Journal of Coastal Research	9	3	766–784	Fort Lauderdale, Florida	Summer 1993

Denmark's Great Belt Link

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ABSTRACT

LARSEN, O.D., 1993. Denmark's Great Belt link. *Journal of Coastal Research*, 9(3), 766–784. Fort Lauderdale (Florida), ISSN 0749-0208.

The 3.5 billion US dollar Great Belt fixed link will join the eastern and western parts of Denmark presently connected by ferry services. The link is the largest per capita investment in a civil engineering project in any country today. It consists of 8 km long twin bored rail tunnels, a 6.6 km long combined rail and road bridge, and a 6.8 km long high level road bridge with a world record suspension span.

The fixed link is going to cross the main channel for exchange of waters between the Baltic Sea and the North Sea and therefore the Public Works Act requires compensation measures ensuring a zero environmental influence on the Baltic waters.

The requirements to safe passage of the important international navigation between the Baltic Sea and the North Sea gave decisive input to the design of the crossing structures.

ADDITIONAL INDEX WORDS: *International navigation, rail tunnel, suspension bridge, design criteria, causeway, embankments.*

INTRODUCTION

An important step to improve the northern European transportation network is the present construction of the 3.5 billion US dollar Great Belt fixed link in Denmark. It will later be completed with fixed links to Sweden and Germany for full interconnection of Scandinavia with Central Europe, and thus stimulate economical growth (Figure 1).

The 18 km wide Great Belt is divided into two main channels, east and west, by the tiny island of Sprogø. Therefore, it was obvious to take advantage of this island as an integrated part of a fixed link project, a "stepping stone" between east and west across the water.

The link consists of three major structures: a bored railway tunnel (HAWLEY and BENNICK, 1992) and a high level motorway bridge (OSTENFELD and JACOBSEN, 1992) across the eastern channel, and a low level dual mode bridge for railway and motorway (HOMMEL and FRIES, 1992) across the western channel (Figure 2).

A shareholding company, Great Belt A.S., was founded by the Danish state as a Client organization with the objective to build and operate the link. The company will be responsible for design and construction and later for the operation of the motorway part of the link, while the Danish State Railways will operate the rail link. The railway connection is due to open for traffic in 1995 and the motorway part late in 1997.

The project is financed by government guaranteed commercial loans to be paid back via user tolls. The payback period is estimated to be 15 years for the road connection, whereas the rail crossing is to be paid by the Danish State Railways over 30 years in fixed installments per year.

In terms of size and complexity, the link is an international construction project. However, the design is primarily Danish with COWIconsult as the lead partner in the three major consultant joint ventures which also comprise Mott MacDonald in the tunnel project; Carl Bro Group and Leonhardt, Andrä und Partner in the West Bridge project; and Rambøll, Hannemann & Højlund in the East Bridge project.

ENVIRONMENTAL CONSIDERATIONS

A design criterion laid down in the Public Works Act includes a so-called "zero environmental influence" of the marine environment in the Baltic Sea by an obligation to ensure the complex water flow exchange with different salinities through the Great Belt in spite of the blocking effect of embankments and bridge piers.

This aim is achieved through dredging and by a design that favours short ramps, long spans, hydraulically shaped piers, and streamlining artificial islands.

92074 received and accepted 5 October 1992.

Earthworks and causeways were put out for tender under the constraint that they should be built of the dredged materials. Part of the soils to be dredged were excellent for this purpose, whereas soft organic materials unsuitable for construction works were deposited in sedimentation basins placed within the confines of the Sprogø island (HANSEN and LARSEN, 1991).

The extent of compensation dredging was determined by an advanced two-layer mathematical model (MØLLER and NIELSEN, 1991). A field measurement programme, comprising 14 stations, will be carried out to calibrate and verify the mathematical model in two phases:

(1) Approximately halfway through construction, dredging measurements will be taken to adjust the dredging quantities and depths.
(2) When the dredging is completed, a second set of measurements will be taken to confirm that the compensation is adequate.

To determine local effects, a Biological Monitoring Program has been undertaken (RANDLØV and JENSEN, 1991). The program comprises field studies of herring spawning, the growth rate for mussels, and monitoring the plumes and sedimentation from dredging.

The Client has employed an international board of experts to advise on these important environmental issues.

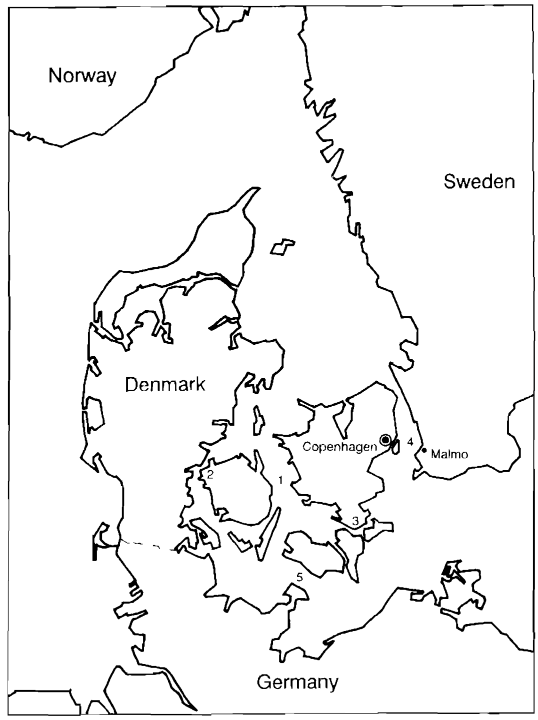

Figure 1. (1) Great Belt link. (2) Little Belt Bridge. (3) Farø Bridges. (4) Øresund Crossing. (5) Fehmern Belt Crossing.

RISK ANALYSES

A series of "overall" risk analyses has been undertaken to establish risk acceptance criteria and assure adequate and consistent safety level for the entire link.

The safety aspects considered were service disruption and risk of fatalities. The acceptance criteria required that the probability of disruption of a duration of more than one month should not exceed a specified level, and the risk level for a fatality for crossing the fixed link shall be comparable to the risk of crossing by ferry.

The "overall" risk was assured systematically by analyzing individual risk elements such as train accidents, fire and explosion, ice loads, ship collision, and many other factors.

The analyses are followed by risk management throughout the design and construction period to ensure that the objectives are met. A risk element of particular importance is the extensive navigation in the Great Belt.

Approximately 20,000 ships per year traffic the international eastern channel between the Baltic Sea and the North Sea. In spite of a separation arrangement in north and south oriented channels, 61 ship accidents have occurred in the area since 1974, such as groundings, ship-ship collisions, and ship-lighthouse collisions.

A comprehensive investigation programme of the future interaction between the ship traffic and the East Bridge structures has thus been carried out. The objectives were to maintain the navigation conditions at the same level after the bridge is built, and to provide a probabilistic basis for bridge design against ship collision based on an accepted maximum risk of bridge disruption.

Manoeuvring simulations were performed using computer based simulators, and included two ships in loaded and ballasted conditions, a 100 m long 40,000 DWT car carrier, and a 300 m long 150,000 DWT tanker. The modelling of the manoeuvring behaviour included propulsion forces, rudder forces, effects from wind, currents, and water depths between keel and sea bottom.

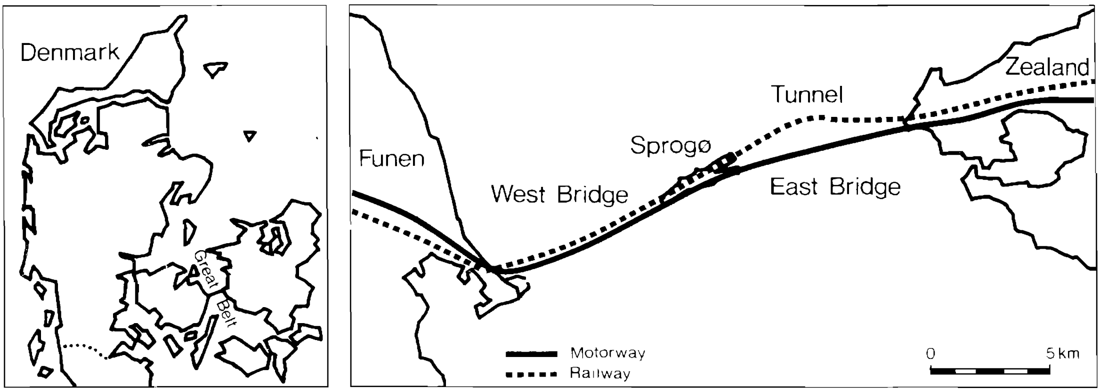

Figure 2. The Great Belt fixed link.

Bridge spans from 800 to 1,800 m were investigated in combination with navigation route alternatives ranging from the existing one with an angle of 68° with the bridge line, to a route perpendicular to the bridge line. The ships were navigated through the different bridge spans by pilots with up-to-date piloting expertise in the Great Belt.

