions in mm, elevations in m)

impressive and has justified the great care and time in developing that particular shape.

The main reinforcement cage for the pier top was simply orthogonal, with the main tie steel between the two bearing locations at right angles to the main pier shaft steel. The actual curved shape of the pier top was then formed with a layer of much lighter reinforcement, which had been pre-assembled in a

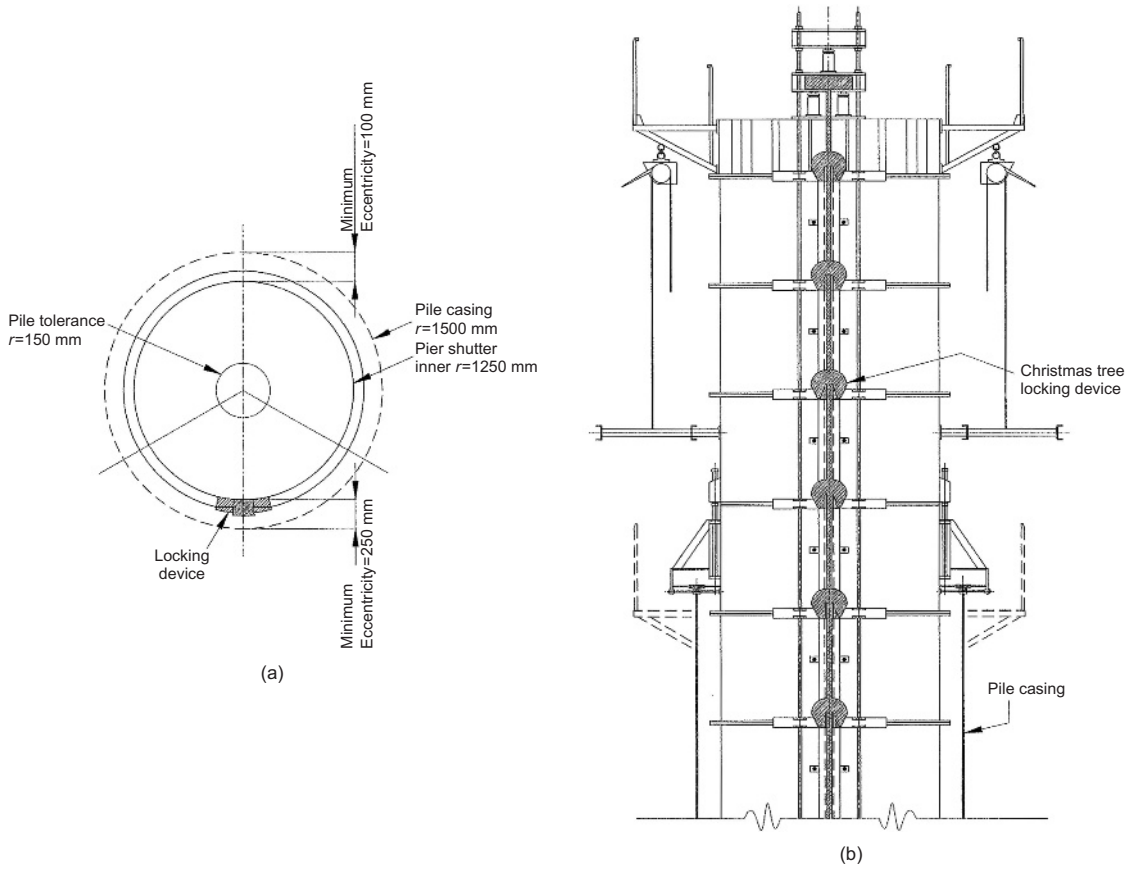

Figure 28. Pier shutter within pile casing: (a) plan; (b) section

Figure 29. Pier top details

jig. The pier concrete was a C40/50 mix with 510 kg of cementitious content, 50% slag replacement, a water–cement ratio of 0·38, water-reducing agents, retarders and pumping aids. The target flow was 650 mm, with 400 mm for up to 3 h. The actual 28 day strengths were in the range 65-80 N/mm^2. The cycle time to complete each pier was around one week.

The pier tops were carefully dimensioned to suit the provision of the launching bearings, launching jacks, lateral guides and the space needed for the permanent bearings, and jacks to replace them. The permanent bearings were rubber pots that were all installed on completion of the launch. Typical piers have one guided bearing and one that is free to slide in all directions. The four central piers all have fixed bearings, which hold the bridge longitudinally against differential friction forces and provide the restraint to the ship impact loads. Given the number of bearings in the whole bridge, the maximum differential friction was taken as 2·5%, which generated a load of about 5 MN. All bearings were held in place by friction alone, except at piers 12 and 15, where a positive connection was also provided for the ship impact load case.