The pilots concluded that:

(1) A bridge span of 1,600–1,800 m in the existing navigation route would not change the present conditions.
(2) A bridge span of 1,400 m would maintain the present conditions provided the crossing angle be altered to 76°.
(3) A bridge span of 1,200 m or less would reduce the navigation conditions considerably.

The pilot's evaluations were of significant importance in the clarification and verification of the overall design requirements to the bridge arrangement. Experience gathered from bridges worldwide have confirmed these design requirements.

Observations of the distances which ships tend to keep between each other under different navigational conditions have been analyzed mainly by Japanese researchers and generalized in terms of ship domain. These observations indicate that aversion manoeuvres normally are the result of ship encounters with overlapping domains.

An analysis of recorded ship collisions with 26 North American bridges showed a marked reduction of collision frequency for bridges with spans fulfilling the domain theory (FRANDSEN *et al.*, 1991).

The required length of the navigation span was also studied based on the ship domain theory and the objective to avoid encounters with overlapping ship domains. The study concluded that a span length of about 1,500 m would be needed in agreement with the result of the manoeuvring simulations.

An improved ship-bridge collision simulation model has been established. Based on ship traffic, bridge arrangement and bridge impact capacity, the model can estimate frequencies of ship-bridge collisions and bridge collapse due to ship collisions.

Requirements for bridge impact capacities have been established on a probabilistic basis using the above mentioned ship collision model. A more refined formulation of the design loads to be applied to the bridge structures in various collision situations has been facilitated by detailed Finite Element Method calculations.

Investigations of methods to reduce the risk of ship collision especially to the West Bridge have resulted in a decision to implement a Vessel Traffic Service system (OLSEN *et al.*, 1992) for the entire Great Belt area. Intended for the operational period of the bridges, the system will be able to give guidance to the ship traffic and detect whether aberrant vessels leaving the main route could endanger the bridges.

Altogether the investigations have provided basis for the following decisions:

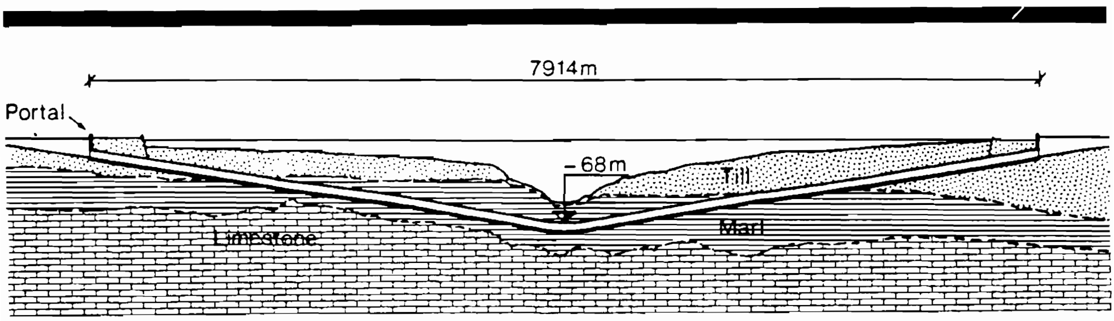

Figure 3. Geological cross section in tunnel alignment.

(1) Length of the East Bridge main span to be 1,624 m.
(2) Navigation route crossing the East Bridge will be straightened from a crossing angle of 68° to 78°, the nearest bends will be located 2,300 m from the bridge axis, and dredgings to a depth of 19 m for south going ships and 17 m for north going ships will be performed.
(3) Types and positions of navigation marks at the East Bridge.
(4) Impact strength requirements to the East Bridge structure.
(5) Implementation of a Vessel Traffic Service system for the Great Belt.

THE EAST TUNNEL

The east tunnel between Zealand and Sprogø was tendered in 1988. Immersed tunnel solutions as well as bored tunnels were considered. Given the difficult conditions at the site, crossing the deep 9 km wide channel would be a challenging task for either tunnelling methods. Bored tunnels were finally selected for economical and environmental reasons, and the construction contract was awarded to MT Group.

The 8 km long twin tunnels carry the rails 75 m below sea level at the deepest point (Figure 3). The tunnels are bored except for cut-and-cover sections at each end totalling 0.5 km. The cut-and-cover sections are terminated by portal buildings in open ramps 15 m below sea level.

Alignment and Geology

Water depths along the tunnel alignment increase gradually from each side to about 20 m, and then in the central area more steeply to a maximum of about 55 m. The alignment is dictated by the location of Halsskov point and reef on the eastern side, and the need to pass north of the existing island of Sprogø, where the transition from tunnel to bridge is sited on reclaimed land.

To take advantage of a saddle in the channel, the alignment is curved to the north (Figure 2), which also provides sufficient length to accommodate reasonable gradients. The curvatures and gradients have to be appropriate for a railway design speed of 160 km/h.

The channel has been formed in layered strata of tills underlain by marl. The tunnel passes through the tills and the marl, but does not enter the underlying limestone.

The tills are essentially very stiff to hard clays with meltwater deposits and boulders. The underlying marl varies from weak marl stone to a moderate weak marl stone. It is expected to be fissured and jointed.

Tests on the ground water indicate a potentially aggressive environment with sulphate and chloride concentrations similar to those found in seawater.

Another critical consideration is the minimum ground cover to the tunnel in the deepest part of the channel. At this point the marl is not overlain by the till, and vertical fissuring may be present. After some further detailed site investigations, a cover of approximately 15 m has been adopted.

The alignment and geotechnical constraints result in a maximum gradient of 1.56% with vertical curve radii of 12.5 km and minimum horizontal curves of 4 km. The two tunnels are 25 m apart, with an internal diameter of 7.7 m. Every 250 m, the tubes are connected by cross passages.

A tunnel boring machine study indicated that

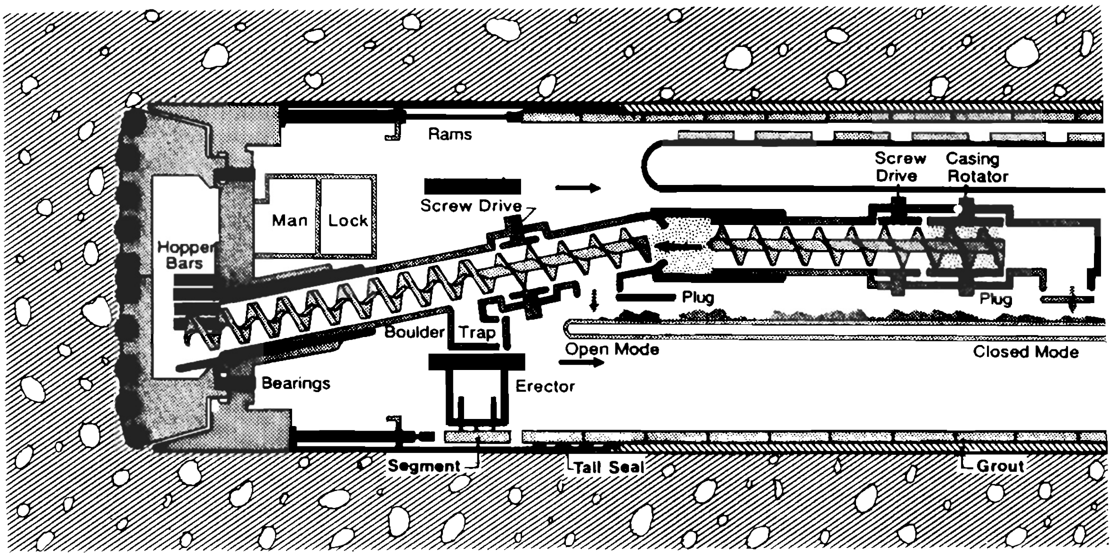

Figure 4. Earth pressure balance Tunnel Boring Machine.

the main tunnels could be excavated using full-face tunnel boring machines (TBM's) (Figure 4).

Main Tunnel Lining

The main tunnel lining needs to be designed as thin as possible to minimize the overall cost of construction, both in terms of weight and cost of the lining and to minimize the excavated diameter. The tunnel has been designed to carry the full hydrostatic head of up to 80 m, and the lining to take the reaction of about 9,000 tonnes from the tunnelling machine.

The precast concrete lining consists of 1.65 m wide rings with 6 segments plus key (Figure 5). The lining is 400 mm thick. Each ring has an overall taper of 20 mm for correcting plane and for negotiating curves, with straights being built by alternating left-handed and right-handed rings.

The standard segment is a solid unit to take the machine trust, with minimally sized bolt pockets for curved bolts. The radial joints have slightly convex surfaces, with provision for a gasket towards the outer face, and for internal caulking. Synthetic rubber gaskets have been specified, and are required to withstand a working pressure of minimum 0.8 MPa. The circumferential joint is flat, with the same gasket and caulking provisions, and a thin packer to help distribute localised stresses.

Two-dimensional finite element analyses modeled the ground/lining interaction in the tills, in the marl and in the till/marl interface. Loads due to yield of the ground were found to be small, but an overriding minimum ground load of two tunnel diameters was adopted. Distortions causing bending moments in the ring were generally found to be governed by construction tolerances rather than by ground loads.

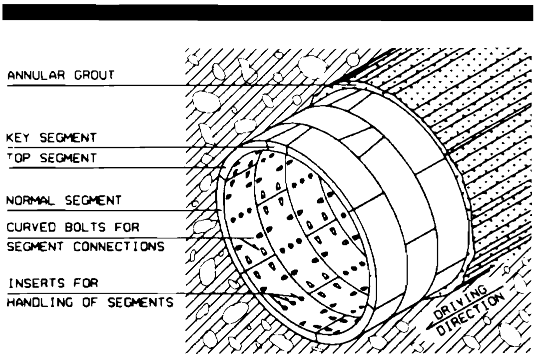

Figure 5. Bored tunnel lining.

Every 250 m, a set of 6 special opening rings is installed in each of the bored main tunnels—in all 29 opening sets in each tunnel—so that 2.0 m wide and 3.35 m high openings can be formed later for the cross passages.

Approximately 60,000 concrete segments are needed for the project, and a factory was built in Halsskov on Zealand. The plant is able to produce up to 200 segments a day. The segments are oven cured for a minimum of 8 hours at a maximum temperature of 55° C. The segments are then stored under cover for 28 days, after which gaskets are fitted followed by a minimum of further 28 days of storage before installation.

The segments are reinforced to avoid excess cracking during handling due to stress concentrations at joints and at bolt pockets. A fully welded reinforcement cage is employed, which gives a very modest steel proportion of 80 kg/m^3 of concrete. A high degree of automation is being used in the factory. The reinforcement is blast-cleaned, cut, bent and welded, with all of these operations either automatically or semi-automatically controlled.

The segments have to be manufactured to very high tolerances to avoid excessive stress concentrations with the very high loadings involved. Segments are checked for dimensional accuracy at 130 points using automatic laser devices, and information is analyzed by computer. One in every 20 segments is checked in this way.

Durability of Lining

The required design life of the tunnel is 100 years, and the durability of the lining in the submarine environment is an important consideration. The external environmental conditions along the tunnel with fairly permeable ground, high water pressures and a potentially aggressive environment as well as the internal environmental conditions with warm, humid, polluted air of varying pressure and velocity and possible spills, necessitated special measures against deterioration of the lining.

A multi-stage protection strategy was applied for the precast concrete lining segments, including the following elements:

(1) An annular grout with high binding capacity for chlorides and sulphates;
(2) Segments of very dense, high strength concrete with fully gasketted joint seals, ensuring a water tight lining;
(3) Epoxy coating of welded reinforcement cages;
(4) Possibility of future cathodic protection of welded reinforcement cages.

A number of protective measures for the reinforcement were considered. The concrete cover for the steel is a compromise between structural requirements and durability objectives, and 35 mm was eventually adopted.

The bid documents provided for either external coatings to the segments or the use of fusion-bonded epoxy coating to the reinforcement. In this case, the reinforcement is being coated with epoxy, which requires the welded cages to be blast cleaned, heated to 260° C, dipped in the fluidized bed for 4 seconds, and then cured and stored. The cages emerge from the fluidized bed with a very even coating.

So far as we know, the fluidized bed technique for epoxy coating has not been used before for three dimensional reinforcement cages. Therefore, during 1987–1988 COWIconsult and Mott McDonald initiated a series of trials. The first involved dipping model cages. Pull-out tests were carried out to check the effect of the bond in concrete, followed by structural testing of quarter-size segments. These tests showed that the epoxy coating did not have any significant effect on the structural performance of the joint and the lining.

Cross Passages

The 4.5 m diameter cross passages (Figure 6), were designed to be lined with bolted cast iron liner plates, but shotcrete primary support followed by cast-in-place concrete permanent lining was included as an alternative in the bid documents. Use of the cast iron lining was decided at contract award, but the contractor later decided to use the alternative method. Detailed construction methods for the cross passages will be decided on a case-by-case basis after probing through the holes provided in the linings for that purpose and for subsequent ground treatment.

The current proposal for construction of cross passages, which is not yet started, considers construction of the first two cross passages under the protection of a clay blanket on the Halsskov side with shotcrete primary support and cast-in-place unreinforced concrete permanent lining. Thereafter, cross passages in the lower till are proposed using cast iron liners, with a pilot near the crown and enlarging after further ground treatment. In

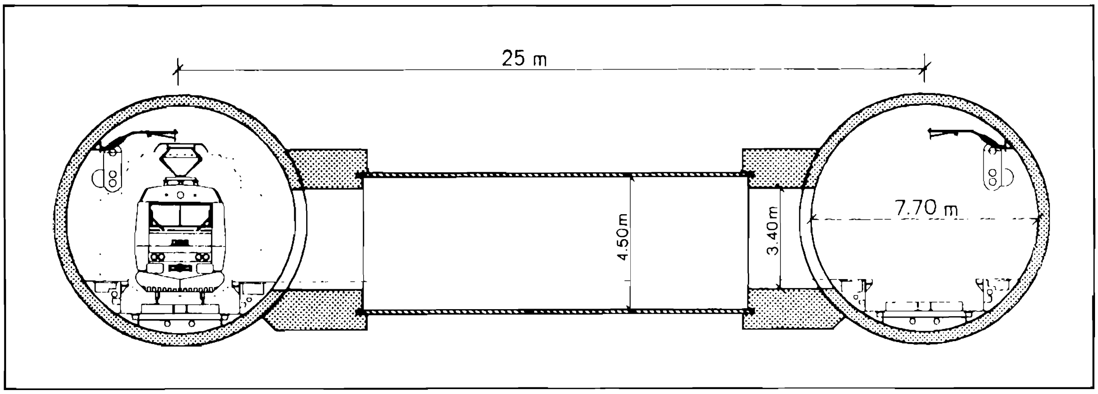

Figure 6. Main tunnels with cross passage.

the marl, the shotcrete and cast-in-place concrete method is likely to be chosen.

Tunnel Construction Progress

Tunnelling was halted in May 1991 for extensive repairs to all the TBMs' front screw conveyor bearings. Shortly after restart of the Sprogø machines in September, 1991, the sea inundated the machines.

Tunnelling from Halsskov restarted in January, 1992 after the added protection of a clay blanket; by April, both of the Halsskov machines had driven about 550 m and were each progressing at a maximum rate of 50 m per week. After dewatering and machine repair, the Sprogø TBM's restarted in July, 1992.

THE WEST BRIDGE

The 6.6 km long West Bridge will carry two railway tracks and two three-lane carriageways across the western channel of the Great Belt, from the tiny island of Sprogø to the shore near the existing ferry harbour on the island of Funen (Figure 7).

The tender designs had three alternative types of superstructures: a double deck composite girder, triple independent concrete girders side by side, and a single steel box girder. All three bridge solutions shared a common gravity founded, sand filled caisson substructure topped by pier shafts of varying layout.

The overall concept for the tree solutions was based on full span prefabrication of girders and prefabrication of complete caissons in a reclaimed harbour area and large capacity marine equipment for the transportation to the site and the final installation.

Tender evaluation resulted in selection of an alternative design with two haunched concrete box girders with a typical span of 110.4 m, reduced to 81.75 m at abutments and expansion joints. The total length was subdivided into six continuous girders of 1,047 m and 1,157 m, respectively, requiring seven expansion joints with movements up to 1,400 mm.

In June 1989, the Client Great Belt A.S. awarded a lump-sum contract to European Storebælt Group, ESG, for the design and construction.

CCL joint venture have been entrusted to carry out an independent design check, and technical services during construction.

Soil Conditions

Danien limestone constitutes the base to any depth important for the foundation. It is partly topped by marl and calcarenite from the Selandien strata, followed by glacial and post glacial deposits. The glacial layers are upper till, lower till, and the so-called Knudshoved till. Whereas the latter deposit is an extremely hard clay and sandy till, the lower till consists of alternating layers of clay till, sandy till, and melt water sand. The upper till is predominantly clay till where the undrained shear strength has a tendency to decrease with depth.

The ground conditions facilitate direct foundation, and generally the substructures are designed for foundation on stone bed layers at or

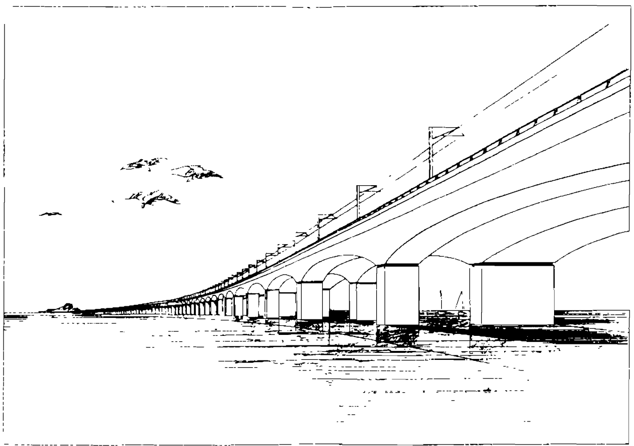

Figure 7. The West Bridge.

slightly below the seabed level. However, for some piers, it was necessary to excavate material of the weak upper till down to 15 m below seabed and place the foundations on competent lower till.