5.6 Deck

Reinforcement cages were prepared in specially designed steel jigs alongside the casting area, with the whole bottom mat and web steel lifted as one piece in approximately 12 m lengths (see Figure 30). The top mats were also lifted as one piece in similar lengths. The deck concrete was a C50/60 mix with 480 kg of cementitious content, 36% slag replacement, a water–cement ratio of 0·37, water-reducing agents and accelerators. The target flow was 550 mm, with 500 mm for up to 50 min. The actual 28 day strengths were in the range 80–95 N/mm^2. The concrete top surface was initially mist cured to prevent drying out, followed by the application of wet hessian for a further week.

The concrete was always batched using heated water to aid the early strength gain, and the whole casting area was maintained at around 25–30°C. The concrete was then monitored using a temperature-matched curing system and maturity meters, backed up with a second system of temperature data loggers. Cubes were heated to follow the same heat gain profile as the deck and were then tested at regular intervals to prove the actual strengths required for mould stripping, launching and stressing. Typical early age strengths were 10–12 N/mm^2 for striking of the formwork (achieved at about 12 h), 20 N/mm^2 for launching (at about 18 h), 25 N/mm^2 in the span for stressing of the launch cables (at about 2 days) and 35 N/mm^2 in the pier unit for stressing the same cables (achieved at about 3 days, although the cables were not stressed until the unit was well over a week old).

The typical two-week cycle for each 45 m span was actually achieved after about five spans and was then refined to achieve an optimum cycle of eight working days, although nine working days was the norm (see Figure 31).

The fixing of the launching prestress ducts, the threading of the cables, and their stressing and grouting followed in the sections of deck away from the casting area, but still while over the skidding beams. The launching cables were staggered over two spans such that 50% of the cables were stressed in any cycle, giving a gradual build-up of the axial force from zero to 50% to 100%, before the deck left the casting area. The continuity cables were then installed on completion of the launch in a

Figure 30. Lifting of the deck reinforcement cage

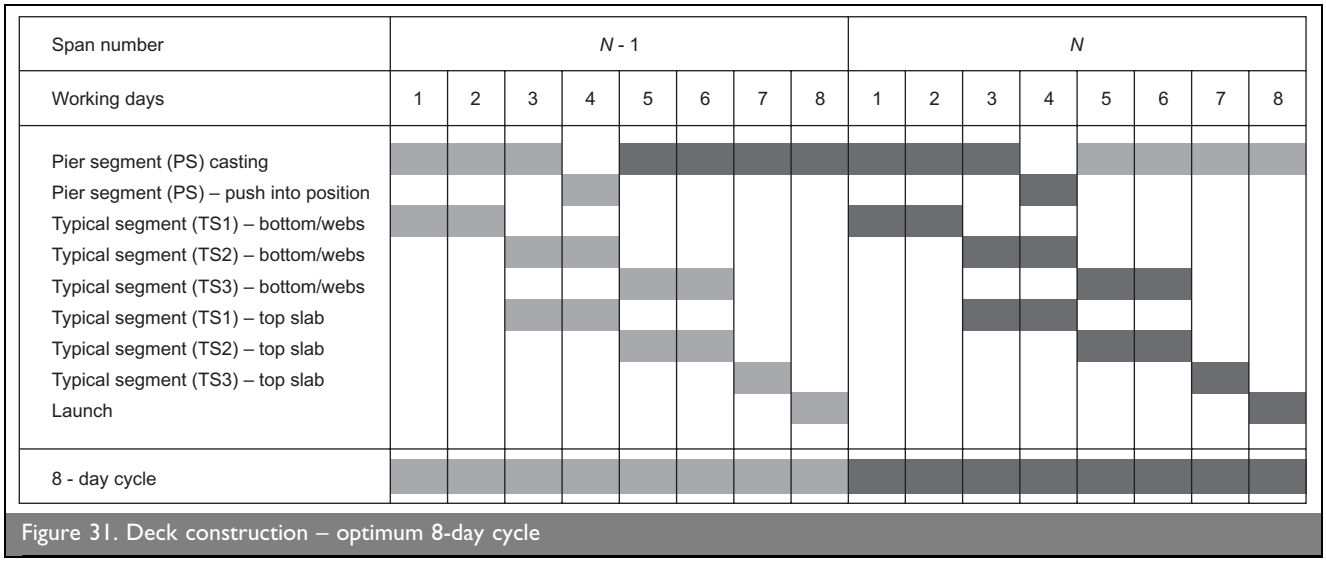

Figure 31. Deck construction – optimum 8-day cycle

sequence to suit the removal of the temporary piers and the installation of the permanent bearings (see Figure 32).

5.7. Safety

The methods of design and construction, with factory-like conditions in the casting area and the piers built within the pile casings with no marine cofferdams, ensured that the safety of the workforce was always highly controlled. Nearly 1–4 million hours were completed with no major accidents, which is a tribute to the chosen construction processes, the care taken in their preparation and the vigilance of the site team.