Detailed Design

In the design, dead loads; train, traffic and other loads; wind loads; wave and current loads; impact loads from a 2,000 DWT ship; and ice loads were considered in the various limit states.

The design requirements are based on a partial safety factor concept as defined in Danish Standards and are asking for limitation of stresses and crack width under serviceability limit state load combinations and adequate load carrying capacity in ultimate and accidental limit states.

The total bridge structure consists of a total of 324 prefabricated units, comprising 62 caissons, 124 pier shafts, and 114 standard and 24 special bridge girders.

The caissons are slipformed. From a base plate, they are built up in seven transverse and four longitudinal walls. The outer cells are topped with cover slabs, and a massive plinth caps the inner cells (Figure 8). Dimensions 34 m × 22.5 m maximum and 29.4 m × 17.5 m minimum at the base, and 31.1 m × 6.4 m at the top. The base plate is 1.2 m or 1.15 m thick and prestressed in the transverse direction. The walls are 350 mm to 400 mm thick, and the plinth is 2 m deep.

The pier shaft for the roadway is 12.55 m long and 5 m wide and consists of two different sections and a head. The lower section, which is more robustly designed because of the risk of local ship impacts and ice pressure, has 850 mm thick external walls and two stiffening walls. The upper part is a rounded-off rectangular section with 500 mm thick walls only.

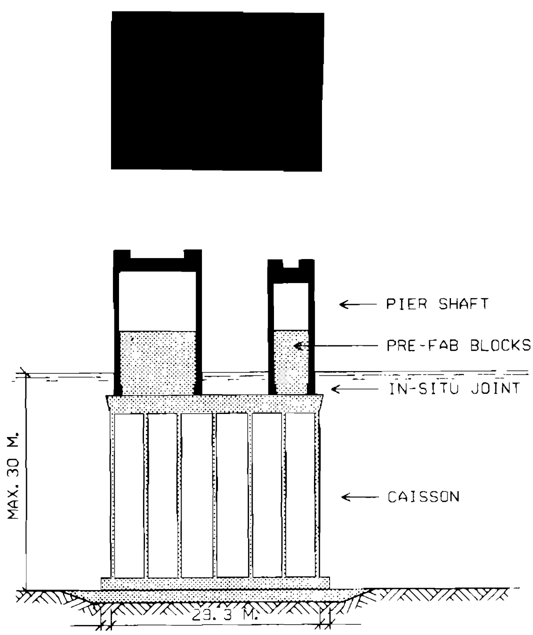

Figure 8. Caisson and pier shafts.

The 7.2 m long railway shafts are of similar width and layout as the roadway shafts, but are stiffened by one internal wall.

The shafts are up to 23 m high and attain a maximum element weight of 1,800 tonnes.

The pier shafts are connected to the caissons at level −3.5 m by casting the joint in the dry within a removable cofferdam on top of the caisson. This method was developed due to architectural requirements to avoid transition steps on the pier shafts above the water surface, see Figure 8.

The 25 m wide roadway is carried by a box girder, 12.5 m wide and 7.3 m deep at the pier. The deck is transversally prestressed with tendons each 600 mm, and the girder requires 19 tendons per web longitudinally. The tendons are concentrated near the web in the deck. The mid-span section is 3.8 m deep and requires 9 tendons per side in the bottom slab only. Throughout the girder length, the vertical webs are 500 mm thick.

The more sturdy 13 m wide railway girder deck is supported by a 7.1 m wide box, 8.7 m deep at the pier. The prestressing is arranged similarly to the road girder with 1,200 mm spacing in the deck and 18 tendons longitudinally. At mid-span, the section is 5.1 m deep and requires 8 longitudinal tendons per side.

All tendons are of the Losinger type, with 4 or 22, 15 mm diameter strands each.

All girders are subdivided to be prefabricated in five parts: three 18 m and two 27.2 m with a total length of 108.4 m. The closure joint left open between cantilevered girders is thus 2 m wide, and will be closed by *in-situ* concreting after installation. These joints represent the only cast-in-place concrete for the superstructure.

The girders weigh about 5,700 tonnes and 4,300 tonnes for the road and rail, respectively.

Prefabrication Onshore

The 324 elements are cast in five production lines at a reclaimed area named Lindholm in Nyborg Fjord about 4 nautical miles from the Funen abutment. The maximum element weight is 7,100 tonnes (caissons).

The elements are cast, moved, stored, and loaded out on piled production lines without use of heavy gantry cranes or drydocks. Sliding surfaces are established to move all elements.

There are two batching plants and sufficient storage area for aggregates which are shipped in from Norway.

The further transportation and installation is performed by a large purpose-built heavy-lift catamaran crane vessel, "The Swan", with overall dimensions 94 × 65 m and 74 m high.

The catamaran crane vessel was delivered to the site in January, 1991, from Rotterdam. The pontoons had been fabricated in Spain and towed to Holland for erection of the crane structure. The vessel then had to undergo special testing before certification. Lifting tests using a ballasted caisson were performed at the site.

The nominal lifting capacity of the vessel is limited to 6,900 tonnes. The few heavier caissons will be first lifted out and placed in the offshore construction yard where final casting of cover slabs and counterforts takes place and the cofferdams are mounted. The finally equipped heavy caissons will then be transported in a partly submerged condition.

Erection Offshore

Soil is excavated by a bucket dredger that can reach down to −34 m. A jack-up rig is then moved in to clean the foundation bed, place and compact crushed stone, and level the top layer to exacting tolerances.

The Swan lifts out a caisson from the jetty, transports it to the final position, and lowers it into the stone bed, where it will rest without any grouting underneath.

The Swan then places the pier shaft, which is cast to the caisson inside the temporary cofferdam. The caissons are filled with sand, the excavation area backfilled, and a two-layer scour protection placed around the caisson on the seabed.

Bearings are placed on top of the pier shaft, but not grouted immediately. The bearings are designed for 50 and 10 MN vertical and horizontal forces, respectively, due to ship collision risks. They are of the pot type and guided by externally arranged guides. The temporary girder supports consist of three jacks on either side of the bearings and a central guiding structure to facilitate the correct positioning of the girder later on.

The girder is transported by the Swan and slowly lowered onto the temporary supports, the joints between girder pairs are closed, and the remaining tendons added, prestressed, and grouted. Then the bearings are connected to the piers and girders resulting in simply supported spans with two cantilevers.

Starting from the pier with the longitudinally

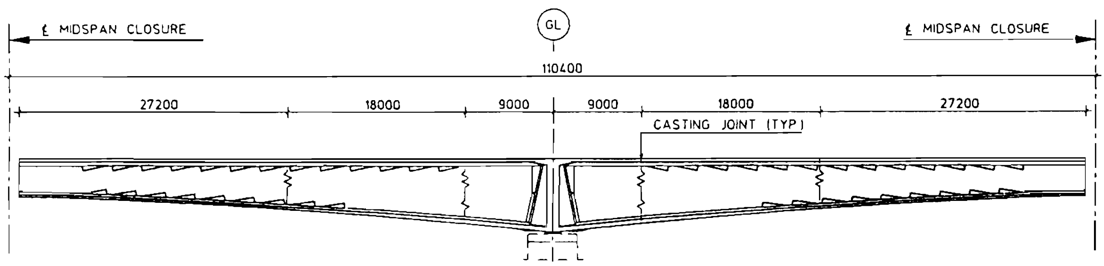

Figure 9. Longitudinal section in superstructure.

fixed bearing, girder pairs are connected subsequently by moving each pair into a predeterminate position before joining, to introduce the required presetting for movements due to creep and shrinkage. Within the limitations of prefabrication, the bearings can be used with a zero preset only.

Time Schedule

The current time schedule anticipates that railway operation will start in 1995. This can be achieved by the present production and erection rate by which ESG produces 1 span (one caisson, two shafts, and two girders) in two weeks.

THE EAST BRIDGE

The high level motorway bridge across the eastern channel of the Great Belt will be the landmark of the link spanning the international shipping route between the Baltic and the North Sea (Figure 10). Following an intensive study of cable stayed and suspension bridge concepts, a suspension main span of 1,624 m was selected.

The bridge will carry four lanes of motorway and two emergency lanes from Zealand to the island of Sprogø using 2 percent grades to allow a 65 m clearance under the main span. The bridge will land on a protruding embankment leading to Sprogø island and the low level West Bridge.

The total length of the suspension bridge is 2,694 m and the approach bridges total approximately 4 km: 2,544 m for the eastern approach and 1,552 m for the western approach (Figure 11).

COWIconsult developed the conceptual design, and the tender design was elaborated by a Danish consultant joint venture, CBR, comprising COWI-

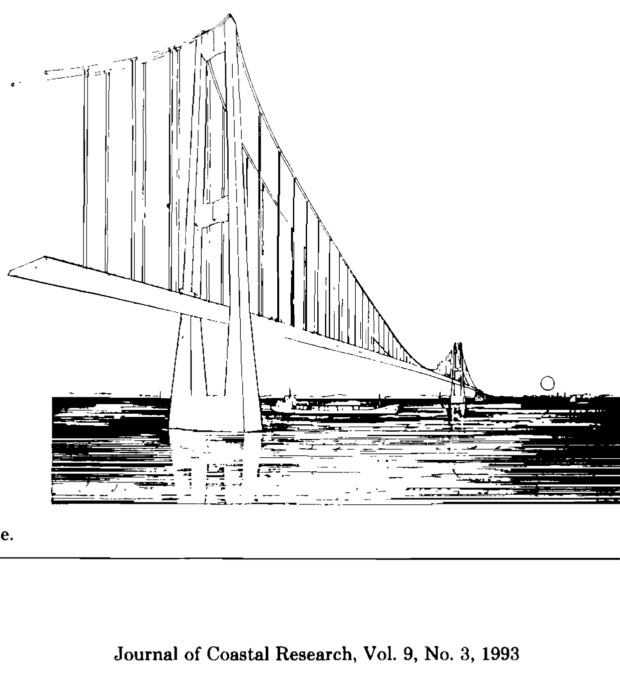

Figure 10. The East Bridge.

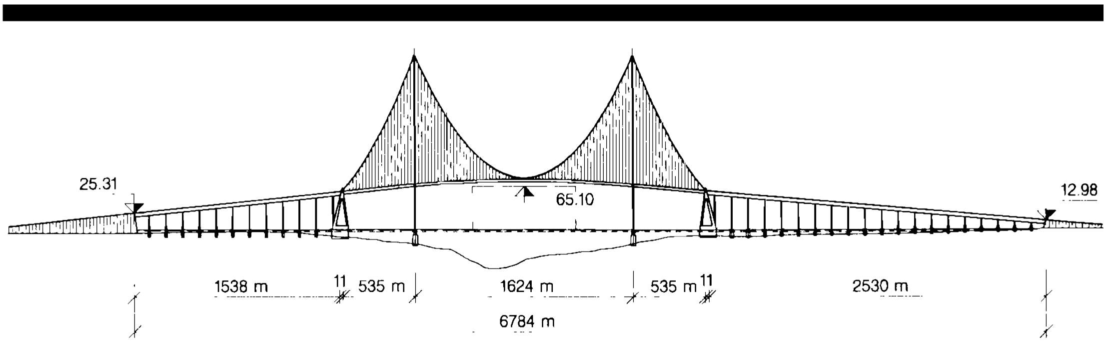

Figure 11. East Bridge elevation.

consult, B. Højlund Rasmussen and Rambøll & Hannemann. The aesthetical evaluation has been made in collaboration with the architects Dissing & Weitling and the landscape architect Jørgen Vesterholt.

Construction of the East Bridge commenced in October, 1991, by signing construction contracts with two international consortia: The GBC joint venture for the substructures, and CMF Sud S.p.A. for the steel superstructures. At the same time, the consultant joint venture was continued for detailed design.

Aerodynamic Investigations

During tender design, 16 different road girder box section configurations were outlined and subjected to wind tunnel section model test at the Danish Maritime Institute.

Wind tunnel testing determined the critical wind speed for flutter for the selected box girder shape to be 74 m/sec and 72 m/sec without and with a windscreen on the girder, respectively, which is safely above the design critical wind speed of 60 m/sec.

Aerodynamic investigations for the detailed design of the bridge are being continued in a new wind tunnel facility dedicated to aeroelastic full bridge model tests. The tunnel cross section is 14 × 1.6 m, which permits testing of a 1:200 scale model under simulated turbulent wind conditions.

Erection of the box girder suspension bridge presents particular aerodynamic stability problems due to temporary lack of torsional rigidity between box sections and low mass (no superimposed dead load) in combination with full exposure of the girder surface to the wind.

Enhancement of critical wind speeds through careful selection of erection sequence or provision of eccentric ballast has been studied experimentally and theoretically. The studies continue during the design phases in cooperation with the superstructure contractor.

Girder

The girder cross section is arranged as an aerodynamically shaped fully welded closed box section with an orthotropic steel top deck (Figure 12). The inside bulk heads are arranged as a truss system, proposed as a variant modification by the contractor. The trusses are located at 4 metre distances and support the road deck, side panels, and bottom panels.

The main span is designed with an optimal main cable sag corresponding to $\frac{1}{9}$ × span length. Vertical hangers each 24 m support the girder. The bridge is arranged with an unusual innovative articulation allowing the girder to be continuous over the full length of 2.7 km between the two anchor blocks. The traditional expansion joints at the tower positions are thus avoided. Compared to a system with joints at the pylons, analyses have further indicated an approximately 25% reduction in the longitudinal movement of the girder from asymmetric traffic load.

Hydraulic buffers between the anchor blocks and the girder partly restrain longitudinal short term movements. The buffers allow for slow horizontal movements up to ±1 m and free rotation of the girder. Remaining portion of the theoretical movement will be eliminated by restraining forces in the girder. If free movements were allowed, the extreme horizontal movement from the charac-

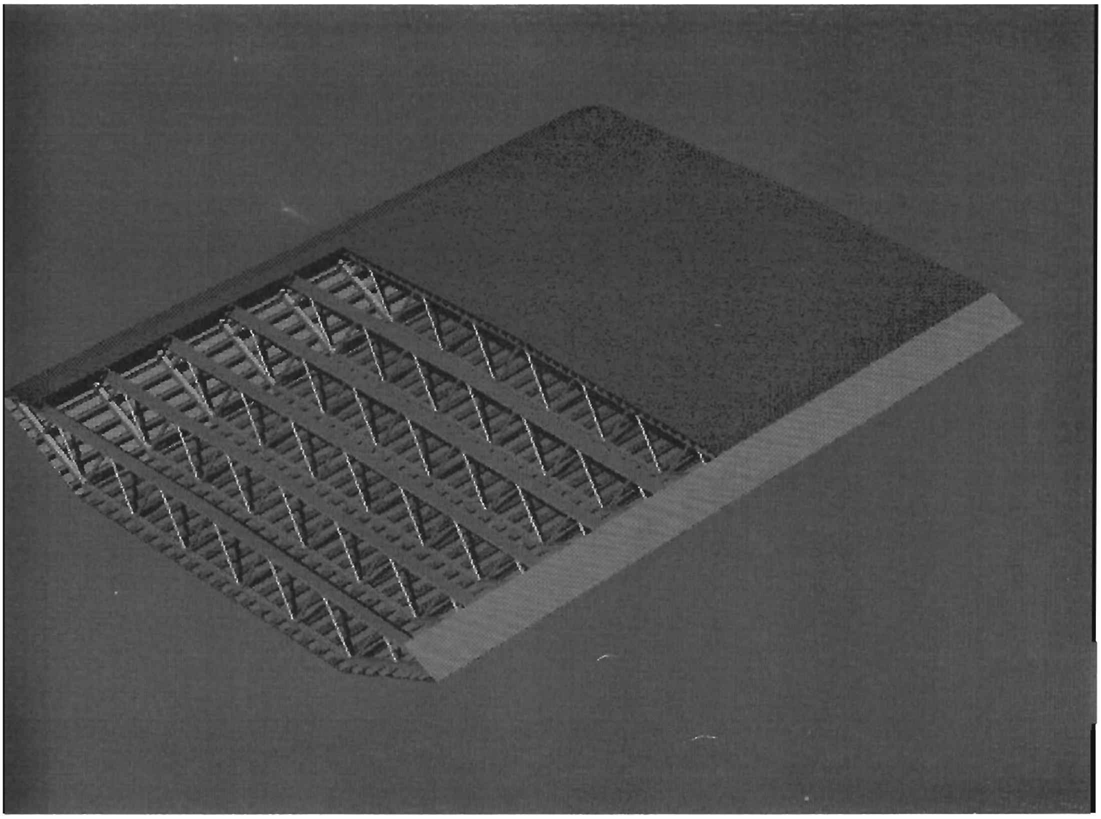

Figure 12. Suspension bridge girder cross section.

teristic traffic load at the expansion joints would be ±1.8 m.

The continuous girder concept leads to reduced installation and maintenance costs for the expansion joints at the anchor blocks, and improved stiffness and aerodynamic stability. This arrangement has also permitted elimination of the traditional cross beams on the pylons immediately below the girder, and thus a considerable aesthetic improvement has been reached.

With a completely smooth exterior surface and all panel stiffeners arranged inside, the box girder is suitable for rationalized repetitive fabrication of panel production, box assembly, and barging to the bridge site.