5.8. Finishes

All exposed concrete surfaces were impregnated with a water-based water repellent. The parapet edge beams were precast in 3 m lengths and attached to the deck with an in situ stitch. They remain jointed throughout and provide no structural capacity. The option to include them within the launch was considered but the additional weight of nearly 2500 t and the need to generate a final line and level that was satisfactory, determined that all the edge beams were better installed after the launch was completed.

Access to the deck was provided at the abutments and by

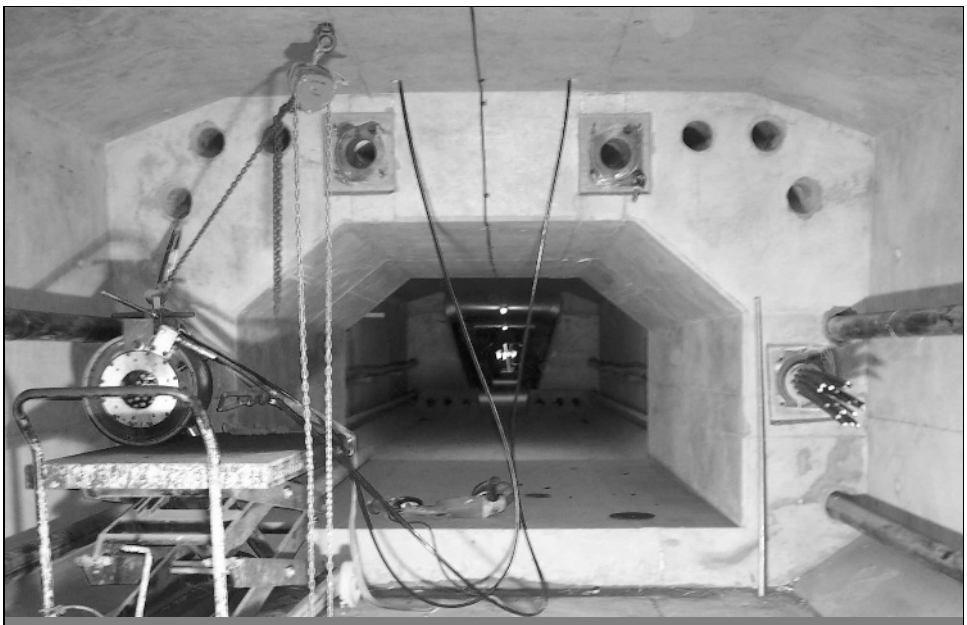

Figure 32. Prestressing details

manholes in the main webs, which then linked the three cells. The inside of the box was ventilated by 150 mm diameter holes in the webs. The carriageway has no highway lighting but the inside of the deck was provided with access lighting. Service ducts were also provided inside the box.

The parapets were 1.5 m high, performance class N2/B/W3 to BS EN 1317.[19] They had aluminium posts at 3 m centres with a four-rail system to suit the footpath and the cycleway.

The bridge was continuous over its whole length with expansion joints only at its ends. These joints were provided within the depth of the carriageway surfacing and accommodated movements up to 800 mm.

The whole bridge deck was drained with recycled plastic kerb drains that carried all the water to the four corners of the bridge. The water was then carried through stainless steel pipes into the highway drainage system behind the abutments.

Figure 33. Completed bridge

6. CONCLUSIONS

This major crossing of the Firth of Forth has been completed on time by a design and construction team that has worked closely together, as well as with the client and his team. The 1188 m long bridge was incrementally launched from one end to become the second longest launched bridge in the world. This was achieved safely within a difficult marine environment by a series of innovations that reduced materials, time and risk. The use of single piles, of up to 3.85 m diameter into strong sandstones, to support a wide bridge deck was a great success, as was the use of partial prestressing to cut the prestress tonnage by over half. The team worked to develop ideas from previous projects and to create new ideas to ease the construction tasks. The project was completed with minimal environmental impact through a design and construction process that had addressed these issues at every turn. The bridge was opened by the first minister of Scotland, Alex Salmond, on 19 November 2008, and was named as the Clackmannanshire bridge (see Figure 33).

ACKNOWLEDGEMENTS

Many people have contributed to this paper. Unfortunately, limits on space allow only a small number to be noted here, but the authors' thanks go to everyone involved. In particular, the authors must thank:

David Mustard, Graham Porteous and Steven Brown of Transport Scotland
Iain Murray and Helen Bradley of Jacobs
John Osborne of Morgan Est
Pierre Villard, Francis Guinchard and William Paschetta of Vinci Construction Grands Projets
Paul Mullins and Richard Davies of Benaim
Ken McDowall and Bob Barclay of Fairhurst
Matthew Lawman of Alexander Associates
Ron Yee of Yee Associates
Peter Clutterbuck of Fugro Seacore
Paul Bottomley of Freyssinet

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PRINCIPAL BRIDGE QUANTITIES

Deck concrete 13,000 m^3
Deck reinforcement 3,200 t
Deck prestress 440 t
Substructure concrete 8,500 m^3
Substructure reinforcement 1,000 t

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