The interior box surfaces are unpainted and protected by dehumidification of the inside air volume. About 80% of the total steel surface is thus unpainted with substantial cost and time savings as a result.

Cable System

The length of the main cable will be approximately 3,000 m. The steel area for each cable will be 0.42 m^2 leading to an outer diameter of 0.82 m.

The construction contract is based on using the aerial spinning (AS) method, whereas the tender documents allowed either AS or the more recent prefabricated strand method to be used. Each cable will comprise 37 stands, each with 504 galvanized wires, 5.38 mm in diameter, or altogether 18,648 wires. The amount of steel in the main cables is 19,700 tonnes.

When the air spinning is finished, an outer surface protection of the cable is performed by a wrapping wire laid in a zinc paste. Close to the pylon top the cable passes through a steel cover ensuring the required protection.

At the anchorage chambers, the 37 strands are

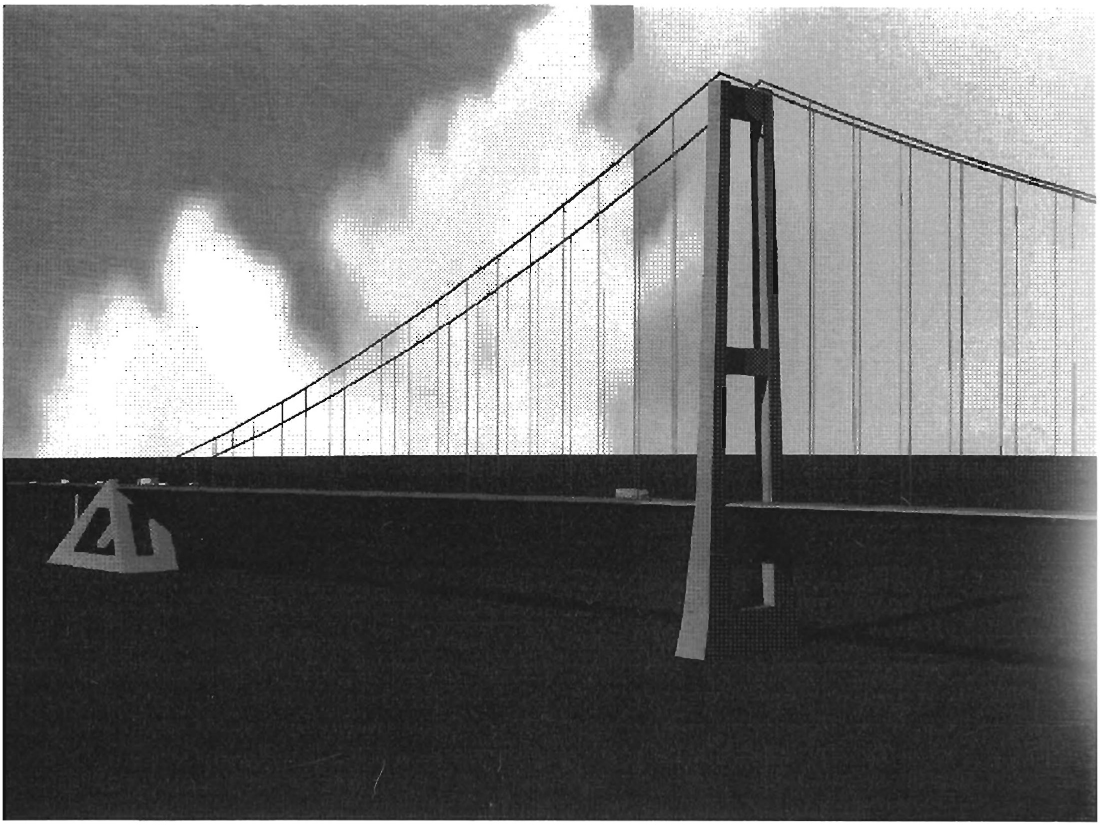

Figure 13. Pylon.

splayed out in horizontal and vertical planes to distribute the large load concentration. Each strand is anchored in a strand shoe which via bolts is fixed to the cast in anchorage in the concrete.

Tenders (bids) were invited for hanger cables of the parallel wire type and the locked coil type, but the construction will apply parallel wires.

Each hanger consists of two strands spaced 0.5 m in the longitudinal direction of the bridge. The nominal sectional area of one strand is typically 26 cm^2. The galvanized wires are protected by an 8 mm thick plastic sheath.

The upper hanger socket is fixed to the cable clamp by a pin connection. At the lower end, the socket supports a steel tube anchorage which is built into the girder diaphragm and thus in a corrosion protected area.

Two cast steel parts form the upper and lower half of the cable clamp. Prestressed bolts connect the parts and ensure the required friction capacity to resist the inclined hanger load.

To deviate the cable at the pylon tops and at the anchor blocks, the cable is supported by saddles with a curvature of approximately 7 m.

The saddle consists of an upper cast steel trough welded to a supporting plate steel structure with bottom dimensions designed for distribution of the heavy load into the concrete surfaces.

The saddles at the pylon tops are fixed to the pylon whereas the anchor block saddles are of the rocker type; *i.e.*, allows for angular rotations.

Pylon saddles weigh 100 tonnes each, and anchor block saddles 130 tonnes each.

Pylons

Rising 254 m above sea level, the pylon has slightly tapered legs with a rectangular, hollow cross section above the girder (Figure 13). Below

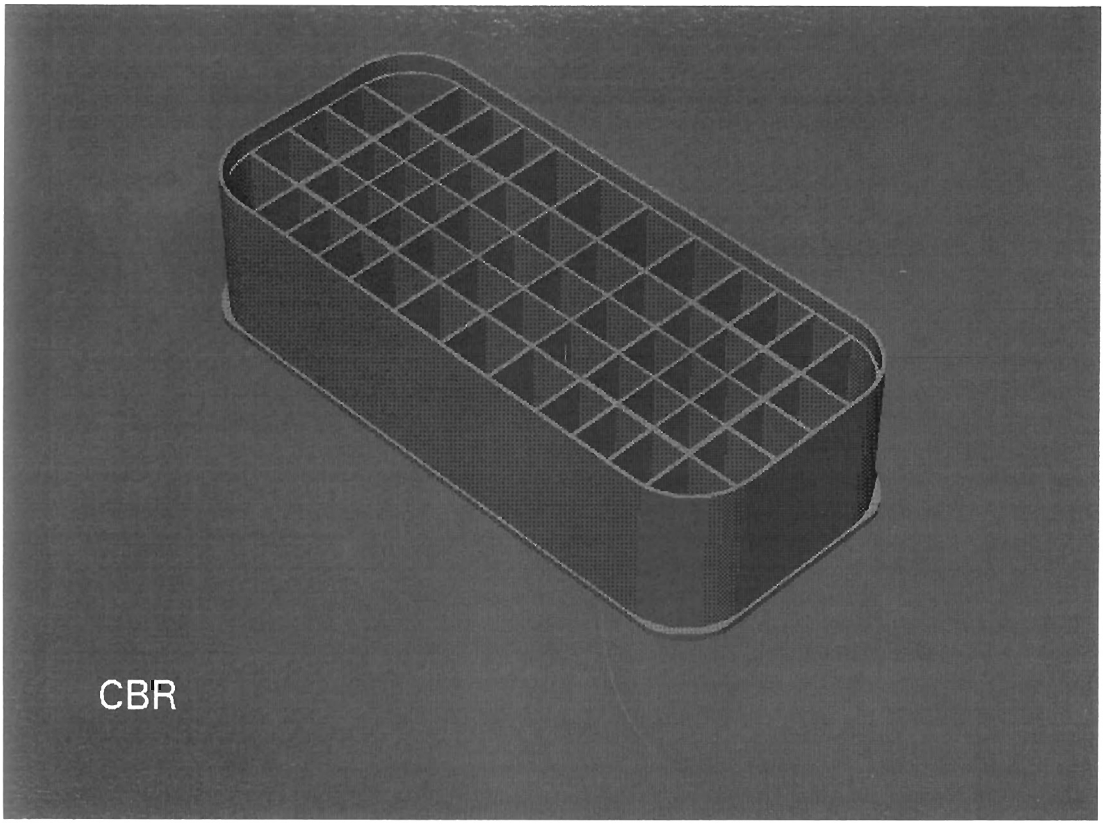

Figure 14. Pylon caisson.

girder level, the leg cross section taper is increased to achieve adequate proportions to resist the large horizontal forces from wind on the girder and accidental ship impact loads.

Both pylons will be supported by foundation caissons placed in water approximately 20 m deep. After excavation of an approximately 10 m deep layer of soft till, a trimmed bed of 5 m crushed stone will be placed.

The cellular 78 × 35 m caisson, 20 m high, and weighing 30,000 tonnes (Figure 14), will be cast in a dry dock off site in Kalundborg and towed about 20 nautical miles to the bridge site, placed on the stone bed, and sand filled. The caisson is equipped with 0.5 m high skirts which will penetrate 0.3 m into the stone bed.

The contact between the stone bed and the caisson will be secured by grouting below the base, the skirts serving as grout confining barriers.

The corners of the caisson are rounded to minimize water drag and reduce the effects of ship impact.

The foundations are designed to resist impact from ships up to 250,000 DWT without permanent deformations or movements. The design impact load is 670 MN for such a vessel.

The lower 21 m of the pylon above the caisson is designed as a monolithic structure with heavily reinforced 1.2 m thick walls to resist impact loads of 400 MN.

The pylon legs are hollow. The exterior dimensions vary from 8 × 9 m at the girder level to 6.5 × 7.5 m at the top. The wall thicknesses vary from 1.5 m to 2.0 m.

The foundation works below the caissons will consist of:

(1) Dredging with a bucket dredger (which minimizes risk of disturbance of boulder clay).
(2) Clearing away siltation left from the exca-

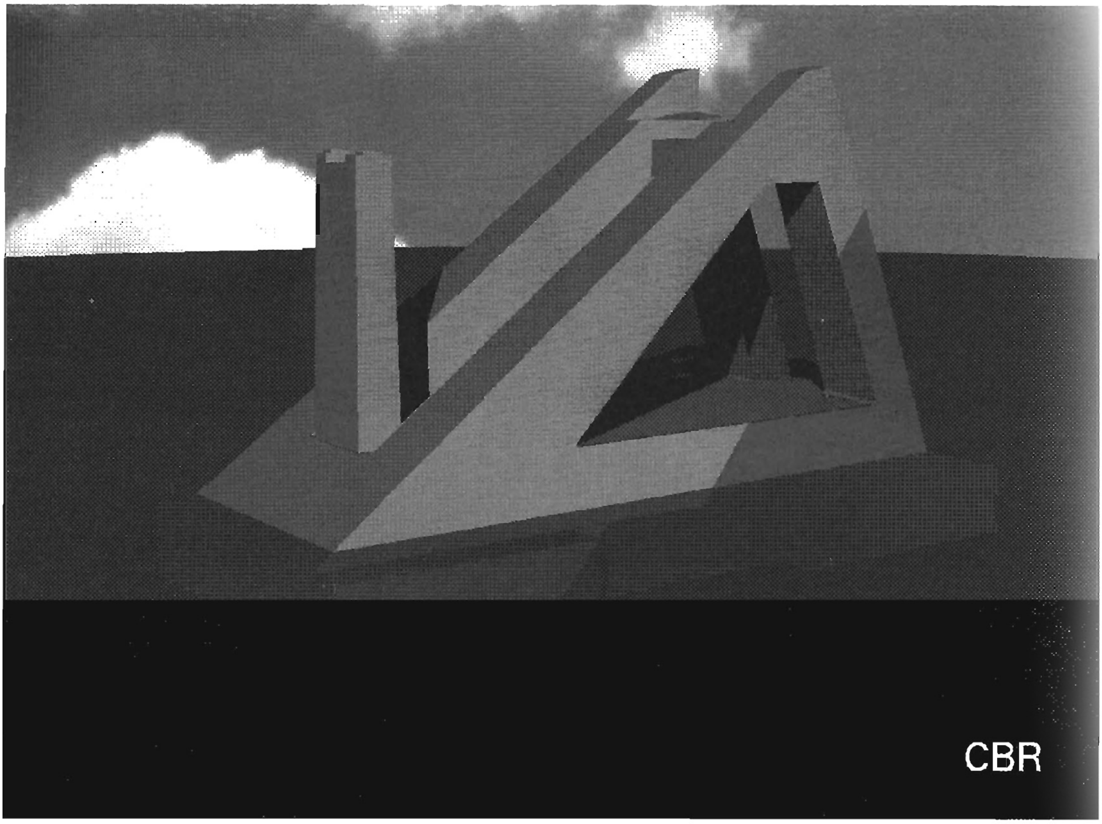

Figure 15. Anchor block.

vation with a dumping vessel equipped with a suction device.
(3) Placing the crushed stone material with a side dumping vessel.
(4) Compacting by plate vibration from a floating barge equipped with a crane and the vibration units.
(5) Screeding the foundation top layer to 5–10 cm tolerance.

When flooding the dry dock in the Kalundborg harbour area and before tow-out and transport of the caissons, an exit channel down to level −13 m has to be dredged.

The caisson, which has a 11.5 m draft, will be towed to the site by pulling and steering tugs as done for GBS offshore platforms.

A mooring pontoon will be anchored at the bridge site on a pre-installed mooring system (anchors), and the pontoon will be the guidance for placing the caissons. Pump systems for lowering and trimming will be pre-installed on the caissons. Ballasting with water will be used to reach the required depth.

Anchor Blocks

Located at a water depth of approximately 10 m, the anchor blocks shall resist cable forces of 600 MN (Figure 15). Excavation to 25 m below sea level is necessary for the construction works to construct a wedge shaped foundation base suitable for large horizontal loading.

The soil conditions are stiff, preconsolidated glacial boulder clay below a thin post-glacial deposit. It was found that the stability of the natural soil deposit was less critical than the risk of sliding along a thin weakened zone of the excavated boulder clay surface at the interface. Therefore, two

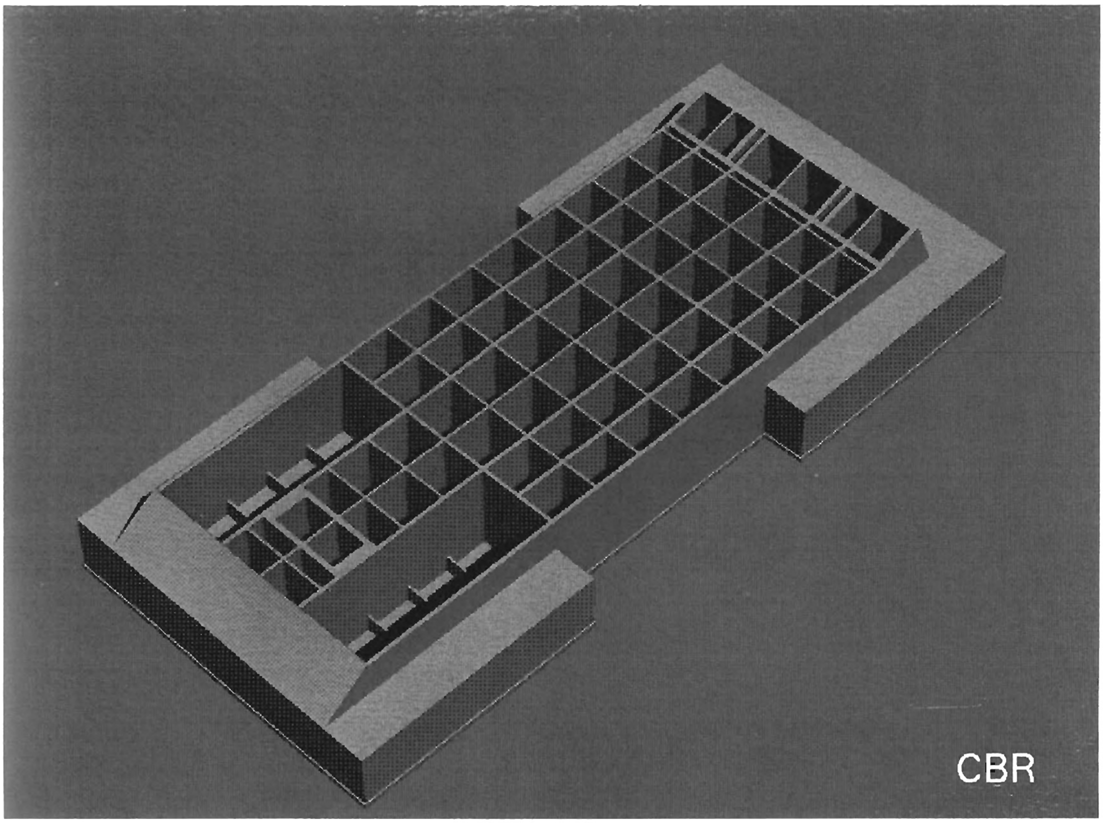

Figure 16. Anchor block caisson.

wedge shaped stone beds with an inclination of 16° are designed to reduce the shear stress in the interface gravel/boulder clay. The resulting force from the pull of the cable and the weight of the anchor block has, likewise, an inclination of approximately 16° with the vertical, and thus perpendicular to the interface.

Reconsolidation is achieved at the two inclined excavation surfaces, and shear strength of the remoulded clay is improved. The contact between the stone beds and the main foundation areas at the front and the rear ends of the caisson will be secured by grouting below the base, whereas the narrower interconnection portion of the caissons will be separated from contact with the underlying strata.

Extensive non-linear FEM-analyses have been used to demonstrate the carrying capacity of the soil.

Each anchor block caisson covers 6,100 m^2 and weighs 36,000 tonnes in the transport stage. They contain approximately 100 cells which will be ballasted with sand and the heavier olivine (Figure 16). Construction work involves basically the same type of operations as for the pylons.

On top of the caisson, inclined legs reach above the bridge girder to accept the main suspension cables into an interior saddle point. Between the apexes of the legs, a cross beam is provided where the expansion joint between the main bridge and the approach spans is arranged.

Artificial islands will be constructed around the anchor blocks to ease the water flow around the large structures (improve drag coefficient). The islands are also favourable as a protection against ship impact, and act furthermore as depots for surplus soil dredged in the Belt.

As the offshore concrete works on the anchor blocks and pylons are very intensive, a floating construction site will be arranged around them. It will consist of a 90 × 27 m batching plant barge, work/storage barges for reinforcing steel, shutterings and form work, offices, locker rooms and canteens, etc.

The concreting activities around the pylons and anchor blocks will last almost two years.

Approach Bridge Substructures

The requirement that any blocking effect of the bridge piers or embankments in the Belt must be compensated for by dredging the sea bed to accomplish unchanged water exchange between the North Sea and the Baltic Sea leads to deeper bridge pier foundation, and worse: possible access by stray ships of larger draft and impact on the bridge piers, ships that would otherwise have grounded.

The piers have been designed for impact from vessels ranging from 4,000 DWT near the abutment up to 60,000 DWT close to the anchor blocks based on the probabilistic analysis.

The sea bed is between level −3 m and −10 m; but due to the compensation dredging works, most of the piers are designed for foundation levels about −9.5 m.

The offshore conditions at the bridge site favour a maximized production on shore and minimum critical path time at sea. Therefore, the design is focused on repetitive prefabrication of large units.

In the selected concept for the piers, a large bottom caisson is placed on a prepared bed of crushed stones. The caisson will be prefabricated in a dry dock and towed or barged to the site for placing.

Underneath the caissons, grouting will ensure a uniform bearing pressure to prevent deformations and cracks in the caissons of limited height and minimize settlement as well as improve accidental ship collision carrying capability.

To achieve weight without increase of dimensions, heavy fill of olivine or iron ore will be filled into both caissons and pier shafts.

Nine pier caissons and all the lower and upper shafts, 18 respectively, will be precast outside the dock at Kalundborg harbour, 30 nautical miles from the bridge site. The caissons have a total weight up to 2,600 tonnes, lower pier shafts from 400 to 2,000 tonnes, upper pier shafts from 400 to 1,700 tonnes.

Both piers and shafts will be loaded onto a transport barge by means of a special sliding device and/or sheerlegs.

The transport barge will be towed by tugs to the site escorted by extra tugs and guard vessels in the channel. The piers and shafts are lifted by sheerlegs and placed in position. The lower shafts will be placed on top of the piers, and the upper shafts on top of the lower part.

Approach Bridge Superstructures

Due to the ship impact requirement, optimization of the superstructure indicates the need for fairly long spans. The heavier reaction on the piers from the longer spans is beneficial for the ship impact resistance. Furthermore, fewer piers means lower probability of a ship hitting a pier and minimizes water flow obstruction. The construction contract is thus based on 193 m full steel spans erection with expansion joints only at the abutments and at the anchor blocks (max. continuous length 2.5 km).

The girder is designed as a closed steel box girder with a central longitudinal bulk head (Figure 17). The material is high-strength steel with a yield point of 400–420 Mpa.

As for the suspension bridge girder, all stiffeners on the straight steel panels are placed internally in the box girder for a smooth external surface which facilitates painting and easy maintenance, whereas the interior will be protected against corrosion by dehumidification. Each approach bridge will be divided into interior sections corresponding to four or five spans, closed at the ends by diaphragms with sealed doors to permit separate dehumidification plants to operate.

The 2,300 tonnes full span girder sections can be transported on a barge and lifted onto the top of the pier.

Time Schedule

The East Bridge is due to open for traffic in 1997. Completed on time, it will hold the world record span length for a short period.

CONTRACTORS AND CONSULTANTS

East Tunnel

MT Group, a consortium of the following contractors: Monberg & Thorsen (Denmark), Campenon Bernard (France), Dyckerhoff & Widmann (Germany), Kiewit Construction Company (USA) and SOGEA (France).

Consultants, Joint Venture COWI/MOTT: COWI-

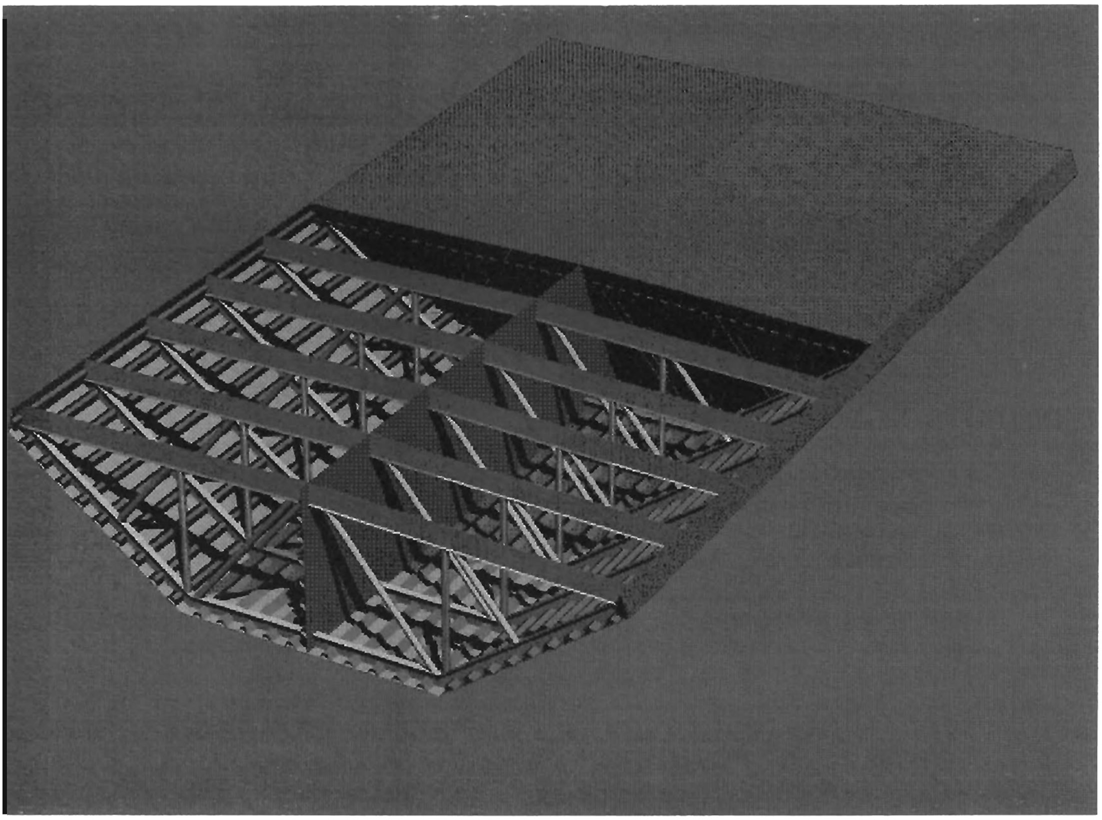

Figure 17. Approach bridge girder cross section.

consult (Denmark) and Mott MacDonald Group (UK).

West Bridge

European Storebælt Group, a consortium of the following contractors: Højgaard & Schultz (Denmark), Ballast Nedam (Holland), Taylor Woodrow (UK), Losinger (Switzerland), C.G. Jensen (Denmark), and Per Aarsleff (Denmark).

Consultants, Joint Venture CCL: COWIconsult (Denmark), Carl Bro Group (Denmark), and Leonhardt, Andrä und Partner (Germany).

East Bridge

Great Belt Contractors, a consortium of the following contractors (for the substructures): Hochtief (Germany), Wayss & Freytag (Germany), Hollandische Beton- en Waterbouw (Holland), KKS Entreprise (Denmark), and E. Pihl & Søn (Denmark).

Firm of contractors CMF Sud, Italy, in association with Steinman Boynton Gronquist & Birdsall, USA (for the superstructures).

Consultants, Joint Venture CBR: COWIconsult (Denmark) and Rambøll, Hannemann & Højlund (Denmark).

Sprogø Island

CD Joint Venture, a consortium of the following contractors: Jan de Nul Dredging (Belgium) and Amsterdam Ballast Dredging (Holland).

Consultants, HLD Joint Venture: Hostrup-Schultz & Sørensen (Denmark), LIC Engineering (Denmark), and Danish Hydraulic Institute (Denmark).

Railway Installations

DSB (Danish State Railways).

Consultants: DSB/Danish Road Directorate.

Consultants of mechanical installations, Joint Venture COWI/MHAI: COWIconsult (Denmark) and Mott MacDonald Group (UK).

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